Ocean Structures

Construction, Materials, and Operations

Srinivasan Chandrasekaran
Arvind Kumar Jain

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Preface

The basis for the development of this book, *Ocean Structures: Construction, Materials, and Operations*, grew while the authors were successfully completing their instruction of a few online courses in India and abroad. The demand from graduate students and practicing engineers to develop a full-length textbook on ocean structures and materials has also been felt by the authors during their teaching careers of approximately 25 years. This motivated the authors to develop this textbook, which follows a classroom model, describing the concepts through clear explanations, illustrations, and tutorials. The subject of this textbook is widely taught as a core course in many engineering disciplines: ocean engineering and naval architecture, civil engineering, applied mechanics, offshore structural engineering, and petroleum engineering at both the undergraduate and postgraduate levels. This book will also serve as self-reading material as complementary lecture slides are included with this book. Illustrations are provided to explain the concept and improve understanding of the subject matter. This book offers chapters on different forms of ocean structures including coastal protection structures. Construction methods, materials used for construction, and repair and rehabilitation methods discussed in this book are of a very high value. Recent geometric forms of offshore structures developed by the authors as a part of research are included in the content. This book can also be used as a reference for research scholars. It attempts to enrich the reader's knowledge of ocean structures and materials including health monitoring of offshore structures, which is a relatively new attempt. This book also exposes readers to the various important aspects of ocean structures. It presents the concepts of material selection, analysis, choice of structural form, construction methods of repair, and rehabilitation of ocean structures in detail. Recent research studies and a couple of case studies, which were successfully completed by the authors, are also included. As ocean structures are one of most expensive infrastructures, it is imperative to understand the different types of ocean structures, the materials used for construction, basis analysis, and design to enhance capacity building in this domain of professional practice. The subject covered in this book is therefore of societal importance and of interest to the engineering community.

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Srinivasan Chandrasekaran, PhD, is a professor in the Department of Ocean Engineering, Indian Institute of Technology Madras, India. He has 25 years of teaching, research, and industrial experience, during which time he has supervised many sponsored research projects and offshore consultancy assignments, both in India and abroad. Dr. Chandrasekaran's active areas of research include dynamic analysis and design of offshore platforms, development of geometric forms of compliant offshore structures for ultra-deepwater oil exploration and production, subsea engineering, rehabilitation, and retrofitting of offshore platforms, structural health monitoring of ocean structures, seismic analysis and design of structures, and risk analyses and reliability studies of offshore and petroleum engineering plants. He has published 130 research papers in international journals and refereed conferences organized by professional societies around the world. Dr. Chandrasekaran has authored seven textbooks, which are popular among the graduate students of civil and ocean engineering. These textbooks are recommended as reference material for classroom studies, as well as research in many universities in India and abroad. He is a member of the American Society of Civil Engineers (ASCE). He has been a Visiting Fellow of the Ministry of Italian University Research (MiUR) for 2 years, during which time he also participated in the International Program on Earthquake Engineering (ReLuis).

Arvind Kumar Jain, PhD, is a professor of structural engineering in the Department of Civil Engineering, Indian Institute of Technology Delhi, India. He has 30 years of teaching, research, and industrial experience. Dr. Jain has authored several papers published in international journals of a high impact factor. He is the former head of the Department of Civil Engineering, IIT Delhi. His research interests include structural dynamics, offshore structures, tall structures, and reliability engineering. He is a member of the American Society of Civil Engineers (ASCE) and the International Society of Offshore and Polar Engineers (ISOPE). Dr. Jain has undertaken a large number of consultancy projects in the areas of tall buildings, earthquake analysis, water tanks, treatment plants, communication towers, bridges, and more. He has visited numerous countries and is on the board of several organizations, including the All India Council for Technical Education (AICTE), the Council of Scientific and Industrial Research (CSIR), and the Union Public Service Commission (UPSC).

1 Ocean Structures

Ocean structures are unique in design and geometric form apart from the types of environmental loads they are subjected to. In addition to the complexities that arise from their functions and operational conditions, their structural forms are as interesting as they are highly innovative. Similarly, structures that are built for coastal protection have a variety of functional variations with a high degree of common geometric forms. In this chapter, different types of ocean structures that are built to cater to a variety of functional requirements are discussed from a structural engineering perspective. A brief introduction to various terminologies related to oil and gas exploration is also presented.

1.1 INTRODUCTION

Ocean structures are of multidisciplinary interest, attracting professionals with backgrounds in civil and structural engineering and naval architecture. They are also of interest to graduate students of mechanical, electrical, and chemical engineering, and furthermore to students with chemistry and physics backgrounds. This chapter provides an overview of different types of ocean structures, which are generally deployed for oil and gas exploration at sea. Various structural systems that are deployed for shallow water, medium water, deepwater, and ultra-deepwater vary in their geometric configurations. It is rather fascinating to know that structural systems deployed at different water depths are not similar. Under environmental loads, their structural actions vary widely due to their compliancy.

1.2 OFFSHORE INDUSTRY

To understand the basis for designing ocean structures, it is vital to understand the early era of the offshore industry. For example, see Figure 1.1, which refers to a historic photograph of Huntington Beach, California (Chandrasekaran, 2013a). The photo shows several drilling rigs deployed along the coast; a few towers supporting the drilling rigs are also seen. One can also readily observe that most of the structures are built of either steel or wood. Structural configurations consist of truss elements, which are assumed to be the most primitive type of support systems for offshore drilling operations. Furthermore, drilling rigs are located very close to the coast, ensuring that oil was explored at a shallow depth during early days (Chandrasekaran, 2013b).

A similar thing happened in Summerland, California. Figure 1.2 shows a series of drilling rigs, located along the beach side in Summerland. The basic form of the drilling derrick is a truss type, which supports a group of drilling rigs in series. Figure 1.3 shows a photo of production platforms in Summerland. The structural form appears to be stiff and rigid, conveying a meaning that such an insensitive system is required to support offshore drilling activities (William et al., 1984, 2011).

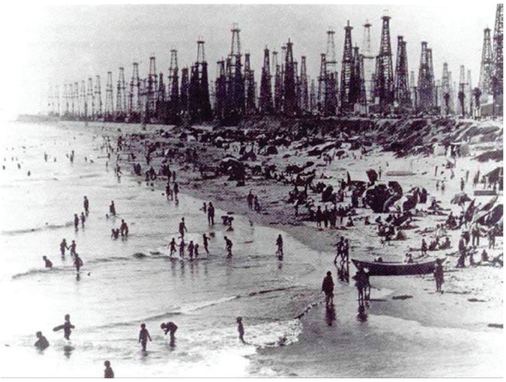


FIGURE 1.1 Huntington Beach, California.

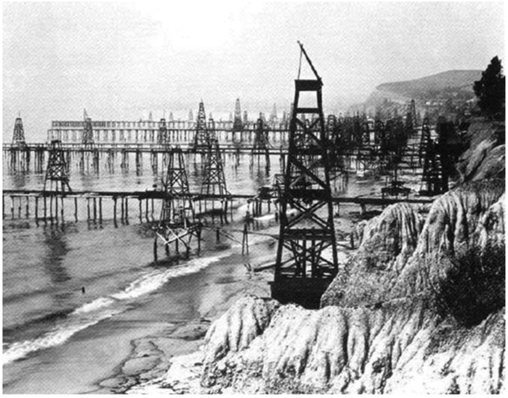


FIGURE 1.2 Summerland, California.

Figure 1.4 shows a typical drilling platform, commissioned in Venezuela, whereas Figure 1.5 shows drilling platforms deployed in the Caspian Sea. A taller truss system with two deck levels, one for drilling operations and the other for maintenance of service, is shown as the structural configuration. One can also notice a few cantilever members extending out to facilitate the operation circumferentially, and the material for construction is essentially steel. It is evident that as the drilling platforms move away from the coast toward seaside, structural configuration becomes more complex (Adams and Baltrop, 1991). This includes an increase in the height of the drilling derrick, member sizes, and so on. One can infer from the above example that a stiff system is preferred to alleviate the environmental loads encountered. It is also evident (see Figure 1.5) that offshore structures are connected to the coast for transporting the explored hydrocarbons for further processing. As they are deployed very close to the coastline, pipelines and barges were not required to transport oil from the production unit to the processing unit. Because of the paucity of space on the drilling platforms, upstream activities, such as drilling and production, and downstream activities, such as processing, were carried out offshore and onshore,

FIGURE 1.3 Production platforms in Summerland, California.

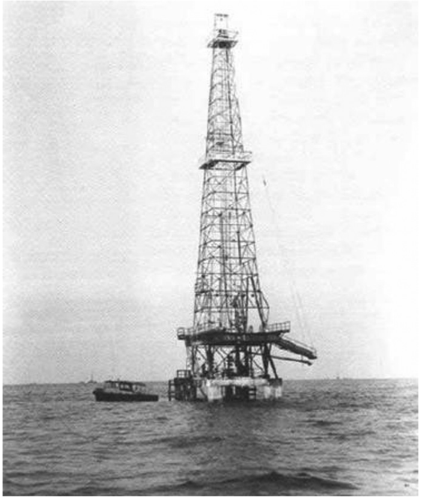


FIGURE 1.4 Lake Maracaibo, Venezuela.

respectively (Chandrasekaran, 2015a,b). It is also important to note that these drilling platforms have very little storage capacity (Adrezin et al., 1996).

The offshore industry originated as early as 1891, when one of the earliest oil wells was drilled at Grand Lake, St. Mary, Ohio. A chronology of drilling activities is given as follows:

- 1891: The first oil well was drilled at Grand Lake, St. Mary, Ohio
- 1896: The first submerged oil well in saltwater was drilled in the Summerland Field, extending to the Santa Barbara Channel in California

FIGURE 1.5 Drilling platforms in the Caspian Sea.

- 1900: Early submerged drilling activities occurred on the Canadian side of Lake Erie
- 1910: Caddo Lake in Louisiana
- 1920: Drilling started from concrete platforms in Lake Maracaibo, Venezuela
- 1923: The oldest subsea well was recorded in Infield's offshore at Bibi Eibat Well, Azerbaijan
- 1930: A Texas company developed the first mobile steel barges for drilling
- 1937: Pure Oil Company, which later became a part of Chevron Corporation and Superior Oil Company, and then a part of ExxonMobil Corporation, used fixed-type platforms to develop a field in 4.2 m water depth at one mile off the coast of Calcasieu Parish, Louisiana
- 1946: Magnolia Petroleum Company, which later became a part of ExxonMobil, commissioned a drilling platform in 5.4 m water depth at 18 miles off the coast of St. Mary Parish, Louisiana
- 1947: Superior Oil Company erected a drilling/production platform in 6 m water depth at Vermilion Parish, Louisiana
- 1947: Kerr-McGee Oil Industries, which later became a part of Anadarko Petroleum Corporation, Phillips Petroleum (Conoco Phillips), and Stanolind Oil & Gas (BP), completed their historic Ship Shoal Block 32 well in October 1947

Table 1.1 provides a list of fixed offshore platforms constructed worldwide.

As the table indicates, a large group of platforms were initiated in the United States and Europe. To correlate the investment to an economic perspective, it is important to

TABLE 1.1
Fixed Offshore Platforms around the World

S. No.	Platform Name	Water Depth (m)	Location
		North America	
1	East Breaks 110	213	United States
2	GB 236	209	United States
3	Corral	190	United States
4	EW910-Platform A	168	United States
5	Virgo	345	United States
6	Bud Lite	84	United States
7	Falcons' Nest production	119	United States
8	South Timbalier 301	101	United States
9	Ellen	81	United States
10	Elly	81	United States
11	Eureka	213	United States
12	Harmony	365	United States
13	Heritage	328	United States
14	Hondo	259	United States
15	Enchilada	215	United States
16	Salsa	211	United States
17	Cognac	312	United States
18	Pompano	393	United States
19	Bullwinkle	412	United States
20	Canyon Station	91	United States
21	Amberjack	314	United States
22	Bushwood	210	United States
23	Hebron	92	Canada
24	Hibernia	80	Canada
25	Alma	67	Canada
26	North Triumph	76	Canada
27	South Venture	23	Canada
28	Thebaud	30	Canada
29	Venture	23	Canada
30	KMZ	100	Mexico
		South America	
1	Peregrino Wellhead A	120	Brazil
2	Hibiscus	158	Trinidad and Tobago
3	Poinsettia	158	Trinidad and Tobago
4	Dolphin	198	Trinidad and Tobago
5	Mahogany	87	Trinidad and Tobago
6	Savonette	88	Trinidad and Tobago
7	Albacora	165	Peru

(*Continued*)

TABLE 1.1 (*Continued*)
Fixed Offshore Platforms around the World

S. No.	Platform Name	Water Depth (m)	Location
		Australia	
1	Reindeer	56	
2	Yolla	80	
3	West Tuna	85	
4	Stag	49	
5	Cliff Head	37	
6	Harriet Bravo	24	Australia
7	Blacktip	50	Australia
8	Bayu-Undan	80	Australia
9	Tiro Moana	102	New Zealand
10	Lago	200	Australia
11	Pluto	85	Australia
12	Wheatstone	200	Australia
13	Kupe	35	New Zealand
		South America	
1	Peregrino Wellhead A	120	Brazil
2	Hibiscus	158	Trinidad and Tobago (Continental shelf of South America)
3	Poinsettia	158	Trinidad and Tobago (Continental shelf of South America)
4	Dolphin	198	Trinidad and Tobago (Continental shelf of South America)
5	Mahogany	87	Trinidad and Tobago (Continental shelf of South America)
6	Savonette	88	Trinidad and Tobago (Continental shelf of South America)
7	Albacora	***	Peru
		Asia	
1	QHD 32-6	20	China
2	Peng Lai	23	China
3	Mumbai High	61	India
4	KG-8 Wellhead	109	India
5	Bua Ban	92	Thailand
6	Bualuang	60	Thailand
7	Arthit	80	Thailand
8	Dai Huang Fixed Wellhead	110	Vietnam
9	Ca Ngu Vang	56	Vietnam
10	Chim Sao	115	Vietnam
11	Oyong	45	Indonesia
12	Kambuna	40	Indonesia
13	Gajah Baru	54	Indonesia

(*Continued*)

TABLE 1.1 (*Continued*)
Fixed Offshore Platforms around the World

S. No.	Platform Name	Water Depth (m)	Location
14	Belumut	61	Malaysia
15	Bukha	90	Oman
16	West Bukha	90	Oman
17	Al Shaheen	70	Qatar
18	Dolphin	62	Qatar
19	Zakum Central complex	24	United Arab Emirates
20	Mubarek	61	United Arab Emirates
21	Sakhalin I	54	Russia
22	Lunskoye A	48	Russia
23	Molikpaq	30	Russia
24	Piltun-Astokhskoye-B	30	Russia
25	LSP-1	13	Russia
26	LSP-2	13	Russia
27	Gunashli Drilling and Production	175	Azerbaijan
28	Central Azeri	120	Azerbaijan
29	Chirag PDQ	170	Azerbaijan
30	Chirag-1	120	Azerbaijan
31	East Azeri	150	Azerbaijan
32	West Azeri	118	Azerbaijan
33	Shah Deniz Production	105	Azerbaijan
		Europe	
1	Brage	140	Norway
2	Oseberg A	100	Norway
3	Oseberg B	100	Norway
4	Oseberg C	100	Norway
5	Oseberg D	100	Norway
6	Oseberg South	100	Norway
7	Gullfaks A	138	Norway
8	Gullfacks B	143	Norway
9	Gullfacks C	143	Norway
10	Sleipner A	80	Norway
11	Sleipner B	80	Norway
12	Sleipner C	80	Norway
13	Valhall	70	Norway
14	Ekofisk Center	75	Norway
15	Varg Wellhead	84	Norway
16	Hyperlink	303	Norway
17	Draugen	250	Norway
18	Statfjord A	150	Norway
19	Statfjord B	290	Norway

(*Continued*)

TABLE 1.1 (*Continued*)
Fixed Offshore Platforms around the World

S. No.	Platform Name	Water Depth (m)	Location
20	Statfjord C	290	Norway
21	Beatrice Bravo	290	United Kingdom
22	Jacky	40	United Kingdom
23	Ula	40	United Kingdom
24	Inde AC	70	United Kingdom
25	Armada	23	United Kingdom
26	Auk A	88	United Kingdom
27	Fulmar A	84	United Kingdom
28	Clipper South	81	United Kingdom
29	Clair	24	United Kingdom
30	East Brae	140	United Kingdom
31	Lomond	113	United Kingdom
32	East Brae	86	United Kingdom
33	Alwyn North A	126	United Kingdom
34	Alwyn North B	126	United Kingdom
35	Cormorant Alpha	126	United Kingdom
36	Dunbar	145	United Kingdom
37	Nelson	121	United Kingdom
38	Schooner	100	United Kingdom
39	Andrew	117	United Kingdom
40	Forties Alpha	107	United Kingdom
41	Forties Bravo	107	United Kingdom
42	Forties Charlie	107	United Kingdom
43	Forties Delta	107	United Kingdom
44	Forties Echo	107	United Kingdom
45	Eider	159	United Kingdom
46	Elgin	93	United Kingdom
47	Elgin PUQ	93	United Kingdom
48	Franklin	93	United Kingdom
49	Babbage	42	United Kingdom
50	Alba North	158	United Kingdom
51	Alba South	138	United Kingdom
52	Judy	80	United Kingdom
53	Amethyst	30	United Kingdom
54	Buzzard	100	United Kingdom
55	Brigantine BG	29	United Kingdom
56	Brigantine BR	29	United Kingdom
57	Cecilie Wellhead	60	Denmark
58	Nini East	62	Denmark
59	Nini Wellhead	58	Denmark
60	South Arne	60	Denmark
61	Galata	34	Bulgaria

understand the significance of Global Strategic Petroleum Reserves. Crude oil inventories held by the government of a particular country are called strategic reserves, which are generally used in case of an energy crisis. This is an important index of the net import cover, which means that the offshore industry is closely associated with the economic growth of any country. Hence, engineering updates with respect to offshore industries are a direct link to the economic growth of the nation.

1.3 FIXED-TYPE PLATFORMS

The most primitive type of offshore platforms was of a fixed type, where the base of the platform was fixed to the seabed. The essential reason for such a geometric configuration is that they are stiff and relatively insensitive to wave loads (Mei, 1966, 1983; Mei et al., 1974; Moan and Sigbjørnson, 1977; Moe and Verley, 1980; Moses and Stevenson, 1970). Designers are convinced that such structures are well suited for oil exploration due to their insignificant response under wave loads (API RP WSD, 2005; BS6235, 1982; Morison, 1953; Morison et al., 1950). Construction methods are also supportive as they are mostly installed in shallow water depths. Table 1.2 provides the details of fixed-type offshore platforms that are commissioned at various water depths. The table shows that fixed-type offshore structures are not preferred for greater water depths mainly due to (1) increased costs at greater water depths and (2) increased complexities during construction, installation, and commissioning. Interestingly, they tend to attract more forces due to their increased stiffness, resulting in members with a larger cross section and thickness, making the construction of platform more expensive (Bea et al., 1999). As a common point of observation, note that steel is the most commonly used construction material, whereas reinforced concrete is a remote alternative.

Such structures are generally very stiff, due to which they tend to attract more forces. However, mechanical properties will cause a lot of rigidity in the structure. Structural systems with flexible geometry would result in efficient dispersal of forces, instead of attracting them (Stansberg et al., 2002). Even though fixed-type platforms are popular and are increasingly common for oil exploration in shallow waters, they gradually become obsolete. A few important factors that forced the engineering community to revisit the choice of geometric configurations are (1) the cost of construction for deepwaters, (2) the downtime for commissioning the platform, and (3) reusability.

Among the fixed-type offshore platforms constructed in the world, a few deepwater platforms commissioned in the United States are Bullwinkle (412 m), Pompano (393 m),

TABLE 1.2
Fixed Platforms in Different Water Depths

S. No.	Water Depth (m)	Number of Platforms
1	<100	70
2	101–200	43
3	200–300	8
4	>300	8

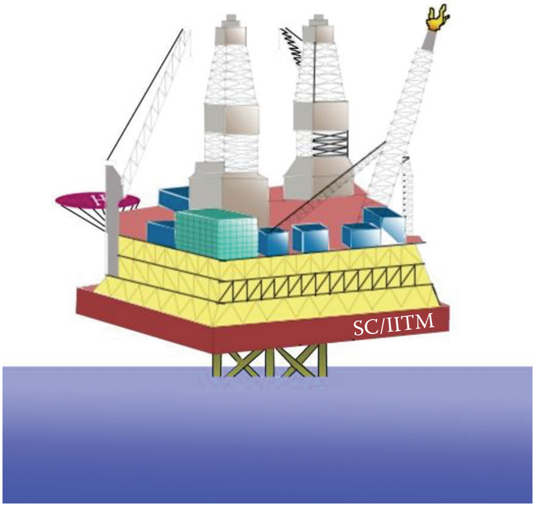


FIGURE 1.6 Bullwinkle platform.

and Harmony (365 m). Fixed-type platforms are preferred for oil exploration in shallow waters. LSP-1 is the shallowest platform, commissioned in Russia at 13 m water depth; other shallow platforms include South Venture (23 m, Canada) and QHD 32-6 (20 m, China). Figure 1.6 illustrates a schematic view of the Bullwinkle platform, located in Manatee Field, in the Gulf of Mexico. The height of the platform tower is about 529 m, and the weight is approximately 77,000 tons. Designed for a production capacity of 59,000 barrels of oil per day (BOPD), the construction time taken by Heerima contractors was about 36 months (1985–1988). One can readily estimate the commercial value of the platform in terms of its return on investment, knowing the cost of one barrel of oil (158.98 L) (Chandrasekaran, 2013a).

The Pompano platform was constructed in 393 m water depth and commissioned in 1994. Weighing about 38,000 tons, the platform is supported by 12 piles in four groups to yield 60,000 BOPD. Figure 1.7 shows the Hibernia platform located in Canada in 80 m water depth. The platform is constructed with reinforced cement concrete (RCC) and commissioned in 1997 with a production capacity of 50,000 BOPD. One of the important factors of this design is the concrete base with large ridges all along the circumference to counteract iceberg impact. This is a gravity-based structure (GBS), which rests on the seabed by its own weight. Except skirt piles that are required to improve soil lateral stability, GBS platforms do not require pile foundations. Because of their increased self-weight, the lateral resistance is relatively higher (Chakrabarti, 1980). This results in an increased payload capacity and provision of overloading (Chakrabarti, 1971, 1984, 1987, 1990). Large-diameter RCC columns support the deck, which is designed with multitier functionalities.

This is a classic example where the offshore community used concrete as an alternate construction material. It was realized at a later stage that performance on concrete in terms of corrosion resistance and durability is superior to steel, which is otherwise susceptible to corrosion in marine environments. A Troll A platform

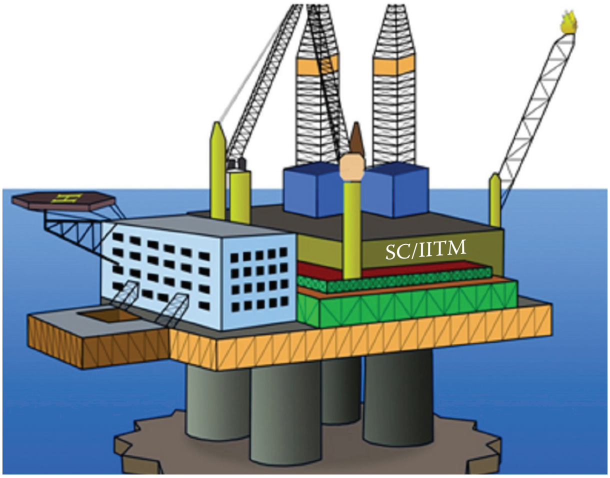
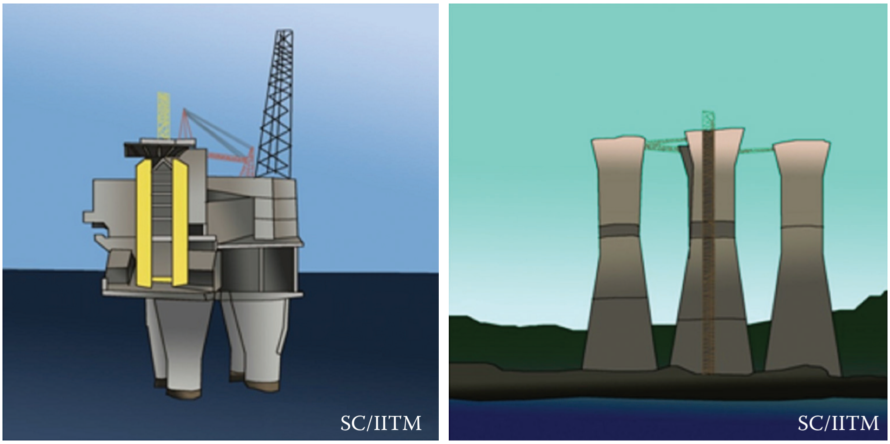


FIGURE 1.7 Hibernia platform.



(a) (b)

FIGURE 1.8 (a) Troll A platform; (b) the towers without topside.

was constructed parallel off the coast of western Norway in 1996. Figure 1.8 shows a schematic view of the Troll A platform, which set a Guinness Record for being the largest offshore gas platform in the world. Commissioned in the water depth of 303 m, the platform deck is supported by a tower 472 m high. About 245,000 m^3 of concrete and 785,000 tons of reinforcement steel were used to construct the Troll A platform (Chandrasekaran, 2015a,b). Figure 1.9 shows a list of fixed-type offshore platforms commissioned in water depths of more than 300 m (about 1000 ft) along with the year of commissioning, and water depths at which they are located. Most of the platforms are similar in the structural geometry and functional characteristics that continued to dominate until the late 1990s (Chandrasekaran, 2013c, 2015a,b).

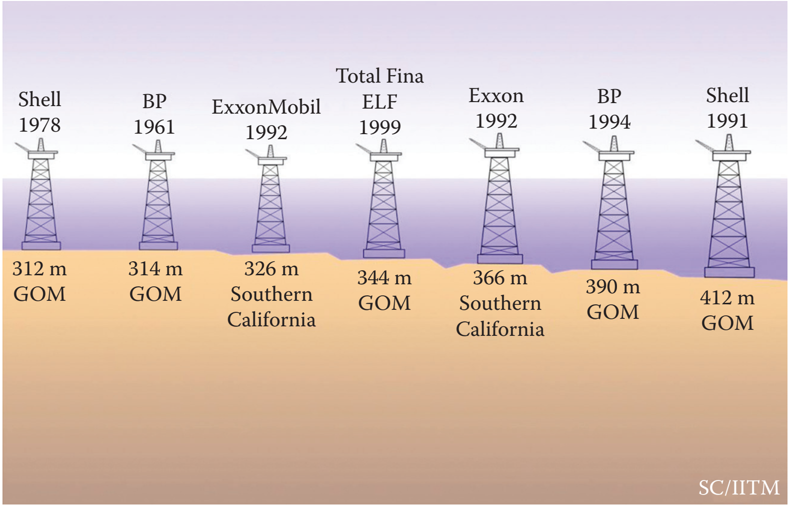


FIGURE 1.9 Fixed platforms installed at a water depth of more than 300 m.

1.4 JACKET PLATFORMS

Steel jacket-type fixed platforms consist of a tower that supports the superstructure. A tower is of truss configuration, whose transparency to waves enables it to reduce the encountered loads. Steel jacket platforms are also called template structures, as the legs of the jacket are fixed on the prelaid steel template on the seafloor. These installations are built for various applications: (1) drilling, (2) preparing water or gas for injection into the reservoir, (3) processing oil and gas, (4) cleaning the produced water for disposal into the sea, and (5) accommodation facilities. Steel jackets are prefabricated in offshore construction yards and then transported to the site using barges. Figures 1.10 and 1.11 show a schematic view of the jacket fabrication and transportation on a barge. Upon reaching the site, they are upended as shown in Figure 1.12. The figure also shows a schematic view of the jacket during installation, where special tools are deployed as shown in Figure 1.13 (Chandrasekaran, 2013a,b).

Upon successful installation of the jacket legs, the top deck is then installed. Prefabricated deck modules are transported to the site and fixed on the jacket legs; this process is called deck mating, as shown in Figure 1.14. Subsequently, the building module will be towed and installed as shown in Figure 1.15. The figure also shows a schematic view of the jacket platform after commissioning.

Figure 1.16 shows a line diagram of a typical jacket platform. It also shows an assembly of different tubular members forming a jacket, hence the name. The topside of the platform is supported on a deck, which is fixed to the jacket legs. The topside consists of an administrative block, a control room, power generator units, transformer units, test burners, support vessels, rig helicopters, rig heliports, platform heliports, cabins for working, a rig office, a game room, rest rooms, decompression chambers, and so on (CAP 437, 2010; HSE, 2010).

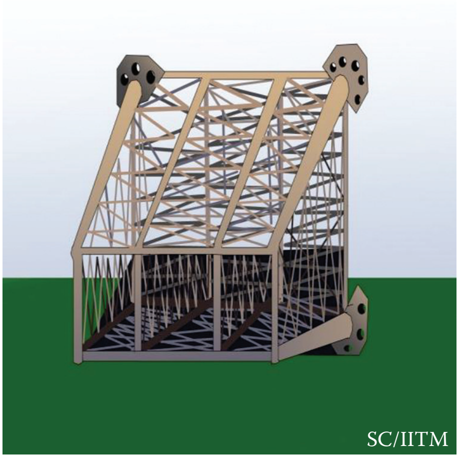
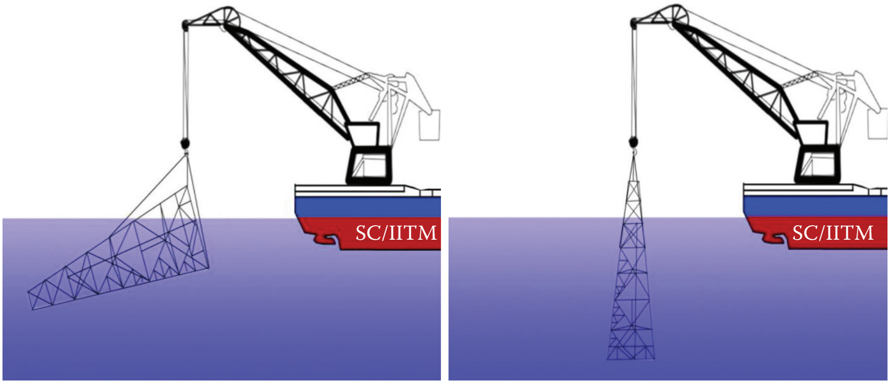


FIGURE 1.10 Jacket fabrication.

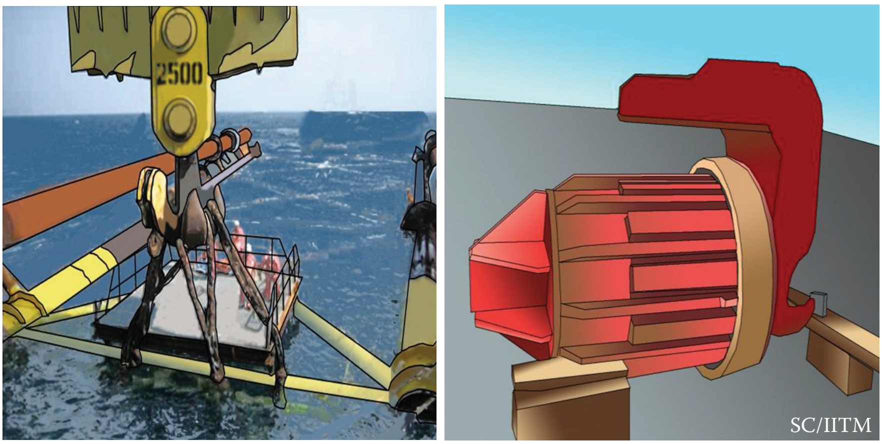
FIGURE 1.11 Jacket transport.

A steel jacket platform is one of the fixed-type platforms that rests on pile foundations. It is one of most common types of fixed structures that exists worldwide. The substructure or jacket is fabricated using a steel tubular section and is welded and subsequently pinned to the seafloor using steel piles. Piles are driven through pile guards located on the outer members of the jacket. They are thick steel pipes about 2 m in diameter, which can penetrate as deep as 100 m into the seabed. As steel is susceptible to high corrosion, one of the common maintenance practices is cathodic protection. It is suitable for water depths of 150–250 m, depending on the wave climate of the site. The jacket surrounds the piles and holds the pile extension in position from the mud line to the deck substructure. It supports and protects the



(a) (b)

FIGURE 1.12 Jacket upending: (a) first stage of upending; (b) upending complete.



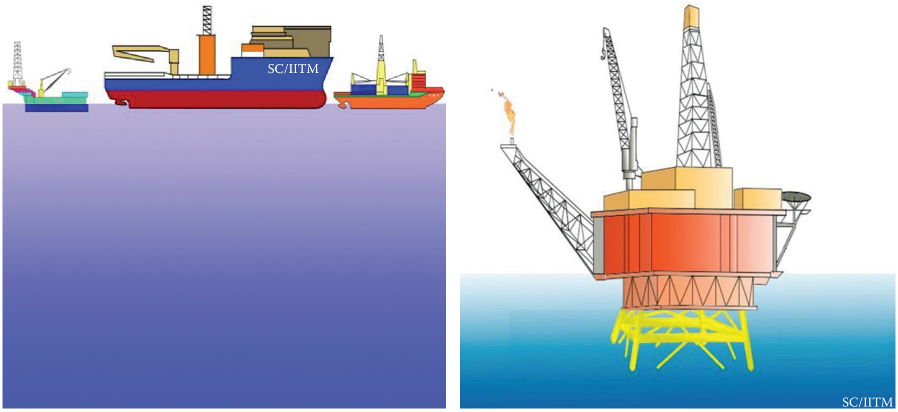
(a) (b)

FIGURE 1.13 (a) Jacket installation; (b) lifting tool.

well conductors, pumps, sumps, and risers, hence the name *jacket*. The jacket legs serve as guides for driving piles, and, therefore, it is called template structure. Soil condition that is suitable for jacket platforms is clay, as pile driving is comparatively easier. Jacket platform consists of a helideck, which is a raised level of a platform used for facilitating helicopter landing. Solar panels are also mounted just below the helideck to facilitate auxiliary power for the platform. A flare boom is a long truss that supports a vent or a flare line. The topside or deck structure has an upper part, which is generally above the reach of the highest wave height. This houses most of the mechanical equipment used for production drilling. As jacket platforms are suitable for production drilling, they are likely to become permanent installations. Topside generally equips machineries related to mechanical, electrical, piping, and

(a) (b)

FIGURE 1.14 Deck mating: (a) installation of deck; (b) construction of building module.



(a) (b)

FIGURE 1.15 Final installation: (a) building module being towed; (b) platform commissioned.

instrumentation processes. It also contains a doghouse, living quarters, workshops, and battery rooms. A jacket structure is the supporting frame of the platform and is designed for encountering lateral forces from the waves.

A crane pedestal is a large structural tube that supports an offshore crane for lifting purposes. It also functions as a diesel storage tank, because its diameter is very large. Piles are one of the major structural members that are driven through hollow leg tubes to embed the steel jackets below the seabed. Pile embodiment is governed by the capacity of soil to withstand platform loads. Skirt piles are required when the soil is very weak, and the existing number of piles formed in the geometry is not adequate. They are run closer to the main piles as a cluster of two, three, or four and

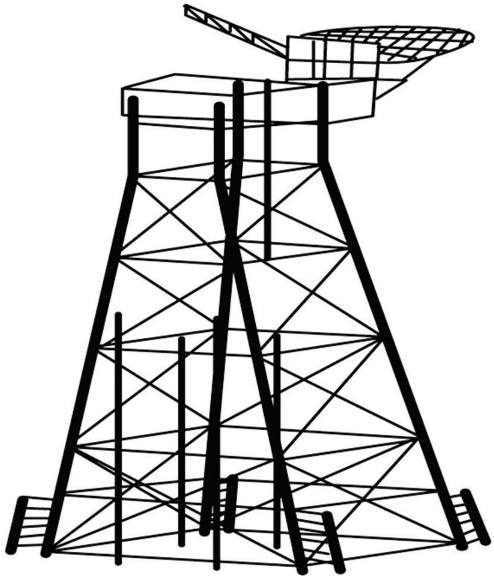


FIGURE 1.16 Jacket platform.

are actually in groups. The transition piece is an important structural member, which is in the form of a cone that links the topside with that of a jacket. A cone-shaped design is preferred as the leg size of the topsides is smaller in diameter compared to that of the jacket legs. Conductors are long, hollow, straight, or curved tubes that embed into the seabed through which drilling is performed. To support such long tubes, conductor framings are provided. Risers are long slender tubes that carry crude oil, or partially processed oil to another location for further processing. They are generally clamped to the jacket legs as the lateral support system. Boat landings, barge bumpers, and riser guards are required for berthing supply vessels. They are used to facilitate smooth berthing. Barge bumpers are equipped with shock shells, which are mounted on each side of boat landing to reduce the vessel impact on the jacket platform. A riser guard is an alternate protective structure, which is used to protect the oil-carrying risers from the impact. It reduces the accidental impacts caused on the jacket platforms. The launch truss is one of the vital components used for the installation of jacket structures. Sometimes, jacket structures are very large and cannot be lifted even with large cranes. Permanent structures such as launch trusses are provided on one side of the jacket to facilitate the loading out to the barge. If the jacket is designed for buoyancy, then the jacket is launched in sea after reaching its destined position for natural append and leveling. When the jacket is launched, it floats due to its buoyancy. The jacket legs are sequentially flooded to make it upright, which is known as appending. A mud mat is the bottommost frame of the platform which helps in the stability against lateral forces. These are useful for creating stability of the platform even before the piles are driven. It is similar to a large raft that is made out of timber. It helps the platform to sink deeper because the soil is too soft near the top layer of the seabed.

Essentially, jacket platforms are meant for production. They are steel-framed tubular structures attached to the seabed with piles driven to the seafloor. Constructed in sections and transported to the site in pieces, the design lifetime of jacket platforms varies from 10 to 25 years. The advantages of jacket platforms are as follows: (1) capability to support large deck loads; (2) ease in construction, which can be fabricated in sections and transported, making installation simple; (3) capable of supporting large field for long-term production; (4) highly stable under lateral loads due to pile foundation; and (5) having little effect from the seafloor scouring in comparison with GBS platforms. However, the cost of jacket platforms increases exponentially with the increase in depth. In addition to high initial and maintenance costs, the complete structure is not reusable. One of the main disadvantages is that they are highly susceptible to corrosion. Corrosion protection measures, which are adopted in the design stage itself (see, e.g., sacrificial anode method), make jacket platforms expensive (Chandrasekaran and Bhattacharyya, 2011; Chandrasekaran and Saha, 2011; Jin et al., 2007; Rackwitz, 1977).

1.5 GRAVITY-BASED STRUCTURES

Gravity platforms have a very specific design objective. They are meant for the production of oil from the reservoirs. Gravity-based structures consist of a large reinforced concrete bottom mounted on the seabed. They resist lateral loads using their self-weight, as the colossal weight is very high (Clauss et al., 1992; Clauss and Birk, 1996). These platforms are not attached to the seabed through piles but rest on their own weight, hence the name gravity base. They are suitable for a medium water depth of up to 350 m. Concrete gravity-based structures are constructed with the base as a reinforced concrete structure. The design of the base includes plenty of void spaces, termed as caissons, which initiate natural buoyancy to the geometry (Dawson, 1983; Hove and Foss, 1974). This enables the structure to float to the field development location during installations. Once the location is reached, these void spaces are flooded, enabling the platform to settle down on the seafloor. Once the bottom tower is mounted, topside modules are lifted and placed in position. Void spaces are used as compartments for storing the explored crude oil. Sometimes, they are also permanently filled with iron ore ballast to maintain stability during operation. A Hibernia is a classic example of GBS platform, shown in Figure 1.7.

Figure 1.17 illustrates a variety of GBS platforms. As shown, the common structural characteristic of these platforms is that they all have a caisson base on which they rest. These caisson bases are very large in diameter and height, and the void spaces are used to store the crude oil during exploration. These platforms are very stable under lateral loads (Tromans et al., 2006). Because of the increase in cost with an increased water depth, these platforms are limited to about 300 m water depth. Figure 1.18 shows other GBS platforms: the Brent Platform, the Ninian platform, the TSG or the Maureen platform. It can be readily observed that these platforms have a large set of caisson bases on which they rest. Topside details of the platforms are almost similar; they have provision for storage unlike fixed-type tower structures that are built in steel. Gravity-based platforms have many salient merits in the offshore context. They support large deck loads and have a very high possible reuse of the material. Construction and testing of these platforms

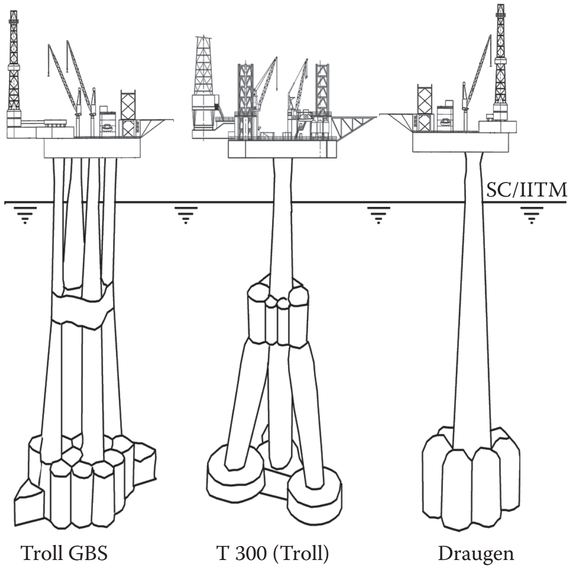


FIGURE 1.17 GBS platforms.

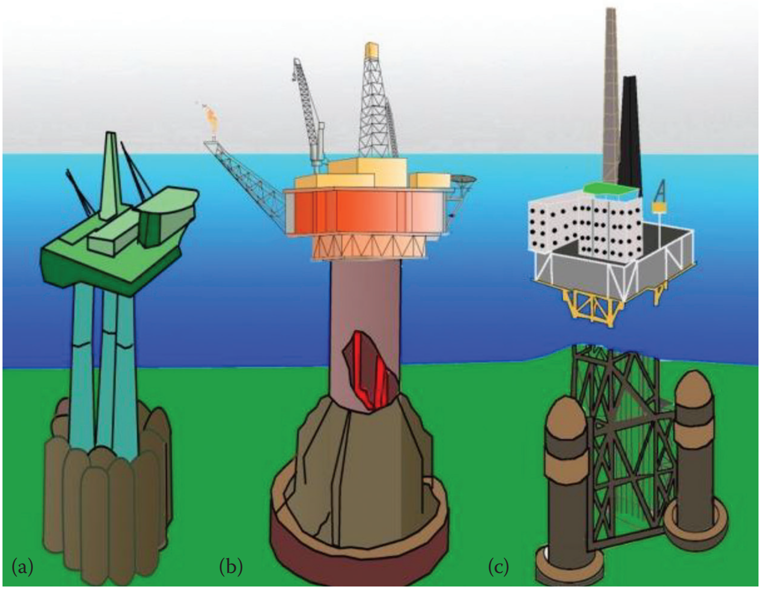


FIGURE 1.18 (a) Brent platform; (b) Ninian; (c) TSG (Maureen).

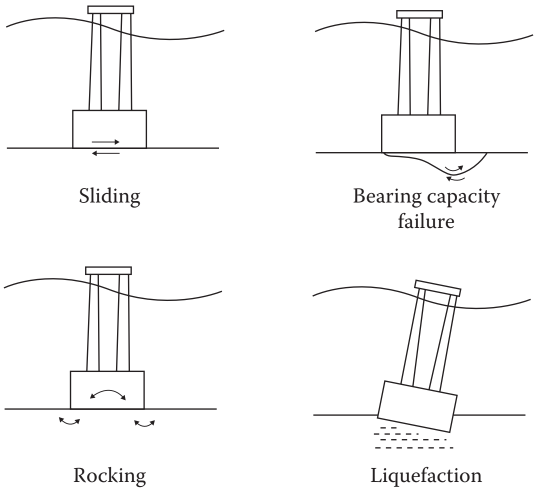


FIGURE 1.19 Geotechnical problems associated with GBS platforms.

are generally completed in casting yard where a good degree of quality control can be exercised; they are subsequently floated or towed to the site for installation. In addition, they support a large field of oil exploration and are useful in long-term production. In fact, GBS platforms are best suitable to support a large number of wells with high (oil) yielding capacity. Being apart as they have large storage capacity, they are more tolerant to overloading and seawater exposure in comparison with steel jacket platforms.

Gravity-based structures have some demerits as well. Cost increases exponentially with the increase in water depth. Because of a high colossal weight, they can cause foundation settlement. Geotechnical problems are very specific and critical. Figure 1.19 shows various geotechnical problems associated with GBS platforms. Sliding can occur if there is no proper installation of skirt piles and dowel rods. Though the colossal weight of the structure is very high, the platform can slide on the clay bottom due to the seafloor scour. Its very high colossal weight can also result in bearing the capacity failure, inducing the high stress concentration created at the foundation level on the seafloor (Young et al., 1975). Differential settlement at the foundation on the seafloor can also result in rocking. This can damage the foundation system and the caissons of the platform easily, and the extended damage could be very severe. When there is water entrapment below the larger area of the foundation of these platforms, it can result in soil liquefaction. Once liquefaction occurs, it can cause a differential settlement with the caissons of the platform, which can cause serious damage to the platform (Hoeg, 1976; Hoeg and Tong, 1977).

To improve their lateral stability, GBS platforms are provided with steel skirt piles. They act as an erosion-resistant member while improving the grouting of caisson base by providing accessibility. They improve transverse resistance of the platform against sliding. In addition to steel skirts, dowels are also provided. Dowels extend about 4 m below the level of steel skirts, which help prevent damage to steel skirts. An explicit disadvantage is that they are subjected to very high seafloor scour (Scheidegger, 1963). They may require more reinforcing steel than the total steel that is required to construct a

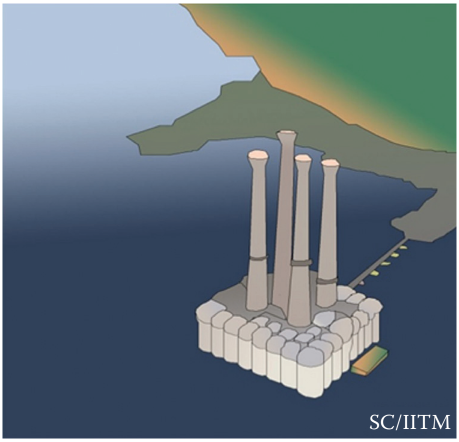


FIGURE 1.20 Gravity platform at Ardyne Point, Scotland.

steel jacket structure, which has direct implication for the cost. GBS platforms have some merits in comparison with that of a steel platform. Because of their insensitivity to lateral loads, greater safety for people onboard and topside facilities are ensured (Schuêller and Choi, 1977). Towing to the site with a deck is convenient, making the installation easier. This also minimizes the installation time and cost (Neviele, 1997). As concrete possesses better durability characteristics in the marine environment, GBS platforms have a lesser maintenance cost. Adjustable crude oil capacity and capability to support larger deck areas are shown as functional merits. Risers are protected as they are placed inside the central shaft. Large caissons create possible access to the seafloor from the cell compartments in the foundation, making the structural monitoring effective and healthy. Figure 1.20 depicts a gravity platform constructed at Ardyne Point on the west coast of Scotland. It also shows the tower and caisson base built to support the topside of 100 × 100 m. Caissons 56 m high have a storage capacity of about 1 million barrels, which support 116 m high towers as shown in the figure. Large numbers of caissons remain void when they are floated. Subsequently, they are ballasted to achieve a specific draft during installation. On completion of installation, the ballast material on alternate caissons is emptied to enable the storage of crude oil that is explored from the west coast. The construction was undertaken in a dry basin before the platforms were floated out for completion. Tugboats towed the completed structures to the installation site.

1.6 JACK-UP RIGS

Drilling rigs are used for exploration under the sea or soil. The more commonly used are land-based rigs, which are deployed to explore water. A schematic view of a jack-up rig is presented in Figure 1.21. It consists of legs to support the hull; legs are essentially steel lattice towers. Topside details are similar to those of the GBS or jacket platform, whereas the variation is significant only in its geometric form. They are similar to a cargo vessel in its design, which is either self-propelled or towed. Although they remain afloat, the legs are lifted up and the hull will be facing the sea.

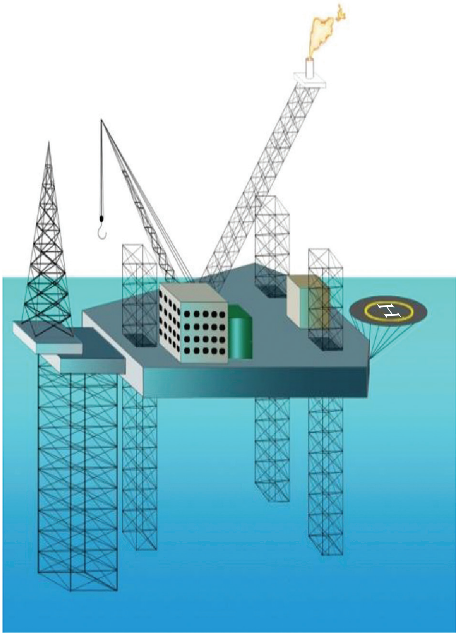


FIGURE 1.21 Jack-up platforms (rigs).

Depending on the design drought, the rig will remain afloat so that it can be conveniently towed from one location to another for exploration/production drilling. The name jack up is due to the fact that legs will be pulled up while they are transported from one site to another. On reaching the installation site, these legs are driven into the seabed for better stability. Its mobility is the significant change in its geometric design in comparison with those of the fixed-based structures.

The vital components of jack-ups are derrick, draw works, drill floor, drill pipe, drill string, cantilever boom, legs, living quarters, helipad, hull, and the spud can. The primarily function of the jack-up rigs is exploratory drilling. A barge with movable legs and a rig is towed to the site, and the legs are jacked down into the seabed; the platform hull is raised for operational engagement. It is suitable for a shallow water depth (90–140 m). Some salient advantages include mobility, stable when elevated, low cost, high efficiency, and reduced downtime to start the exploratory drilling process. A few demerits are that operational convenience strongly depends on the weather window. It is restricted only to shallow water depth. Other issues could arise due to initiation of seabed scoring on the site of installation. In case of the site under soil liquefaction, it can even cause collapse of the platform. These platforms do not have storage capacity. Jack-up rigs are capable of operating in a harsh environment up to a wave height of about 24 m and a wind speed up to 100 knots (1 knot is about 1.85 km). A jack-up rig is floated to the installation site similar to that of a moving barge. Legs are all kept on the lifted position, which is called towing position. Subsequently, on reaching the site, the legs are fixed to the seabed, whereas the deck is lifted up to a comfortable height to have the desired freeboard from the high tide level. The rig will be preloaded to

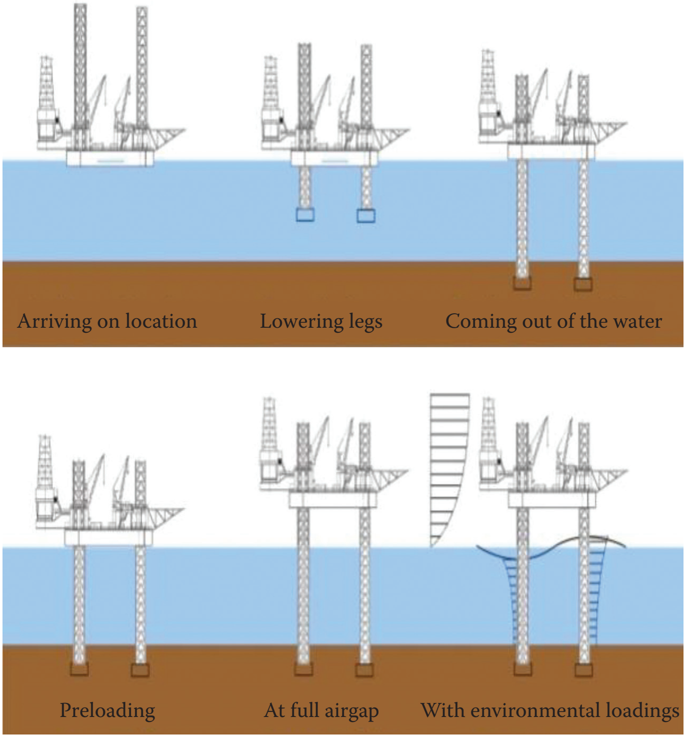


FIGURE 1.22 Jack-up platforms.

test whether the foundation of the legs has reached the desired level of lateral stability. Once the preloaded test is completed, the deck is further lifted to have a clear air gap during operation. An air gap is provided to ensure that the deck is not interfered with the tides during operation. This is the final stage of commissioning the jack-up rig; on completion, the rig is ready for exploratory drilling. Figure 1.22 shows a schematic view of a jack-up rig that has been towed to the site, installed, commissioned, and then made ready for exploratory drilling operations.

The foundation of a jack-up rig becomes important to ensure its stability against lateral loads caused by waves and wind. Lattice tower-type legs of jack-up rigs are supported on a spud can, which is a shallow, conical underside footing of the legs. Figure 1.23 shows a schematic view of a spud can used in the foundation of each of the legs of jack-up rigs. Spud cans are suitable for stiff clay and sand but not for rocks. The depth of penetration of the conical portion is about 2 m. Once the spud can is fixed firmly, it accumulates soil particles in its void space as it is placed inverted. It then requires a high pullout force to extract it from the seabed.

Initially, in the early 1960s, jack-up rigs were deployed up to 30 m water depth, but subsequently they were attempted even at 170 m. As they are assumed to be suitable for deepwaters at a later stage, accidents resulting in capsizing of jack-up rigs occur while they are towed or transported. When the legs are lifted up, their height and thin lattice structure cause roll and pitch motions to the vessel on towing.

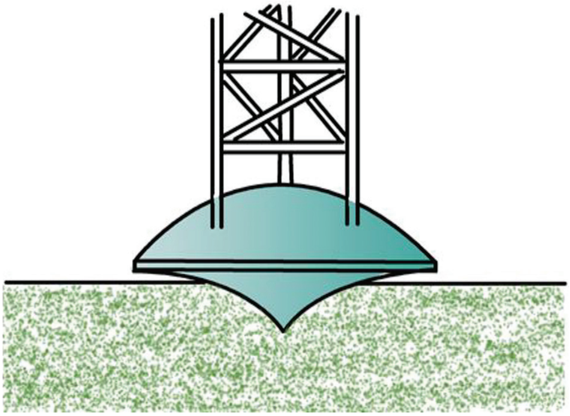


FIGURE 1.23 Spud can.

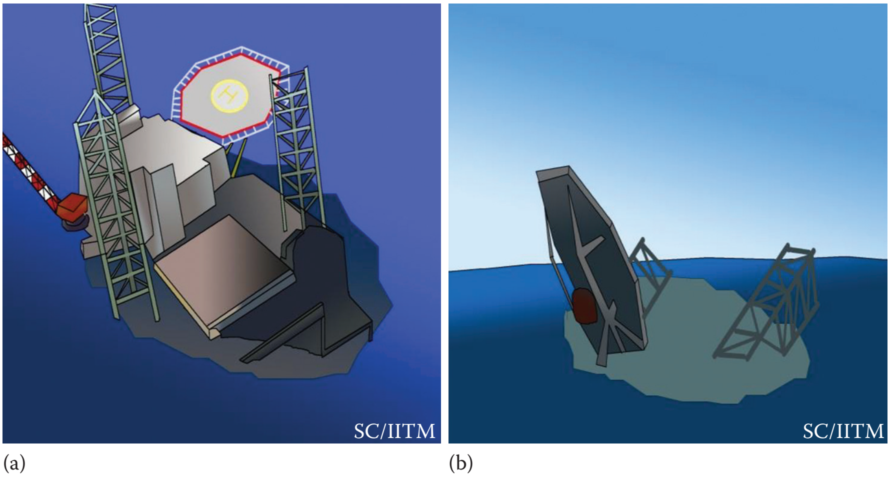


FIGURE 1.24 Capsizing of jack-up rigs: (a) topside collapse; (b) complete capsizing.

Figure 1.24 shows a schematic view of the jack-up rig, when capsized. Under such conditions, one may lose the complete topside facility.

1.7 COMPLIANT-TYPE PLATFORMS

Compliancy refers to movement. Hence, it refers to those kinds of structures that have the capability to move along with the external forces acting on the structure. Compliancy induces flexibility to the structure (Chakrabarti, 1990, 1994; Kim and Zou, 1995; Kjeldsen and Myrhaug, 1979). As the structure becomes flexible, it responds to external forces by notwithstanding the forces alone but by undergoing large displacement as well. Compliant offshore structures are drilling platforms that are deployed in deep sea for oil exploration, whereas earlier types of offshore platforms are meant for shallow water depth only (Chandrasekaran and Jain, 2002a,b). Compliant platforms are designed for drilling, especially in deep sea. They are connected to the seafloor by allowing them to move freely under the action of current waves and wind. It is important to note that the methods by which compliant structures are attached to the seafloor are significantly different from that of the fixed type. In the latter type, they are connected to the seabed using piles that make to fix to the seabed firmly, whereas the former type is connected using cables or tethers. These structures, therefore, strongly rely on the restoring buoyancy force to maintain the stability under the encountered lateral loads (Chandrasekaran and Gaurav, 2008; Chandrasekaran and Sharma, 2010; Papoulis and Pillai, 1991; Pavia et al., 1977). These structures avoid resonance by operating at a frequency well below that of the ocean wave's frequency. This is considered to be one of the greatest design advantages. As the natural frequencies are well away from the band of operational frequency of ocean waves, these structures do not resonate under the external action of environmental loads (Chandrasekaran and Pannerselvam, 2009; Chandrasekaran and Parameswara Pandian, 2011). Compliant offshore structures provide flexibility, which is preferred to exploit energy at deepwaters (Chandrasekaran and Bhattacharyya, 2011; Madhuri, 2013; Niedzwecki and Huston, 1992; Niedzwecki et al., 2000; Nordgren, 1987; Roitman et al., 1992; Sellers and Niedzwecki, 1992; Shaver et al., 2001; Tigli, 2012; Yashima, 1976).

The structural form of compliant platforms is also significantly different from that of the fixed type. Although the latter resists load by their self-weight (in case of GBS) or by steel jacket, the former type resists the lateral loads by undergoing large displacements in the direction of wave loads; this is the degree of compliancy imposed through the design. As no jacket-like structural form is extended from the topside till the seabed, compliant platforms are economical for an increased water depth (Chandrasekaran and Saha, 2011). Figure 1.25 presents a typical compliant platform.

1.8 GUYED TOWERS

Guyed towers are compliant-type offshore platforms that are deployed for both drilling and production activities. They are viable to operate at a water depth from 180 to 600 m. They consist of a top deck, which houses the necessary electric and mechanical equipment necessary for drilling. The top deck is supported by the steel truss-type tower, which is similar to a jacket in form (Chandrasekaran et al., 2006a,b, 2007b,c). Unlike steel jackets, these towers are supported on spud cans, which are similar to those of jack-up rigs that enable rotational motion at the base. To ensure quick and reliable recentering, guy wires are attached to the tower at the top middle third of the height of the tower. The point at which the guy wire is attached to the tower is known as the fair-lead point, and the point at which the guy wires touch the seabed is known as the touchdown point. Guy wires are similar to catenary cables, which on lateral movement of the tower offer a horizontal pull in the opposite direction to that of the wave action. This ensures recentering of the tower. Usually, multiple guy lines are attached to the tower circumferentially. One end of the guy lines are attached to the clump weights, which in turn are connected to the drag anchors. Drag anchors hold these guy lines to the seabed. The other end of the guy line passes through the fair-lead and is connected to the tower. The topside of the tower is equipped with heavy-duty hydraulic jacks to impose tension

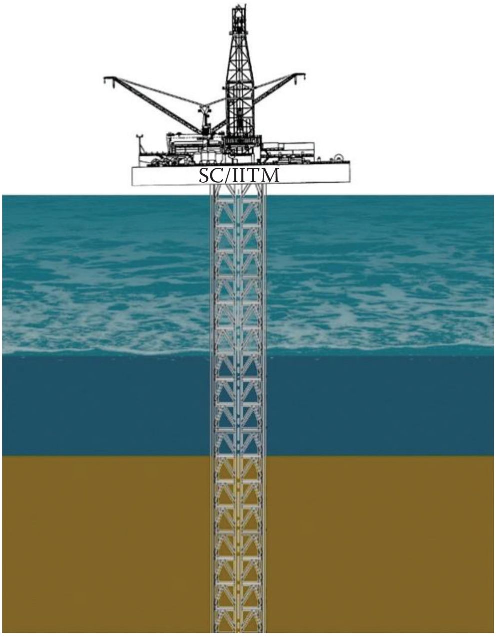


FIGURE 1.25 Compliant towers.

in the guy lines. Guy lines restrain the surge/sway motion of the tower. The spud can offer a support connection, which is position-fixed and rotation-free, treated as a pinned beam in the analysis (Chandrasekaran et al., 2010a,b,c). The foundation of the guyed tower is simple as less horizontal reaction is counteracted by the spud can. The spud can offer only stability against lateral sliding and does not offer resistance against lateral forces; as lateral loads are resisted by the guy lines, the foundation becomes simpler.

1.9 ARTICULATED TOWERS

Articulation is a term related to rotation. The tower is made free to rotate by providing a hinged connection at the bottom of the tower; thus, compliancy is induced in the structural form (Chandrasekaran et al., 2010b,c). Articulated towers are offshore platforms that are connected to the seabed using universal joints, which imposes free rotation at the connection point. Universal joints offer position restraint, but no restraint against rotation. One of the major disadvantages of this structural form is that it induces a single-point failure, which is pivoted at the universal joint. Unlike other types of offshore platforms discussed earlier, articulated towers fail at the universal joint (Helvacioglu and Incecik, 2004; Herbich, 1991). Failure is mainly due to large fatigue imposed on the joint by extensive rotation. Figure 1.26 shows a schematic view of the articulated tower. As shown in the figure, the tower consists of a buoyancy chamber located at the top one-third of the tower and a ballast chamber at the bottom. This configuration ensures a shift in the center of gravity of the platform to the bottom, conforming to increased stability. The tower consists of a shaft, which rests on the universal joint at its

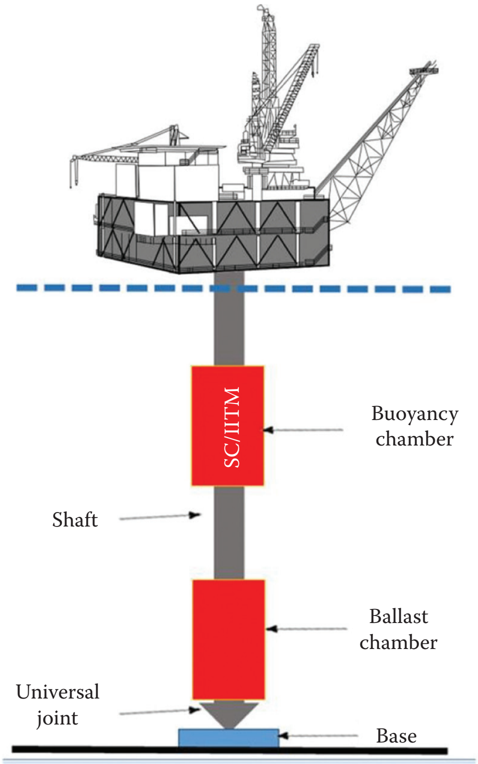


FIGURE 1.26 Articulated tower.

bottom. The central column contains a upper shaft and a lower shaft. The central shaft should be either a single tubular column or a truss-type structural system. Compliancy of the articulated tower helps to avoid the concentration of high moments at the bottom. It shifts the high stresses caused by external forces to the tower or the central shaft. The buoyancy chamber ensures the recentering capability of the tower while the tower undergoes a pendulum action, pivoting at the universal joint. Unlike the guyed towers, the articulated tower restores its stable position by variable submergence caused by the buoyancy chamber (Choi and Lou, 1991; Marthinsen et al., 1992).

As rotation is permissible at the base, it results in a simple foundation system. Any disturbance due to lateral load is restored by the buoyancy force, which is achieved by the dynamic change in the water plane area of the buoyancy chamber. Figure 1.27 shows a single anchor leg mooring system, which is one of the common applications of articulated towers. These platforms are useful for anchoring large vessels in open sea, as shown in the figure. Articulated towers are deployed for small fields and commissioned at a water depth up to 200 m. Explored crude oil is moved up the deck of the tower, which is subsequently transferred to a tethered tanker for processing and storage. Articulated towers have a limited storage capacity except the buoyancy chambers. Shuttle tankers are used to transfer the explored oil from the offshore to the onshore for further processing. Articulated tower is a low-cost structural form, which has a large

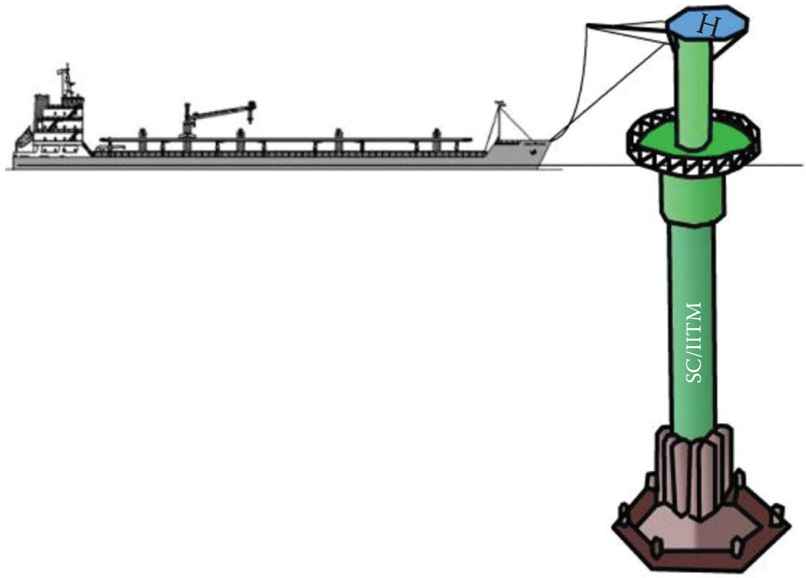


FIGURE 1.27 Single anchor leg mooring system.

restoring (moment) capacity due to high center of buoyancy. Risers are protected by the tower from the action of external loads. It attracts less force due to its compliancy.

The natural period of these towers is larger than that of the encountering waves. It is in the range of 40–75 s, whereas a typical wave period in open sea is around 6–15 s. A shift in the natural period of the tower from that of the encountering waves results in a lower dynamic amplification factor in comparison with that of fixed offshore structures. Because of its lesser weight, the tower can be easily towed to the installation site after prefabrication. As the structure is supported by the universal joint at the bottom, decommissioning of the platform is simple as the foundation system does not have either spud cans or piles. Articulated towers cannot operate in bad weather as a large surge will be imposed on the tower, causing a significant rotation at the bottom. This in turn will invoke a quick restoration due to variable submergence of buoyancy chambers causing discomfort to people onboard. In addition, they are limited to small fields only. The most undesirable feature of an articular tower is the fatigue of the universal joint. Large numbers of repeated cycles of rotation result in the fatigue of the universal joint as restoration of the tower is due to the rotation of the universal joint at the bottom. This induces a single point of failure.

1.10 TENSION LEG PLATFORMS

A tension leg platform (TLP) is designed with excess buoyancy, in comparison with its weight (Zeng et al., 2007a,b; Rana and Soong, 1998; Sarpkaya and Isaacson, 1981). For a structural form whose weight is much lower than the buoyancy force, the platform will have a tendency to be pushed up when it is installed (Faltinsen et al., 1995; Hogben and Standing, 1974; Logan et al., 1996; Martin and Dalrymple, 1988; Patel and Witz, 1991).

Exceedance of the buoyancy force will be counteracted by imposing pretension in tethers, which are used to anchor the platform to the seabed (Demirbilek, 1990; Donley and Spanos, 1991; Mercier, 1982; Yoneya and Yoshida, 1982; Yoshida et al., 1984). Borrowing the facts of cost implication that arise from tower-like platforms with an increase in water depth, the structural form of TLP is free from the tower structure (Chen et al., 2006; Karimirad et al., 2011; Low, 2009; Marshall, 1969; Marshall and Bea, 1976; McCamy and Fuchs, 1954; Meyerhof, 1976; Yan et al., 2009; Younis et al., 2001). It consists of large diameter pontoons and column members, which are helpful in excessive buoyancy as the displaced volume will exceed its weight (since the pontoons and columns are hollow tubes). Because the legs of the platform will be imposed with high pretension, the name tension leg is associated with the platform. Commissioning of the platform is simpler in comparison with the earlier structural forms of offshore structures. Because the buoyancy exceeds the weight of the platform, it remains free floating. This enables easy towing of the prefabricated platform to the installation site (Gadagi and Benaroya, 2006; Masciola and Nahon, 2008; Thiagarajan and Troesch, 1998; Vannucci, 1996; Venkataramana et al., 1993). On reaching the installation site, the topside will be loaded with the excess weight, making the tethers slackened. Once the tethers are properly anchored to the seabed with the pre-laid anchorage system, the excess topside weight is removed. An increase in buoyancy on the removal of excess weight on the topside will now be transferred to the tethers (Chen et al., 2006; Ertas and Lee, 1989; Gie and de Boom, 1981; Kim et al., 2007; Moharrami and Tootkaboni, 2014; Tabeshpour, 2013; Tabeshpour et al., 2006). They will pull down the legs of the platform to hold down in position. Taut mooring systems, otherwise known as tension legs, tethers, or simply tendons, are tubular pipes. Depending on the magnitude of initial pretension, even simple wires or set of cables can be also used. Figure 1.28 shows a schematic view of the TLP, indicating the vital components.

A TLP is a unique type of offshore platform, which is hybrid in nature. Hybrid structures have two distinct sets of natural periods of vibration, which are far from each other. In the innovative development of structural forms for offshore structures, TLPs are relatively new and novel due to their form-dominated design concept (Kareem, 1985; Kareem and Datton, 1982; Kareem and Sun, 1987). The platform alleviates the encountered environmental loads mainly from its compliancy characteristics and not from its strength. TLPs are unique because of this special characteristic. The two groups of natural periods are stiff and flexible. Out of six degrees of freedom (surge, sway, heave, roll, pitch, and yaw), displacement degrees of freedom on the horizontal plane (surge and sway) and rotational freedom (yaw) are highly flexible. Rotation degrees of freedom (roll and pitch) and vertical displacement (heave) are stiff, restricting the displacements. Typical periods in soft degrees of freedom range from 80 to 120 s, and those in stiff degrees of freedom range from 2 to 5 s. Figure 1.29 illustrates the TLP mechanics; displacement along the wave direction causes an offset, which subsequently induces vertical displacement in the heave direction, called *setdown* (Munkejord, 1996; Murray and Mercier, 1996; Muren et al., 1996; Natvig, 1996; O'Kane et al., 2002).

With reference to Figure 1.29, TLP mechanics can be explained. A TLP in its static position will remain in equilibrium. Any excess buoyancy will be accounted by initial pretension in the tethers. Under the equilibrium condition, the platform remains vertical and the tethers will always be in tension. Under the action of lateral forces caused

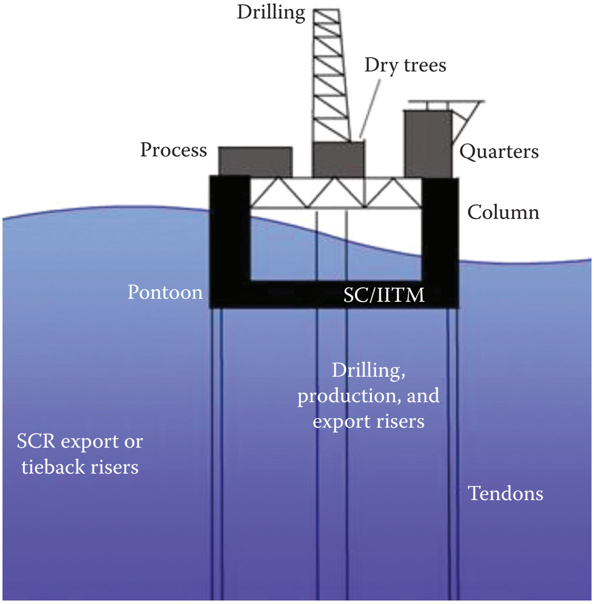


FIGURE 1.28 Tension leg platform.

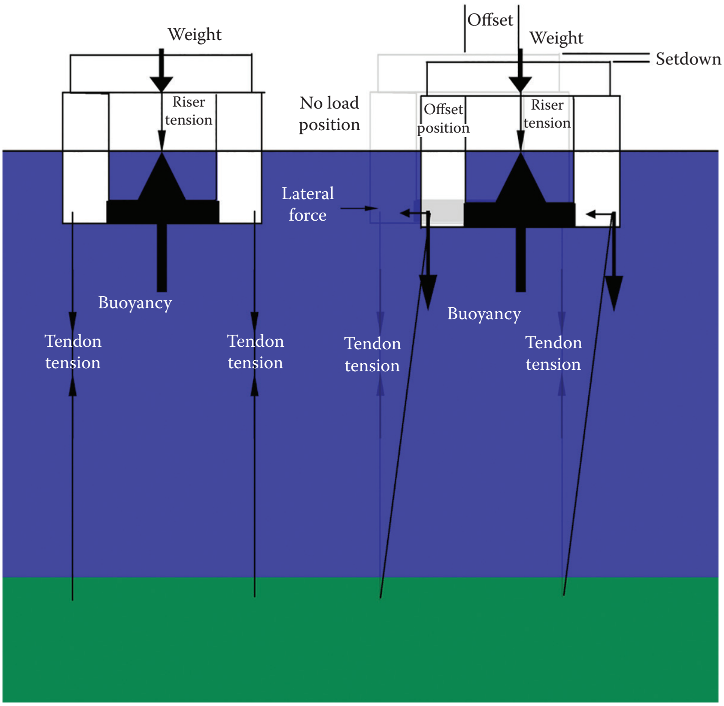


FIGURE 1.29 TLP mechanics.

by waves or wind (on the superstructure), the platform is displaced along the *X*- or *Y*-axis, as the case may be. This will induce the vertical displacement in the heave direction, indicating that there is a strong coupling between surge/sway and heave degrees of freedom (Kawanishi et al., 1993; Kim et al., 2007). Setdown will cause a change in the water plane area, which in turn affects buoyancy forces. Additional tension will now be imposed on the tethers. On the displaced position, when the platform moves to the right as shown in the figure, the vertical component adds to the weight to counteract buoyancy. The horizontal component of the large pretension will counteract the lateral force acting on the platform (Reddy and Arockiasamy, 1991). Thus, the horizontal component of the tethers induces a restoring force, whereas the vertical component improves stability. Hence, the tether tension will be under continuous variation and is therefore called dynamic tether tension variation. Unlike the articulated towers where the buoyancy chamber helps in recentering, all members of a TLP do not contribute directly to restore the lateral force caused by wind or waves. It is only the component of a very large pretension that counteracts lateral force through which the platform is brought back to the normal position (Spanos and Agarwal, 1984).

TLPs possess a lot of merits as offshore platforms (Chandrasekaran et al., 2007f; Jain, 1997; Ker and Lee, 2002; Kurian et al., 2008; Leonard and Young, 1985). They have high mobility. Once the pretension of the tethers is released, which is done by de-ballasting, they get slackened (Booton et al., 1987; Demirbilek, 1990; Kareem and Zhao, 1994; Kobayashi et al., 1987; Koo et al., 2004). At this state, the tethers can be easily removed from the foundation system, which will now make the complete platform self-buoyant and enable free floating. This also enhances its reusability. However, it is stable because of the minimum vertical motion, and hence TLPs are a highly stable structural form (Chandrasekaran and Jain, 2004, 2007c; Jefferys and Patel, 1982; Kim et al., 2007; Patel and Lynch, 1983; Patel and Park, 1995; Rijken and Niedzwecki, 1991). As no tower-like structure or shaft is extended through the water depth, it has a very marginal increase in cost with the increase in water depth. Except the length of the tethers, member dimensions and platform size remain unaltered even for deeper waters because the platform has to be designed with very excessive buoyancy (Chandrasekaran and Gaurav, 2008). There are problems associated with TLPs, which are their demerits. TLPs have phenomenally a high initial cost (capital expenditure [CAPEX]); the majority of the cost goes to the subsea installation. Installation and commissioning of TLPs at higher water depths is expensive as they require special construction expertise (Chandrasekaran et al., 2006a,b, 2008, 2011; Donley and Spanos, 1991). From a structural engineering point of view, fatigue induced on tension legs due to continuous variations in the tether tension will result in tether failure (Amr et al., 2013; Haritos, 1985; Kareem, 1985; Mekha et al., 1996; Taflanidis et al., 2008, 2009). Unlike earlier types of structural forms, TLPs do not collapse but remain afloat due to the excess buoyancy by design (Adrezin and Beneroya, 1999; Ahmad, 1996; Chandrasekaran and Koshti, 2013; Jefferys and Rainey, 1994). TLPs are complicated in terms of maintenance for subsea systems, which also makes them expensive. They have practically no storage except that available from column members or pontoons (Vickery, 1990, 1995). Hence, they need to be associated with storage vessels all through the operation (Arnot et al., 1997; Bar Avi, 1999; Bar Avi and Benaroya, 1996; Chandrasekaran et al., 2004, 2006a,b, 2007a,b; Perryman et al., 1995).

Table 1.3 provides a summary of TLPs constructed in different parts of the world. As shown in the table, the majority of investment is concentrated in the Gulf of Mexico. Table 1.4 presents a summary of TLPs constructed at different water depths, TLPs are mostly preferred for deepwater exploration, unlike the fixed-type structures (Paik and Roesset, 1996). Tables 1.5 and 1.6 provide a list of the deepest and shallowest TLPs constructed worldwide, respectively. A schematic view of a Neptune TLP, constructed in 2007 at a water depth of 1295 m is shown in Figure 1.30. Designed for

TABLE 1.3
TLPs Constructed Worldwide

S. No.	Platform Name	Water Depth (m)	Location
		United States	
1	Shenzi	1333	United States
2	Auger	872	United States
3	Matterhorn	869	United States
4	Mars	896	United States
5	Marlin	986	United States
6	Brutus	1036	United States
7	Magnolia	1433	United States
8	Marco Polo	1311	United States
9	Ram Powell	980	United States
10	Prince	454	United States
11	Neptune	1295	United States
12	Ursa	1222	United States
13	Morpeth	518	United States
14	Alllegheny	1005	United States
15	Jolliet	542	United States
		Europe	
1	Snorre A	350	Norway
2	Heidrun	351	Norway
		Africa	
1	Okume/Ebano	500	Equatorial Guinea
2	Oveng	280	Equatorial Guinea

TABLE 1.4
TLPs at Different Water Depths

S. No.	Water Depth (m)	Number of Platforms
1	250–500	5
2	501–1000	7
3	<1500	7

TABLE 1.5
Deepest Platforms

S. No.	Platform Name	Water Depth (m)	Location
1	Magnolia	1433	United States
2	Shenzi	1333	United States
3	Marco Polo	1311	United States

TABLE 1.6
Shallowest Platforms

S. No.	Platform Name	Water Depth (m)	Location
1	Oveng TLP	280	Equatorial Guinea
2	Snorre A	350	Norway
3	Heidrun	351	Norway

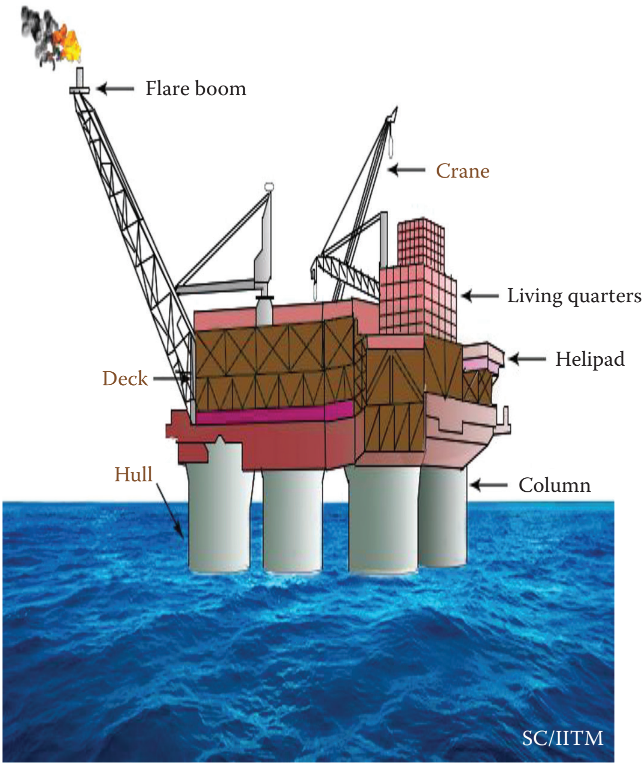


FIGURE 1.30 Neptune TLP.

a production capacity of 50,000 BOPD, six tendons are used in groups to support the platform. Tethers are anchored by six piles with a diameter of 2.4 m to the seafloor.

1.11 SPAR PLATFORMS

A spar consists of a single larger diameter, vertical cylinder, which supports the deck. The cylinder is weighted at the bottom by a chamber, which is filled with a denser material, to lower the center of gravity, thereby improving stability (Agarwal and Jain, 2002; Finn et al., 2003; Montasir and Kurian, 2011; Montasir et al., 2008; Newman, 1963; Ran et al., 1994; Sun et al., 1995; Zhang et al., 2007). Spars are anchored to the seabed by a spread mooring system. It can be with either a chain-wire-chain or a chain-polyester-chain composition. The structural form of spar platforms are of three types: classic spar, truss spar, and cell spar. Figure 1.31 shows a typical spar platform.

A classic spar has a cylindrical hull with a heavy ballast at the bottom of the cylinder. A truss spar has a shorter cylinder called hard tank. The truss structure is used to connect the bottom of a hard tank, which is further connected to a soft tank, housing ballast material. This is the most common type of spar used in offshore exploration. A cell spar has a large central cylinder surrounded by smaller cylinders of alternating lengths. A soft tank is attached to the bottom of the longer cylinder to house the ballasting material. Table 1.7 shows spar platforms constructed worldwide. Table 1.8 provides a summary of the types of spar platforms, and Table 1.9 provides a

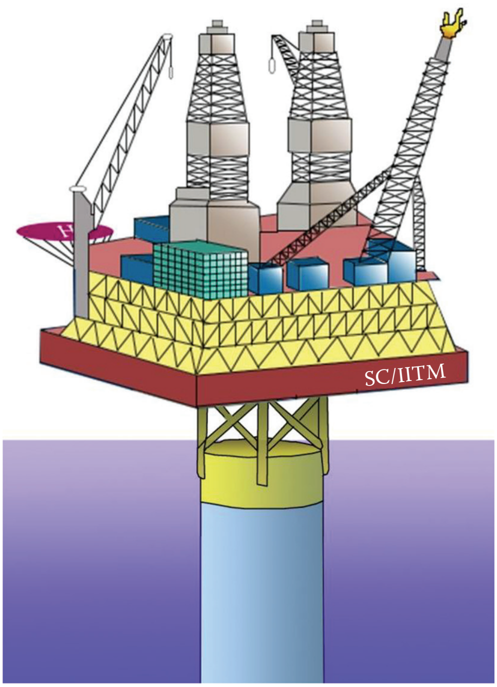


FIGURE 1.31 Spar platform.

TABLE 1.7
Spar Platforms Constructed Worldwide

S. No.	Platform Name	Water Depth (m)	Location
1	Constitution Spar	1554	United States
2	Mad Dog	1311	United States
3	Gunnison	945	United States
4	Perdido	2377	United States
5	Front Runner	1066	United States
6	Tahiti	1339	United States
7	Devils Tower	1710	United States
8	Holstein	1324	United States
9	Boomvang	1052	United States
10	Nansen	1120	United States
11	Neptune	588	United States
12	Horn Mountain	1653	United States
13	Red Hawk	1615	United States
14	Genesis	790	United States
15	Medusa	762	United States
16	Hoover Diana	1471	United States
		Asia	
1	Kikeh		Malaysia

TABLE 1.8
Summary of the Types of Spar Platforms

S. No.	Spar Type	Number of Platforms
1	Classic spar	3
2	Truss spar	13
3	Cell spar	1

TABLE 1.9
Summary of Spar Platforms

S. No.	Water Depth (m)	Number of Platforms
1	750–1000	4
2	1000–1500	8
3	1500–2000	4
4	>2000	1

TABLE 1.10
Deepest Spar Platforms

S. No.	Platform Name	Water Depth (m)	Location
1	Perdido	2377	United States
2	Devils Tower	1710	United States
3	Horn Mountain	1653	United States

TABLE 1.11
Shallowest Spar Platforms

S. No.	Platform Name	Water Depth (m)	Location
1	Neptune	588	United States
2	Medusa	762	United States
3	Genesis	790	United States

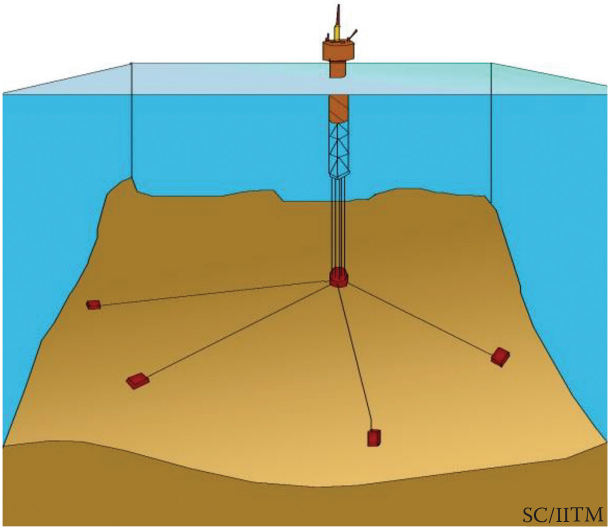


FIGURE 1.32 Perdido spar.

list of spar platforms constructed at various water depths. As shown in the table, spar platforms are a highly preferred structural form for deepwater oil and gas exploration. Tables 1.10 and 1.11 present lists of spar platforms constructed at the deepest and shallowest water depths, respectively. Figure 1.32 shows a schematic view of the Perdido spar platform, installed in 2008 at 2377 m water depth. Polyester rope mooring lines are used in the installation of the platform.

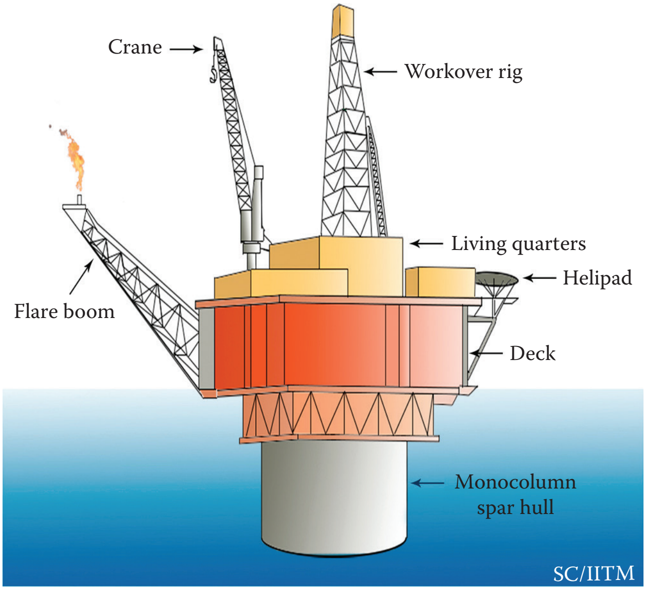


FIGURE 1.33 Horn Mountain.

A schematic view of Horn Mountain, a monocolumn spar platform commissioned in a water depth of 1652 m is illustrated in Figure 1.33. Designed for a production capacity of 65,000 BOPD, this platform was commissioned in 2002 and was successful in its exploration.

1.12 SEMISUBMERSIBLES AND DRILL SHIPS

Semisubmersibles are the floating structures used for exploration and production. They are towed to the site, ballasted, and moored (anchored). They have large vertical columns, which are connected to large pontoons and columns that support the deck structure and equipment. Semisubmersibles are compliant-type drilling structures, which are among the oldest offshore exploratory rigs used for oil exploration in deep seas (Copson, 1985; Witz et al., 1986). They typically operate in wetlands and swamps, standing in the water depths up to 30 m. Submersibles include posted barges, bottle types, arctic types, and inland barges. A semisubmersible rig floats on the water surface when moved from one drilling site to another. When it reaches the destination, certain compartments are flooded to submerge the lower part of the rig to the seafloor. The lower part of the rig rests on the seafloor to enable drilling operation. With the base of the rig in contact with the seabed, it has good resistance to lateral forces. Wave loads have a little effect on the structural motion of semisubmersibles. They typically have two or more air-filled steel floats, called pontoons. There can be either one or two large-sized, air-filled floats on which the rig rests. They are held in position by massive anchors. Because the pontoons are usually submerged a few feet below the water surface, they are called semisubmersibles (Isaacson, 1982; Isaacson et al., 1998, 2000; Liagre and Niedzwecki, 2003). They have good stability of operation during drilling as the topside weight is balanced by the equivalent water plane area of the submerged part. They have a very good stability compared to drill ships (Wilson, 2003). This is due to the fact that drill ships float while drilling is carried out, whereas semisubmersibles rest on the seafloor during their operation. Semisubmersibles can be easily towed from one site to another using one or two towboats. Some semisubmersibles are also equipped with propellers. They have inbuilt power units, which can be used to propel them from one site to another.

The operational depth of the semisubmersibles varies from 90 to 1000 m. High mobility with a high transit speed of about 10 knots makes these platforms highly versatile. As their structural form is similar to that of ships or any other large floating vessels, they remain stable and show minimal response under waves. Their seakeeping characteristics resemble ships that are designed to withstand forces arising during critical sea states; semisubmersibles have large deck areas for production, processing, and storage. However, high initial cost and operational expenditure make them an elusive choice only when no other platform is found suitable. Limiting dry dock facilities to repair and difficulties in handling the mooring systems make their choice as drilling and production platform more exclusive. Figure 1.34 shows a schematic view of Blind Faith, which is one of the deepest semisubmersibles operating in U.S. waters. A total of 48 semisubmersibles have been constructed so far. More than 50% are located in Brazil. Sixteen platforms are commissioned in Europe and seven

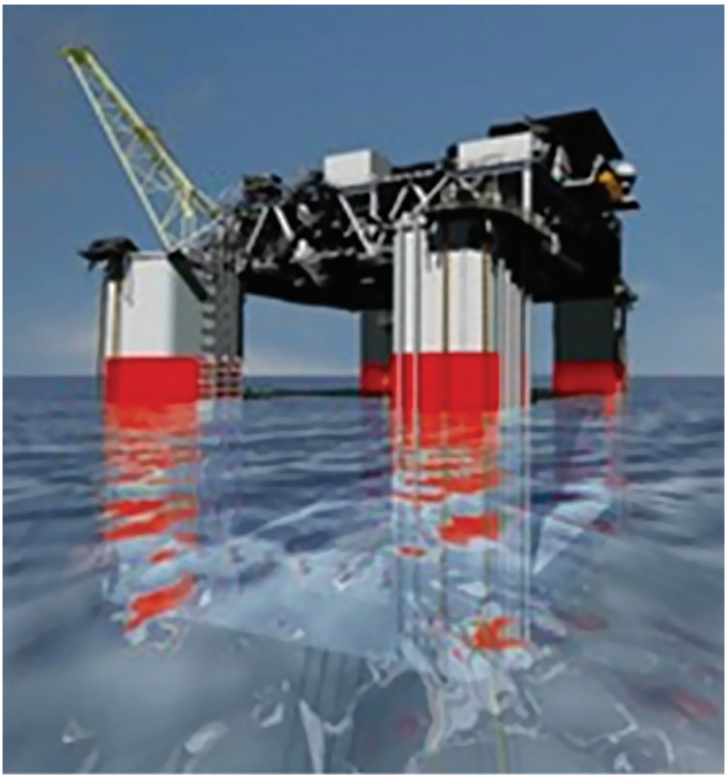


FIGURE 1.34 Blind Faith semisubmersible.

in North America. Table 1.12 presents a summary of semisubmersibles commissioned worldwide. Table 1.13 provides a list of semisubmersibles constructed at different water depths. As shown in the table, semisubmersibles are preferred for different water depths varying from shallow waters to deepwaters. Tables 1.14 and 1.15 provide lists of semisubmersibles commissioned worldwide in deep and shallow waters, respectively.

TABLE 1.12
Semisubmersibles Commissioned Worldwide

S. No.	Platform Name	Location	Water Depth (m)	Year of Commissioning
		Europe		
1	Argyll FPU	United Kingdom	150	1975
2	Buchan A	United Kingdom	160	1981
3	Deep sea Pioneer FPU	United Kingdom	150	1984
4	Balmoral FPV	United Kingdom	150	1986
5	AH001	United Kingdom	140	1989
7	Janice A	United Kingdom	80	1999
8	Northern producer FPF	United Kingdom	350	2009
9	Asgard B	Norway	320	2000
10	Kristin FPU	Norway	320	2005
11	Gjoa	Norway	360	2010
12	Veslefrikk B	Norway	175	1989
13	Troll B FPU	Norway	339	1995
14	Njord A	Norway	330	1997
15	Visund	Norway	335	1999
16	Troll C FPU	Norway	339	1999
17	Snorre B FPDU	Norway	350	2001
		United States		
1	Innovator	North America	914	1996
2	Nakika	North America	969	2003
3	Atlantis	North America	2156	2006
4	ATP Innovator	North America	914	2006
5	Thunder Horse	North America	1849	2008
6	Blind Faith	North America	1980	2008
7	Thunder Hawk	North America	1740	2009
8	P-09	Brazil	230	1983
9	P-15	Brazil	243	1983
10	P-12	Brazil	100	1984
11	P-21	Brazil	112	1984
12	P-22	Brazil	114	1986
13	P-07	Brazil	207	1988
14	P-20	Brazil	625	1992
15	P-08	Brazil	423	1993
16	P-13	Brazil	625	1993

(*Continued*)

TABLE 1.12 (*Continued*)
Semisubmersibles Commissioned Worldwide

S. No.	Platform Name	Location	Water Depth (m)	Year of Commissioning
17	P-14	Brazil	195	1993
18	P-18	Brazil	910	1994
19	P-25	Brazil	252	1996
20	P-27	Brazil	533	1996
21	P-19	Brazil	770	1997
22	P-26	Brazil	515	2000
23	P-36	Brazil Campos Basin	1360	2000
24	P-51	Brazil Campos Basin	1255	2001
25	SS-11	Brazil	145	2003
26	P-40	Brazil	1080	2004
27	P-52	Brazil	1795	2007
28	P-56	Brazil	1700	2010
29	P-55	Brazil	1707	2012
		Asia		
1	Tahara	Indian Ocean	39	1997
2	Nan Hia Tiao Zhan	South China Sea	300	1995
3	Gumusut Kakap	Malaysia	1220	2011

TABLE 1.13
Semisubmersibles at Various Water Depths

S. No.	Water Depth (m)	Number of Platforms
1	<100	1
2	100–200	11
3	201–500	16
4	501–1000	9
5	1001–2000	10
6	>2000	1

TABLE 1.14
Semisubmersibles in Deepwater

S. No.	Platform Name	Water Depth (m)	Location
1	Atlantis	2156	United States
2	Blind Faith	1980	United States
3	Thunder Horse	1849	United States

TABLE 1.15
Semisubmersibles in Shallow Water

S. No.	Platform Name	Water Depth (m)	Location
1	Janice A	80	United Kingdom
2	P-12	100	Brazil
3	P-21	112	Brazil

1.13 FLOATING, PRODUCTION, STORAGE, AND OFF-LOADING PLATFORMS

Other types of offshore compliant platforms are floating, production, storage, and off-loading platforms (FPSOs). FPSOs are typically converted or newly built tankers that produce and store hydrocarbon, which are subsequently transported by other vessels to the terminals or deepwater ports. They are relatively insensitive to water depth as they are floating systems (White et al., 2005). Where an off-loading system is not available, platforms are named as floating production systems (FPSs). The structural forms of offshore structures are significantly modified from bottom supported to completely floating systems in the recent past. Reasons are mainly their versatility and decreased downtime for commissioning and deinstallation, if necessary. The universal term FPS refers to all production facilities that float rather than that are structurally supported by the seafloor. The term is also frequently used to describe the general category of floating production facilities that do not have on-site storage. This includes TLP, spars, semisubmersibles, and shipshape vessels. Floating, storage, and off-loading systems (FSOs) are another type of platforms without any production facility. Like FPSOs, they are actually typically converted or newly built tankers, which are essentially used as storage or off-loading systems. They differ from FPSOs because they do not carry processing equipment, which are essentially required for production. Essentially, they are floating storage units and can also be used as off-loading systems. Off-loading refers to the transfer of the produced hydrocarbon from an offshore facility into shuttle tankers or barges, which subsequently transfer the contents to onshore facility for processing. Today, nearly all FPSOs are installed at a depth of more than 1000 m. Table 1.16 presents a summary of FPSOs installed in different parts of the world. Most of them are commissioned in a water depth greater than 500 m, as shown in Table 1.16. Table 1.17 provides a list of FPSOs at various water depths; and Table 1.18 provides a list of FPSOs commissioned in deepwater. Figure 1.35 shows a schematic view of Greater Plutonio FPSO. As shown in the figure, an FPSO is similar to any typical offshore production platform housing all necessary equipment. It houses all kinds of equipment that are required for production such as drilling derrick, living quarters, helipad, ballast tank, and complicated machineries used for power generation and processing. One can also infer that an FPSO is similar to a conventional barge or tanker, whose hulls are modified to carry out the desired operations.

TABLE 1.16
Details of FPSOs Commissioned Worldwide

S. No.	Platform	Water Depth (m)	Location
	Australia and New Zealand		
1	Maersk Ngujima-Yin	400	Australia
2	Stybarrow Venture	825	Australia
3	Pyrenees Venture	200	Australia
4	Glass Dowr	344	Australia
5	Front Puffin	110	Australia
6	Crystal Ocean	170	Australia
7	Ningaloo Vision	380	Australia
8	Cossak Pioneer	80	Australia
9	Umurao	120	New Zealand
10	Raroa	102	New Zealand
	North America		
1	Terra Nova	95	Canada
2	Sea Rose	122	Canada
3	Yuum K'ak'naab	100	United States
	Egypt		
1	Zaafarana	60	Egypt
2	PSVM	2000	Angola
3	Kizomba A	1241	Angola
4	Kizomba B	1163	Angola
5	Pazfor	762	Angola
6	CLOV	1365	Angola
7	Girassol	1350	Angola
8	Dalia	1500	Angola
9	Gimboa	700	Angola
10	Kuito	414	Angola
11	Petroleo Nautipa	137	Gabon
12	Knock Allan	50	Gabon
13	Abo	550	Nigeria
14	Bonga	1030	Nigeria
15	Armada Perkasa	13	Nigeria
16	Armada Perdana	350	Nigeria
17	Erha	1200	Nigeria
18	Usan	750	Nigeria
19	Agbami	1462	Nigeria
20	Akpo	1325	Nigeria
21	Ukpokiti	***	Nigeria
22	Kwame Nkrumah MV 21	***	Ghana
23	Sendje Ceiba	90	Equatorial Guinea
24	Aseng	945	Equatorial Guinea

(*Continued*)

TABLE 1.16 (*Continued*)
Details of FPSOs Commissioned Worldwide

S. No.	Platform	Water Depth (m)	Location
25	Zafiro	915	Equatorial Guinea
26	Chinguetti Berge Helene	800	Mauritania
27	Baobab	1219	Cote d'Ivoire
	Europe		
1	Huntington	91	United Kingdom
2	BW Athena	134	United Kingdom
3	Global Producer III	140	United Kingdom
4	Bleo Holm	105	United Kingdom
5	Aoka Mizu	110	United Kingdom
6	Kizomba	1341	United Kingdom
7	Hummingbird	120	United Kingdom
8	Petrojarl Foinaven	461	United Kingdom
9	Maersk Curlew	76	United Kingdom
10	Schiehallion	400	United Kingdom
11	North Sea Producer	125	United Kingdom
12	Caption	106	United Kingdom
13	Norne	380	Norway
14	Alvheim	130	Norway
15	Petrojarl I	100	Norway
16	Skarv	391	Norway
17	Goliat	400	Norway
18	Asgard A	300	Norway
19	Petrojarl Varg	84	Norway
	South America		
1	Cidade de Rio das Ostras	977	Brazil
2	Cidade de Sao Mateus	763	Brazil
3	P-63	1200	Brazil
4	Frade	1128	Brazil
5	Cidade de Victoria	1400	Brazil
6	Peregrino	120	Brazil
7	Espadarte I	1100	Brazil
8	Espadarte II	850	Brazil
9	Golfinho	1400	Brazil
10	Marlim Sul (South)	1430	Brazil
11	Espirito Santo	1780	Brazil
12	Cidadde de Angra	2149	Brazil
13	Cidade de Niteroi MV18	1400	Brazil
14	Cidade de Santos MV20	1300	Brazil
	Asia		
1	Bohai Shi Ji	20	China
2	Bohai Ming Zhu	31	China

(*Continued*)

TABLE 1.16 (*Continued*)
Details of FPSOs Commissioned Worldwide

S. No.	Platform	Water Depth (m)	Location
3	Song Doc MV19	55	Vietnam
4	Ruby II	49	Vietnam
5	Ruby Princess	50	Vietnam
6	Arthit	80	Thailand
7	Bualuang	60	Thailand
8	Anoa Natuna	253	Indonesia
9	Kakap Natuna	88	Indonesia
10	Dhirubhai I	1200	India

TABLE 1.17
FPSOs at Various Water Depths

S. No.	Water Depth (m)	Number of Platforms
1	<100	19
2	101–500	30
3	501–1000	10
4	1001–2000	22
5	>2000	1

TABLE 1.18
FPSOs in Deepwater

S. No.	Platform Name	Water Depth (m)	Location
1	Cidade de Angra	2149	Brazil
2	PSVM	2000	Angola
3	Espirito Santo	1780	Brazil

Figure 1.36 shows a typical production system deployed in an FPSO. Different layouts of subsea trees and flow lines are attached to the FPSO, as shown in the figure.

Figure 1.37 illustrates the processing system of an FPSO. It also shows the different components of the FPSO. For example, one can see the flare stack, gas turbine, and an offtake system that has off-loading facility. The central moonpool houses the flexible risers that enter the production systems. It contains a main control room, a helideck, and living quarters on the top deck. The middle deck contains machinery rooms comprising high- and low-voltage rooms, engine and boiler rooms on the upper hull, and on the lower hull as well. The bottommost hull of the FPSO has thruster rooms and azimuth thrusters on either side of the FPSO. The chain mooring system is attached to the lowest hull of the vessel.

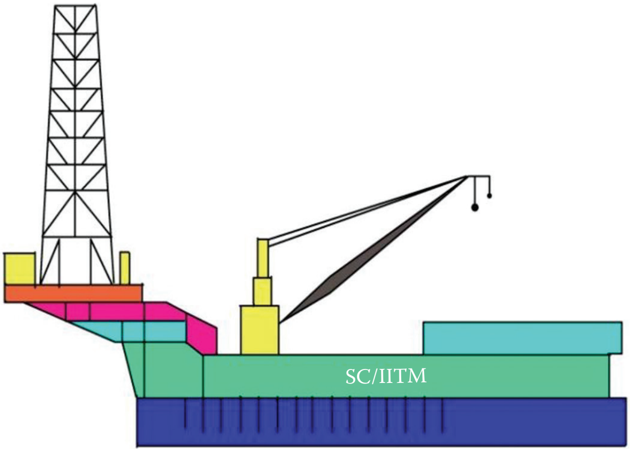


FIGURE 1.35 Greater Plutonio FPSO.

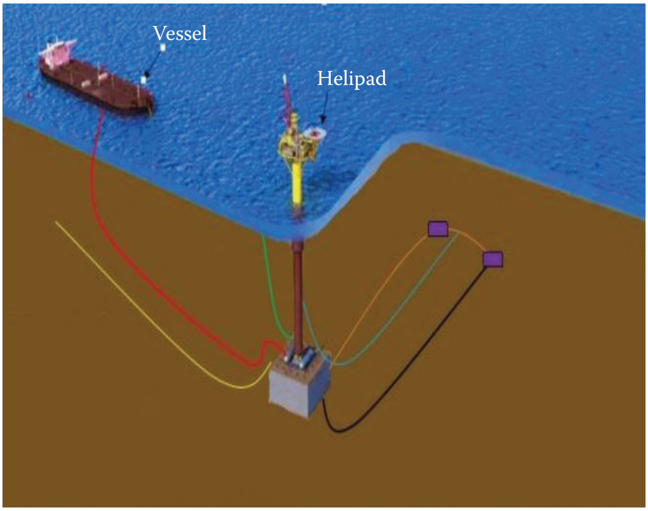


FIGURE 1.36 Production system of an FPSO.

The hull of an FPSO is typically shaped like a ship with a monohull structure. A typical FPSO can be characterized as a tanker with the dimensions as follows: The length varies from 200 to 400 m, whereas the breadth varies from 30 to 60 m and the height varies from 20 to 30 m. Of those systems deployed to date, most of them have conversions of smaller and older tankers. In general, FPSOs are converted modules of old tankers, which have been used for production storage and off-loading facilities in the sea. One of the major advantages of converting the existing vessel to an FPSO is time saving. One can use it rapidly for the first production. The storage capacity of an FPSO depends on many parameters: ship size, availability, and size of the offtake vessel

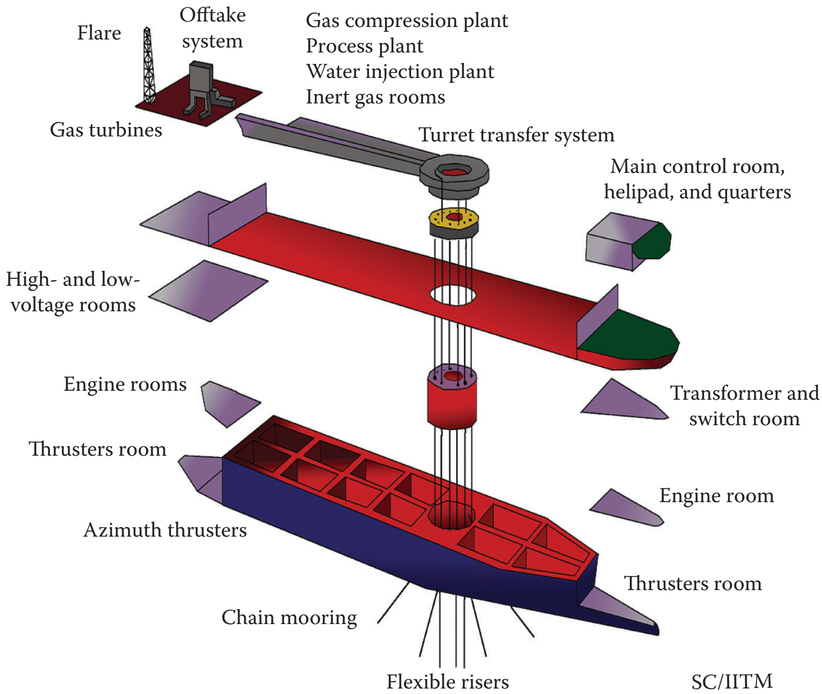


FIGURE 1.37 Processing system of FPSO.

through which the stored oil can be transported; projected downtime; and cargo destination. The projected downtime is the time of operation at which the FPSO will remain in the sea for production. Table 1.16 provides a summary of the FPSOs commissioned in different parts of the world. Table 1.17 provides details of FPSOs commissioned in different water depths. As shown in the table, FPSOs are generally preferred for deepwater and isolated locations where the other types of drilling and production platforms cannot be commissioned for technical reasons. Tables 1.18 and 1.19 provide details of FPSOs commissioned in deep and shallow waters, respectively.

There are two options for station-keeping of an FPSO. The majority of the existing FPSOs employ a fixed mooring system, which uses anchors; anchor lines hold the FPSO in position. A few of them also use dynamic positioning system (DPS), which employ a series of thrusters and a positioning technology. The type of the

TABLE 1.19
FPSOs in Shallow Water

S. No.	Platform Name	Water Depth (m)	Location
1	Armada Perkasa	13	Nigeria
2	Bohai Shi Ji FPSO	20	China
3	Bohai Ming Zhu FPSO	31	China

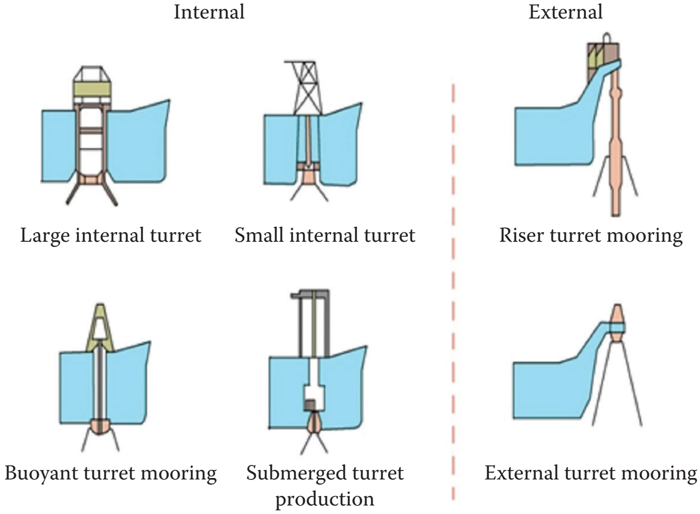


FIGURE 1.38 Turret mooring systems.

DPS depends on the satellite and GPS receivers that are actually deployed for station keeping. The fixed mooring system can be either permanent or temporary. Most FPSOs deploy a permanently moored system. Actually, they are designed to remain at the location throughout the anticipated environmental situations. They are not decommissioned even though the weather can become rough. There are very few cases in which they have been designed to be disconnected under severe conditions such as typhoons and hurricanes or icebergs. Turret mooring systems are generally equipped for an FPSO. It can be either internal or external, as shown in Figure 1.38. Internal mooring systems are further classified as large internal turret mooring system, small internal turret mooring system, buoyant turret mooring system, and submerged turret production system, whereas external mooring systems are classified as riser turret mooring system and external turret mooring system, as shown in the figure.

Alternatively, the FPSO also uses a submerged turret production system, where the turret mooring system is submerged in the FPSO vessel itself, whereas in the earlier cases, they are housed on the top hull. In the case of the external mooring system, Figure 1.38 shows that the production risers are cantilevered away from the FPSOs, whereas in case of the internal mooring system, drilling takes place through the moonpool, and hence the name internal mooring system or external mooring system. Figure 1.39 shows the anatomy of a turret mooring system. The figure shows that a swivel stack is located at the top with its access from the top hull. It is attached to a rotating crane, which is used to lay risers or mooring lines.

The turret cylinder is equipped in the moonpool. Turret annulus is a component with an angular covering of the turret mooring, which rests on the main bearing. The chain stopper located at the bottom of the swivel stack prevents the backflow of the chains or the mooring lines into the turret system. However, in the case of the disconnectable mooring system, one can demobilize the personnel and assets during

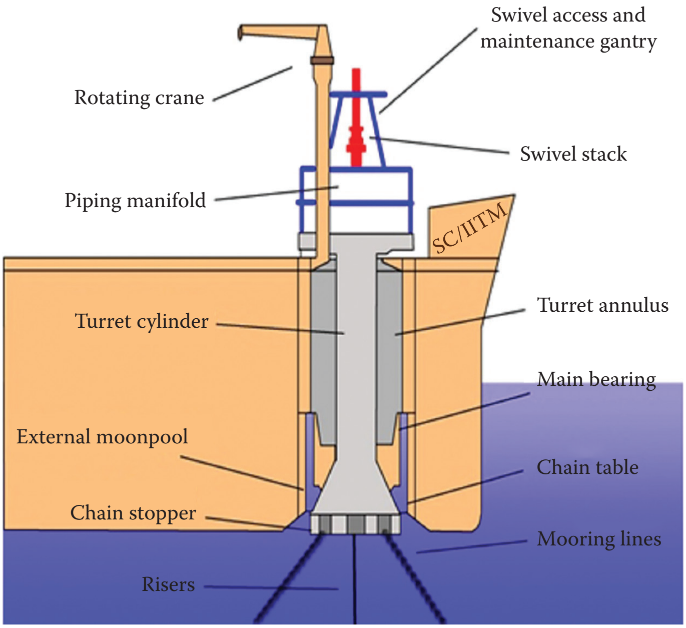


FIGURE 1.39 Anatomy of a turret mooring system.

emergency. When the mooring system or riser is connected to the FPSO or any floating system, there will be a coupled action imposed on the floating system by the risers. This dominates the structural response of the floating platform in addition to its response under lateral loads. In case of extreme emergency, the mooring system is generally disconnected from the FPSO to prevent severe damage imposed by the risers.

1.14 RISERS

Risers are used to transport the produced fluid from the production equipment located on the seafloor to the processing equipment located on the hull of the FPSO. Gas export lines (used in addition to shuttle tanker operations) will also exit the FPSO in a similar manner as that of risers. The riser system associated with an FPSO can be integrated into the mooring system for turret mooring systems and must be accounted for in the mooring system design. If the FPSO mooring is a fixed-point system, as used for semisubmersibles, risers can be hung off the side of the facility. The design basis for power supply to the FPSO focuses on three categories: The main power supply that includes all electrical functions during normal operations; the essential power supply that includes the startup of essential services and the shutdown of facilities as needed; and the emergency power supply that includes life support during a "survival at sea" situation. In addition to the conventional power generation needed for production processing, an FPSO may need power for the thrusters, which can be used in lieu of the mooring system.

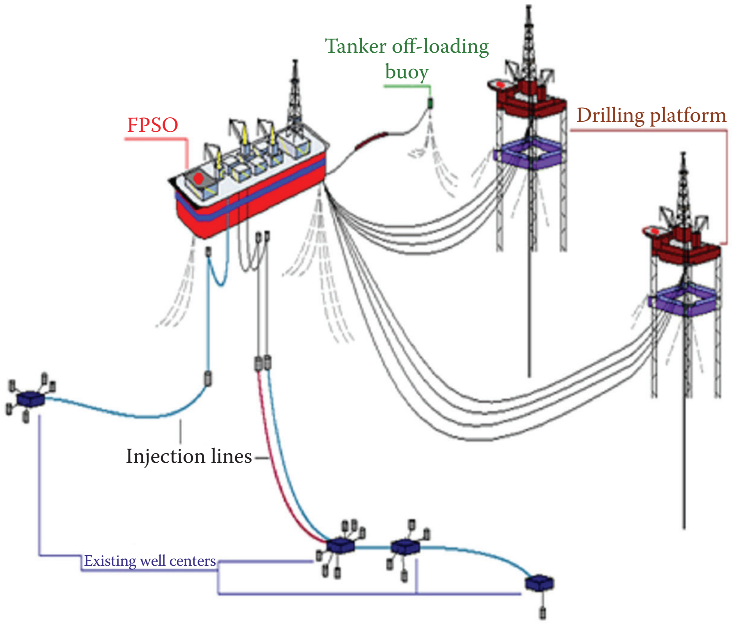


FIGURE 1.40 Offtake system.

1.15 OFFTAKE SYSTEMS

Offtake essentially means off-loading. Off-loading is a term related to the transfer of the produced oil hydrocarbon from a barge or floating system to another location. Figure 1.40 shows a typical offtake system, which connects the FPSO to different drilling wells.

As the FPSs may not have a larger capacity to store the explored hydrocarbons, a large volume of explored oil and gas need to be transported. The liquid hydrocarbons from an FPSO are off-loaded into a shuttle tanker, which further transports it to the onshore. Offtake systems actually include equipment associated with storage tanks to the shuttle tanker, mooring lines, buoys, and transfer hoses. Mooring lines are used for holding down the off-loading system during operation, whereas buoys are used to transfer crude oil from the FPSO to the off-loading system using transfer hoses. Common offtake systems are tandem offtake system, side-by-side offtake system, single-point offtake systems, and remote systems. Figure 1.41 shows a schematic view of the offtake system in tandem and Figure 1.42 shows that of the system side by side.

1.16 DRILLING PLATFORMS

There are two types of basic offshore drilling platforms: (1) movable drilling rigs and (2) permanent drilling rigs. The former are typically used for exploration purposes, whereas the latter are used for the extraction and production of oil and/or gas. Figure 1.43 shows a schematic view of a typical drilling platform, whereas Figure 1.44 shows a permanent drilling rig.

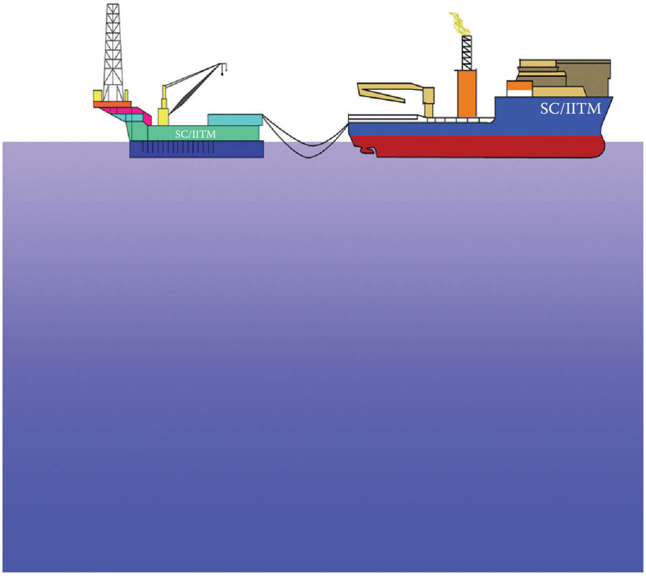


FIGURE 1.41 Offtake system in tandem.

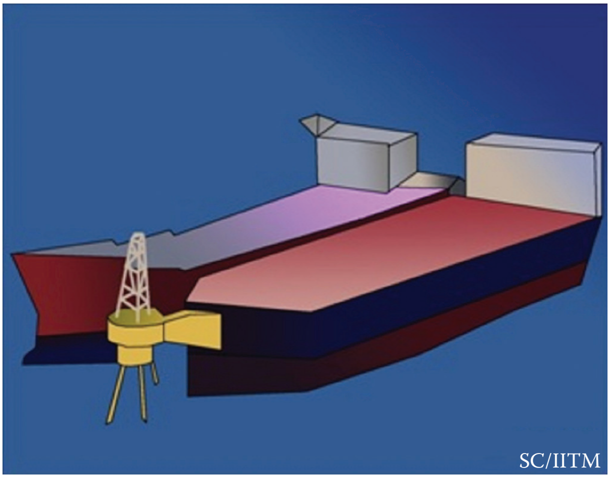


FIGURE 1.42 Offtake system—side by side.

Drill ships are designed to carry out drilling operations. These are large vessels that are specially designed to carry drilling operations in deep sea locations (DNV-RP-F205, 2010). A typical drill ship will have a drilling platform and derrick located in the middle of its deck. In addition, drill ships contain a moonpool that extends right through the hull. Figure 1.45 shows a comparison of floating platform and drill ship. Drill ships are deployed to drill in ultra-deepwaters, which can often be quite turbulent. They use a DPS and are equipped with electric motors on the

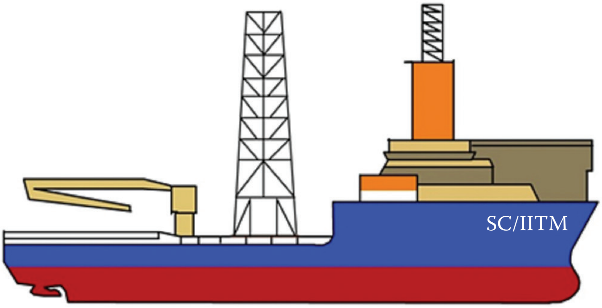


FIGURE 1.43 Drilling platform.

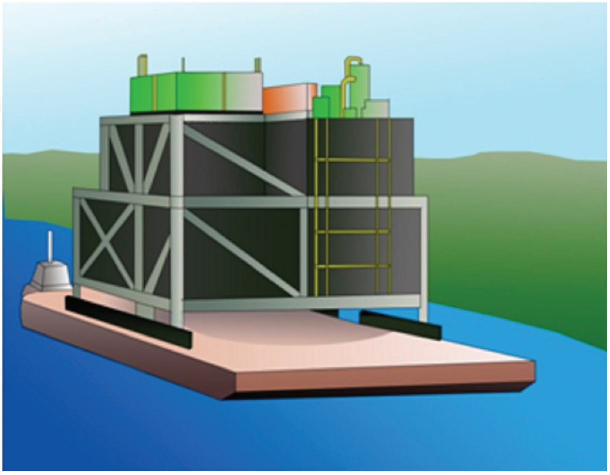


FIGURE 1.44 Permanent drilling rig.

underside of the ship's hull, which is capable of propelling the ship in the desired direction (Chakrabarti, 1998, 2005).

Figure 1.46 illustrates the details of a drill ship. Drill ships are preferred for deepwater drilling in remote locations with a moderate weather environment. High mobility and large load-carrying capability are salient advantages. In comparison with semisubmersibles, drill ships are advantageous because of their conventional ship-shaped hull. They can be subjected to longer periods of downtime under wind and wave actions. Drill ships are used in smoother waters of the world, whereas semisubmersibles can drill in the most hostile environments. Drill ships are susceptible to wave action; criticality in the response is extremely important because the vessel is connected to the seabed by a riser, and the drill string is in contact with the bottom of the borehole. Drill ships are designed to carry the drilling platforms to great distances offshore and in ultra-deepwaters. A large derrick is mounted permanently above the moonpool, which is similar to that of a land rig. Drill ships can drill holes up to 3000 m deep and are typically used for exploration activities, especially

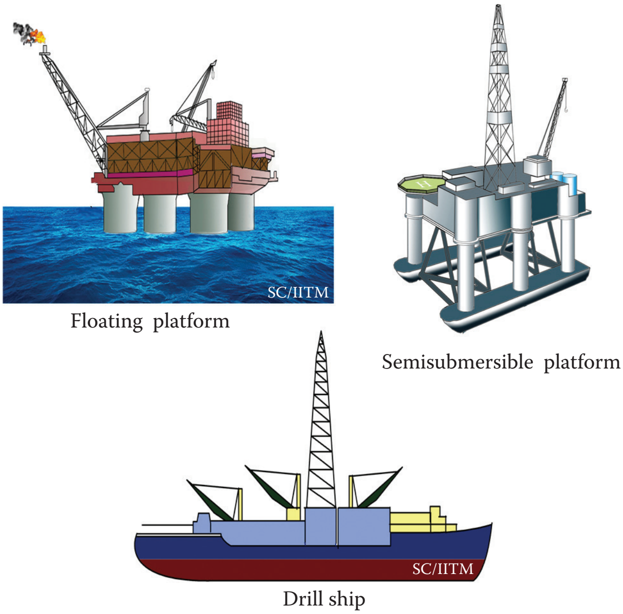


FIGURE 1.45 A drill ship and floating platforms.

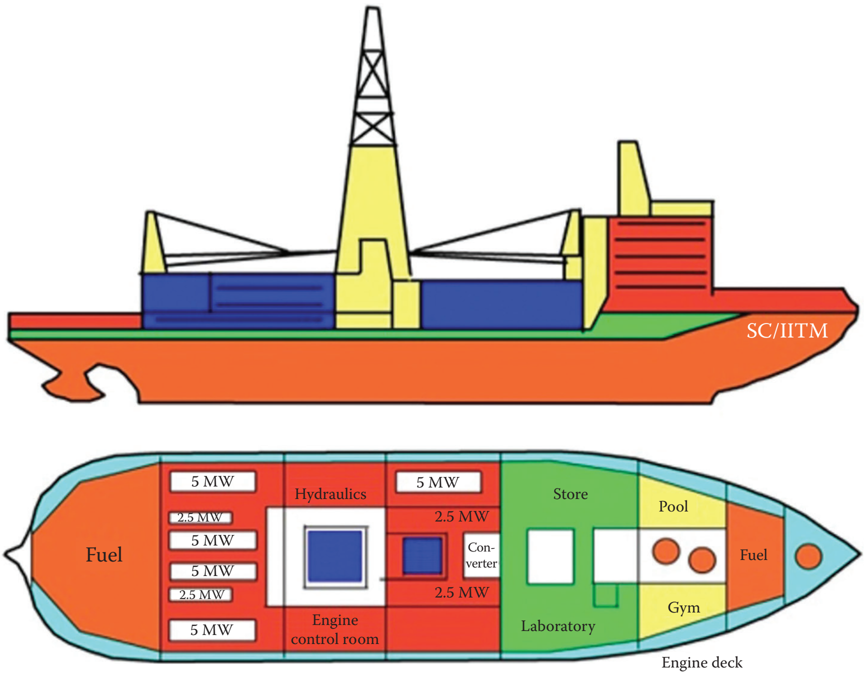


FIGURE 1.46 Schematic views of a drill ship.

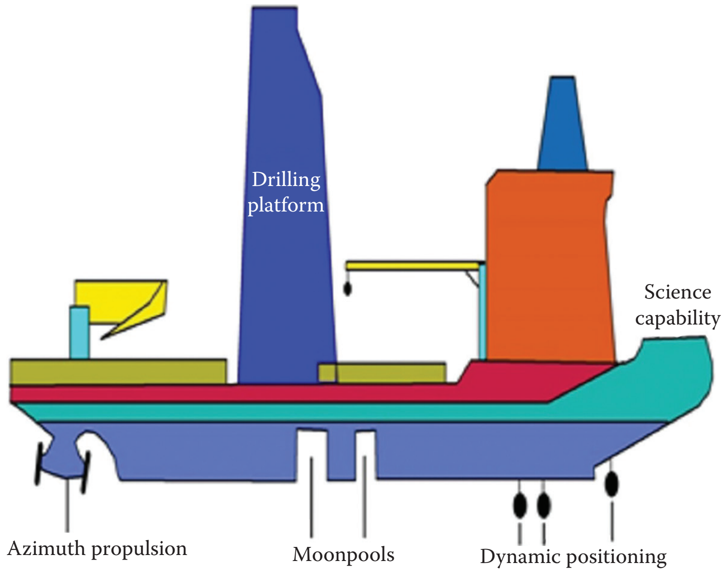


FIGURE 1.47 Dynamic positioning system.

in remote locations. As they are highly mobile and self-propelled, they are popular in isolated locations. They are also relatively unstable and liable to be tossed by waves and currents under excessive roll/pitch motion.

Drill ships are generally an adaptation of a standard sea-going vessel with a monohull form. Modifications are done on the substructure to add a moonpool and a few cantilevers from which the drilling operations may be carried out. These vessels are equipped with an additional means of positioning the unit over the drill center. This is required to establish close contact with the borehole in the seabed. Most of the drill ships are equipped with DPS, which use computers to detect whether the ship has strayed too far from its desired location. When necessary, they activate thrusters to move the ship back into place. Figure 1.47 shows a schematic view of a drill ship fitted with a DPS. Propeller motors are integrated with the computer system of the ship. It uses the satellite positioning technology in conjunction with the sensors located on the drilling template. It ensures that the ship is directly above the drill site at all times. In the most common case, drilling platform is equipped with anchor lines. A mooring system usually has about 8–12 anchor lines for each platform. However, in water depths more than 1000 m, a mooring system becomes uneconomical or impracticable. An alternative solution is the DPS. The DPS controls platform displacements in all the horizontal degrees of freedom. It is composed of a controller, a sensor system, a thruster system, and a power system. The sensor system feeds the controller with information about the platform positioning and environmental parameters that arise from wind, current, and waves. The controller commands the action of thrusters, installed on the bottom of the platform hull, which in turn generates forces and moment needed to counteract the environmental forces.

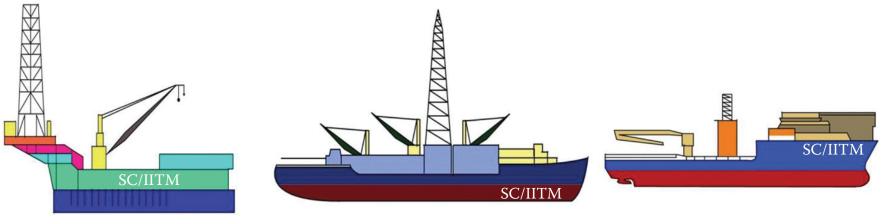


FIGURE 1.48 Drill ships with drilling platforms.

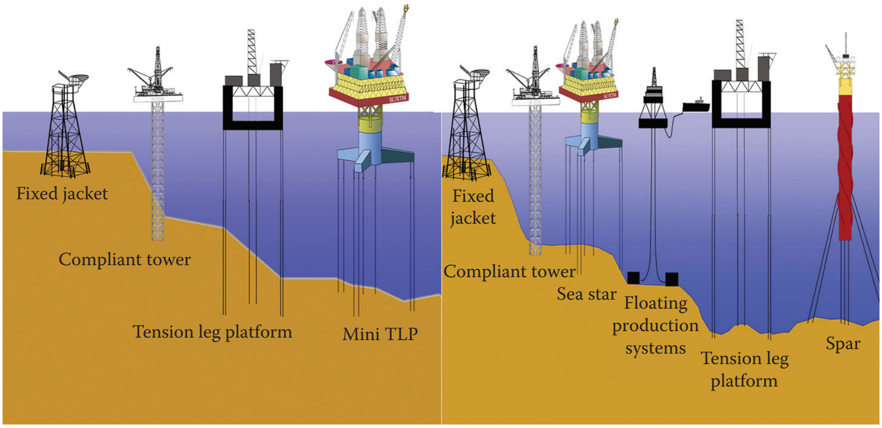


FIGURE 1.49 Different forms of offshore structures.

This helps to keep the vessel at the reference point. It keeps the platform within a tolerance radius of about 2%–6% of the water depth.

Figure 1.48 shows to different types of drill ships equipped with drilling platforms that are commonly deployed in deep sea exploration.

Figure 1.49 shows to different structural forms of offshore structures that are attempted in the past at different water depths. The figure shows that the structural form of deepwater is preferred to be highly compliant due to several advantages as discussed above. Figure 1.50 shows different structural forms of floating offshore structures. Figure 1.51 depicts various locations in the world that deploy offshore structures for oil and gas exploration.

As shown in Figure 1.51, worldwide statistics show that there are about 400 jack-up rigs, 170 semisubmersibles, and about 40 drill ships. Considering the Southeast Asia segment, jack-up rigs are deployed to the maximum. India uses predominately jack-up rigs and a very less number of semisubmersibles and drill ships, whereas the Middle East uses jack-up rigs to the maximum extent. Similarly, on the West African coast, jack-up rigs are substantially high in deployment compared to semisubmersibles and other types of platforms.

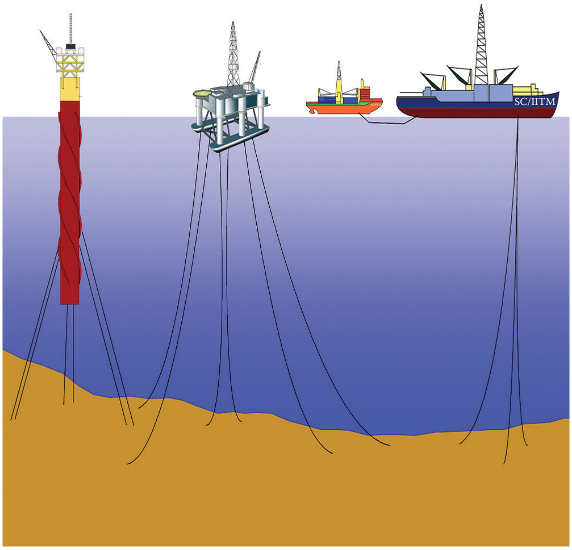


FIGURE 1.50 Different forms of offshore floating structures.

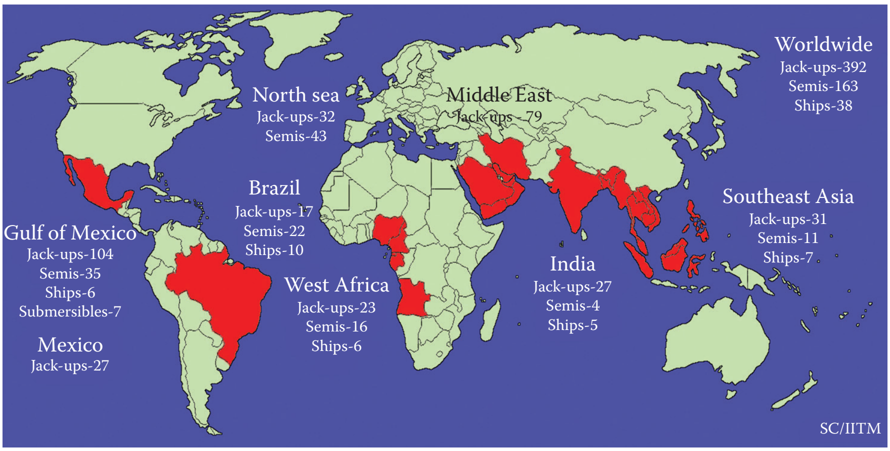


FIGURE 1.51 Drilling platforms worldwide.

1.17 PETROLEUM AND NATURAL GAS

Petroleum and natural gas deposits are found in sedimentary rock basins, where tiny sea plants and animals died millions of years ago. Petroleum products are available in barrels of crude oil with a composition of 19.5 gallons of gasoline, 9.2 gallons of fuel oil, 4.1 gallons of jet fuel, 2.3 gallons of asphalt, 0.2 gallons of kerosene, 0.5 gallons of lubricants, and 6.2 gallons of other products, which makes it about 42 gallons.

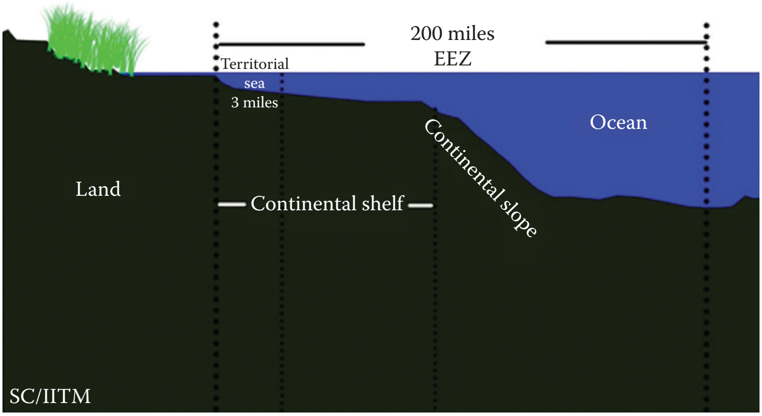


FIGURE 1.52 Exclusive economic zone.

In the presence of proper temperature and pressure, these plants and animals eventually turned into hydrocarbons. Oil and gas are made mostly of hydrogen and carbon, named as hydrocarbons. These hydrocarbons flow into empty spaces in the surrounding rocks, called traps. An oil-soaked rock, which is similar to that of a wet sponge, is formed. These traps are covered with a layer of solid rock, or a seal of salt or clay that confines oil and gas from escaping to the surface. Drilling is done through these formations to explore oil/gas.

The exclusive economic zone (EEZ) shown in Figure 1.52 is a commonly defined boundary for offshore drilling. As per the classical definition of the EEZ, the territorial region is up to 3 miles from the shore. The continental shell is about 20–25 miles, beyond which it is called a continental slope. The EEZ lies about 200 miles away from the offshore. For example, the U.S. EEZ extends by about 3.9 billion acres under water. Compared with the land area, which is only about 2.3 billion acres, the EEZ serves about 30% of the U.S. gas and oil reserves in the respective offshore basins.

1.18 OIL AND GAS EXPLORATION: STEPS AND EFFORTS

Exploration for oil and gas is a time- and effort-intensive process, which relies on the collection and detailed analyses of extensive geologic information. Surveying and mapping of the surface and subsurface are carried out to study the geologic features. Seismic reflection methods are used to identify the location of hydrocarbon traps. The potential of geologic formation is estimated to compute the economically producible oil and/or gas. The best locations to drill are then identified to carry out exploratory drilling to test the hydrocarbon traps. Exploration and delineation wells are drilled to determine the area and thickness of the oil and/or gas reserve. Delineation wells are wells that are drilled outward from a successful wildcat well to determine the boundaries of productive formation. A *Wildcat well* is an exploratory oil well drilled in land, which is not known to be an oil field.

Logging and coring wells are drilled to measure the permeability, porosity, and other properties of the geologic formations encountered. Once a potential location is identified, one or more exploratory wells are drilled to provide information on the composition of the underground rock layers and their geological and geophysical properties. Well logging refers to performing tests during or after the drilling process. This is carried out to allow geologists and drill operators to monitor the progress of the well drilling. This is useful to gain a clear picture of the subsurface formations and to identify specific rock layers, in particular those that represent the target zones for further exploration.

1.19 OIL AND GAS WELL DRILLING

In the present state of art of drilling, almost all oil and gas wells are drilled using rotary drilling. In rotary drilling, a length of steel pipe, called drill pipe, with a drill bit on its end is rotated to cut a hole; this is called the wellbore. As the well goes deeper, additional sections of drill pipes are added to the top of the rotary drill string. Rotary drilling uses a steel tower to support the drill pipe. If the tower is part of a tractor trailer and is jacked up as a unit, it is called a mast. Alternatively, if it is constructed on-site, it is called a derrick. Towers constructed of structural steel are mounted on the derrick floor, where most of the drilling activity occurs. Figure 1.53 shows a typical drilling stack. The major systems of an operational rotary drilling rig comprise a power supply unit, a hoisting system, a rotating system, and a circulating system.

The top of the drilling rig mast is called the crown block. The crown block is connected to the drilling rig and the drilling casing. The outside of the drilling pipe is called the drilling casing. Draw works will supply the necessary liquid at a desired pressure to perform drilling.

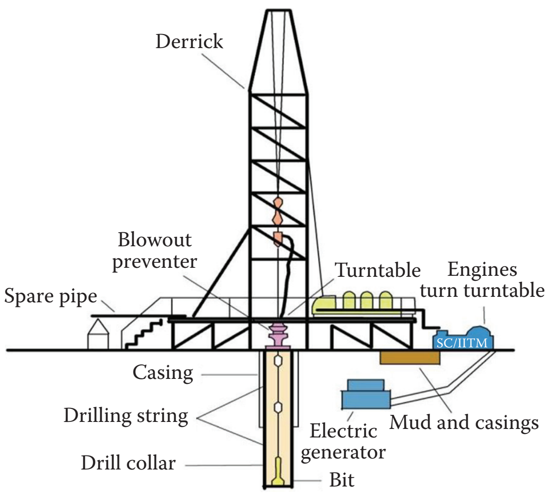


FIGURE 1.53 Oil and gas well drilling stack.

1.20 OFFSHORE DRILLING

One of the major differences between onshore and offshore drilling is the nature of the drilling platform. Figure 1.54 shows a schematic view of an offshore drilling rig. In addition, in offshore drilling, the stack passes through the water column before entering the seafloor. Offshore wells have been drilled in waters as deep as 3000 m. Offshore drilling requires the construction of an artificial drilling platform. The form of the drilling platform depends on the characteristics of the well to be drilled. Offshore drilling also involves the use of a drilling template that helps connect the underwater drilling site to the drilling platform located at the water surface. This template typically consists of an open steel box with multiple holes, depending on the number of wells to be drilled. The template is installed on the seafloor by first excavating a shallow hole and then cementing the template into the hole. It provides a stable guide for accurate drilling while allowing for the movement in the overhead platform due to wave and wind actions. Directional drilling techniques were employed in the 1970s. Normally, wells are drilled vertically; however, there are many occasions when it is helpful to drill at an angle. Directional wells are drilled straight to a predetermined level and are then gradually curved. By changing the direction of the drill bit in small increments of not more than 2–3° at a time, it is possible to drill many wells into a reservoir from a single offshore platform. Figure 1.55 shows a schematic view of directional drilling. Directional wells may also be deflected from a shoreline to reach a reservoir under nearby water. These are very useful in avoiding fault lines, which can cause whole problems. They can also be used in instances where it is undesirable to set a rig in a given spot because of an obstruction or for environmental reasons.

Once a well is drilled and tested, a decision must be taken whether to complete the well or plug it. The rock porosity and permeability of the target reservoir may indicate the potential flow of oil and gas from the drilled well. If it does not justify the cost to complete the well, it is plugged with concrete in several places and the well is abandoned; otherwise, the well is completed. In such cases, production casing is run down

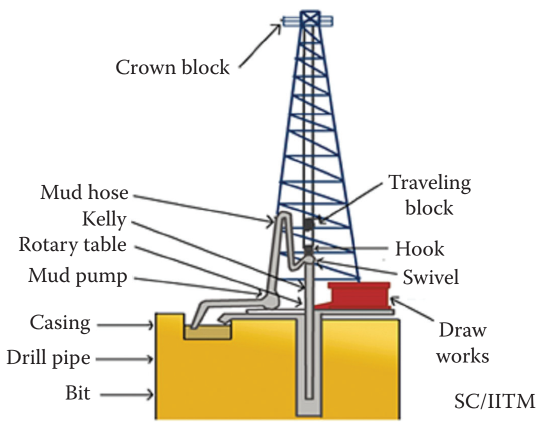


FIGURE 1.54 Offshore drilling.

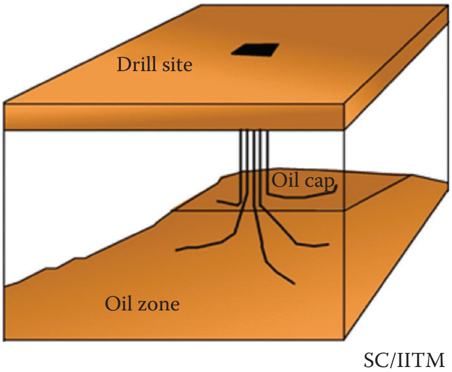


FIGURE 1.55 Directional drilling.

the hole and cemented. Once the casing is in place, a perforating gun is lowered into the wellbore to blast holes through the casing and cement, and into the reservoir. These holes are made to establish communication between the reservoir and the production casing. Tubing is then lowered into the casing. A plug is set above the perforations as a barrier between the production casing and the tubing. This allows the earth's natural pressure to push hydrocarbons to the wellbore and to the surface through the tubing unless a pump is necessary to raise the fluids to the surface, which is called secondary recovery. Figure 1.56 shows a scheme of secondary recovery.

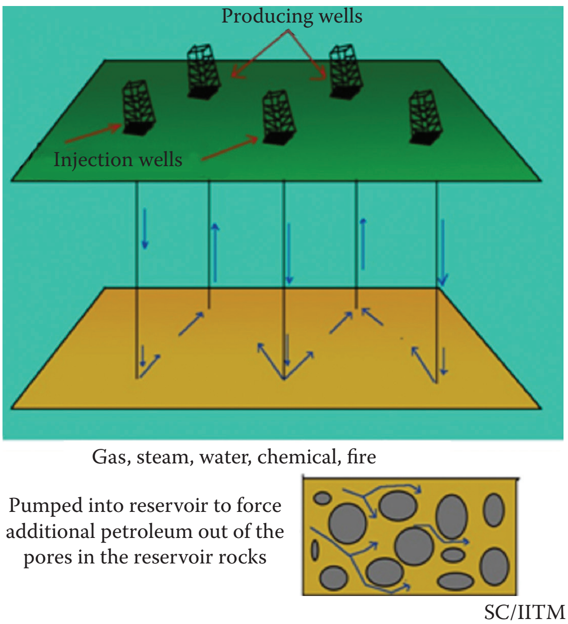


FIGURE 1.56 Secondary recovery.

1.21 SUBSEA PRODUCTION SYSTEMS

Subsea systems are multicomponent seafloor systems. They allow the production of hydrocarbons in water depths where conventional fixed or bottom founded platforms cannot be installed. They comprise an array of subsea wells, manifolds, and central umbilical. Figure 1.57 illustrates the different layouts of subsea systems: a single-well satellite, a multiwell satellite, a cluster-well satellite, a template, and a combination of the above.

A multicomponent system consists of a subsea production tree, pipeline and flow line, subsea manifold, umbilical, host facility, termination units, production risers, templates, and jumpers. A subsea production tree is an arrangement of valves, pipes, fittings, and connections placed on top of a wellbore. Orientation of the valves can be in the vertical bore or the horizontal outlet of the tree valves, which can be operated by electrical or hydraulic signals; alternatively, they are also operated by a remotely operated vehicle (ROV). A schematic view of the valve, which is operated by the ROV, is shown in Figure 1.58.

The arrangement of the valves in the production tree dictates the type of tree: vertical bore or horizontal bore. Figure 1.59 shows the type of production trees. Pipelines and flow lines, which are part of the multicomponent system, are conduits for transporting the fluid from one location to another. Pipelines are piping, risers, and appurtenances installed for the purpose of transporting oil, gas, sulfur, and produced waters between two separate facilities. The length and size of a pipeline or flow line depend on its purpose. Pipe lengths can range from 1 m to 100 km and are typically 450 mm in diameter. Flow lines are piping installed within the confines of the platform or manifold. They are installed for the purpose of mixing the subsea manifold or routing into the processing equipment.

Figure 1.60 shows a typical subsea manifold. It is a gravity-based seafloor structure that consists of valves, pipes, and fittings. It serves as a central gathering point for production from subsea wells and redirects the combined flow to the host facility. A subsea manifold may not be needed for all subsea designs, for example, in field developments where individual production trees are directly tied

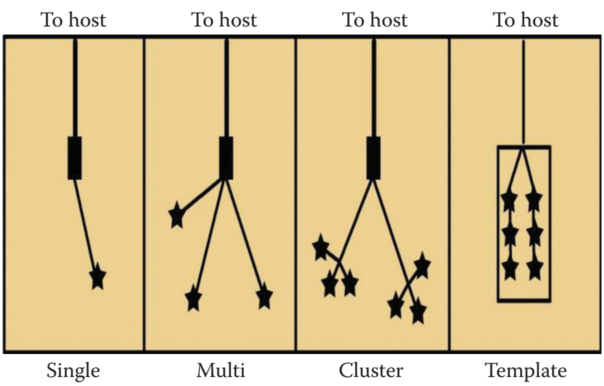


FIGURE 1.57 Different layouts of subsea systems.

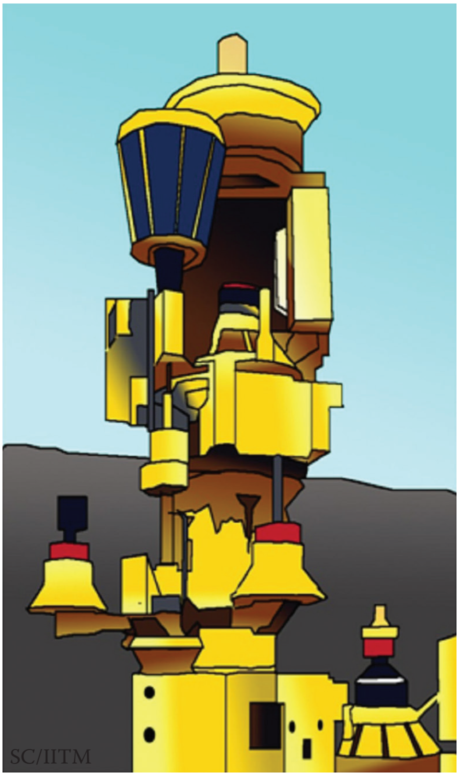


FIGURE 1.58 Valve operated by an ROV.

to the host facility. A manifold arrangement can be any shape, but normally is rectangular or circular. In general, it is either a stand-alone structure or integrated into a well template. The manifold may be anchored to the seafloor with piles or skirts that penetrate the mud line. Although the size of the manifold is governed by the number of wells, its pattern depends on how the wells are integrated into the system. A typical subsea manifold will have dimensions of 24 m in diameter and 9 m high above the seafloor.

Figure 1.61 provides a schematic view of umbilical and jumpers. The umbilical is a bundled arrangement of tubing, piping, and/or electrical conductors in an armored sheath, which is installed from the host facility to the subsea production system. An umbilical is used to transmit the control fluid and/or electrical current necessary to control the functions of the subsea production and safety equipment (tree, valves, manifold, etc.) (Nordic Committee for Building Regulations, 1977; Norwegian Petroleum Directorate, 1985; OCS, 1980). Dedicated tubes in an umbilical are used to monitor pressures and inject fluids (chemicals such as methanol) from the host facility to critical areas within the subsea production equipment.

Electrical conductors transmit the power to operate subsea electronic devices. Dimensions typically range up to 200 mm in diameter. The umbilical includes multiple tubings normally ranging in size up to 25 mm. The number of tubes depends on

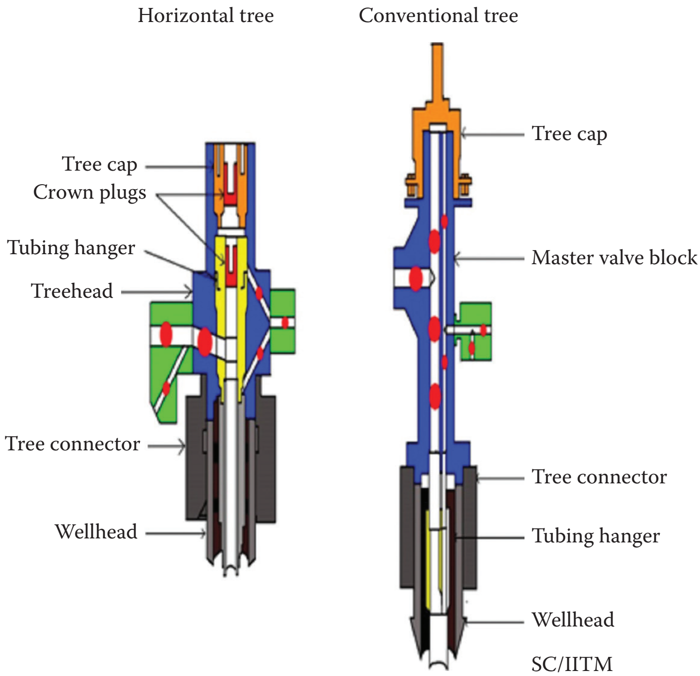


FIGURE 1.59 Types of production trees.

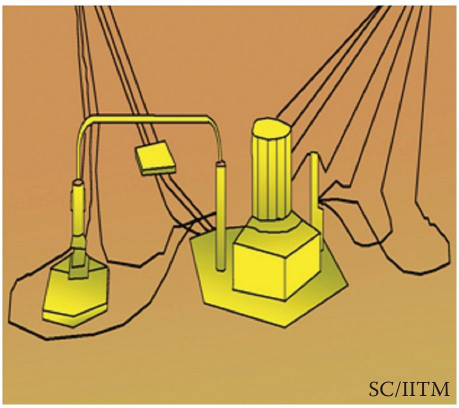


FIGURE 1.60 Subsea manifold.

the complexity of the production system. The length of an umbilical is defined by the spacing of the subsea components and the distance these components are located from the host facility. A typical host facility can be any one of the various types of platforms used for developing offshore hydrocarbon fields, including fixed jacket-type platforms, TLPs, spars, FPSs, FPSOs and off-loading systems. The type of host

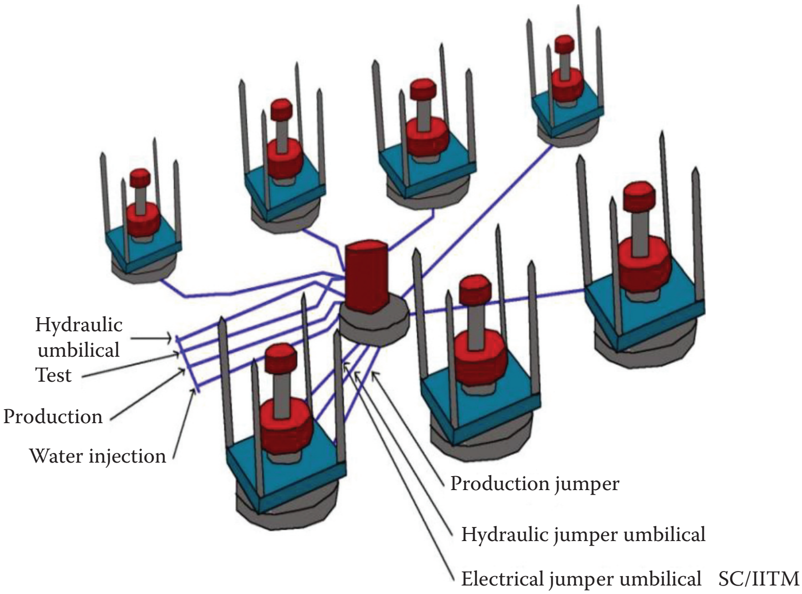


FIGURE 1.61 Umbilicals and jumpers.

facility used for the subsea production system depends on the water depth, the type of field development, the reserve base, and the distance from the infrastructure, but is also largely governed by economic considerations. The termination unit is used to facilitate the interface of the umbilical, pipeline, or flow line with that of the subsea equipment. It has a number of analogous names, including pipeline end manifold, umbilical termination assembly, electrical distribution structure, and flow line lay-down sled. It can be used for electric and/or hydraulic control applications. It is generally equipped with an installation arm to brace it during the lowering process. It is positioned near subsea manifolds, production trees, and flow line, alternatively incorporated into the design of manifolds and templates.

A production riser is a portion of the flow line that resides between the host facility and the seabed adjacent to the host facility. They are usually 3–12 inches in diameter, whose length is governed by the water depth and riser configuration. Risers can either be vertical or assume a variety of wave forms; they can be either flexible or rigid. They can also be contained within the area of a fixed platform or floating facility, run on the seafloor, as well as run partially in the water column. Figure 1.62 shows the different layouts of risers. A template is a fabricated structure that houses the subsea equipment. Templates can be of any shape but are typically rectangular. Dimensions range from 10 to 150 in. length, 10 to 70 in. width, and about 10 to 70 in. height. Templates can accommodate multiple trees in tight clusters, manifolds, pigging equipment, termination units, and chemical treatment equipment. Figure 1.63 shows a schematic layout of a template on subsea equipment. Jumpers are pipe

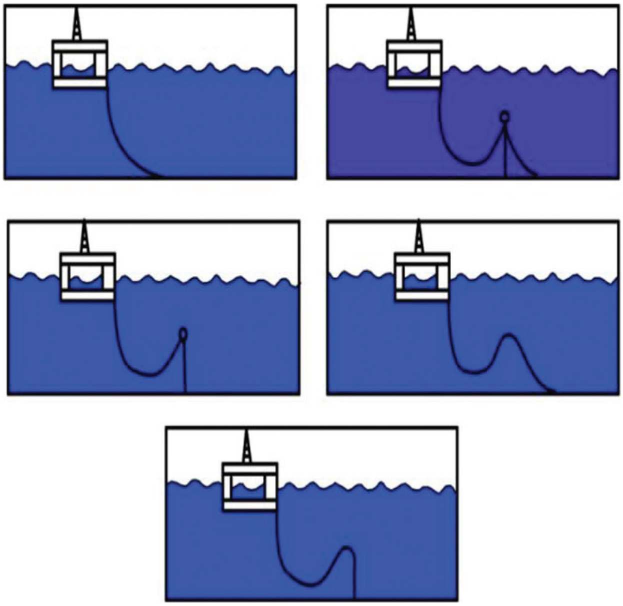


FIGURE 1.62 Different layouts of risers.

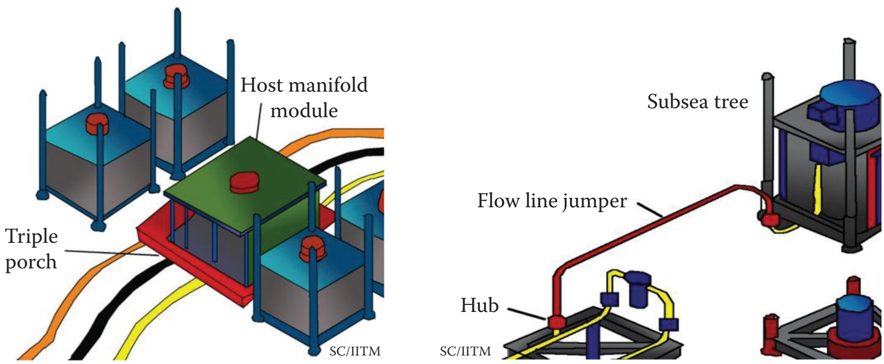


FIGURE 1.63 Template on subsea equipment.

spools typically ranging up to 0.5 m in diameter and 45 m in length. They are used to connect various subsea components. They are beneficial when connected to satellite wells through connections of small diameter production lines (3–6 inch), well testing lines (3–6 inch), hydraulic fluid lines (1 inch), and chemical service lines (1 inch) to the manifold. The offset distance between the components (trees, flow lines, manifolds, etc.) governs the jumper length and characteristics. Flexible jumper systems provide versatility, unlike rigid jumper systems, which limit the space and handling capability (Figure 1.64).

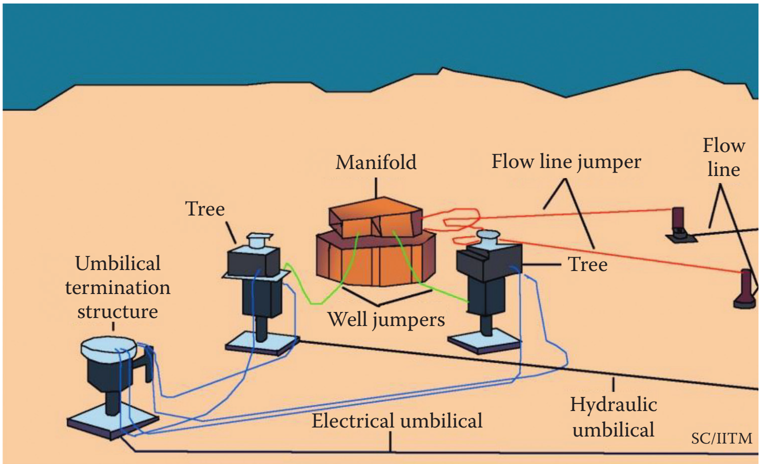


FIGURE 1.64 Jumpers.

1.22 COASTAL STRUCTURES

Various functions of coastal structures are as follows: (1) to protect the shore from wave attacks, (2) to prevent erosion and other similar damage caused to the shore from the wave action, (3) to retain sand for longshore transport, (4) to reduce inlet filling, and (5) to hold down and protect mooring vessels in position. Further extended functions are associated with their usefulness in coastal defense schemes. They are also very helpful in preventing flooding of the hinterland, sheltering the harbor basins, providing or stabilizing navigation channels at the inlets, and also protecting water intake and outfall systems (Devon and Jablokow, 2010; Mogridge and Jamieson, 1975; Requejo et al., 2002; Sadehi, 1989, 2001, 2007; Yip et al., 2000). There are different types of coastal structures depending on their function: sea dikes, seawalls, revetments, bulkheads, and groins.

1.23 SEA DIKES

A sea dike is essentially constructed to prevent or control flooding of low-lying coastal areas by sea. The principal function of a sea dike is to separate the shore line from the hinterland. This is achieved by constructing high impermeable structures. Figure 1.65 shows schematic views of sea dikes. The essential purpose of the sea dike constructed along the Vietnam coast is to protect the hinterland from erosion.

Figure 1.66 presents some sea dikes constructed near Westkapelle in the Netherlands. It also shows a very long coastline protected by the construction of sea dikes. The geometric form is trapezoidal in shape to maintain the desired slope that can limit erosion. Very long in length and high in cross section, they have a massive form of structural geometry. Sea dikes are built as a mound of fine materials such as sand and clay with a gentle seaward slope in order to reduce the wave run-up, and the erodible effect of the wave surface of the dike is armored with grass, asphalt, stones, or concrete slabs (Madsen, 1974, 1983; Madsen et al., 2006; Richey and

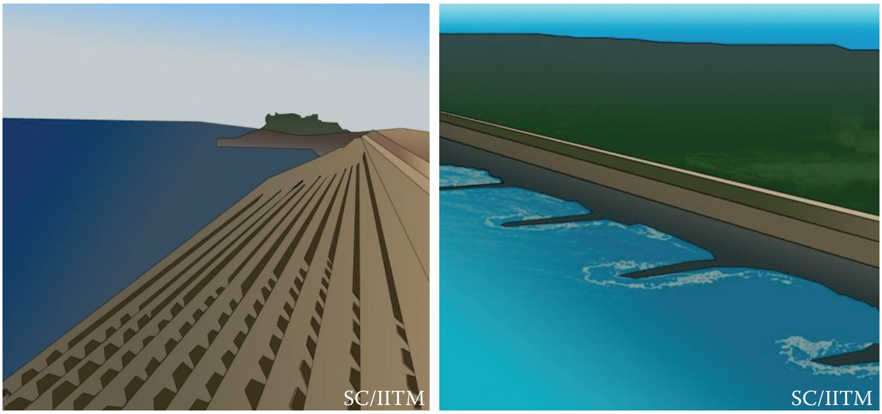


FIGURE 1.65 Sea dikes: (a) constructed along the Vietnam coast; (b) sea defense system in the Netherlands.

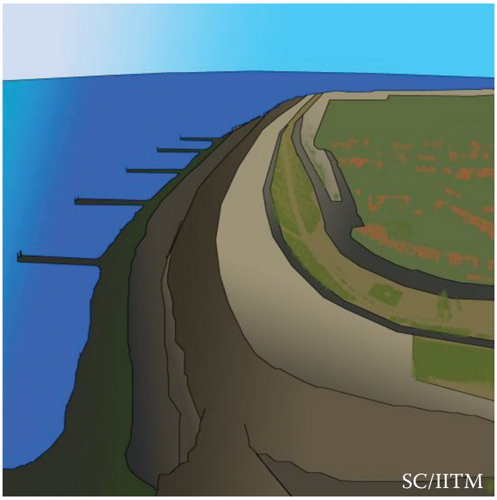


FIGURE 1.66 Sea dikes in the Netherlands.

Sollitt, 1969; Sahoo et al., 2000). Figure 1.67 shows the common geometric forms of sea dikes. Figure 1.68 shows a asphalt armored sea dike, constructed in the North Sea coast of Denmark.

They are low-permeability (watertight) structures built for protecting low-lying areas against flooding (Williams et al., 2000a,b; Williams and Li, 2000). Fine materials such as sand, silty sand, and clay are used for the construction. Seaside slope is usually very gentle in order to reduce wave run-up and wave impact. Steepness of the rear slope is based on the orientation of planes of slip failure and erosion by piping. Steeper slopes require stronger armoring. Sea dikes act as flood protection systems. They protect the low-lying land areas from the wave action. They are

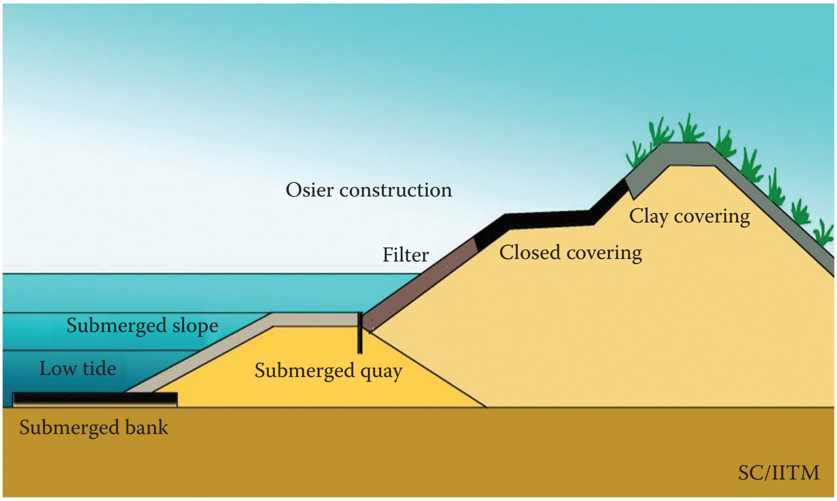


FIGURE 1.67 Geometric form of sea dikes.

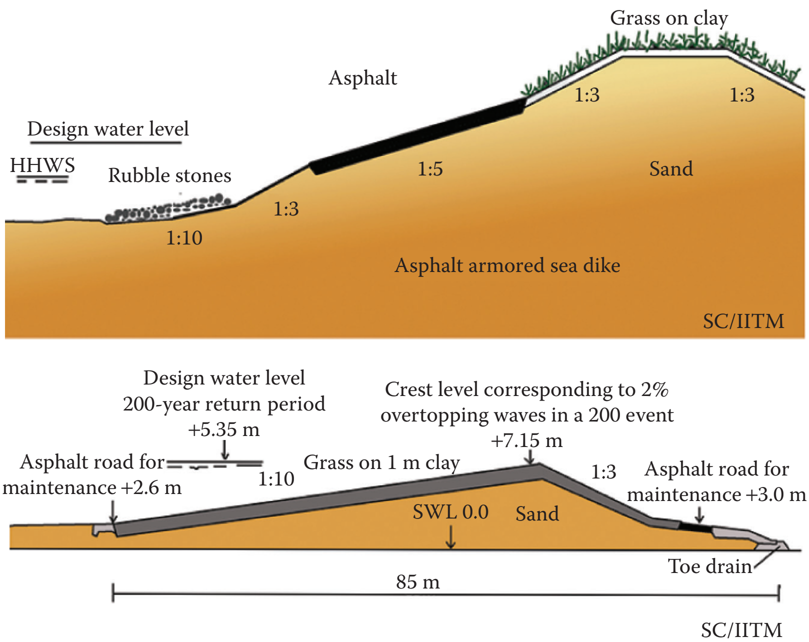


FIGURE 1.68 Asphalt armored sea dike, the North Sea coast of Denmark.

constructed along the coastline to act passively by preventing wave overtopping over the dike's crest. The choice of the location of sea dike depends on the sea statistics at the specific location. Sometimes, naturally formed dunes can also act as sea dikes. Figure 1.69 shows a schematic view of sand dunes. They are formed by plantation or vegetation of grass on the green segments.

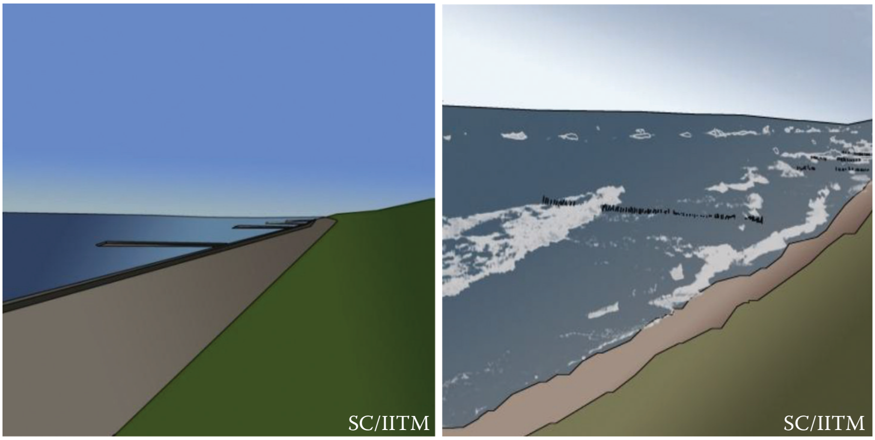


FIGURE 1.69 Natural dunes.

The initial cost of construction of sea dikes is high because they are constructed for a longer length along the shoreline. The material used for construction and the geometric form of sea dikes are designed to be massive so that they resist the lateral loads by their self-weight, without any additional anchorage. The height of sea dikes should suffice the requirement of the sea state at that coastline. Unfortunately, sea dikes also have negative environmental impacts. They actually decrease the landscape value along the coastline as the landscape beauty of the coastline is significantly affected by the construction of sea dikes. The slope of sea dikes needs to be monitored on a continuous basis, failing which they will be under severe erosion. Slope stability treatment of sea dikes are relatively expensive and invoke a periodic investment. Sea dikes are not a very popular mode of shoreline protection. One of the greatest demerits of the construction of sea dikes along the coastline is that it prevents access to the coast, unless a passage is made across the sea dike. This means that the coast and the land behind the sea dike are separated by the structure, which is generally undesirable.

1.24 SEAWALLS

The principal objective of the construction of a seawall is to protect the coastal land and structures along the coast from flooding and overtopping. Seawalls are constructed where sea has a direct impact on the coast. They are reinforced for a certain stretch of length to protect them from the wave action. Figure 1.70 shows a seawall at Malecón, Havana. They are typical onshore structures with the principal function of preventing flooding of land structures behind due to storm surges and waves (Dean and Dalrymple, 2000; Zhong and Wang, 2006). They are constructed parallel to the shoreline and strengthen part of the coastal profile. They are used to protect promenades, roads, and houses placed seaward of the crest edge of the natural beach profile. Figure 1.71 shows a schematic view of a seawall constructed to protect the habitat along the shoreline.

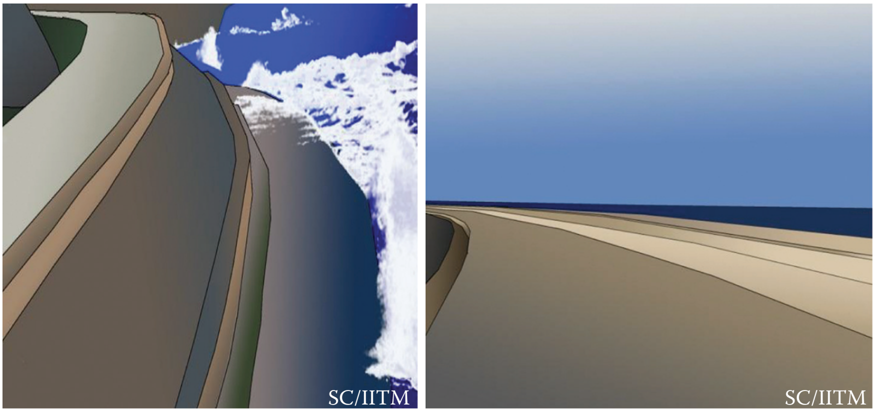


FIGURE 1.70 Seawall at Malecón, Havana.

FIGURE 1.71 Seawalls protect houses along the coastal line.

Construction of seawalls leads to a wide variety of environmental problems (Kriebel, 1992; Linton and McIver, 2000; Li et al., 2006; Losada et al., 1995; Macaskill, 1979; Silva et al., 2003; Sollitt and Cross, 1972; Song and Tao, 2007; Teng et al., 2004; Terret et al., 1968). For example, it disrupts the sediment movements and affects the transport patterns of sediments. The cost of the construction of seawalls is very high, which is one of the undesirable characteristics of a seawall (Urashima et al., 1986). Seawalls reflect the instant wave energy back to the sea and therefore reduce the impact of energy on the coastal sides. Reflection induced by the construction of seawalls causes severe environmental problems as it lowers the sand level of the fronting beach. It therefore accelerates the erosion of the adjacent unprotected coastal areas. This is one of the major impacts caused by the construction of seawalls on coastal sides.

Highly vulnerable to toe scour, which causes instability to the wall and to protect them from such problems, they are constructed together with groins. Wave slamming, surface run-up, and overtopping are critical actions responsible for structural failure of the seawall. They are classified as sloping-front and vertical-front structures. Sloping-front structures are constructed as flexible rubble-mound structures. They have flexibility to overcome toe and crest erosion. The stability of slope depends on the intact toe support; the loss of toe support will result in significant damage of the armor layer, which also results in partial/complete failure of the armored slope. Figure 1.72 shows a schematic view of sloping-front, rubble-mound seawall. Figure 1.73 shows the schemes that are deployed for strengthening of seawalls (Twu and Lin, 1991; Wang and Ren, 1993, 1994). There are different types of seawalls. Vertical seawalls are constructed to reflect the wave energy of standing waves under storm conditions. These are very early type of seawalls constructed along the coastal side. They deflect the wave energy away from the coast very effectively, but undergo expensive damage within a short period of their service life.

Figure 1.74 shows a vertical front seawall. Moreover, vertical seawalls can be easily overtopped by high wave energy, making them less efficient. Figures 1.75 and 1.76 show schematic views of vertical seawalls constructed at Saint-Jean-de-Luz, France, and Stanley Park, Vancouver, respectively. Seawalls are also constructed in other geometric forms (see, e.g., stepped seawall shown in Figure 1.77 constructed at Wheelers Bay, England. Such forms have a better lateral stability but occupy more ground coverage; in addition, they are esthetically better.

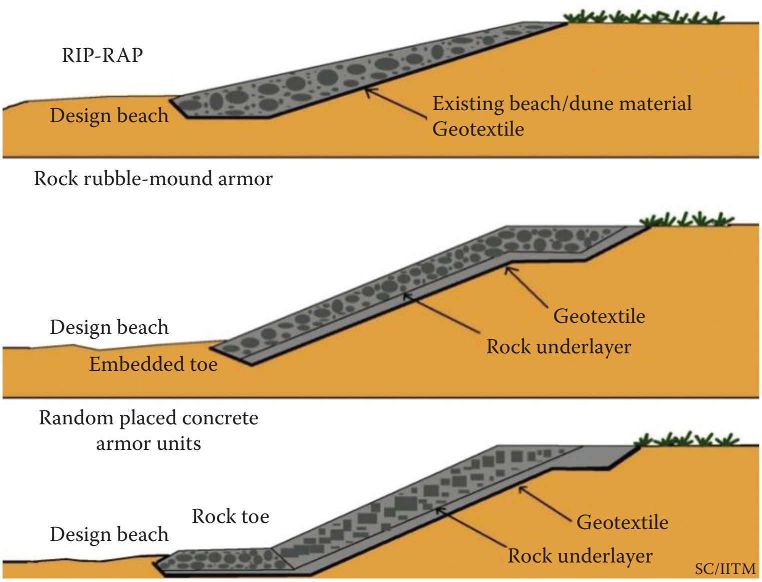


FIGURE 1.72 Sloping front, rubble-mound seawall.

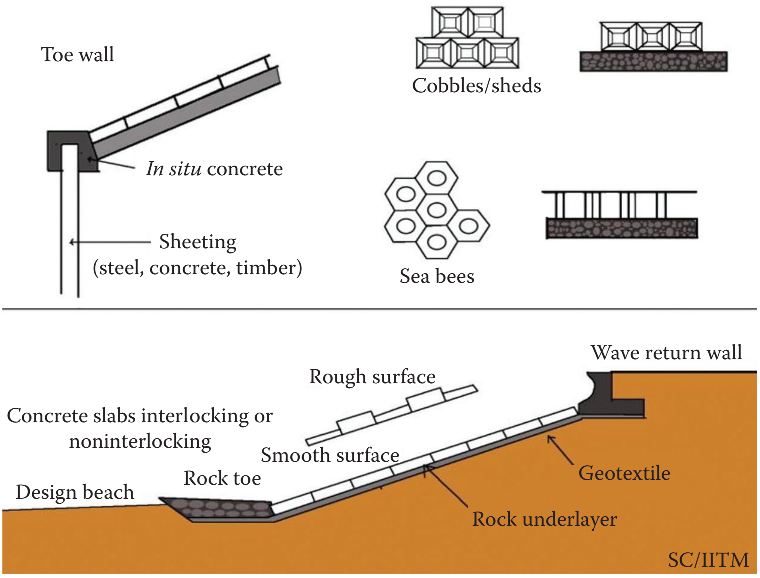


FIGURE 1.73 Strengthening of toe walls/return walls of seawalls.

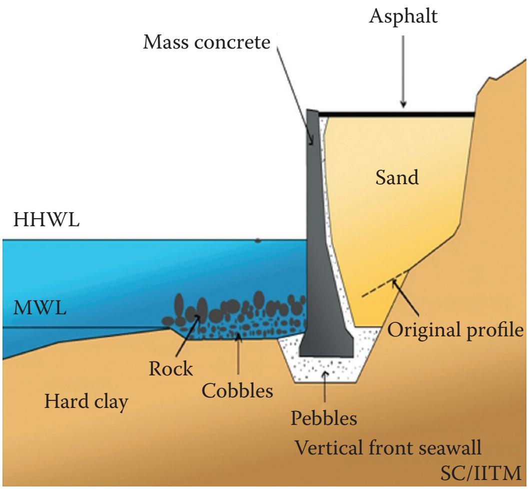


FIGURE 1.74 Vertical front seawalls.

Curved seawalls are designed to enable waves to break or to dissipate energy. The curved profile helps in preventing the overtopping of waves; the concave geometry induces dissipative element. It redirects most of the incident waves and results in low reflected waves. Deflected waves can cause serious scouring problems at the base of the wall, which is seen as one of the serious demerits of this structural form

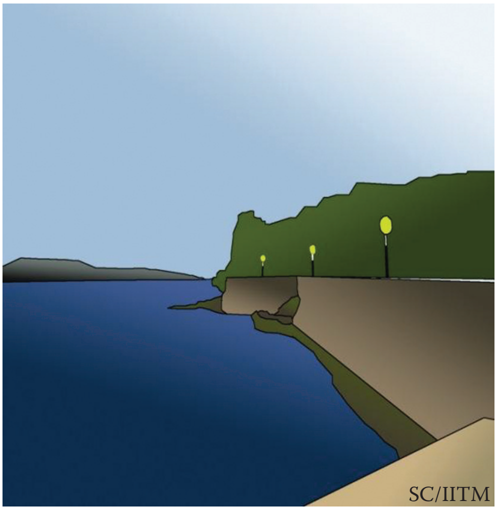


FIGURE 1.75 Vertical seawall at Saint Jean de Luz, Sainte-Barbe, France.

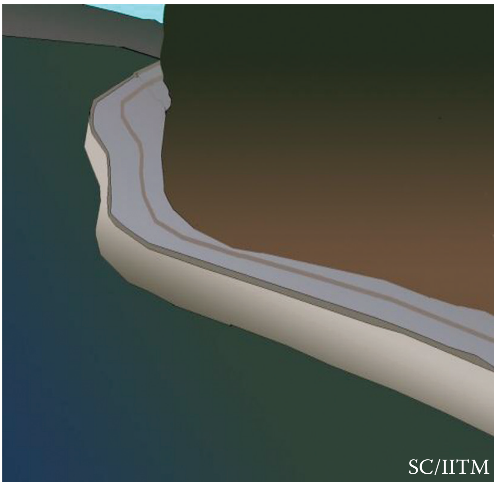


FIGURE 1.76 Vertical seawall at Stanley Park, Vancouver, Canada.

of seawalls (Tanimoto et al., 1984, 1992). Figure 1.78 shows a schematic view of a curved seawall constructed in Wheelers Bay. Dependence of seawalls proved to be unsuccessful. For example, during the 2011 tsunami in Kamaishi, one of the largest seawalls (2 km long) could not protect the city. Nuclear power plants at Daiichi and Daini are washed off. It is also shown in literature that the construction of seawall results in sea-level raise (International Panel on Climate Change [IPCC], 1997). It increases the mean water level and the height of waves.

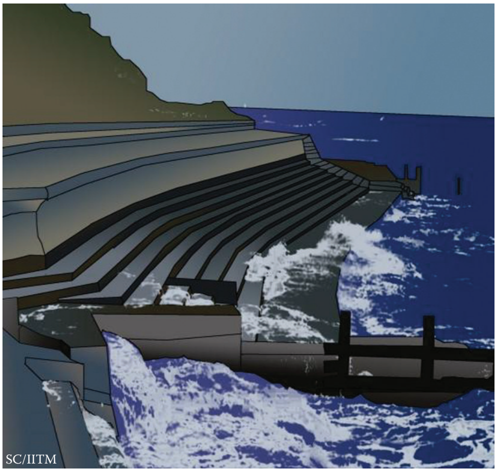


FIGURE 1.77 Stepped seawall at Wheelers Bay.

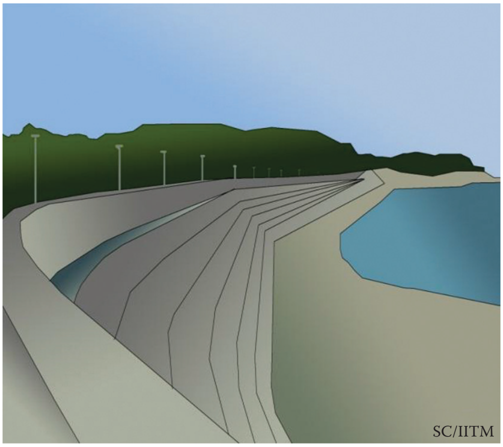


FIGURE 1.78 Curved seawall at Wheelers Bay.

1.25 REVETMENTS

Revetments are onshore structures, constructed with the main objective of protecting the shoreline from erosion. The slope profile, when reinforced with stone cladding, absorbs the energy of incoming water. The principal function of onshore structures is to protect the shoreline from erosion. Constructed with cladding of stone, concrete, or asphalt to armor the sloped natural shoreline profiles, it typically consists of stone cladding or asphalt lining to protect the slope of natural coast line; it reinforces the beach line to some length against erosion. Figure 1.79 shows the cross-sectional details of revetment, whereas Figure 1.80 shows schematic

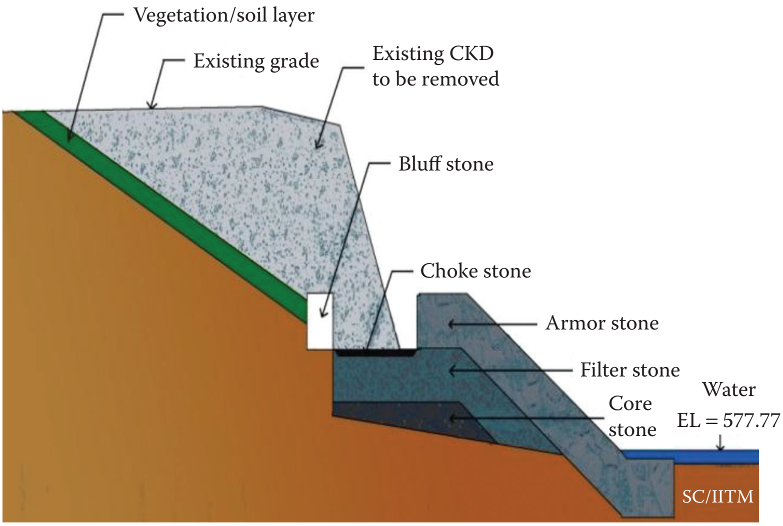


FIGURE 1.79 Cross-sectional details of revetment.

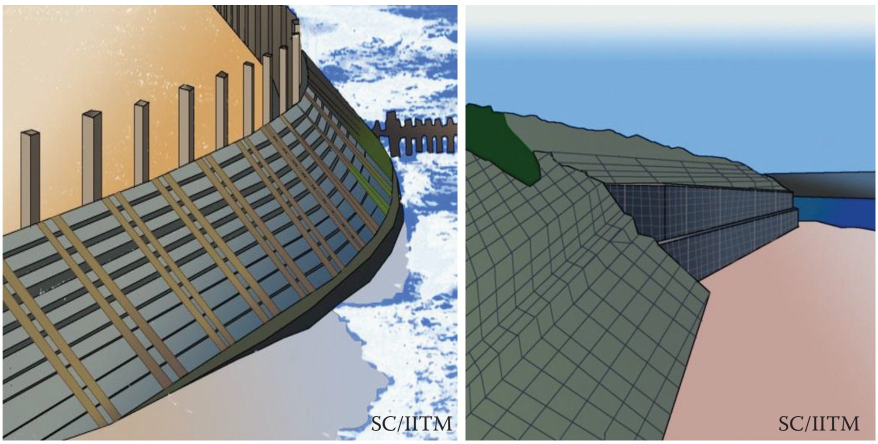


FIGURE 1.80 Revetments of different geometric forms.

views of revetments constructed in different geometric forms. Figure 1.81 shows a schematic view of rock revetment constructed in Duluth, Minnesota. Revetments are used as a low-cost solution for coastal erosion. They are a type of retaining walls constructed to strengthen the slope of the coastline to protect from erosion. They are a strong and cheap solution for coastal protection. They are effective in absorbing the wave energy but have a relative shorter service life (only about 30–40 years). Considering the service life, the initial investment on revetment construction is too high.

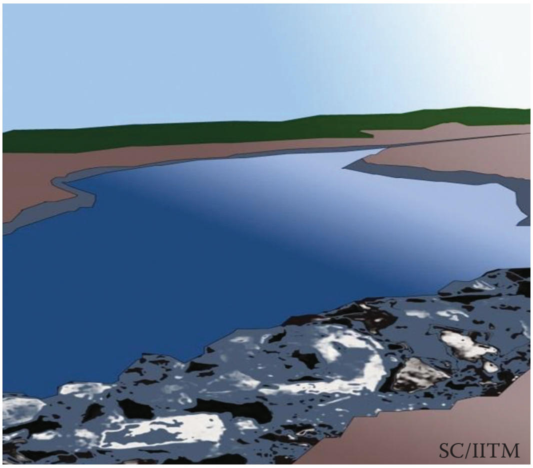


FIGURE 1.81 Rocky revetment, Duluth, Minnesota.

1.26 BULKHEADS

Bulkheads are typically smaller coastal protection structures that are designed to retain shore material under less severe wave conditions in comparison with seawalls. Primarily intended to retain the slope or prevent sliding of land behind, they are fundamentally soil-retaining structures. In most of the cases, they are vertical walls anchored with tie rods. Figure 1.82 shows the common structural form of a bulkhead.

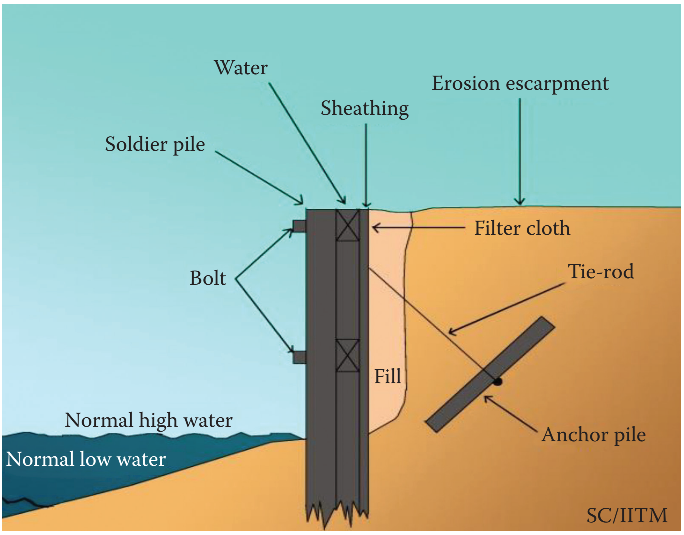


FIGURE 1.82 Structural forms of bulkheads.

They are commonly deployed in mooring facilities in harbors and marinas to minimize the exposure to wave action. They are effective in reinforcing some portion of the beach profile but significantly influence the change in the beach profile. Increased wave reflection caused by the bulkheads results in increased resuspension of sand in water in front of bulkheads. This results in serious coastal drift and also affects coastal habitat seriously. Figure 1.83 shows a schematic view of a steel bulkhead constructed in Bolinas Lagoon, California. Steel is coated with epoxy and tar for enhanced protection. Figure 1.84 shows another view of the same bulkhead. Figure 1.85 shows bulkheads constructed to protect coastal habitat along the shoreline.

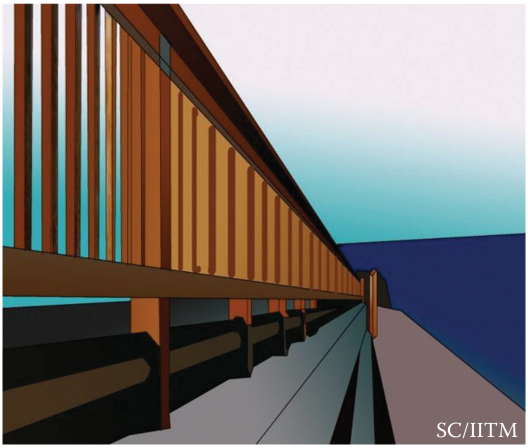


FIGURE 1.83 Steel bulkhead at Bolinas Lagoon, Stinson Beach, California.

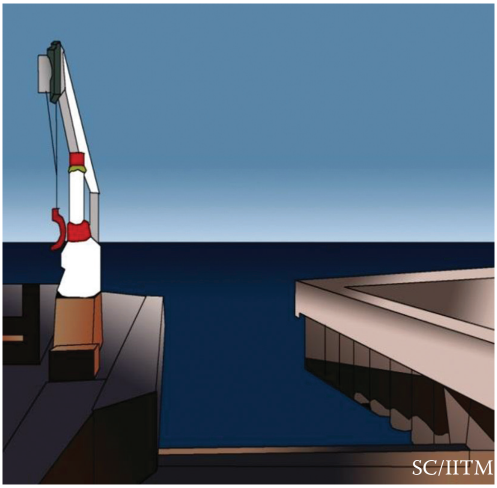


FIGURE 1.84 Bulkhead at Bolinas Lagoon (coated with epoxy and tar).

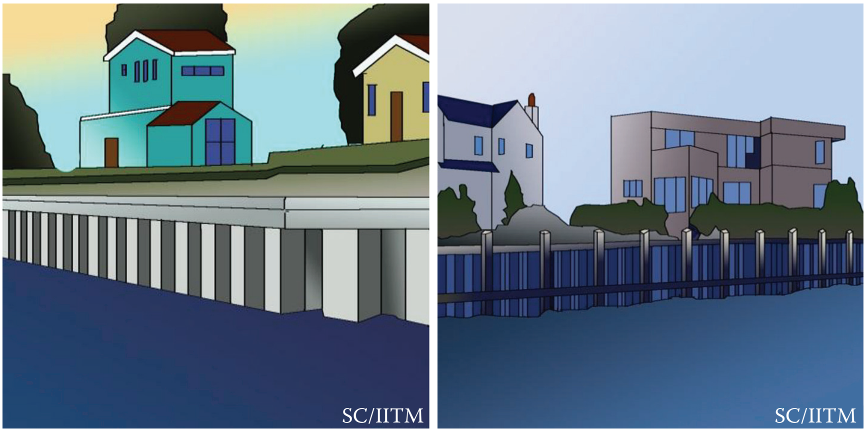


FIGURE 1.85 Bulkheads protect the coastal habitat.

1.27 GROINS

Groins are structures built to stabilize a stretch of naturally or artificially nourished beach against erosion. Protection is offered against longshore loss of beach material. Groins are narrow, long structures constructed perpendicular to the shoreline. Their primary advantages are accretion of beach material on the updrift side and erosion on the downdrift side. Other than preventing beach erosion, they reduce longshore sediment transport. They also significantly reduce the wave height on the leeward side. They are generally constructed in series. A series of groins constructed on the shoreline create a sawtooth-shaped shoreline within the groin field. Figure 1.86 shows a schematic view of a series of groins and also indicates different geometric forms of groins. Groins, if constructed, create differentials in the beach level on their either side. They also create a complex wave pattern and current in the vicinity of their construction. They create a major impact on the river morphology and causes autonomous degradation of river. Although their primary function is to prevent

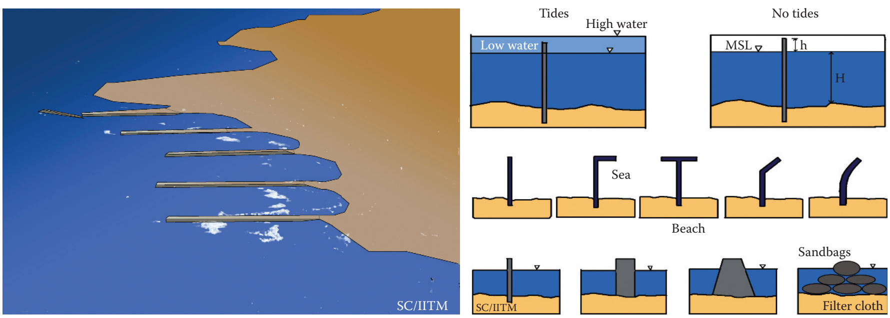


FIGURE 1.86 Groins built with different plan view shapes (straight, T-head, L-head, straight groins with pier head, wing, and tail groins).

beach erosion, they also reduce longshore transport sediment. The building up of material in groin bays provides protection against erosion to the coastline. They are used to hold artificially nourished beach material and prevent sedimentation or accretion in a downdrift area by acting as a barrier to longshore transport. Groins of shorter lengths are recommended for areas with severe erosion potential. They can also be nonperpendicular to the shoreline, can be curved, and have a shore-parallel T-head at their seaward end. In most cases, groins are built with sheet-pile or rubble-mound constructions.

Rubble groins have reduced the risk of scouring and the formation of strong rip currents. Groins must protrude some distance into the zone of littoral transport. The projection dimension is determined by a surf zone width classified as long or short, depending on how far across the surf zone they extend, and also classified as high or low, depending on the possibility of sediment transport across the crest. Terminal groins extend far enough seaward to block all littoral transport. Permeable groins allow sediment to be transported through the structure. Low and permeable groins have the benefit of reduced wave reflection and less rip current formation compared with high and impermeable groins.

Groins are useful in deflecting strong tidal currents away from the shoreline. The actual change in the shoreline will be governed by (1) the orientation of the groins, (2) the length and height, (3) permeability, and (4) spacing. Groins are commonly constructed as sheet-pile or rubble-mound constructions and are usually connected to seawalls. This will help to avoid outflanking by back scour and also enables stable back-beach features. Groins are classified as long or short depending on how far they are extended across the surf zone.

Figure 1.87 shows groins constructed as a post-tsunami measure along the southern coast of India. Groins that transverse the entire surf zone are termed as long and those that extend only to a part of it are termed as short. They are also classified as high or low depending on the possibility of sediment transport across the crest.

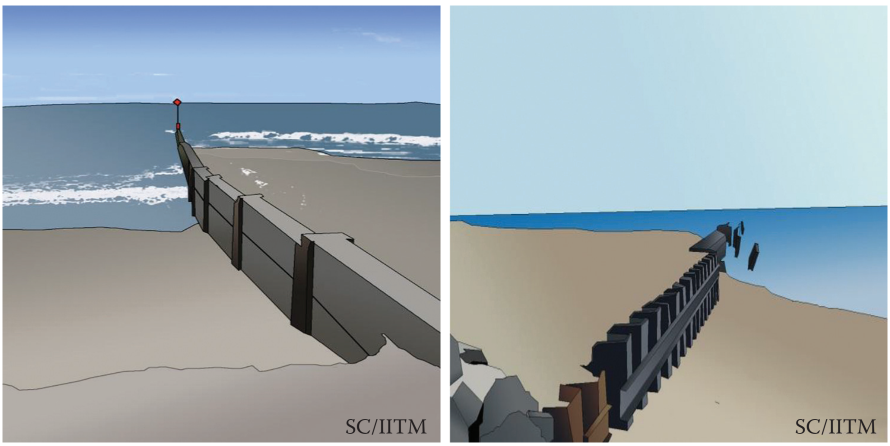


FIGURE 1.87 Groins along the southern coast of India.

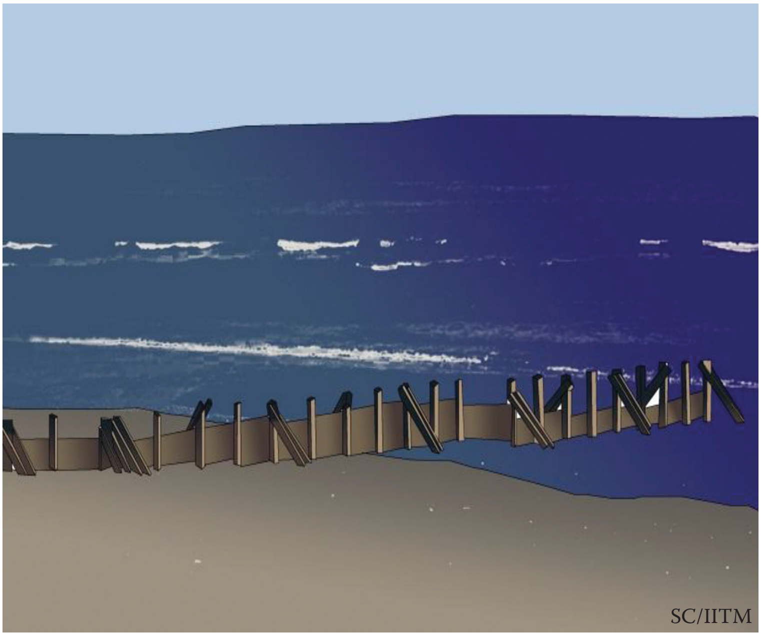


FIGURE 1.88 Groins on the east coast of England.

Permeable groins allow sediment transport through the structure, enabling a well-distributed retaining effect along the coast. Figure 1.88 shows a schematic view of groins on the east coast of England. Figure 1.89 shows groins constructed along the coast of Chennai, India, as a post-tsunami measure. Figure 1.90 shows the activities of shore transportation that occur in the presence of groins.

1.28 BREAKWATERS

Breakwaters are built to reduce the wave action in the leeward area of a structure. Wave action is reduced through a combination of reflection and dissipation of the incoming wave energy (Aburatani et al., 1996; Jarlan, 1961; Liu et al., 2012; Kakuno, 1983; Kenny et al., 1976). Breakwaters are also constructed to protect the coastline against tsunami waves. They create calm waters, which are useful for harbor areas (Chen et al., 2002; Tanimoto and Takahashi, 1994; Tanimoto and Yoshimoto, 1982). Calm waters are used for safe mooring and loading operations, handling of ships, and so on. One of the main objectives of the construction of breakwaters is to provide shelter for harbor basins, harbor entrances, and water intake systems against waves and current. Breakwaters dissipate the wave energy and reflect it back into the sea. They help to improve the maneuvering conditions at harbor entrance by directing the current appropriately. Areas with differing levels of wave disturbances are initiated by the presence of breakwater, which helps in harbor functioning. They are relatively short and non-shore-connected near-shore breakwaters with the principal

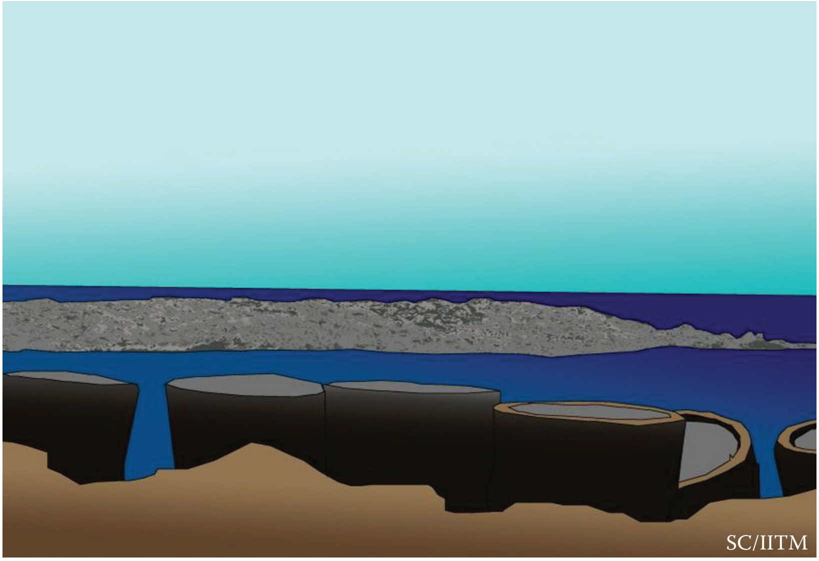


FIGURE 1.89 Groins near Ennore Expressway, Chennai, India.

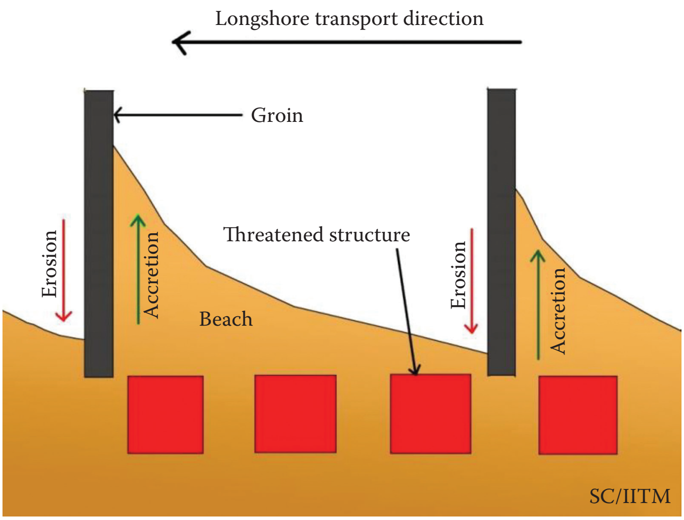


FIGURE 1.90 Longshore transport due to groins.

function of reducing beach erosion. They are built parallel to the shore, just seaward of the shoreline in shallow water depths. Multiple detached breakwaters spaced along the shoreline can provide protection to substantial shoreline frontages. Gaps between the breakwaters are in most cases on the same order of magnitude as the length of one individual structure. Breakwaters reflect and dissipate some of the incoming wave energy reducing the wave heights in the lee of the structure and reducing shore erosion.

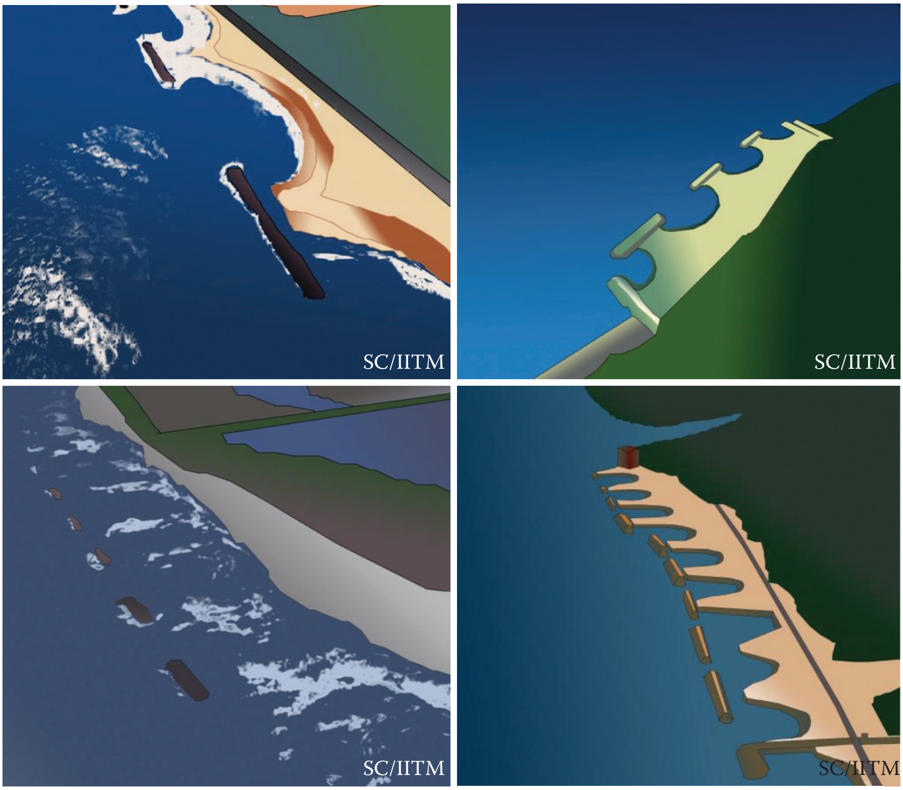


FIGURE 1.91 Detached breakwaters.

Detached breakwaters are normally built as rubble-mound structures with low crest that allow a significant overtopping during storms at high water.

Breakwaters can be of two types of geometric configurations: detached and shore connected. The cost of breakwaters increases with the severity of wave climate and water depth. The layout of breakwaters depends on the direction of storm waves, the net direction of current, littoral drift, and maneuverability of vessels at the harbor. Figure 1.91 shows schematic views of detached breakwaters.

Detached breakwaters are built parallel to the coastline, located inside or very close to the surf zone. They are built with rubble-mound structures. They are similar to groins in their structural form, but have a lower crest, which allows overtopping during the storm. Submerged breakwaters are also preferred as they do not spoil the view of the coastline, but they cause very serious nonvisible hazard to swimmers and boats (Suh and Park, 1995; Sulisz, 1985). Figure 1.92 shows a longshore transport in the presence of detached breakwaters. Detached breakwaters on the coastline protection induce a few noticeable hydrodynamic impacts. Breakwaters shelter the adjacent areas partly; it is very simple to understand that longer the breakwater, better the shelter. Submerged and floating breakwaters provide less shelter.

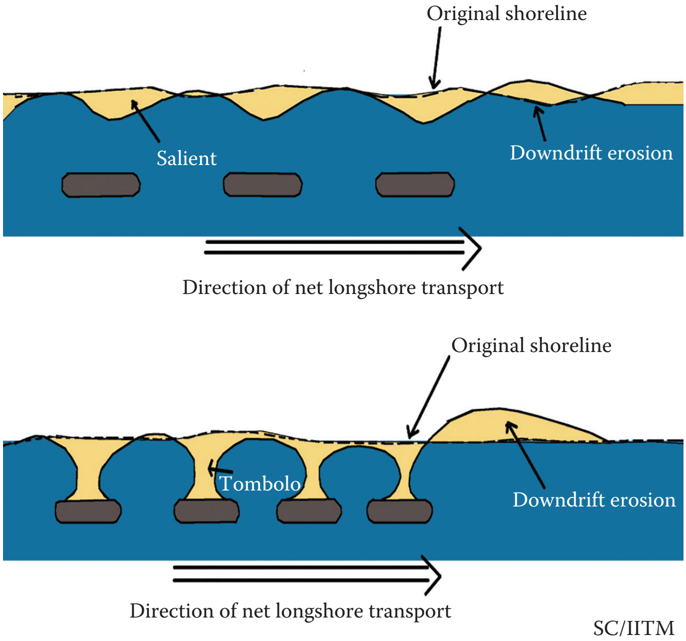


FIGURE 1.92 Longshore transport in the presence of detached breakwaters.

Overtopping waves in the case of submerged breakwaters induces an additional supply of water to the areas behind the breakwater.

Undesirable currents are generated by a wave setup in the sheltered areas, which causes eddies. Longshore currents are partly blocked resulting in their diversions. Diversion of longshore currents causes local erosion effects in the areas that are very close to the head of the breakwaters (Fugazza and Natale, 1992). Trapping of sand, which is an eventual part of the areas where breakwater is constructed, will result in leeward side erosion. Breakwaters actually trap sand, which will cause serious coastal impact. Swimmers are sometimes tempted to use the sheltered areas, but the circulation of the current that is present in these areas because of the intervention of construction of breakwaters can be dangerous for swimmers.

Rubble-mound breakwaters are the most common type constructed worldwide; the most commonly applied type of breakwater of a simple shape is a mound of stones. Geometric parameters useful for the design of rubble-mound breakwaters are shown in Figure 1.93. These consist of a homogeneous structure of stones. They are large enough to resist displacements due to wave forces. If designed to be permeable, it may result in penetration of waves and sediments present in the area. Conventional rubble-mound structures consist of a core of finer material covered by big blocks forming the so-called armor layer. To prevent finer material from being washed out through the armor layer, filter layers are usually provided. The lower part of the armor layer is supported by a toe beam except in cases of shallow water structures. Concrete armor units are used as armor blocks in areas with rough wave climates or at sites where a sufficient amount of large quarry stones is not available.

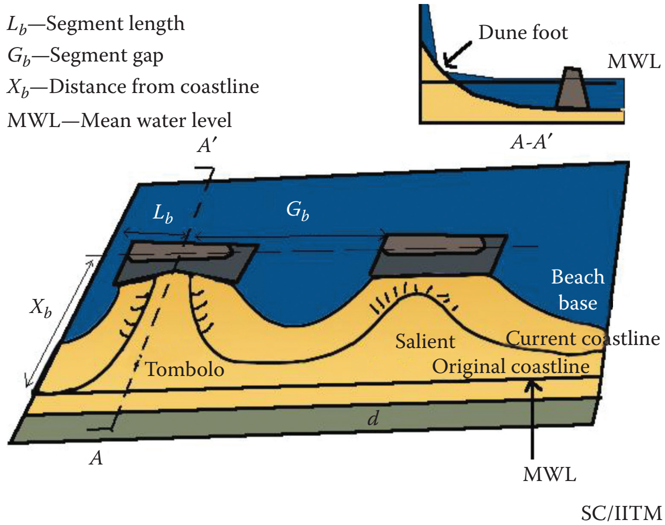


FIGURE 1.93 Rubble-mound breakwaters: geometric parameters.

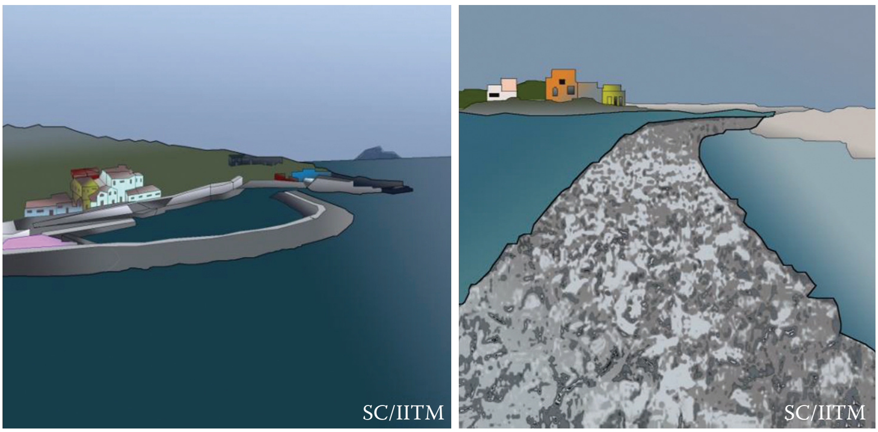


FIGURE 1.94 Rubble-mound breakwaters.

Figure 1.94 shows schematic views of rubble-mound breakwaters constructed for coastal protection.

Reef breakwaters are built parallel to the coast as long or short submerged structures. The main objective is to reduce the wave action by forcing the wave breaking over the reef rubble-mounded, narrow-crested geometry. Figure 1.95 shows the cross-sectional detail of reef breakwaters. They regulate the wave action by refraction and diffraction. If submerged, they pose serious hazards to swimmers. The design principle of reef breakwaters is similar to that of the rubble-mound structures with submerged crests. The main objectives are to prevent beach erosion and to reduce the

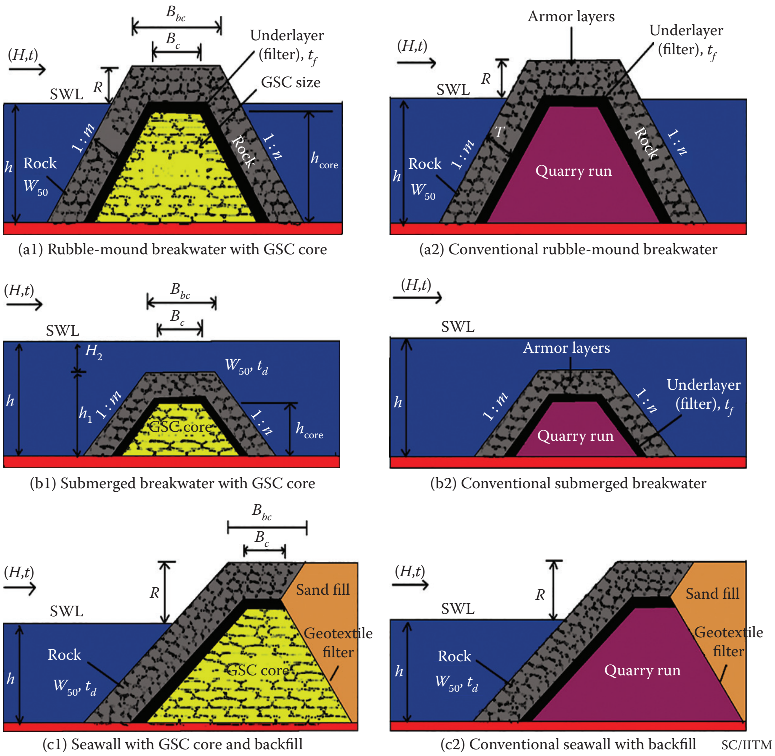


FIGURE 1.95 Cross section of reef breakwater.

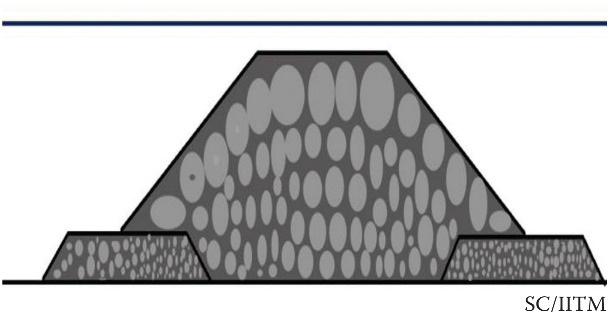


FIGURE 1.96 Reef breakwater.

wave heights at the shore (Issacson et al., 1998, 2000). They are constructed parallel to the coast, in the form of long or short submerged structures. They are designed to be stable or may be allowed to reshape under the wave action. Besides triggering wave breaking and subsequent energy dissipation, reef breakwaters can be used to regulate wave action by refraction and diffraction. Figure 1.96 shows a schematic view of reef breakwater.

Floating breakwaters are constructed at sites that have mild wave climate with short-period waves. They are effective for small-craft harbors or marinas. They are best suitable for sites with poor soil conditions that cannot afford bottom-supported breakwater. For increased water depth more than 6 m, bottom-supported breakwaters become expensive. Floating breakwaters are more suitable for sites where frequent ice formation occurs; they can be towed alongside and ice can be removed. Although their layout is simpler and faster, improvements are required on visual impact (Issacson, 1982). Figure 1.97 shows a schematic view of floating breakwater in Fezzano, Italy.

There are different types of floating breakwaters: box type, pontoon type, mat type, and tethered float type. These are the most common types of floating breakwaters that are normally constructed in different sites worldwide. The box type is the most frequently used type of floating breakwater. It generally consists of RCC modules. Pontoon-type breakwaters, as shown in Figure 1.98, have different types of geometry. The alternate type of breakwater has two cylinders that are connected by a metal frame with a wooden sheet between them; which are assembled to form a floating breakwater. Twin lock-type breakwaters also exist where there are two pairs of locks connected as a deck. The main advantage of this type of breakwater is that the deck is an open wooden frame, which can be used for some inspection purposes as well (Darwiche et al., 1994).

Floating-type breakwaters have a specific advantage when one is deploying the pontoon type (Williams and Li, 1998). They are effective because the overall width of pontoon-type breakwater is less than that of the wave length; practically, it is half of the wave length. This results in the significant attenuation of wave height. It means that the wave height reaching the foresite of the breakwater is significantly reduced as the overall width of pontoon-type breakwater is less than that of the wave length. Mat-type breakwaters are essentially used as low-cost modules. They are much cheaper compared to the pontoon type or the box type.

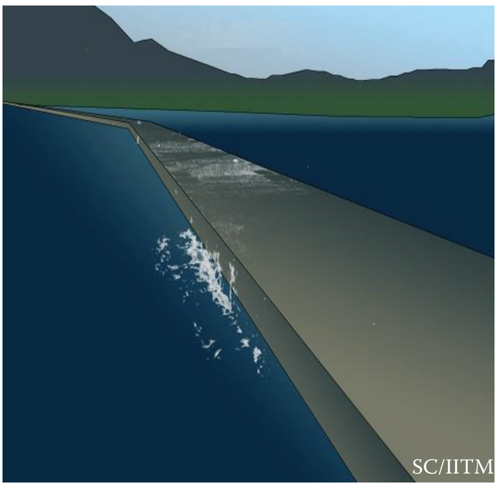


FIGURE 1.97 Floating breakwater, Fezzano, Italy.

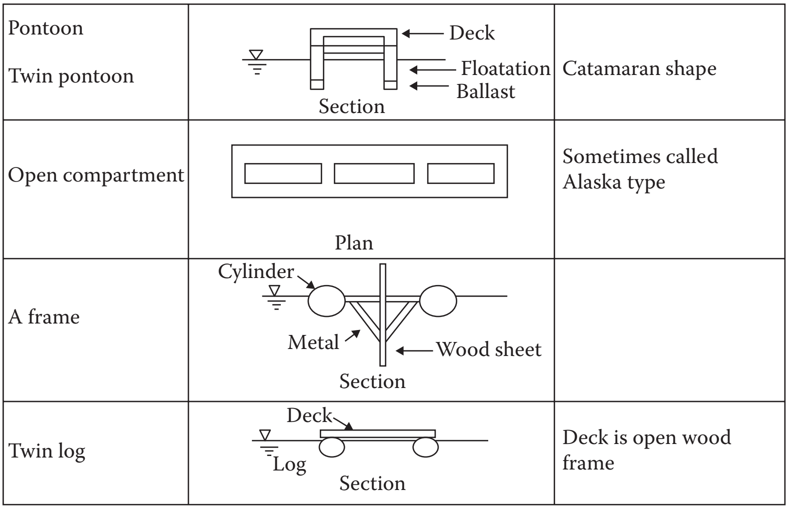


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FIGURE 1.98 Pontoon type.

Mat-type breakwaters can be easily transported from one location to another. These types of breakwaters can be constructed with unskilled labor; special construction equipment is not required. They reflect less and dissipate more wave energy, which is seen as one of the main advantages. Tethered breakwaters are anchored to the seafloor using tethers. They contain sphere-type floats, one or in series, which are anchored to the seabed using tethers. Series of them rather than a single one are generally installed. These types of breakwaters do not have a significant advantage in controlling wave action on the coastal site, and are not used very often in practice.

1.29 SUBMERGED SILL

A submerged sill is a special type of reef breakwater built near the shore. It is principally used to prevent beach erosion and to retard the offshore movement of sediments. It introduces a structural barrier at a point on the beach profile that interrupts the onshore sand movement. A submerged sill introduces discontinuity in the beach profile. They are built as rock-armored, rubble-mound structures. Few prefabricated units are also commercially available to be used as submerged sills. Submerged sill is a special version of reef breakwater, which is built near the shore and used to retard offshore sand movements by introducing a structural barrier at one point on the beach profile. The sill may also interrupt the onshore sand movement. A sill introduces a discontinuity in the beach profile so that the beach behind it becomes a perched beach as it is at a higher elevation and thus wider than adjacent beaches. Submerged sills are also used to retain beach material artificially placed on the beach profile behind the sill. They are usually built as rock-armored, rubble-mound structures, or commercially available prefabricated units. Submerged sills are invisible hazards to swimmers and boats.

1.30 BEACH DRAINS

The essential function of the beach drain is to prevent beach erosion. It helps in accumulating beach material on the drained portion of the beach. It is constructed at an elevation just beneath the lowest seasonal elevation of the beach profile in the swash zone. Backwash speed and groundwater outflow from the beach zone can be reduced by pumping water from the beach drains. This enables the beach material to settle on the foreshore slope. Beach drains are constructed similar to normal drains that consist of granular filters. Drain pipes are connected to shore-normal pipelines leading to a sump in the upper part of the beach.

1.31 JETTIES

They are used for stabilization of navigation channels at tidal inlets. They are shore-connected structures, generally built on both sides of the navigation channel and perpendicular to the shore, extending into the sea. Confining the stream or tidal flow, it is possible to reduce channel shoaling and decrease dredging requirements (Yu and Chwang, 1994). Extended offshore of the breaker zone, jetties improve the maneuvering of ships by providing shelter against storm waves similar to breakwaters. Figure 1.99 shows a schematic view of Carlsbad Jetty, California. The main function of jetties is to reduce channel sloshing and decrease dredging requirements. They also help in arresting the cross-currents and thus improve navigation on defined channels. They prevent longshore drift and slow down beach erosion. Figure 1.100 shows a schematic view of a jetty constructed to improve navigation. These are used for stabilization of navigation channels at river mouths and tidal inlets.

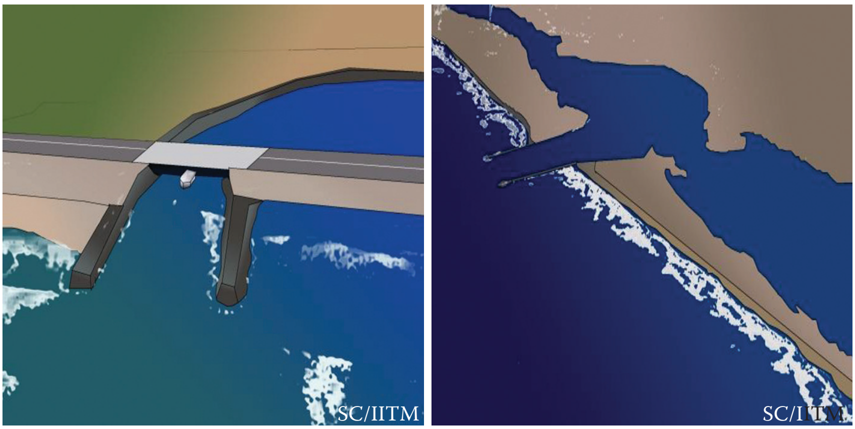


FIGURE 1.99 Carlsbard Jetty, California.

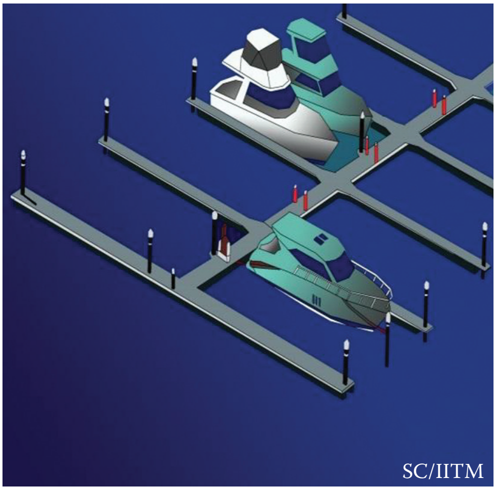


FIGURE 1.100 Jetty to improve navigation.

1.32 TRAINING WALLS

Training walls are constructed to direct the flow, and they also improve the mooring conditions in an estuary. They help in directing the littoral drift away from an area of potential deposition (Yu, 1995). Most commonly, they are constructed using sheet piles. Figure 1.101 shows a schematic view of training walls constructed in Tweed River entrance in Queensland, Australia.

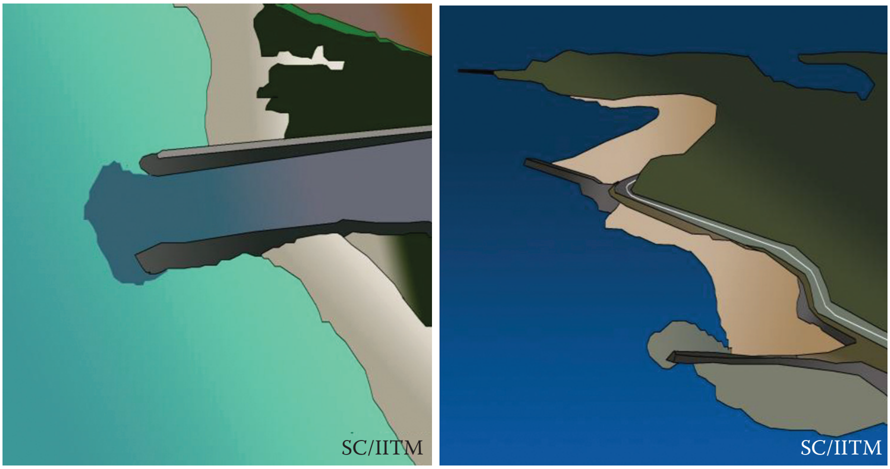


FIGURE 1.101 Training walls in Queensland, Australia.

1.33 STORM SURGE BARRIERS

Storm surge barriers protect estuaries against storm surge flooding and wave attack. They prevent saltwater intrusion during high water episodes. They contain series of movable gates that normally stay open and are generally closed when storm surge exceeds the permissible level. They are constructed in concrete and rest on pile foundations.

2 Environmental Loads on Ocean Structures

Behavior of ocean structures under environmental loads is highly complex, not because mathematical models are unable to predict but because of integration of geometric form with that of the response, as in the case of compliant structures, in particular. As it is realized that the design of offshore structures is essentially form dominated, it is imperative to quantify the loads that they encounter during their service life. A variety of environmental loads, being quantified by different theories and empirical relationships, make their understanding confusing. This chapter deals with the estimate of environmental loads on ocean structures along with the explanation of relevant theories used to quantify them.

2.1 INTRODUCTION

Different types of environmental loads act on offshore structures. Figure 2.1 shows a variety of environmental loads that act on ocean structures. They are classified as follows: (1) permanent loads or dead loads, (2) operating loads or live loads, (3) other environmental loads including earthquake loads, (4) construction and installation loads, and (5) accidental loads (Adams and Baltrop, 1991; Chandrasekaran, 2015a,b). Permanent and operating loads govern the design of buildings constructed on land. Environmental loads and other loads that arise during various stages of construction and installation dominate the design of offshore structures (Agarwal and Jain, 2002; Anagnostopoulos, 1982; Coppleand and Capanoglu, 1995; Røren and Furnes, 1976; Soding et al., 1990; Stansberg et al., 2004). For example, earthquakes, which are regarded as accidental loads in general, are classified as environmental loads in the design of offshore structures (Amr et al., 2013; API RP 2T, 1997; API RP WSD, 2005). Environmental loads include wind, waves, current, tides, earthquakes, temperature, ice, seabed movement, and marine growth. Their characteristic parameters, defining design load values, are determined in special studies based on available data (Anagnostopoulos, 1982). According to the U.S. and Norwegian regulations, the mean recurrence interval for the corresponding design event must be 100 years, whereas according to the British codes, it will be 50 years or greater (Naess and Moan, 2013).

2.2 WIND FORCES

Wind forces act on the superstructures of offshore structures. The superstructures of coastal structures are not wind-sensitive as wind force does not influence up to a datum reference height of about 10 m from mean sea level (MSL). As no superstructures on coastal jetties fall under this category, the wind effect on them is negligible.

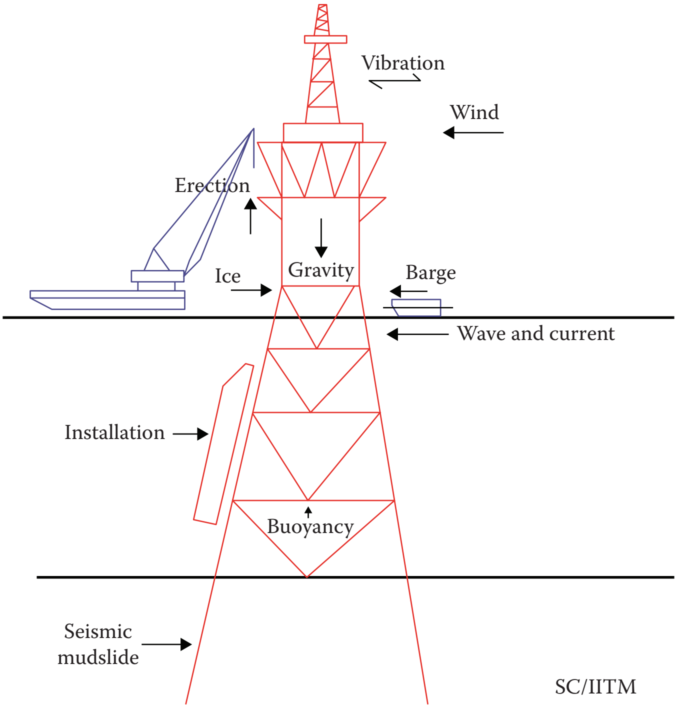


FIGURE 2.1 Environmental loads on ocean structures.

However, in case of offshore structures, drilling masts, flare stacks, living quarters, and upper decks are significantly taller with respect to their lateral dimensions in plan; hence, they become wind sensitive. Codes such as API RP 2A (2005) and API RP 2T (1997) distinguish the differences between global and local wind load effects. For global wind effects, the values of mean 1-h average wind speed are to be combined with the extreme waves and current. However, for local wind effects, the values of extreme wind speeds are to be used without regard to waves. Wind loads are generally taken as static, unless otherwise the ratio of the height to the least horizontal dimension of the platform is greater than 5 (Kareem, 1985). In such cases, flow-induced cyclic wind loads due to vortex shedding also become important. The commonly used engineering approach to estimate wind forces on offshore structures assumes that the force will be proportional to the exposed area and the square of the wind velocity. Hence, the wind force on a plate, orthogonal to the wind flow direction, can be determined by the net wind pressure as follows:

$$p_w = \frac{1}{2}\rho_a C_w v^2 \tag{2.1}$$

where:

ρ_a is the mass density of air (1.25 kg/m^3)
C_w is the wind pressure coefficient
v is wind speed

The total wind-induced force on the plate is given by the following equation:

$$F_w = p_w A \tag{2.2}$$

where:
p_w is wind pressure
A is the effective area of exposure

If the plate has an angle (θ) with respect to the wind direction, then the appropriate projected area normal to the flow direction should be used in the above equation. Typical values of the wind pressure coefficient, determined under controlled stationary wind flow conditions, are in the range of 0.7–1.2 for cylindrical members (Chandrasekaran, 2015a,b). The mean wind component, which is the static component of wind, is used in most design cases. As the name suggests, it leads to a static analysis. Other components of wind velocity are the gust components, which are dynamic in nature. The gust component is generated by turbulence of the flow field in all three spatial directions. Wind velocity, as a sum of both the components is given by

$$v(t) = \bar{v} + v(t) \tag{2.3}$$

where:
$\bar{v}$ is the mean wind velocity
$v(t)$ is the gust component

The spatial dependence of the mean component is only through the vertical coordinate, whereas $v(t)$ is homogeneous in both space and time. Wind force is experienced in the directions parallel and normal to that of the wind direction. The former is known as "drag," whereas the latter is known as "lift force." They are given by

$$\begin{aligned} F_D &= \frac{1}{2}\rho C_D \bar{v}_z A \\ F_L &= \frac{1}{2}\rho C_L \bar{v}_z A \end{aligned} \tag{2.4}$$

Wind spectrum above the MSL is given by 1/7th power law:

$$v_z = V_{10}\left[\frac{z}{10}\right]^{\frac{1}{7}} \tag{2.5}$$

where:
v_z is the wind speed at an elevation of z m above MSL
V_{10} is the wind speed at 10 m above MSL where 10 m is called as the "reference height"

The power law is purely empirical and most widely used. It is tested with the actual field measurements and found to be in good agreement. Alternatively, to obtain the approximate gust wind component, one can multiply the mean wind speed with the average gust wind factor (F_g = 1.35–1.45). For design of members under wind loads, most of the international codes prefer quasi-static analysis (Chandrasekaran et al., 2013; Kareem, 1985). Very slender and flexible structures are wind prone; for members under wave action, deamplification takes place in flexible structures due to their compliant nature (Davenport, 1961; Simiu and Leigh, 1984). However, spatial variations of wind velocity are handled using *aerodynamic admittance function*. Wind spectra, as applied to offshore structures are expressed in terms of circular frequency and are given by

$$S_u^+(\omega) = fG_u^+(f) \tag{2.6}$$

Davenport spectrum

$$\frac{\omega S_u^+(\omega)}{\delta \bar{U}_p^2} = \frac{4\theta^2}{\left(1+\theta^2\right)^{4/3}} \tag{2.7}$$

Harris spectrum

$$\frac{\omega S_u^+(\omega)}{\delta \bar{U}_p^2} = \frac{4\theta}{\left(2+\theta^2\right)^{5/6}} \tag{2.8}$$

where the variable θ is given by

$$\theta = \frac{\omega L_u}{2\pi \bar{U}_{10}} = \frac{\delta L_u}{\bar{U}_{10}}, \quad 0 \prec \theta \prec \infty \tag{2.9}$$

where:

L_u is the integral length scale (= 1200 m for Davenport and 1800 m for Harris spectrum)

δ is the surface drag coefficient referred to $\bar{U}_{10}$

For offshore locations, $\delta = 0.001$. For large floating structures, the following spectra are recommended (Chandrasekaran, 2015a,b):

Kaimal spectrum

$$\frac{\omega S_u^+(\omega)}{\sigma_u^2} = \frac{6.8\theta}{\left(1+10.2\theta\right)^{5/3}} \tag{2.10}$$

where σ_u^2 is the variance of $U(t)$ at a reference height of 10 m.

API (2000) spectrum

$$\frac{\omega S_u^+(\omega)}{\sigma_u(z)^2} = \frac{\left(\omega/\omega_p\right)}{\left[1+1.5\left(\omega/\omega_p\right)\right]^{5/3}} \tag{2.11}$$

where:

ω_p is the peak frequency

σ_z^2 is the variance of $U(t)$, which is not assumed as independent

2.3 WAVE FORCES

Wave loading on ocean structures is the most important of all environmental loads. Determination of these forces requires a solution of two separate, though interrelated problems (Boaghe et al., 1998; Buchner and Bunnik, 2007; Ertas and Lee, 1989). The first one is the computation of sea state using an idealization of the wave surface profile; in this case, water particle kinematics is given by an appropriate wave theory (Buchner et al., 1999; Ertas and Eskwaro-Osire, 1991). The second one is the computation of wave forces on individual members and on the total structure. Two different analyses are seen in the literature: (1) single design wave analysis and (2) random wave analysis.

2.3.1 Single Design Wave Analysis

In this approach, a regular wave, which is termed as a "design wave," of a known given height and period is defined. Usually, a maximum wave with 100-year return period is chosen. Forces induced by the design wave are computed using a higher order wave theory. It is important to note that no dynamic behavior of the structure is considered in such cases (Bea et al., 1999; Ertas and Lee, 1989). Static analysis is appropriate when the dominant wave periods are well above the period of the structure. This is applicable to the case of extreme storm waves acting on shallow water structures (Buchner and Bunnik, 2007; Freudenthal and Gaither, 1969; Guo et al., 2005).

2.3.2 Random Wave Analysis

In this approach, a statistical analysis is carried out based on the appropriate wave scatter diagram, chosen for the location of the ocean structure. Appropriate wave spectra are defined to perform the analysis in the frequency domain. With statistical methods, the most probable maximum force that occurs during the lifetime of the structure is calculated using linear wave theory (Fjeld, 1977; Furnes, 1977). In case dynamic analyses are required to be carried out for extreme wave loadings on deepwater offshore structures, appropriate wave spectra are used to generate random waves (Bar Avi, 1999; Bar Avi and Benaroya, 1996; Boaghe et al., 1998).

2.3.3 Wave Theories

Wave theories describe wave kinematics (water particle velocity and acceleration) based on the potential theory. Waves are assumed to be long crested and described by a two-dimensional flow field. They are characterized by different parameters: wave height (H), wave period (T), water depth (d), wave number (k), circular frequency (ω), and cyclic frequency (f). The common wave theories seen in the literature are linear (Airy) wave theory, Stokes fifth-order theory, solitary wave theory, Cnoidal theory, and Deans' stream function theory. Figure 2.2 refers to various parameters that are common for different wave theories. Ocean surface waves are generated at any offshore site by the drag of wind on the water surface (Buchner et al., 1999; Stansberg et al., 2002).

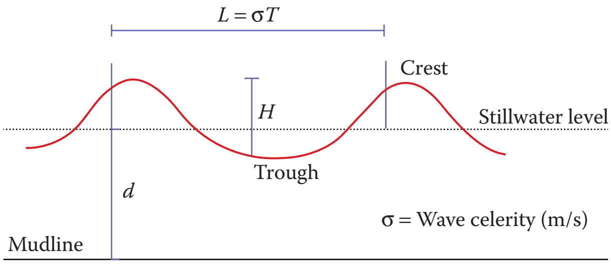


FIGURE 2.2 Parameters used in different wave theories.

It is necessary to relate the surface wave data to the water particle kinematics and pressure beneath the waves. This is usually done by using the appropriate wave theory (Dawson, 1983). Airy (1842) presented a relatively simple theory of wave motion. He assumed a sinusoidal wave form whose height (H) is smaller than its wave length (λ) and water depth (d). Although not strictly applicable to typical design waves used in offshore structural engineering, Airy's theory is useful for preliminary force estimates.

Sea surface elevation is given by

$$\eta(x,t) = \frac{H}{2}\cos(kx - \omega t) \tag{2.12}$$

where $k = 2\pi/\lambda$ and $\omega = 2\pi/T$.

Horizontal and vertical water particle velocities, at any location (x,y) and time (t) are given by

$$\dot{u}(x,t) = \frac{\omega H}{2}\frac{\cosh(ky)}{\sinh(kd)}\cos(kx - \omega t) \tag{2.13}$$

$$\dot{v}(x,t) = \frac{\omega H}{2}\frac{\sinh(ky)}{\sinh(kd)}\sin(kx - \omega t) \tag{2.14}$$

Horizontal and vertical water particle accelerations are given by

$$\dot{u}(x,t) = \frac{\omega^2 H}{2}\frac{\cosh(ky)}{\sinh(kd)}\sin(kx - \omega t) \tag{2.15}$$

$$\dot{v}(x,t) = -\frac{\omega^2 H}{2}\frac{\sinh(ky)}{\sinh(kd)}\cos(kx - \omega t) \tag{2.16}$$

Airy's theory is valid till the MSL only. As the submerged length of the member changes continuously with the passage of waves, this will attract additional forces. This is known as the "variable submergence effect." To compute the water particle kinematics up to the actual level of submergence, stretching modifications suggested by various researchers are used (Chandrasekaran, 2015a,b). One of the limitations

TABLE 2.1
Classification of Water Waves According to Relative Depth

Classification	d/L	kd	$\tanh(kd)$
Deepwater	1/2 to ∞	π to ∞	~1
Transitional	(1/20) to 1/2	(π/10) to π	$\tanh(kd)$
Shallow water	0 to (1/20)	0 to (π/10)	~kd

of Airy's linear theory is that the velocity potential does not satisfy Laplace's equation but satisfies dynamic free surface boundary conditions. Therefore, in many physical situations, linear theory, even with stretching modifications, is not adequate to describe water particle kinematics completely. Progressively definable by their wave height and period in a given water depth, when wave height becomes larger, simple treatment may not be adequate. Hence, some higher order theories are used to obtain better free surface and water particle kinematics expressions. These theories become nonlinear and allow the formulation of waves that are not purely sinusoidal. Table 2.1 shows the classification of water waves according to the relative water depth.

2.3.4 Stokes Fifth-Order Wave Theory

Lord Stokes (1880) investigated the mechanics of water waves of finite height. Stokes' theory expands the wave in series form and determines the coefficients of individual terms so as to satisfy the appropriate hydrodynamic equations for finite amplitude waves. An extension of this method of the fifth order has been made by Skjelbreia and Hendrickson (1961). Because of the slowness of the convergence in the series for shallow water, this theory is considered to be valid in the regime where (d/λ) is greater than 0.1. According to Stokes fifth-order nonlinear wave theory, sea surface elevation is given by

$$\eta(x,t) = \frac{1}{k}\sum_{n=1}^{5} F_n \cos\left[n(kx - \omega t)\right] \tag{2.17}$$

$$F_1 = a \tag{2.18}$$

$$F_2 = a^2 B_{22} + a^4 B_{24} \tag{2.19}$$

$$F_3 = a^3 B_{33} + a^5 B_{35} \tag{2.20}$$

$$F_4 = a^4 B_{44} \tag{2.21}$$

$$F_5 = a^5 B_{55} \tag{2.22}$$

Constants, denoting the wave profile parameter *vis-à-vis* B22, B24, and so on, depend on the value (kd) and the wave height parameter, a, which is obtained from the following equation:

$$\frac{kH}{2}=\left[a+a^3B_{33}+a^5\left(B_{35}+B_{55}\right)\right] \tag{2.23}$$

Constants for Stokes fifth-order wave theory are given by (Dawson, 1983; Patel, 1989)

$$s=\sinh\left(\frac{2\pi d}{\lambda}\right) \tag{2.24}$$

$$c=\cosh\left(\frac{2\pi d}{\lambda}\right) \tag{2.25}$$

$$B_{22}=\frac{\left(2c^2+1\right)}{4s^3} \tag{2.26}$$

$$B_{24}=\frac{c\left(272c^8-504c^6-192c^4+322c^2+21\right)}{384s^9} \tag{2.27}$$

$$B_{33}=\frac{3\left(8c^2+1\right)}{64s^6} \tag{2.28}$$

$$B_{35}=\frac{\left(88{,}128c^{14}-208{,}244c^{12}+70{,}848c^{10}+54{,}000c^8-21{,}816c^6\right)}{12{,}288s^{12}\left(6c^2-1\right)}+\frac{\left(6264c^4-54c^2-81\right)}{12{,}288s^{12}\left(6c^2-1\right)} \tag{2.29}$$

$$B_{44}=\frac{c\left(768c^{10}-448c^8-48c^6+48c^4+106c^2-21\right)}{384s^9\left(6c^2-1\right)} \tag{2.30}$$

$$B_{55}=\frac{\left(192{,}000c^{16}-262{,}720c^{14}+83{,}680c^{12}+20{,}160c^{10}-7280c^8\right)}{12{,}288s^{10}\left(6c^2-1\right)\left(8c^4-11c^2+2\right)}+\frac{\left(7160c^6-1800c^4-1050c^2+225\right)}{12{,}288s^{10}\left(6c^2-1\right)\left(8c^4-11c^2+3\right)} \tag{2.31}$$

Horizontal and vertical particle velocities are given by

$$\dot{u}\left(x,t\right)=\frac{\omega}{k}\sum_{n=1}^{5}G_n\,\frac{\cosh\left(nky\right)}{\sinh\left(nkd\right)}\,\cos\left[n\left(kx-\omega t\right)\right] \tag{2.32}$$

$$\dot{v}(x,t) = \frac{\omega}{k}\sum_{n=1}^{5} G_n \frac{\sinh(nky)}{\sinh(nkd)} \sin\left[n(kx-\omega t)\right] \tag{2.33}$$

$$G_1 = aG_{11} + a^3G_{13} + a^5G_{15} \tag{2.34}$$

$$G_2 = 2\left(a^2G_{22} + a^4G_{24}\right) \tag{2.35}$$

$$G_3 = 3\left(a^3G_{33} + a^5G_{35}\right) \tag{2.36}$$

$$G_4 = 4\left(a^4G_{44}\right) \tag{2.37}$$

$$G_5 = 5\left(a^5G_{545}\right) \tag{2.38}$$

Wave velocity parameters are given by

$$G_{11} = A_{11}\sinh(kd) \tag{2.39}$$

$$G_{13} = A_{13}\sinh(kd) \tag{2.40}$$

$$G_{15} = A_{15}\sinh(kd) \tag{2.41}$$

$$G_{22} = A_{22}\sinh(2kd) \tag{2.42}$$

$$G_{24} = A_{24}\sinh(kd) \tag{2.43}$$

$$A_{11} = \frac{1}{s} \tag{2.44}$$

$$A_{13} = \frac{-c^2\left(5c^2+1\right)}{8s^5} \tag{2.45}$$

$$A_{15} = \frac{-\left(1184c^{10} - 1440c^8 - 1992c^6 + 2641c^4 - 249c^2 + 18\right)}{1536s^{11}} \tag{2.46}$$

$$A_{22} = \frac{3}{8s^4} \tag{2.47}$$

$$A_{24} = \frac{192c^8 - 424c^6 - 312c^4 + 480c^2 - 17}{768s^{10}} \tag{2.48}$$

$$A_{33} = \frac{\left(13 - 4c^2\right)}{64s^7} \tag{2.49}$$

$$A_{55} = \frac{-\left(2880c^{10} - 72,480c^8 + 324,000c^6 - 432,000c^4 + 163,470c^2 - 16,245\right)}{61,440s^{11}\left(6c^2 - 1\right)\left(8c^2 - 11c^2 + 3\right)} \tag{2.50}$$

$$A_{44} = \frac{\left(80c^6 - 816c^4 + 1338c^2 - 197\right)}{1536s^{10}\left(6c^2 - 1\right)} \tag{2.51}$$

$$A_{35} = \frac{\left(512c^{12} + 4224c^{10} - 6800c^8 - 12,808c^6 + 16,704c^4 - 3154c^2 + 107\right)}{4096s^{13}\left(6c^2 - 1\right)} \tag{2.52}$$

Horizontal and vertical water particle accelerations are given by

$$\ddot{u}\left(X,t\right) = \frac{kc_s{}^2}{2}\sum_{n=1}^{5} R_n \text{sinn}\left(kx - \omega t\right) \tag{2.53}$$

$$\ddot{v}\left(X,t\right) = \frac{-kc_s{}^2}{2}\sum_{n=1}^{5} S_n \text{cosn}\left(kx - \omega t\right) \tag{2.54}$$

Wave speed is given by

$$c_s = \left[\frac{g}{k}\left(1 + a^2c_1 + a^4c_2\right)\tanh kd\right]^{\frac{1}{2}} \tag{2.55}$$

$$c_1 = \frac{\left(8c^4 - 8c^2 + 9\right)}{8s^4} \tag{2.56}$$

$$C_2 = \frac{\left(3840c^{12} - 4096c^{10} + 2592c^8 - 1008c^6 + 5944c^4 - 1830c^2 + 147\right)}{512s^{10}\left(6c^2 - 1\right)} \tag{2.57}$$

Acceleration coefficients are given by

$$R_1 = 2U_1 - U_1U_2 - V_1V_2 - U_2U_3 - V_2V_3 \tag{2.58}$$

$$R_2 = 4U_2 - U_1{}^2 + V_1{}^2 - 2U_1U_3 - 2V_1V_3 \tag{2.59}$$

$$R_3 = 6U_3 - 3U_1U_2 + 3V_1V_2 - 3U_1U_4 - 3V_1V_4 \tag{2.60}$$

$$R_4 = 8U_4 - U_2^{\,2} + V_2^{\,2} - 4U_1U_3 + 4V_1V_3 \tag{2.61}$$

$$R_5 = 10U_5 - 5U_1U_4 - 5U_2U_3 + 5V_1V_4 + 5V_2V_3 \tag{2.62}$$

$$S_1 = 2V_1 - 3U_1V_2 - 3U_2V_1 - 5U_2V_3 - 5U_3V_2 \tag{2.63}$$

$$S_2 = 4V_2 - 4U_1V_3 - 4U_3V_1 \tag{2.64}$$

$$S_3 = 6V_3 - U_1V_2 - U_2V_1 - U_1V_4 - U_4V_1 \tag{2.65}$$

$$S_4 = 8V_4 - 2U_1V_3 + 2U_3V_1 + 4U_2V_2 \tag{2.66}$$

$$S_5 = 10V_5 - 3U_1V_4 - 3U_4V_1 - U_2V_3 + U_3V_2 \tag{2.67}$$

$$U_n = G_n \frac{\cosh(nky)}{\sin(nkd)} \tag{2.68}$$

$$V_n = G_n \frac{\sinh(nky)}{\sinh(nkd)} \tag{2.69}$$

2.4 WAVE SPECTRA

Sea state, in a short-term period, which is typically 3 h, is assumed as a zero-mean, ergodic Gaussian process (Chakrabarti, 1994; Chakrabarti and Tam, 1975; Chakrabarti et al., 1976, 1987). This can be defined completely by a wave spectrum. For the North Sea, JONSWAP spectrum is recommended. For open sea conditions, Pierson–Moskowitz (P–M) spectrum is recommended. In a long-term period, the variation of sea state is slower than the short-term fluctuations (Chakrabarti et al., 1976, 1987). It is often approximated by a series of stationary, non-zero-mean Gaussian processes, which are specified by the significant wave height (H_s) and peak wave period (T_p). A few relevant spectra, applicable in the design of ocean structures, are as follows:

P–M spectrum for wave loads is given by

$$S^+(\omega) = \frac{\alpha g^2}{\omega^5} \exp\left[-1.25\left(\frac{\omega}{\omega_0}\right)^{-4}\right] \tag{2.70}$$

where α is the Phillips constant ($\cong 0.0081$).

Modified P–M spectrum with two parameters (H_s, ω_0) is given by

$$S^+(\omega) = \frac{5}{16} H_s \frac{\omega_0^{\,4}}{\omega^5} \exp\left[-1.25\left(\frac{\omega}{\omega_0}\right)^{-4}\right] \tag{2.71}$$

The International Ship Structures Congress (ISSC) spectrum with two parameters (H_s, $\bar{\omega}$) is given by

$$S^+(\omega) = 0.1107 H_s \frac{\omega^{-4}}{\omega^5} \exp\left[-0.4427\left(\frac{\omega}{\bar{\omega}}\right)^{-4}\right] \tag{2.72}$$

$$\bar{\omega} = \frac{M_1}{M_0} \tag{2.73}$$

JONSWAP spectrum with five parameters (H_s, ω_0, γ, $\bar{\sigma}_a$, $\bar{\sigma}_b$) is given by

$$S^+(\omega) = \frac{\bar{\alpha} g^2}{\omega^5} \exp\left[-1.25\left(\frac{\omega}{\omega_0}\right)^{-4}\right] \gamma^{a(\omega)} \tag{2.74}$$

where γ is the peakedness parameter.

$$a(\omega) = \exp\left[-\frac{(\omega - \omega_0)^2}{2\bar{\sigma}^2 \omega_0^2}\right] \tag{2.75}$$

where $\bar{\sigma}$ is the spectral width parameter and is given by

$$\bar{\sigma}_a = 0.07, \quad \omega \leq \omega_0 \tag{2.76}$$

$$\bar{\sigma}_b = 0.09, \quad \omega \succ \omega_0 \tag{2.77}$$

The modified Phillips constant is given by

$$\bar{\alpha} = 3.25 \times 10^{-3} H_s^2 \omega_0^4 \left[1 - 0.287 \ln(\gamma)\right] \tag{2.78}$$

$$\gamma = 5 \text{ for } \frac{T_p}{\sqrt{H_s}} \leq 3.6 \tag{2.79}$$

$$\gamma = \exp\left[5.75 - 1.15 \frac{T_p}{\sqrt{H_s}}\right] \text{ for } \frac{T_p}{\sqrt{H_s}} \succ 3.6 \tag{2.80}$$

$$H_s = 4\sqrt{m_0} \tag{2.81}$$

where γ varies from 1 to 7.

2.5 WAVE STRUCTURE INTERACTION

Ocean structures exposed to waves experience substantial forces much higher than wind loading (Chakratabarti, 1984, 1987; Chandrasekaran and Jain, 2002a,b, 2004). These forces result from the dynamic pressure variation and water particle motion (Chandrasekaran and Sharma, 2010). Two different cases can be distinguished: (1) large volume bodies and (2) slender bodies. Large volume bodies, which are termed as hydrodynamic compact structures, influence the wave field by diffraction and reflection (Chandrasekaran and Parameswara Pandian, 2011; Chandrasekaran et al., 2007b,c,f; Gadagi and Benaroya, 2006; Gasim et al., 2008). Forces on these bodies have to be determined by numerical calculations based on diffraction theory. However, slender bodies, which are termed as "hydrodynamic transparent," have no significant influence on the wave field. Forces can be calculated in a straightforward manner with Morison's equation. Morison's equation may be applied when $D/L < 0.2$, where D is the diameter of the member under consideration and L is the wave length. For example, jacket platforms, tension platforms, and so on, which are regarded as hydrodynamically transparent, can be analyzed with Morison-type forces; however, gravity-based structures need to be analyzed using diffraction theory (Chandrasekaran, 2013a,b,c, 2015a,b). Wave force per unit length of the member, as per Morison's equation, is given by

$$F = \frac{1}{2}\rho C_d D\dot{u}\left|\dot{u}\right| + \frac{\pi D^2}{4}\rho C_m \ddot{u} \tag{2.82}$$

where:

$(\dot{u},\ddot{u})$ is the water particle velocity and acceleration computed at the axis of the member

ρ is the density of sea water

D is the external diameter of the member, including marine growth

(C_d, C_m) are the drag and inertia coefficients, respectively

For compliant structures such as TLPs, forces due to relative motion are given by

$$F = \frac{1}{2}\rho C_d D\left(\dot{u} - \dot{x}\right)\left|\dot{u} - \dot{x}\right| + \frac{\pi D^2}{4}\rho C_m \ddot{u} \tag{2.83}$$

As seen from the above equation, drag force is nonlinear, which is used in the design wave concept (Chandrasekaran and Koshti, 2013; Chandrasekaran et al., 2004). For determining the transfer function that is required for frequency domain calculations, the drag force needs to be linearized (Glanville et al., 1991). Frequency domain solutions are appropriate for fatigue life calculations, for which the forces due to the operational level waves are dominated by the linear inertia term. Nonlinear formulation and hence the time domain solutions are required for dynamic analyses of deepwater structures under extreme, storm waves, for which the drag portion of the force is the dominant part. Drag and inertia coefficients depend on the wave theory used, surface roughness, and the flow parameters. Appropriate values, as recommended by the codes, are $C_d \sim 0.6$–1.2 and $C_m \sim 1.3$–2.0 (API RP WSD, 2005); additional information on the variation of these coefficients can be found in the Det Norske Veritas (DNV) rules (DNV, 1982). Total wave force on each member is obtained by numerical integration over the length of the member. Fluid velocities and accelerations at the integration points are found by direct application of the selected wave theory.

In addition to Morison's forces, lift forces and slamming forces can be important for local member design and are given by (Chandrasekaran and Koshti, 2013)

$$F_L = \left(\frac{1}{2}\right)\rho C_L D V^2 \tag{2.84}$$

$$F_s = \left(\frac{1}{2}\right)\rho C_s D V^2 \tag{2.85}$$

where (C_L, C_S) are the lift and slamming coefficients, respectively. Lift forces are perpendicular to the member axis; fluid velocity is related to the vortex shedding frequency. However, slamming forces act on the underside of horizontal members, near the MSL; they are impulsive and nearly vertical.

2.5.1 Maximum Wave Force

Consider a case of a surface-piercing cylinder such as a pile of a structure or a leg of a jacket platform. As seen from the above equations, the combined effect of drag and inertia forces varies with time and will be maximum only at one occasion (Chandrasekaran and Bhattacharyya, 2011). In order to find the maximum force, the phase angle at which the maximum force occurs shall be determined first. By substituting velocity and acceleration components and integrating between the limits (from the surface to the seabed, i.e., 0 to $-h$), the total force on the pile is given by

$$F_T = \frac{1}{2}\rho c_d D \frac{\pi^2 H^2}{T^2}\frac{\cos\theta|\cos\theta|}{\sinh^2(kh)}\left[\frac{\sinh(2kh)}{4k} + \frac{h}{2}\right] - c_m\rho\frac{\pi D^2}{4}\frac{2\pi^2}{T^2}\frac{\sin\theta}{k} \tag{2.86}$$

Total force will be maximum under the following condition:

$$\frac{\partial F_T}{\partial \theta} = 0 \tag{2.87}$$

Substituting for velocity and acceleration components in the drag and inertia terms of force equation and differentiating, we get

$$\theta_{\max} = \sin^{-1}\left[-\frac{\pi D}{H}\frac{C_m}{C_d}\frac{2\sin^2(kh)}{\left(\sinh(2kh) + 2kh\right)}\right] \tag{2.88}$$

By substituting, the maximum force can be then computed.

2.6 FLOATING BODY: HYDROSTATIC STABILITY

The static equilibrium of a floating vessel is influenced by weight and buoyancy. Weight of the floating vessel is the product of mass and gravitational acceleration, which acts downward through the center of gravity. Buoyancy is the weight of the displaced volume of water by the vessel computed generally at its equilibrium position. It acts upward through the center of buoyancy. When a vessel is floating freely, the above two forces must be coplanar, collinear, and concurrent; they counteract each other. Stability is defined as the ability of a system to return to its undisturbed position on removal of external load; the higher the value of the righting capacity (moment), the higher is the stability of the vessel (Faltinsen, 1998). Transverse

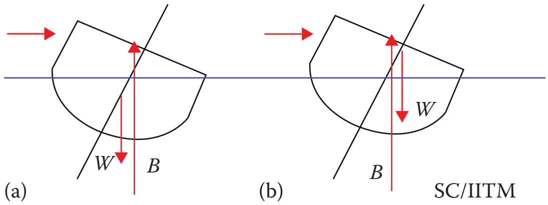


FIGURE 2.3 Stability cases: (a) positively stable; (b) unstable. W is weight and B is buoyancy.

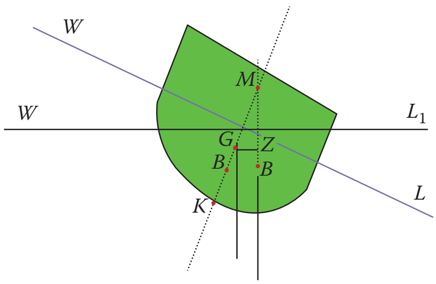


FIGURE 2.4 Righting moment of a vessel.

stability is determined by the points of action of weight and buoyancy. It is the horizontal distance between the relative positions of these two parameters. Figure 2.3 refers to two cases of stability. Case 1 is stable as the net moment tends to right the body, which is termed as "positively stable." Case 2 is negatively stable or unstable as the net moment tends to destabilize the body.

Figure 2.4 refers to the righting moment of a floating vessel and metacenter. With reference to the figure, K is the bottom point or line, termed as the "keel of the vessel," G is the center of gravity, and B is the center of buoyancy. The center of buoyancy shifts from B to B_1 when a vessel rotates by a small angle, which leads to the generation of the moment ($= W \times GZ$). Metacenter (M) is the point of intersection between the line of action of buoyancy force (vertical) and the centerline of the vessel in its inclined position. This will likely be the center of oscillation of the suspended pendulum, whose length is given by GM. For the pendulum to swing in a stable oscillation and return to its original position, the center must be located above the pendulum (Chandrasekaran and Saha, 2011; Thoft-Christensen, 1977; Thoft-Christensen and Baker, 1982; Winterstein, 1988; Wirsching and Light, 1980; Wirsching et al., 2006; Yang and Freudenthal, 1977). Metacentric height is the sum of the distance between the vessel keel and the center of buoyancy (KG) and the distance between the center of buoyancy and metacenter (BM); the moment now becomes $W \times \mathrm{GM} \sin\theta$. When the metacenter is located above the center of gravity, the moment is righting and is stable. If it is located lower than the center of gravity, this will result in overturning and causes instability. For inclinations less than about 15°, BM will be the second moment of area of the water plane cross-sectional area about the middle line and is given by

$$\mathrm{BM} = \frac{I_{XX}}{\nabla} \tag{2.89}$$

For GM greater than zero, the floating system should be positively stable. Therefore, for a submerged vessel to be stable, the center of gravity should geometrically lie below the center of buoyancy. Because the point of action of buoyancy is fixed along the line of gravity and does not change, metacenter is unaltered (remains at point *B*). In such case, condition that GM greater than zero still holds good.

2.7 BUOYANCY FORCES

Pressure loading on fully or partially submerged objects arises from two sources: hydrostatic head above it and the movement of water particles around it under wave action. This induces stresses in the structural members (Chandrasekaran et al., 2007b,c, 2013; Chou, 1980). Additional buoyant force that arises from the hydrostatic pressure is given by

$$p = -\rho g z \tag{2.90}$$

where:

ρ denotes the specific weight of water
z denotes the vertical distance from the stillwater level
negative sign indicates that the measurement axis of z is downward

It is important to note that buoyancy force exists even when wave action is absent and therefore must be accounted for separately. Geometric forms of compliant offshore structures depend heavily on buoyancy forces, which are caused by large displaced volume of hollow, airtight members that are sealed by the welds to avoid water entry (Gurley and Kareem, 1998; Perrettand and Webb, 1980). For example, tubular members of jacket platforms are chosen such that they have a reserve buoyancy of about 10%–15%. Reserve buoyancy is defined as a buoyancy in excess of its weight. To achieve this in design, tubular members are carefully selected such that their buoyancy-to-weight ratio is greater than 1.0, which enables them to float in water during towing (Halkyard et al., 1991). In case a member being supported at its two ends and forced to be submerged by the weight of other members, it will experience an upward force whose magnitude will be equivalent to that of the displaced volume of water. This is called "buoyancy force." Buoyancy force can be calculated by two methods: marine and rational. Marine method assumes that the member is considered to have a rigid body motion. According to this method, the buoyant per unit length is given by

$$W_B = \frac{1}{4}\pi\left(D^2 - \left(D - t\right)^2\right)\left(\rho_{\text{steel}} - 1.025\right) \tag{2.91}$$

where:

D is diameter of member
t is thickness
ρ_{steel} is density of steel

Unlike gravity forces, which are a true body force that acts on every particle of the body, buoyancy is the resultant of fluid pressure acting on the surface of the body. Loads on the members are normal to the member axis and are aligned on the vertical plane containing the member. The magnitude of this distributed load on the member is given by

$$B_B = \frac{1}{4}\pi D^2 \rho_W \cos\alpha \tag{2.92}$$

where α is the angle between the member and its projection on a horizontal plane. Joint loads consist of forces acting in the directions of all the members meeting at a joint. Joint forces act in a direction that would compress the corresponding members if they are acted directly on them, whose magnitude is given by

$$P_B = \rho_w a h \tag{2.93}$$

where:

a is the displaced area and it is equal to the area of the flooded members
h is the water depth at the end of the member under consideration

2.8 CURRENT FORCES

The presence of current in water produces additional forces. There are tidal, circulation, and storm-generated currents. When insufficient field measurements are available, current velocities may be obtained from various sources (Chandrasekaran et al., 2007f; Isaacson, 1978a,b). Current velocity is generally added vectorially to the horizontal water particle velocity to compute the drag force. Because the drag force varies with the square of velocity, as seen in the Morison equation, vectorial addition is likely to increase the forces on the platform significantly. This force is computed using the drag component of the Morison equation with appropriate modifications of fluid kinematics. The presence of current modifies the wave period. This effect is termed as the "Doppler effect." For slender members, cyclic loads induced by vortex shedding may also be important and should be examined. Although the current decreases with the increase in water depth, it steepens the wave profile by changing the wave celerity. Alternatively, the presence of current is accounted by increasing wave height to 10%–15%.

2.9 EARTHQUAKE LOADS

Offshore structures in seismic regions are typically designed for two levels of earthquake intensity: strength level and ductility level (Lee and Juang, 2012; Wilson, 1984). Strength-level earthquakes are assumed with a reasonable likelihood of not being exceeded during the platform's life, by assuming their mean recurrence interval as about 200–500 years. Platforms are designed to respond elastically. For strength-level design, seismic loading may be specified either by sets of accelerograms or by means of design response spectra. Use of design spectra has a number of advantages over time history solutions (base acceleration input). For this reason, design response spectrum is the preferable approach for strength-level designs. If the design spectral intensity (a_{max}) and the characteristic of the seismic hazard at the site are known, Code (API RP WSD, 2005) recommends using this value along two principal horizontal directions and half of this value for the vertical direction. DNV rules, however, recommend a_{max} and $0.7a_{max}$ for the two horizontal directions and half of its value for the vertical direction. The magnitudes of design spectral density and spectral shapes are determined by site-specific seismological studies. However, ductility-level earthquakes are defined as close to the "maximum credible earthquake" at the site. Platforms are designed for inelastic response and to have an adequate reserve strength to avoid collapse. Designs for ductility-level earthquakes will normally require inelastic analyses for which the seismic input must be specified by sets of three-component accelerograms, real or artificial; they should be the representative values of the extreme ground motions that could cause the lateral movement of seabed at the site where ocean structures are commissioned. Characteristics of such motion, however, may still be prescribed by means of design spectra based on site-specific seismotectonic studies.

Fixed-type offshore platforms and coastal structures, which are founded firmly on the seabed, are affected by earthquakes directly (Hsu, 1981; Zhao et al., 2009). However, compliant structures that are position restrained by tethers will be subjected to dynamic tether tension variations under the presence of earthquake forces (Lee et al., 1999; Lee and Wang, 2000). Tethers that are anchored to the seabed will undergo large displacement at these anchoring points along horizontal and vertical directions. Although horizontal displacements will influence the tether tension significantly, vertical displacements may sometimes result in tether slackening (Kawanishi et al., 1993; Kobayashi et al., 1987). Such continuous variations in tether tension are termed as "dynamic tether tension variation" (ΔT_0). As a result, the response of the compliant platform is significantly influenced by the earthquake loads, though the effect is not direct as in the case of fixed-type platforms. Ground acceleration caused by earthquakes exhibits random characteristics due to (1) the nature of the mechanism causing earthquakes, (2) wave propagation, (3) reflection, and (4) deflection. Dynamic tether tension will give rise to a change in stiffness of the platform as stiffness coefficients are the functions of tether tension. Further, they also cause change in buoyancy due to setdown effect. In case of the analysis of compliant structures such as TLPs, stiffness coefficients of the members are altered under the influence of dynamic tether tension variation. The following equation holds good (Chandrasekaran, 2015a,b; Chandrasekaran and Gaurav, 2008; Chandrasekaran and Mayanak, 2016; Chandrasekaran and Madhuri, 2015; Chandrasekaran et al., 2015a,b):

$$\Delta T = \frac{AE}{\ell}\left[x(t) - x_g(t)\right] \tag{2.94}$$

where:

$x(t)$ is the instantaneous response vector of TLP
$x_g(t)$ is the ground displacement vector, which is given by

$$x_g(t) = \begin{Bmatrix} x_{1g}(t) \\ 0 \\ x_{3g}(t) \\ 0 \\ 0 \\ 0 \end{Bmatrix} \tag{2.95}$$

where x_{1g} and x_{3g} are the horizontal and vertical ground displacements, respectively. Acceleration of ground motion can be simulated using Kanai-Tajimi ground acceleration spectrum (K-T spectrum) as follows:

$$S_{\ddot{x}_g\ddot{x}_g}(\omega)=\left[\frac{\omega_g^4+4\xi_g^2\,\omega_g^2\,\omega^2}{\left(\omega_g^2-\omega^2\right)^2+4\xi_g^2\,\omega_g^2\,\omega^2}\right]S_0$$

$$S_0=\frac{2\xi_g\sigma_g^2}{\pi\omega_g(1+4\xi_g^2)} \tag{2.96}$$

where:

S_0 is the intensity of the earthquake
ω_g is the natural frequency of the ground
ξ_g is the damping of the ground
σ_g^2 is the variance of the ground acceleration

These are the three parameters on which the K-T spectrum depends, which need to be chosen for any analytical studies on TLP under seismic action. Studies showed that the dynamic tether tension variations caused by the earthquake forces are on the order of about 65% more than that of the normal values. Even rigid degree of freedom like heave is excited, which may result in the loss of functionality of the platform (Chandrasekaran and Madhuri, 2015; Chandrasekaran et al., 2006a,b, 2011).

2.10 ICE AND SNOW LOADS

Ice is a primary problem for marine structures in the arctic and subarctic zones. Ice formation and expansion can generate large pressure that gives rise to horizontal as well as vertical forces (Chandrasekaran, 2015a,b). In addition, large blocks of ice driven by current, wind, and waves may hit the offshore structures and cause impact loads. Statically applied horizontal ice forces may be estimated as follows:

$$F_i=C_i f_c A \tag{2.97}$$

where:

A is the exposed area of structure
f_c is the compressive strength of ice
C_i is the coefficient accounting for shape, rate of load application, and other factors with usual values between 0.3 and 0.7

Ice formation and snow accumulation increase gravity and wind loads. Various ice conditions that exist in the service life of an offshore platform: level ice, broken ice, ice ridges, and icebergs make the load estimates more complex. Offshore structures show different types of failure under ice loads: creep, cracking, buckling, spalling, and crushing. Ice loads are classified as (1) total or global loads and (2) local loads or pressure. Although global loads affect the overall motion and stability of the platform, local loads affect the members at connections. In the level ice condition, the frequency of interaction between the structure and ice becomes important to quantify the ice loads. Studies show that ice loads on a conical structure are less than that on a cylindrical structure (Sanderson, 1988). This is due to the fact that a well-designed cone shape can change the ice failure mode from crushing to bending. Design ice loads use varying factors for level ice, first-year ridge ice, and multiyear ridge ice; they are 2, 5, and 7, respectively. As ice loads depend on the geometric shape of the structural form, there is a lot of uncertainty in estimating ice loads. Ice force spectrum on a narrow conical structure is given by

$$S^{+}(f) = \frac{A\overline{\mathrm{F}}_0^2\,\overline{\mathrm{T}}^{(-\delta)}}{f^{\gamma}}\exp\left[-\frac{B}{\overline{\mathrm{T}}^{(\alpha)}\mathrm{f}^{\beta}}\right] \tag{2.98}$$

where:

- A and B are constants
- $\overline{\mathrm{F}}_0$ is the force amplitude on the structure
- $\overline{\mathrm{T}}$ is the period of ice
- α, β, γ, δ are constants whose values are typically 0.64, 0.64, 3.5, and 2.5, respectively

The values of constants A and B that account for the shape of ice are typically 10 and 5.47, respectively. Ice period is given by

$$\overline{T} = \frac{L_b}{v} \tag{2.99}$$

where:

- v is the velocity at which ice is traveling
- L_b is the ice-breaking length

Ice-breaking length is typically 4–10 times of that of the thickness of ice. Force amplitude on the structure, caused by impact of ice, is given by

$$\overline{F}_0 = C\sigma_f h^2\left(\frac{D}{L_c}\right)^{0.34} \tag{2.100}$$

where:

- C is the constant
- σ_f is the bending strength of ice (0.7 MPa)
- h is the ice thickness
- D is the diameter of the ice cone
- L_c is the characteristic length of ice

The characteristic length of ice is given by

$$L_c = \left(\frac{Eh^3}{12g\rho_w}\right)^{0.25} \tag{2.101}$$

where:

E is Young's modulus of ice (= 0.5 GPa)

ρ_w is the density of water

2.11 LOADS DUE TO TEMPERATURE VARIATIONS

Offshore structures can be subjected to temperature gradients that produce thermal stresses. To account for the same, the extreme values of sea and air temperature, which are likely to occur during the life of the structure, must be estimated. Relevant data for the North Sea are given in BS6235. In addition, thermal loads are also generated through an accidental release of cryogenic material, which must be taken into account in design as accidental loads (Chandrasekaran, 2015a,b). The temperature of the oil and gas produced must also be considered, while estimating the temperature stresses caused in members.

2.12 MARINE GROWTH

Marine growth is accumulated on submerged members. Wave forces on the members are increased with the increase in surface area caused by marine growth deposition. In addition, it also increases the mass of the platform (Chen et al., 2002, 2006). Marine growth makes underwater inspection of offshore members very difficult. As most of the nondestructive evaluation (NDE) methods demand a prerequisite of a clean surface, deposition of marine growth makes NDE ineffective and inaccurate. It is also seen in the literature that marine growth influences the drag coefficient by increasing its value; though this increase will be marginal but still need to be accounted for. Increase in drag coefficient is mainly due to surface roughness caused by marine growth. Marine growth results in an increase in unit mass of the member causing higher gravity loads; it also reduces the natural frequency of vibration of the platform (Kim et al., 2007). Depending on geographic location, the thickness of marine growth can reach even as high as 0.3 m. In the structural design of offshore members, marine growth is accounted by increasing the diameter of the member for load computations.

2.13 TIDES

Tides affect wave and current loads indirectly by causing variations in the MSL. Tides are classified as astronomical tides and storm surge, as shown in Figure 2.5. Whereas astronomical tides are caused essentially by the gravitational pull of the moon and the sun, storm surge is caused by the combined action of wind and barometric pressure differentials during a storm. The combined effect of both is termed as "storm tide." The range of astronomical tides largely depends on the geographic location and phase of the moon. The maximum of its magnitude, called a "spring tide," occurs during a new moon. Storm surge depends on the return period considered in the design; they vary on the order of 1–3 m. When designing a platform, extreme storm waves are superimposed on the stillwater level. For the design of ancillary elements such as a boat landing, barge fenders, and estimating the upper limits of marine growth, daily variations of astronomical tides are used.

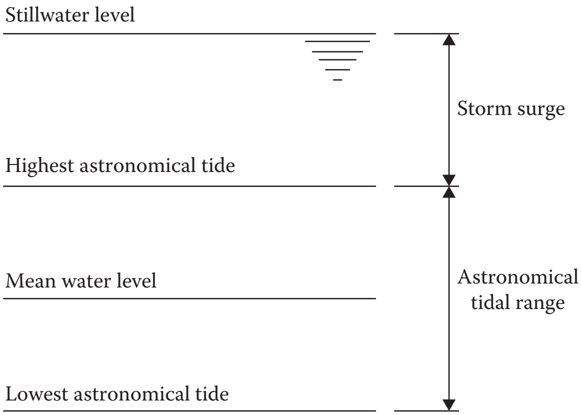


FIGURE 2.5 Astronomical tide and storm surge.

2.14 SEAFLOOR MOVEMENTS

The movement of the seafloor can occur as a result of active geologic processes, storm wave pressures, earthquakes, pressure reduction in the producing reservoir, and so on. Loads generated by such movements affect not only the design of the piles but the jacket as well. Such forces are determined by special geotechnical studies and investigations. Although they have a significant influence on the design of foundation of fixed-type structures such as jacket platforms, their influence on gravity-based structure (GBS) platforms and coastal structures is the most significant. Compliant-type offshore structures such as TLPs are not influenced by seafloor movements, except that the dynamic tether tension variation induced by such movements should be accounted for.

2.15 WIND FORCE ESTIMATE SUMMARY ON A COMPLIANT OFFSHORE PLATFORM

Wind forces are attracted by the superstructure of offshore deck. They are quantified by dividing the projected area of the superstructure into several segmental areas (Kareem and Datton, 1982). Velocity fluctuations are defined at the centroid of these areas. Wind fluctuations depend on both space and time, but spatial fluctuations are normally neglected for compliant-type structures. Direct wind pressure on the superstructure causes translational surge force and moments in the yaw and pitch degrees of freedom. Due to the coupled nature of heave degree of freedom with that of other degrees of freedom, heave motion is also activated. This coupled wind force also influences tether tension variation. It is assumed that the total wind force is said to be concentrated on the aerodynamic center, whereas the mass is lumped at the center of gravity. Difference between these two centers activates pitch and yaw forces in the deck. Wind-induced force is termed as "drag force per unit area," which acts on the projected plane and normal to the direction of mean wind velocity (Simiu and Leigh, 1984). This is given by

$$f\left(y,z,t\right)=\frac{1}{2}\rho_{\text{air}}C_{\text{air}}\left(y,z\right)A_a\left[\dot{u}\left(y,z,t\right)-\dot{x}\left(t\right)\right]^2 \tag{2.102}$$

where:

ρ_{air} is the density of air
$C_{\text{air}}(y, z)$ is the force coefficient at elevation z and at horizontal coordinate y
A_a is the exposed area in the surge direction
$\dot{u}\left(y,z,t\right)$ is the wind velocity in the surge direction varying with time and space location
$\dot{x}\left(t\right)$ is the structural velocity in the surge direction

It is assumed that the directions of wind and surge motion are collinear. The wind velocity is expressed as

$$\dot{u}\left(y,z,t\right)=\dot{u}+\dot{u}'\left(y,z,t\right) \tag{2.103}$$

where:

$\dot{u}$ is the mean wind velocity at 10 m above MSL
$\dot{u}'\left(y,z,t\right)$ is the fluctuating wind velocity

Hence, the force due to wind in N segments above the pth segment, starting from the MSL, is given by

$$F\left(y,z,t\right)=\sum_{p}^{N}\bar{C}_p\left[\dot{u}\left(y,z,t\right)-\dot{x}\left(t\right)\right]^2 \tag{2.104}$$

$$\bar{C}_p=\frac{1}{2}\rho_{\text{air}}C_pA_{ap} \tag{2.105}$$

where:

A_{ap} is the projected area of the pth segment
$\bar{C}_p$ is the aerodynamic force coefficient of the pth segment
N is the total number of segments, considered for the analysis

Mean wind velocity, using the logarithmic law, is given by

$$\bar{u}\left(z\right)=u\left(\text{ref}\right)\left\{\frac{\ln\left[\dfrac{z}{z_0}\right]}{\ln\left[\dfrac{z_{\text{ref}}}{z_0}\right]}\right\} \tag{2.106}$$

where:

z_{ref} is the reference elevation (usually taken as 10 m above MSL)
z_0 is the roughness length

The aerodynamic force coefficient, C_p, with roughness length z_0 over the sea surface is given by

$$C_p = \left[\frac{K}{\ln\left(\frac{10}{z_0}\right)} \right]^2 \tag{2.107}$$

where K is the Von Kármán constant (= 0.4). The above equation gives a conservative estimate of the mean wind velocity component due to the presence of water waves compared to flow above a rigid surface (Ursell et al., 1960). Fluctuating velocity component is estimated by the Fourier synthesis of the wind spectrum suitable for the offshore environment. As deepwater compliant platforms have frequencies of interest at very low values, the ordinates of this spectrum at these frequencies are maximum. The shape of the spectrum in the very low-frequency range has very little effect on the design of land-based structures of fixed type, while it significantly influences the design of compliant structures (Michel, 1999). Fluctuating wind velocity spectrum is given by

$$\left\{ \frac{nS_u(Z,n)}{u_*^2} \right\} = a_1 f + b_1 f^2 + d_1 f^3 \qquad 0 < f \le f_m \tag{2.108}$$

$$= c_2 + a_2 f + b_2 f^2 \qquad f_m < f < f_s \tag{2.109}$$

$$= 0.26 f^{-\frac{2}{3}} \qquad f \ge f_s \tag{2.110}$$

where:

$$u_* = \frac{ku(10)}{\ln\left(\frac{10}{z_0}\right)} \tag{2.111}$$

$$a_1 = \frac{4L_u\beta}{Z} \tag{2.112}$$

$$\beta_1 = 0.26 f_s^{-\frac{2}{3}} \tag{2.113}$$

$$f = \frac{nz}{u(z)} \tag{2.114}$$

where f is a nondimensional frequency.

$$b_2 = \frac{\frac{1}{3}a_1 f_m + \left[\frac{7}{3} + \ln\left(\frac{f_s}{f_m}\right)\right]\beta_1 - \beta}{\frac{5}{6}(f_m - f_s)^2 + \frac{1}{2}(f_m^{\,2} - f_s^{\,2}) + 2f_m(f_s - f_m) + f_s(f_s - 2f_m)\ln\left(\frac{f_s}{f_m}\right)} \tag{2.115}$$

$$a_2 = -2b_2 f_m \tag{2.116}$$

$$d_1 = \frac{2}{f_m^3}\left[\frac{a_1 f_m}{2} - \beta_1 + b_2(f_m - f_s)^2\right] \tag{2.117}$$

$$b_1 = -\frac{a_1}{2f_m} - 1.5 f_m d_1 \tag{2.118}$$

$$c_2 = \beta_1 - a_2 f - b_2 f_s^2 \tag{2.119}$$

In the above expressions, Z_0 is generally taken as 0.001266 m and L_u is taken as 180 m. For offshore decks, $u(10)$ values are generally taken as 35, 40, and 45 m/s, and the corresponding values of current velocity are 1.56, 1.76, and 2.01 m/s, respectively. Wind velocity spectrum, as given by the above equation, covers a wide range of frequencies of compliant offshore platforms. For ease of simulation, the cross-correlation coefficient is assumed to be unity. This means that the wind velocities are assumed to be fully correlated along the height of the superstructure. Thus, the time histories of wind velocities at different heights can be generated with the help of only the wind velocity spectrum defined above. Fluctuating wind represented by Emil Simiu's spectrum is simulated using the Monte Carlo procedure.

2.16 TUTORIALS

Problem 1

A pile of diameter, $D = 0.75$ m, is to be installed in a water depth of 100 m. Wave height H and period T are 6 m and 10 s, respectively. Take drag and inertia coefficients as 1 and 2, respectively. Compute the maximum wave force and moment at the base of the pile.

Solution

Deepwater wave length is given as $L_0 = 1.56T^2 = 156$ m

$$\frac{h}{L_0} = \frac{100}{156} = 0.64 > 0.5$$

Hence, it is a deepwater condition. The wave length L is equal to the deepwater wave length 156 m. Wave number is given by

$$k = \frac{2\pi}{L} = 0.04$$

The phase angle at which the total wave force is maximum is given by

$$\theta_{\max} = \sin^{-1}\left(\pm\frac{\pi D}{H}\frac{C_M}{C_D}\frac{2\sinh^2 kh}{2kh + \sinh 2kh}\right)$$

$$\theta_{\max} = 51.3°$$

The maximum force is given by

$$F = \rho g V\left(\frac{H}{2h}\right)\tan kh * \left[C_M \sin\theta + C_D\left(\frac{H}{4\pi D}\right)\frac{2kh+\sin 2kh}{\sinh^2 kh}\left|\cos\theta\right|\cos\theta\right]$$

$$F_{\max} = 27{,}311.5\,\mathrm{N}$$

where:

$$V = \frac{\pi}{4}D^2 h$$

The maximum overturning moment at the base of the pile is given by

$$M = \rho g V\left(\frac{H}{2h}\right)\left(\frac{1}{k}\right)\tanh kh\left[C_M \frac{kh\sinh kh - \cosh kh + 1}{\sinh kh}\sin\theta\right.$$

$$\left.+ C_D\left(\frac{H}{2\pi D}\right)\frac{(kh)^2 + kh\sinh 2kh - \sinh^2 kh}{\sinh^2 kh}\left|\cos\theta\right|\cos\theta\right]$$

$$M = 2.7\times 10^6\ \mathrm{Nm}$$

The distance at which the maximum force acts from the base of pile is given by

$$y = \frac{2.7\times 10^6}{27{,}311.5} = 98.86\ \mathrm{m}$$

Problem 2

Consider the front view of the offshore structure as shown in Figure 2.6. Determine the forces exerted on member 1–2 for relatively uniform wave-induced motion is described by u = 4 m/s, v = 1.2 m/s, a_x = 1.2 m/s^2, a_y = −1.6 m/s^2. Take drag and inertia coefficients as 1 and 2, respectively. The stillwater level above the seafloor is 29 m. The diameter of the member is 0.6 m.

Solution

The orientation of member is given by

$$\theta = 90°\varnothing = 135°$$

$$C_x = \sin\varnothing\cos\theta = 0$$

$$C_y = \cos\varnothing = -0.707$$

$$C_z = \sin\varnothing\sin\theta = 0.707$$

$$V = \left[u^2 + v^2 - \left(C_x u + C_y v\right)^2\right]^{\frac{1}{2}}$$

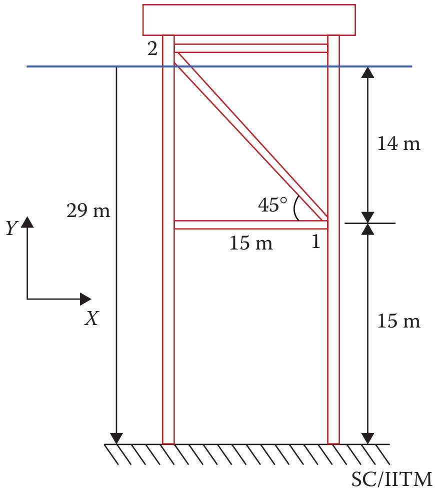


FIGURE 2.6 Front view of the offshore structure.

$$u_n = u - C_x\left(C_x u + C_y v\right) = 4 \text{ m/s}$$

$$v_n = v - c_y\left(c_x u + c_y v\right) = 0.6 \text{ m/s}$$

$$W_n = -C_Z\left(C_x u + C_v\right) = 0.6 \text{ m/s}$$

$$a_{nx} = a_x - C_x\left(C_x a_x + C_y a_y\right) = 1.2 \text{ m/s}^2$$

$$a_{ny} = a_y - C_y\left(C_x a_x + C_y a_y\right) = -0.8 \text{ m/s}^2$$

$$a_{nz} = -C_Z\left(C_x a_x + C_y a_y\right)$$

The magnitude of the velocity normal to the member is given by

$$V = \left(4^2 + 1.2^2 - 0.707 * 1.2\right)^2 . \frac{1}{2} = 4.08 \frac{m}{s^2}$$

$$F_x = \frac{1}{2}\rho C_D D V u_n + \rho C_m \frac{\pi}{4} D^2 a_{nx}$$

$$= 0.5 * 1025 * 1 * 0.6 * 4.08 * 4 + 1025 * 2 * \frac{3.14}{4} * 06.^2 * 1.2 = 5713.9 \text{ N/m}$$

$$F_y = \frac{1}{2}\rho C_D D V v_n + \rho C_m \frac{\pi}{4} D^2 a_{ny}$$

$$= 0.5 * 1025 * 1 * * 0.6 * 4.08 * -0.6 + 1025 * 2 * \frac{3.14}{4} * 0.6^2 * -0.8 = 289 \text{ N/m}$$

where F_x, F_y, F_z are the forces per unit length.

$$F_Z = F_y = 289\,\text{N/m}$$

This is valid as the flow is assumed to be uniform.

Problem 3

Figure 2.7 shows an offshore structure with two piles and one diagonal member. Find the total force on the structure. Wave height and wave length are given as 6 and 90 m in a water depth of 25 m, respectively. The pile has a diameter of 1.2 m and the diagonal member has a diameter of 0.6 m. Assume drag and inertia coefficients as 1 and 2, respectively.

Solution

Wave height $H = 6$ m
Wave length $L = 90$ m
Wave number $K = (2\pi/L)\ 0.06$
Wave frequency $\omega = \sqrt{gk\tan kd} = 0.73$ rad/s
Water depth $h = 25$ m

1. For member 1–3
   The total drag force is given by

$$F_{D1-3} = \frac{\rho C_D D}{32k}(\omega H)^2\left(\frac{\sinh 2ky}{\sinh^2 kh} + \frac{2ky}{\sinh^2 kh}\right)|\cos\theta|\cos\theta$$

where:

$$y = h + \eta = 25 + 3\cos(Kx - \omega t)$$

$$\theta = kx - \omega t$$

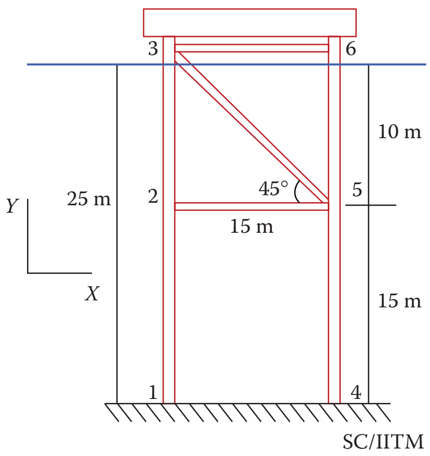


FIGURE 2.7 One-diagonal member.

From subsequent trials, one can find that the wave force is maximum at $\omega t = 6$ and the maximum drag force is given by

$$F_{D1-3} = 44,394.4\,\mathrm{N}$$

Total inertia force is given by

$$F_{I1-3} = \frac{\rho C_M}{2k}\frac{\pi}{4}D^2\omega^2 H\frac{\sinh ky}{\sin kd}\sin\left(kx - \omega t\right)$$

$$F_{I1-3} = 22,043\,\mathrm{N}$$

Hence, the total wave force on the pile 1–3 is given by

$$F_{T1-3} = F_{D1-3} + F_{I1-3} = 44,394.4 + 22,043 = 66,437.4\,\mathrm{N}$$

2. For member 4–6
   X = 15 m
   The total maximum inertia and drag forces are calculated as

$$F_{D4-6} = 6677\,\mathrm{N}$$

$$F_{I4-6} = 78,966\,\mathrm{N}$$

$$F_{T4-6} = 85,643\,\mathrm{N}$$

3. Forces on member 2–6
   For member 2–6, $\theta = \varnothing$ and $\varnothing = 45$

$$C_x = \sin\varnothing\cos\theta = 0.707$$

$$C_y = \cos\varnothing = 0.707$$

$$C_z = 0$$

The horizontal force per unit length acting on a side face diagonal is given by

$$f_x = \frac{1}{2}\rho C_D D V u_n + \rho C_M\frac{\pi}{4}D^2 a_{nx}$$

where:

$$u_n = u - C_x\left(C_x u + C_y v\right)$$

$$= \frac{1}{2}\left(u - v\right)$$

Similarly,

$$a_{nx} = \frac{1}{2}\left(a_x - a_y\right)$$

$$V = \sqrt{\left[u^2 + v^2 - \left(C_x u + C_y v\right)^2\right]}$$

For member 2–6, we have both x and y varying along the member to its intersection with the water surface:

$$x = 10 + \cos\left(kx - \omega t\right)$$

$$y = 25 + 3\cos\left(kx - \omega t\right)$$

$$\text{for } \omega t = 6 \text{ by trial, we get } x = 12 \text{ m}$$

Hence, the elevation of the water surface on the member is 15 + 13 = 28 m, and the length of the member struck by the wave is 13/0.707 = 18.038 m. The wave force per unit length varies along the member, and we estimate the total force by dividing the member length struck by the wave into two segments, each of 9.19 m. The wave force is calculated at the mid-length of each section, and the total force is computed by assuming these values constant over the respective segments.

x (m)	y (m)	u (m/s)	v (m/s)	a_x (m/s²)	a_y (m/s²)	f_x (N/m)
3.24	18.24	1.51	0.62	0.573	–0.88	508.6
3.27	24.27	1.5535	1.5525	1.2636	–1.017	661

$$F_x = \left(508.6 + 661\right)9.19 = 10{,}748.62\,\text{N}$$

The total maximum force on the complete structure is given by

$$F_T = 66{,}437.4 + 85{,}643 + 44{,}394.4 = 196{,}472.8\,\text{N}$$

Problem 4

Find the total horizontal force on the member 1–2 as shown in Figure 2.8 and the moment at the base of the member due to this total horizontal force. The member is inclined at an angle of 30° to the vertical. The wave height and wave period are 6 m and 10 s, respectively. The member diameter is 1.2 m, and the depth of water is 100 m. Assume drag and inertia coefficients as 1 and 2, respectively, and the density of seawater is 1025 kg/m³. Also plot the variation of total force with time.

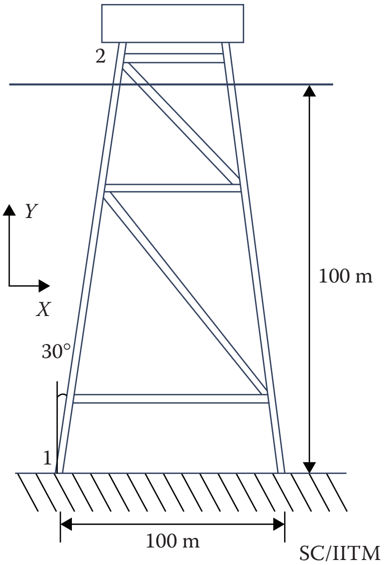


FIGURE 2.8 Horizontal force on member 1–2.

Solution

Given the following:

Pile diameter $D = 1.2$ m
Water depth $h = 100$ m
Wave height $H = 6$ m
Wave period $T = 10$ s

The deepwater wave length is given as $L_0 = 1.568 * T^2 = 156\,\text{m}$

$$\frac{h}{L_0} = \frac{100}{156} = 0.64 > 0.5$$

Hence, it is a deepwater condition. The wave length L is 156 m for deepwater condition.

The wave number is given by

$$k = \frac{2\pi}{L} = 0.04$$

The total force on a pile for a segment dy is given by

$$dF = \frac{1}{2}\rho c_D D |V_n| V_n dy + c_M \rho \frac{\pi D^2}{4} a_{ndy}$$

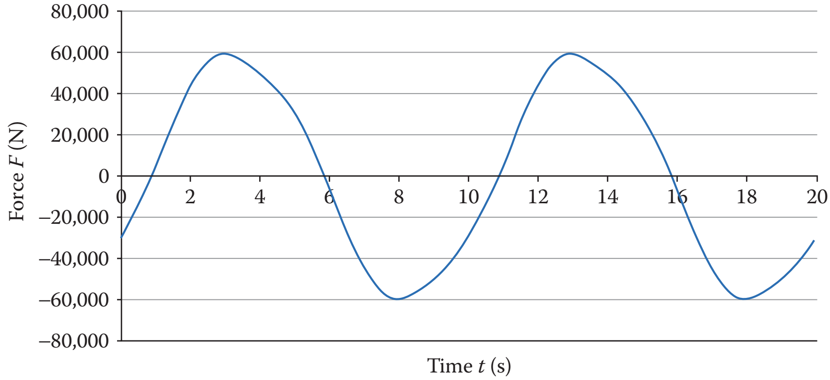


FIGURE 2.9 Total force variations on the member.

where:

$$V_n = u\cos\alpha + v\sin\alpha$$
$$a_n = a_x\cos\alpha + a_y\sin\alpha$$

α = Inclination of the pile to the vertical
u, v = Horizontal and vertical particle velocities from Airy wave theory, respectively
a_x, a_y = Horizontal and vertical particle acceleration from Airy wave theory, respectively

The total force F on the member is calculated by integrating the segment force dF along the length of the pile. The variation of total force F with time is shown below. The corresponding MATLAB® coding used for computation is also given at the end (Figure 2.9).

From Figure 2.9, it has been found that the total force is maximum at $t = 3.3$ s and the corresponding value total force is 59771.77 N. The moment at the base due to the force on the segment dy is obtained as follows:

$$\mathrm{d}M = \mathrm{d}Fy$$

The moment due to total force is calculated by integrating the segment force moment along the length of the member. The moment at the base of the member due to this maximum total force is 4919.544 kN m. The distance at which this maximum total force acts is 82.3 m (= 4,919,544/59,771.77) = 82.3 m from the bottom.

MATLAB Coding for Computing the Forces on Members

```
clc
close all
clear all
H=6;
h=100;
h1=100/cosd(30)
D=1.2;
T=10;
```

```
w=2*pi/T;
cd=1;
cm=2;
rho=1025;
L=1.56*T^2;
k=2*pi/L;
t=0:0.1:100;
dt=1
y=0:dt:h
alpha=30;
z=size(y)
x=y*tand(alpha);
for i=1:z(2)
 u(i,:)=(w*H/2)*(cosh(k*y(i))/sinh(k*h))*cos(k*x(i)-w*t);
 v(i,:)=(w*H/2)*(sinh(k*y(i))/sinh(k*h))*sin(k*x(i)-w*t);
    ax(i,:)=(w^2*H/2)*(cosh(k*y(i))/
    sinh(k*h))*sin(k*x(i)-w*t);
    ay(i,:)=-(w^2*H/2)*(sinh(k*y(i))/
    sinh(k*h))*cos(k*x(i)-w*t);
    Vn(i,:)=u(i,:)*cos(alpha)+v(i,:)*sin(alpha);
    An(i,:)=ax(i,:)*cos(alpha)+ay(i,:)*sin(alpha);
    Fd(i,:)=0.5*rho*cd*D*abs(Vn(i,:)).*Vn(i,:);
    Fi(i,:)=cm*rho*(pi*D^2/4)*An(i,:);
    FT(i,:)=Fd(i,:)+Fi(i,:);
    M(i,:)=FT(i,:)*y(i);
end
Ft=sum(FT);
MI=sum(M);
plot(t,Ft)
```

Problem 5

Figure 2.10 shows the arrangement of members in an offshore structure consisting of four cylindrical members of 2 m in diameter, fixed on the seabed at a water depth of 150 m. All vertical cylindrical members are connected by 1 m diameter pontoon members at the top with a freeboard of 10 m. The wall thickness of the cylindrical members is 44.45 mm. Find the total maximum force on all the four cylinders under a wave height of 10 m and a wave period of 8 s. Neglect the current velocity and relative velocity of cylinders. Take drag and inertia coefficients as 1 and 2, respectively. Assume the lift force coefficient as 1.0. Check the Morison equation applicability. Use Airy wave theory.

Solution

Figure 2.11 shows the wave forces and current forces on member in A row at MSL (zero elevation). Variations of drag, inertia, and total force on the member in A row at MSL are also plotted. Tables 2.2 and 2.3 show the calculation of member forces in A row.

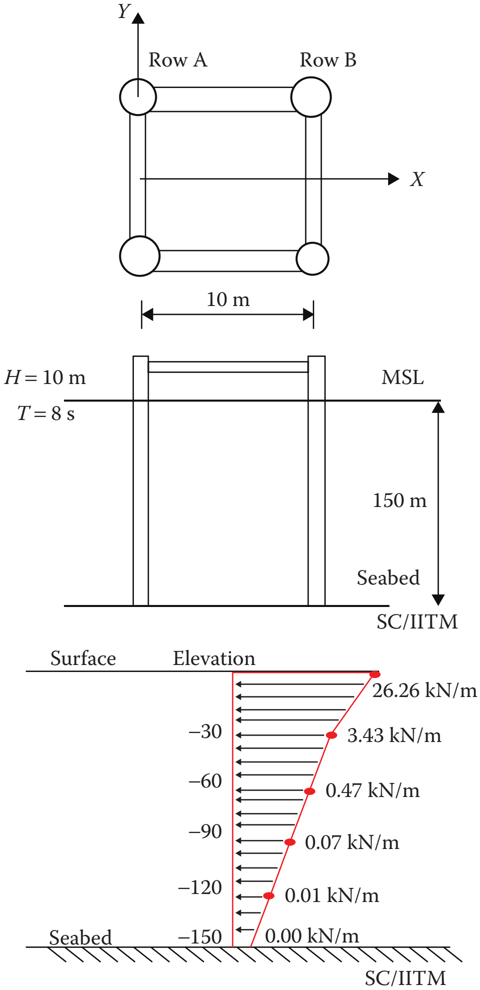


FIGURE 2.10 Wave force and current forces on the member in A row at zero elevation.

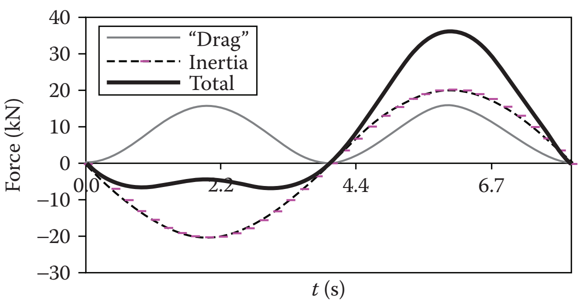


FIGURE 2.11 Variations in drag, inertia, and total force at zero elevation (MSL) on the member in A row.

TABLE 2.2

Calculation of Member Forces in the A Row Pile at MSL

Time Step	x	Horizontal Acceleration	Horizontal Velocity	Vertical Acceleration	Vertical Velocity	F_D	F_I	Vertical Force	Horizontal Force
0.00	0.00	0.00	0.00	3.93	3.93	0.00	0.00	41.89	0.00
0.22	0.00	−0.54	−0.68	3.87	3.87	0.49	−3.52	41.02	−3.03
0.44	0.00	−1.05	−1.34	3.69	3.69	1.88	−6.93	38.45	−5.04
0.67	0.00	−1.54	−1.96	3.40	3.40	4.03	−10.12	34.41	−6.10
0.89	0.00	−1.98	−2.52	3.01	3.01	6.66	−13.02	29.20	−6.36
1.11	0.00	−2.36	−3.01	2.52	2.52	9.46	−15.51	23.23	−6.06
1.33	0.00	−2.67	−3.40	1.96	1.96	12.08	−17.54	16.92	−5.45
1.56	0.00	−2.90	−3.69	1.34	1.34	14.23	−19.03	10.70	−4.80
1.78	0.00	−3.04	−3.87	0.68	0.68	15.63	−19.94	4.96	−4.31
2.00	0.00	−3.08	−3.93	0.00	0.00	16.11	−20.25	0.00	−4.14
2.22	0.00	−3.04	−3.87	−0.68	−0.68	15.63	−19.94	−3.99	−4.31
2.44	0.00	−2.90	−3.69	−1.34	−1.34	14.23	−19.03	−6.93	−4.80
2.67	0.00	−2.67	−3.40	−1.96	−1.96	12.08	−17.54	−8.86	−5.45
2.89	0.00	−2.36	−3.01	−2.52	−2.52	9.46	−15.51	−9.91	−6.06
3.11	0.00	−1.98	−2.52	−3.01	−3.01	6.66	−13.02	−10.29	−6.36
3.33	0.00	−1.54	−1.96	−3.40	−3.40	4.03	−10.12	−10.24	−6.10
3.56	0.00	−1.05	−1.34	−3.69	−3.69	1.88	−6.93	−10.00	−5.04
3.78	0.00	−0.54	−0.68	−3.87	−3.87	0.49	−3.52	−9.76	−3.03
4.00	0.00	0.00	0.00	−3.93	−3.93	0.00	0.00	−9.67	0.00
4.22	0.00	0.54	0.68	−3.87	−3.87	0.49	3.52	−9.76	4.00
4.44	0.00	1.05	1.34	−3.69	−3.69	1.88	6.93	−10.00	8.81
4.67	0.00	1.54	1.96	−3.40	−3.40	4.03	10.12	−10.24	14.15
4.89	0.00	1.98	2.52	−3.01	−3.01	6.66	13.02	−10.29	19.67
5.11	0.00	2.36	3.01	−2.52	−2.52	9.46	15.51	−9.91	24.97
5.33	0.00	2.67	3.40	−1.96	−1.96	12.08	17.54	−8.86	29.62
5.56	0.00	2.90	3.69	−1.34	−1.34	14.23	19.03	−6.93	33.26
5.78	0.00	3.04	3.87	−0.68	−0.68	15.63	19.94	−3.99	35.57
6.00	0.00	3.08	3.93	0.00	0.00	16.11	20.25	0.00	36.36
6.22	0.00	3.04	3.87	0.68	0.68	15.63	19.94	4.96	35.57
6.44	0.00	2.90	3.69	1.34	1.34	14.23	19.03	10.70	33.26
6.67	0.00	2.67	3.40	1.96	1.96	12.08	17.54	16.92	29.62
6.89	0.00	2.36	3.01	2.52	2.52	9.46	15.51	23.23	24.97
7.11	0.00	1.98	2.52	3.01	3.01	6.66	13.02	29.20	19.67
7.33	0.00	1.54	1.96	3.40	3.40	4.03	10.12	34.41	14.15
7.56	0.00	1.05	1.34	3.69	3.69	1.88	6.93	38.45	8.81
7.78	0.00	0.54	0.68	3.87	3.87	0.49	3.52	41.02	4.00
8.00	0.00	0.00	0.00	3.93	3.93	0.00	0.00	41.89	0.00

TABLE 2.3
Calculation of Member Forces in the A Row Pile at MSL

Description	Members in A Row (kN)	Members in B Row (kN)	Remarks
Surge force	1353.8	1349.4	
Sway force	Zero	Zero	Unidirectional wave
Heave force	1666.6	1661.5	
Roll force	Zero	Zero	Sway force is zero
Centroid of the wave force	129.36 m	129.36 m	
Pitch moment	43,100.98 kN m	42,962.16 kN m	

The center of gravity of vertical cylinders = 75 m.
The center of gravity of horizontal cylinders = 160.5 m.
The center of gravity of the structure = 103.5 m.

Problem 6

Find the total force on a cylindrical member of 1.2 m in diameter; the bottom of the member is fixed to the seabed at a water depth of 23 m. Use Stokes fifth-order wave theory. The wave height and wave length are 10 and 115 m, respectively.

Solution

Given the following:

Wave height H = 10 m
Wave length L = 115 m
Wave number K = 0.05
Wave frequency $\omega = \sqrt{gk \tan kd} = 0.67$ rad/s
Water depth h = 23 m

We first determine the wave height parameter a from the following equation:

$$\frac{kH}{2} = a + a^3 F_{33} + a^5 \left(F_{35} + F_{55}\right) = 0.24$$

Wave elevation from the Stokes theory is given by

$$\eta = \frac{1}{k} \sum_{n=1}^{5} F_n \cos n\left(kx - \omega t\right)$$

Figure 2.12 shows a comparison of wave elevation from Stokes fifth-order wave theory and Airy wave theory for the given wave parameters.

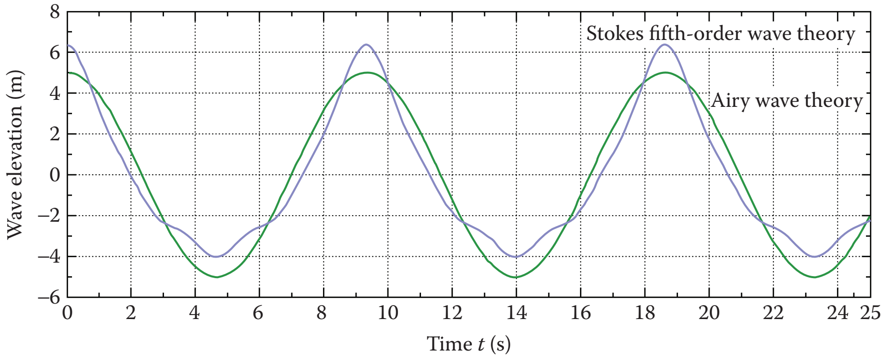


FIGURE 2.12 Comparison of wave elevations.

The velocity and acceleration from Stokes fifth-order wave theory is given by

$$u = \frac{\omega}{k} \sum_{n=1}^{5} G_n \frac{\cosh n(ky)}{\sinh n(kd)} \cos n(kx - \omega t)$$

$$a_x = \frac{kc^2}{2} \sum_{n=1}^{5} R_n \sin n(kx - \omega t)$$

where u and a are the velocity and acceleration coefficients, respectively. The force on the member is calculated using the Morison equation. The member is divided into a number of segments, and the force and moment on each segment are calculated. The total force and moment are then determined as the sum of the force and moment of the individual segment. Figure 2.13 shows the variation of the total force with time.

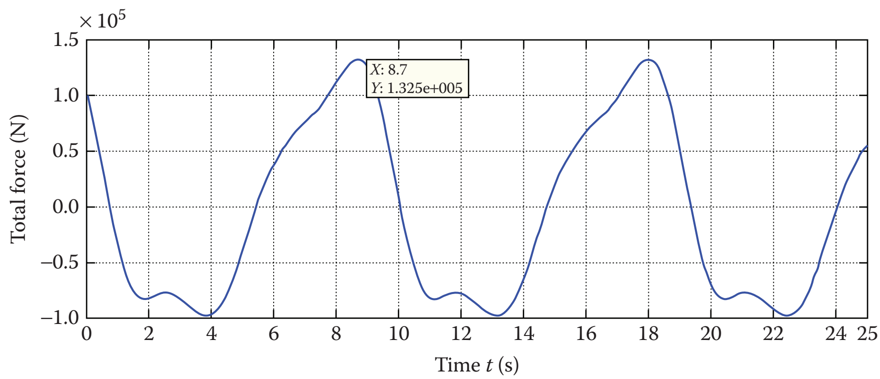


FIGURE 2.13 Total force time history obtained using Stokes fifth-order wave theory.

The magnitude of the total force is 132.5 kN occurring at 8.7 s; this causes the moment at the base as 1.95 kNm, which is acting at a distance of 14.71 m from the fixed base of the cylindrical member (= $1.95 \times 10^6/132.5 \times 10^3$).

Computer Code for Problem 6

```
clc
close all
clear all
format short
H=10;
L=115;
k=2*pi/L;
d=23;
w=sqrt(9.81*k*tanh(k*d))
rho=1025;
D=1.2;
cd=1;
cm=2;
s=sinh(2*pi*d/L);
c=cosh(2*pi*d/L);
%wave profile parameters
b22=c*(2*c^2+1)/(4*s^3);
b24=c*(272*c^8-504*c^6-192*c^4+322*c^2+21)/(384*s^9);
b33=3*(8*c^6+1)/(64*s^6);
b35=(88128*c^14-208244*c^12+70848*c^10+54000*c^8-
21816*c^6+6264*c^4-54*c^2-81)/(12288*s^12*(6*c^2-1));
b44=c*(768*c^10-448*c^8-48*c^6+48*c^4+106*c^2-21)/
(384*s^9*(6*c^2-1));
b55=(1920000*c^16-2627220*c^14+83680*c^12+20160*c^10-
7280*c^8+7160*c^6-1800*c^4-1050*c^2+225)/
(12288*s^10*(6*c^2-1)*(8*c^4-11*c^2+3));
%calculation of a
syms b
a1=solve(-(b+b^3*b33+b^5*(b35+b55))+(k*H/2));
a=a1(1)
F(1)=a; F(2)=a^2*b22+a^4*b24; F(3)=a^3*b33+a^5*b55;
F(4)=a^4*b44; F(5)=a^5*b55
x=0;
t=0:0.1:25;
theta=(k*x-w*t);
for i=1:5
    eta(i,:)=(1/k)*F(i)*cos(i*theta);
end
eta1=sum(eta);                  %stokes fifth order wave
eta2=(H/2)*cos(theta);          %airy wave
plot(t,eta1,t,eta2)  %comparison of stokes and airy wave
```

```
%wave velocity parameters
a11=1/s;
a13=-c^2*(5*c^2+1)/(8*s^5);
a15=-(1184*c^10-1440*c^8-1992*c^6+2641*c^4-249*c^2+18)/
(1536*s^11);
a22=3/(8*s^4);
a24=(192*c^8-424*c^6-312*c^4+480*c^2-17)/(768*s^10);
a33=(13-4*c^2)/(64*s^7);
a35=(512*c^12+4224*c^10-6800*c^8-12808*c^6+16704*c^4-
3154*c^2+107)/(4096*s^13*(6*c^2-1));
a44=(80*c^6-816*c^4+1338*c^2-197)/(1536*s^10*(6*c^2-1));
a55=-(2880*c^10-72480*c^8+324000*c^6-
432000*c^4+163470*c^2-16245)/
(61440*s^11*(6*c^2-1)*(8*c^4-11*c^2+3));
g11=a11*sinh(k*d);
g13=a13*sinh(k*d);
g15=a15*sinh(k*d);
g22=a22*sinh(2*k*d);
g24=a24*sinh(2*k*d);
g33=a33*sinh(3*k*d);
g35=a35*sinh(5*k*d);
g44=a44*sinh(4*k*d);
g55=a55*sinh(5*k*d);
g(1)=a*g11+a^3*g13+a^5*g15; g(2)=2*(a^2*g22+a^4*g24);
g(3)=3*(a^3*g33+a^5*g35); g(4)=4*(a^4*g44); g(5)=5*(a^5*g55);
C1=(8*c^4-8*c^2+9)/(8*s^4);
C2=(3840*c^12-4096*c^10+2592*c^8-1008*c^6+5944*c^4-
1830*c^2+147)/(512*s^10*(6*c^2-1));
Cs=sqrt((9.81/k)*(1+a^2*C1+a^4*C2)*tanh(k*d));
y=0:d;
for j=1:d+1
for i =1:5
   u(i,:)= (w/k)*g(i)*((cosh(i*k*y(j))/
   sinh(i*k*d)))*cos(i*theta);
   U(i)=g(i)*cosh(i*k*y(j))/sinh(i*k*d);
   V(i)=g(i)*sinh(i*k*y(j))/sinh(i*k*d);
end
    R(1)=2*U(1)-U(1)*U(2)-V(1)*V(2)-U(2)*U(3);
    R(2)=4*U(2)-U(1)^2+V(1)^2-2*U(1)*U(3)-2*V(1)*V(3);
    R(3)=6*U(3)-3*U(1)*U(2)+3*V(1)*V(2)-3*U(1)*U(4)-3*V(1)*V(4);
    R(4)=8*U(4)-2*U(2)^2+2*V(2)^2-4*U(1)*U(3)+4*V(1)*V(3);
    R(5)=10*U(5)-5*U(1)*U(4)-5*U(2)*U(3)+5*V(1)*V(4)+5*V
    (2)*V(3);
    for i=1:5
    acc(i,:)=(k*Cs^2/2)*R(i)*sin(i*theta);
    end
    acc1(j,:)=sum(acc);
    u1(j,:)=sum(u);
```

```
    Ft(j,:)=0.5*cd*rho*D*abs(u1(j,:)).*u1(j,:)+cm*rho*(pi/4)*
    D^2*acc1(j,:);
    M(j,:)=Ft(j,:)*y(j);
    end
FT=sum(Ft);
Mo=sum(M);
figure
plot (t,FT)
figure
plot (t,Mo)
```

3 Materials for Ocean Structures

The structural form of ocean structures is unique and expensive by design, installation, commissioning, and operability. Legislation calls for periodic certification of offshore structures. Alternatively, one could rely on the structures to be so well designed and built so that no serious failure develops during his or her working life. Unfortunately, weather conditions and marine growth become more severe than initially predicted values. Wave and current loads have been underestimated in few design cases. Fatigue and corrosion are still debatable subjects. All the above factors result in heavy penalty on the existing weight. Otherwise, this could save a large sum of material costs and ease out the installation procedures. Feedback of actual conditions of structures during their working life would be helpful to do the design successfully. Moreover, marine environment is corrosive in nature, which can cause serious degradation of material and strength of the structural members. Material for construction should be carefully chosen so that the service life of ocean structures is guaranteed. Repair of ocean structures is a multicomplex phenomenon not because of its limited accessibility for repair but also due to its undesirable intervention of the structure for repair and rehabilitation. In this chapter, different types of structural materials that are useful for construction, repair, and rehabilitation are discussed. Recent advancements with respect to repair of concrete structures are also presented.

3.1 INTRODUCTION

Different types of materials are used in the construction of ocean structures. They are used for a variety of purposes: construction, repair, rehabilitation, corrosion protection, and so on. There are different groups such as (1) ferrous and nonferrous and (2) nonmetals, namely, fiberglass, concrete, wood, and glass. In addition, use of composites is on the increasing side in the recent times due to salient advantages they possess in the marine environment. However, concrete can never be ignored as one of the primary materials for the construction of ocean structures. Further, metals can be ferrous and nonferrous: mild steel, copper, aluminum, and brass. Moreover, fiber-reinforced plastics are also used in specific segments of construction of ocean structures. Plastics of thermosetting and thermoplastic resin types, acrylic polythene, and polyvinyl chloride also have their wide applications in the construction of marine structures. Selection of materials for the marine environment is a complex task as no single material characteristic shall rank the suitability of material for offshore applications, in particular. Though it is agreed upon that the basic knowledge of structural characteristics of materials helps an offshore engineer to select appropriate materials for offshore applications, various categorizations of materials having similar structural properties make this task more complicated. Further, a continuous feedback loop of understanding the special requirements of material properties based on the failure in offshore platforms demands a clear understanding of fundamental properties of materials that are commonly addressed by all international codes; advanced properties to make a particular material suitable for offshore applications necessitates advanced learning of those required material properties, making it an unique domain of material research. Figure 3.1 shows the collage of materials for offshore structures. It also shows different types of materials that are available in the market; suitability of these materials to offshore applications categorizes them accordingly. This process of classification of materials can be more precisely addressed after understanding their fundamental properties that make them suitable; later their advanced properties to assess their suitability under particular environmental and loading conditions become important.

Offshore structures are exposed to different environmental loads, and their combinations guide the suitability of materials for the marine environment. For example, wind, waves, ocean currents coupled with thermal gradient, and ice are a few types of environmental loads. In addition, the marine environment also exerts on materials other forces due to chemical, fatigue, stress and corrosion effects, and biofouling effects. Hence, these materials must have properties that ensure survivability (1) in case of any accidents (collision) and (2) in case of excessive loads

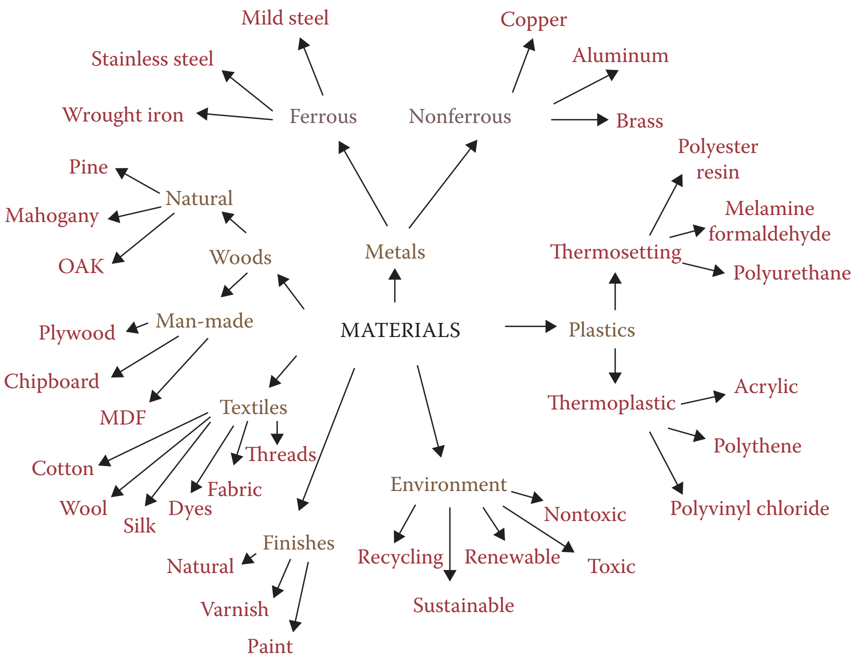


FIGURE 3.1 Materials for marine applications.

during hurricanes; note also that underwater structures have to withstand hydrostatic pressure. Adding to their complexity, structures are also exposed to earthquakes, hurricanes, scouring, and typhoons. Earlier, the major application of materials was only for surface ships; now newly developed ocean systems require materials with special characteristics. Offshore drilling and production platforms, surface buoys, instrument platforms, submarine vehicles, and so on, are the examples of such newly developed ocean systems. As materials are subjected to different types of loads, they require specific properties to sustain these loads and their combinations.

3.2 SELECTION OF MATERIALS

There exists a close relationship between the selection of materials and the type of offshore structures. Various specifications/codes/regulatory agencies guide in the selection process of materials for the marine environment. Referring to the set of recommendations made by the American Bureau of Shipping with respect to the use of materials for surface ships, material selection based on such recommendations is only desirable but not mandatory as it may limit the selection of materials. Under the given wide choice of materials that are available, selection of materials for a specific application becomes very important. Apart from strength, environmental issues related to their recycling characteristics, sustainability, renewable property, and toxic and nontoxic nature are a few important areas of interest to offshore engineering professionals. In the recent times, use of geomembranes and geotextiles for slope stabilization of coastal embankments has also been seen in the literature. As the marine environment has high complexities that influence the performance of materials under the given environmental conditions, the choice of appropriate materials for ocean structures becomes an important engineering decision.

During the selection of materials for ocean structures, the following factors are normally accounted for: (1) physical and chemical properties of materials, (2) cost, (3) fabrication facilities, and (4) maintenance cost. Chosen materials should avoid catastrophic failure. Besides meeting the design requirements, they should withstand hazards (including those arise during operations). During selection, a few material (physical) characteristics are important. Yield strength becomes the first consideration, whereas Young's modulus and ductility follow it. Poisson's ratio is also important as structures are under multiaxis loading. Due to the dynamic nature of environmental loads, fatigue performance and fracture resistance also become equally important. Note that the physical characteristics are presented in the literature based on the data taken from standard specimens. Structure loading in the actual environment differs markedly from those of any tests conducted in controlled laboratory conditions. This warrants change in allowable stress levels for various ocean conditions. This is taken care in design in terms of material allowance, either by increasing the thickness of the members or by using an appropriate factor of safety. Apart from the cost of materials and their availability in the desired cross section and size, it is vital to understand their fundamental characteristics and performance under different environmental conditions to make such an important decision.

3.3 FUNDAMENTAL PROPERTIES

Mechanical properties are generally considered important indices for studying the behavior of metals under loads. For example, strength, hardness, toughness, elasticity, plasticity, brittleness, ductility, and malleability are useful to characterize their behavior under different types of loads. These properties are generally described in terms of the types of forces or stress that the metal must withstand as well as how these forces are resisted. In addition to their homogeneity and isotropic characteristics, their functional degradation in the corrosive environment also needs to be investigated before the choice of material is made. Unlike land-based structures where strength may be one of the most important criteria (in most of the cases, it is the only design criterion), performance-based design of offshore structures makes the material choice inadvertent. Mechanical properties of materials are used as measurements to estimate their behavior under loads. Though it is well known in the literature, it is still advantageous to review it in this section. This will help one to understand the special properties that are required by a material to qualify for offshore applications. Some of the important properties are strength, hardness, elasticity, plasticity, brittleness, ductility, and malleability. These properties are described in terms of the types of forces or stress that the material has to withstand and how are they resisted. Followed by this, the types of forces/stress that are common in offshore structures subjected to environmental loads will become important. Common types of stress are compression, tension, shear, torsion, impact, or a combination of these stresses, such as fatigue. Compression stresses are developed within the material when forces compress or crush the material. For example, a column that supports an overhead beam is in compression, and the internal stresses that develop within the column are compression. Tension (or tensile) stresses develop when a material is subjected to a pulling load; for example, using a wire rope to lift a load or using it as a guy wire to anchor the structure to seabed results in axial tension. "Tensile strength" is defined as the resistance to longitudinal stresses or pull. Shear stresses occur within a material when external forces are applied along parallel lines in opposite directions. Shear forces can separate materials by sliding them apart.

A basic question comes to mind when selecting materials for offshore applications — Under which type of load should one classify the maximum strength? Some materials are equally strong in compression, tension, and shear. However, many materials show marked differences; for example, cured concrete has a maximum strength of 14 MPa in compression but only 3 MPa in tension. Carbon steel has a maximum strength of 386 MPa in tension and compression but maximum shear strength of only 290 MPa; therefore, when dealing with maximum strength, it is always necessary to state the type of loading. Fatigue is induced in members (material) that are stressed repeatedly. Usually, it fails at a critical section at a magnitude of load, which is considerably below its maximum strength in tension, compression, or shear. For example, a thin steel rod can be broken by hand by bending it back and forth several times in the same place; however, if the same force is applied in a steady motion (not bent back and forth), the rod cannot be broken. Tendency of a material to fail under reversal of (nature of) forces at the same point is known as fatigue. Strength is the property that enables a material to resist deformation under load. Ultimate strength of the material is the maximum strength a material can withstand, where tensile strength is a measurement of the resistance to being pulled apart when placed under tensile load. “Fatigue strength” is the ability of materials to resist stresses that are reversal or cyclic in nature and is expressed by the magnitude of an alternating stress for a specified number of cycles. Impact strength is the ability of a metal to resist suddenly applied loads. Hardness is the property of a material to resist permanent indentation. Because there are several methods of measuring hardness, it is always specified in terms of the particular test that is used to measure this property. For example, Rockwell, Vickers, or Brinell are some of the methods of testing harness. Of these tests, Rockwell hardness is the one most frequently used. The basic principle used in the Rockwell test is that a hard material can penetrate a softer one; the amount of penetration is measured and compared to scale. For ferrous metals, which are usually harder than nonferrous metals, a diamond tip is used and the hardness is indicated by a Rockwell “C” number. For nonferrous metals, which are softer, a metal ball is used and the hardness is indicated by a Rockwell “B” number. Toughness is the property that enables a material to withstand shock and to be deformed without rupturing. This is considered to be a combination of strength and plasticity. Elasticity is the ability of a material to return to its original shape on removal of load. Theoretically, elastic limit of a material is the limit to which a material can be loaded and still recover its original shape after the load is removed. However, plasticity is the ability to deform permanently without breaking or rupturing. This property is in converse to that of strength. By careful alloying of metals, combination of plasticity and strength is used to manufacture large structural members that are commonly used in marine construction. Brittleness is the opposite of the property of plasticity. A brittle metal is the one that breaks or shatters before it deforms. Although cast iron and glass are good examples of brittle materials, brittle metals possess higher compressive strength in comparison with their tensile strength. Under the braces of performance-based design of structures, ductility plays an important role, which enables the member to stretch, bend, or twist without cracking or breaking. Malleability is the property that enables a material to deform by compressive forces without developing defects. A malleable material is one that can be stamped, hammered, forged, pressed, or rolled into thin sheets.

Compressive or tensile stresses are commonly generated on structural members in addition to bending, shear, and torsion; impact forces are occasional. Interestingly, the behavior of members and the capacity of material to safely disburse the combination of stresses are the most critical aspects in the design. The combination of various forces results in a complex behavior of the material. One classic example could be dynamic tether tension variations that can arise in tethers of tension leg platform (TLP), which can result in fatigue failure. Ductile materials such as steel are a favorite choice and widely used in offshore construction. The ductile behavior of steel is well understood from the stress–strain curve plotted under uniaxial stress; a typical curve is shown in Figure 3.2. Most ductile metals other than steel do not have a well-defined yield point. For these materials, the yield strength is typically determined by the “offset yield method.” A line is drawn parallel to the initial slope, from 0.2% of the maximum strain value. The point of intersection of this line with the stress–strain curve determines the yield point as shown in the figure. This is also called

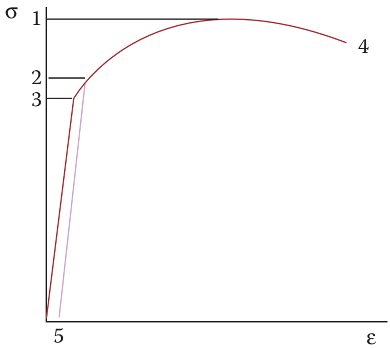


FIGURE 3.2 Stress–strain curve of mild steel: (1) ultimate strength; (2) yield strength; (3) proportional limit; (4) rupture; (5) offset strain (typically 0.2%).

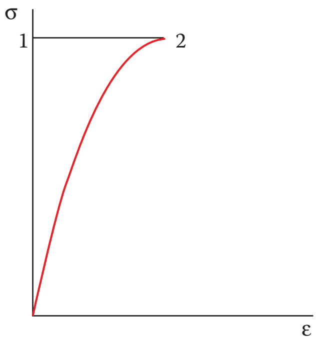


FIGURE 3.3 Stress–strain curve for brittle material: (1) ultimate strength; (2) rupture.

0.2% proof stress. The ratio of the maximum strain to that at yield is called "ductility ratio," which is an important engineering property in the design of offshore structures. This will govern the rotation capacity of plastic hinges, assumed to be formed at critical sections in the plastic analysis. Interestingly, the strength ratio, which is the ratio of ultimate strength to yield strength, is also important for design. However, the ductility ratio governs the decision of choice of materials for offshore structures as large ductility ratio is an index of energy absorption. In case of brittle materials such as concrete or ceramics, a well-designated yield point is not seen. Figure 3.3 shows a typical stress–strain curve for brittle materials. In such cases, rupture and ultimate strength are the same for these members. Area underneath the stress–strain curve with respect to the abscissa is an index of toughness of the material.

3.4 EFFECTS OF THE MARINE ENVIRONMENT ON MATERIALS

Earlier, the major application of materials is only with surface ships. New offshore structures, such as offshore drilling and production platforms, surface buoys, instrument platforms, and submarine vehicles that are developed in the past, require materials with special characteristics. Large deformation induced in the design concept such as TLPs demands high ductility in tether materials. Articulated towers demand more fatigue resistance in the materials used for articulated joints. Offshore structures are exposed to a combination of a variety of environmental loads, as discussed in Chapter 2. The material strength degrades under the effects caused by the environmental loads. Few of them are chemical, fatigue, stress concentration, corrosion effects, and biofouling effects. It is desired that materials must have properties, which ensure survivability in case of any accidents (collision). In case of excessive loads during hurricanes, they should be able to withstand high hydrostatic pressure. Given different combinations of forces acting on the members of the offshore structures, it is imperative to understand the important characteristics that materials should possess to be qualified for construction of ocean structures. There exists a close relationship between the selection of material and the type of ocean structure. Various specifications suggested by codes and other regulatory agencies are only desirable general recommendations but not a requirement as they may impose a serious limitation on the suitability of materials for the marine environment. Materials must have properties that ensure survivability in the marine environment. They should be able to perform their intended function under special conditions: collision or impact, excessive loading that occurs during hurricanes, and special kinds of environmental forces. In addition, materials should withstand a severe hydrostatic pressure. All the above special conditions make the choice of the materials to a highly limited situation. As the algorithm of selection of materials for ocean structures grows complex in nature, one needs to understand the factors based on which materials can be chosen for such special applications. Apart from understanding the list of special properties of materials that are required to sustain the combination of different environmental loads in the critical marine environment, it is important to know about those specific properties, which are demanded from the materials if they have got to qualify for ocean structures. Factors that become vital for selection of materials include the physical and chemical properties of the materials, the cost and availability in large quantity, and the availability of fabrication facilities for the chosen shape, size, and geometry. Materials chosen for marine construction should be easy to fabricate and transport to the offshore site, which is one of the major constraints in material selection. It is also important to note the expected maintenance must be looked upon as the survivability of these kinds of structural systems in the complex ocean environment is very important. Hence, selection of materials is closely governed by the advice or recommendations given by various international codes. These codes indicate a variety of engineering properties that guide the selection for various applications.

Materials should be chosen not to initiate any catastrophic failure; this means that failure, if at all occurs, even at a lower probability, should not be sudden or immediate, or it may lead to loss of assets that are invested on offshore platforms. There can be hazards that arise from operational errors because oil production process is complex. It involves a lot of electromechanical and electrical equipment, which need to be synchronized in a specific format to perform drilling and exploration successfully. Under such a complex operation, materials selected for construction of the platform and other accessories such as pipe lines and rigs should be capable of withstanding the unforeseen hazards besides meeting the design requirements.

3.5 DESIGN CONSIDERATIONS

While selecting a material for marine applications, the following material (physical) characteristics are important in the order of priority as follows: (1) yield strength, (2) modulus of elasticity, (3) Poison's ratio, (4) fatigue performance, and (5) fracture resistance. In addition, the above material properties are only indicative. For example, if the material has a yield value of 250 MPa, it is only an indicative value for a standard specimen. However, in reality, this value may vary depending on the nature of loading and other conditions. Therefore, it is important to understand that there is a significant difference between the material properties under standard test conditions and those in the marine environment. It therefore warrants a change in allowable stress levels for the marine environment and is taken care of through "material allowance." For example, an increase in thickness is seen as allowance in the design to take care of uncertainties that are caused by change in the conditions in the ocean environment. Alternatively, one can also use partial safety factor for materials to account for such uncertainties.

3.6 STEEL CLASSIFICATION

Steel is a common material used for marine construction. It is also one of the widely used materials in ocean structures. It is classified in several ways to make it suitable for particular set of applications in the ocean environment; in fact, classification enables us to identify their areas of application. Steel is classified based on its composition, manufacturing methods, finishing methods, microstructure, strength, heat treatment, and the product form. Based on the composition, it is classified as carbon, low alloy, or stainless steel. Based on the manufacturing methods, it is classified as electric furnace or open hearth process. Based on the finishing methods, it is classified either as hot rolling or as cold rolling. Based on the microstructure, it is classified as ferritic, pearlitic, or martensitic. Based on the strength required, different codes classify them in many ways. Based on heat treatment carried out on steel to achieve desired characteristics, it is classified as annealing, quenching, and tempering. Based on the product form, it is classified as bars, plates, sheets, strips, tubes, and other desired shapes named after the shape of the cross section as L, Tee, and so on.

Classification of steel based on strength is intrinsic in the design codes. Depending on the component of the member and type of load combinations, codes classify them according to their yield strength. Looking at the further level of classification based on carbon content, steel is classified as low, medium, high, and ultrahigh carbon steel. Low carbon steel contains less than (or equal to) 0.3% of carbon and does not contain other elements such as chromium, cobalt, and nickel. Medium carbon steel has a percentage of carbon varying from 0.3 to 0.6, whereas high carbon steel has a content varying from 0.6% to 1%. Ultrahigh carbon steel has about 1.25%–2% of carbon content. Carbon content influences the strength of steel; low carbon steel is also referred to as low-strength steel whose yield strength is less than 415 MPa. This type of steel is widely recommended for the hull structure of platforms, fittings, tanks, instrument ancillaries, and buoys. Medium-strength steel has a medium percentage of carbon whose yield strength is about 1035 MPa. It is widely used for fabricating icebreakers and buoys for arctic regions. High-strength steel has a yield strength greater than 1035 MPa. For example, maraging steel has a yield strength in the range of 1–2 GPa. This type of steel is relatively ductile and manufactured by heat treatment to improve its specific properties such as ductility. The major standards that are used for offshore construction are prEN 10225, BS 7191, and Material Data Sheets of NORSOK, which is Norwegian standard, applicable primarily in European countries. American Petroleum Institute (API) standards are primarily used in American and Asian regions, although there is no bar of using any specific standard on any part of the world. It is important to note that most of the classification of steel and its applications, as recommended by various codes, corresponds to each other. For example, NORSOK refers to prEN 10225, which itself is based on BS 7191.

3.7 GROUPS OF STEEL

Steel is grouped according to the strength level and welding characteristics. Group I refers to steel with a specified minimum yield strength (SMYS) of 280 MPa or less and carbon equivalent of 0.4% or less. Group II refers to steel with SMYS less than 360 MPa and carbon equivalent of 0.45% and higher. Using this steel for ocean structures requires the use of low hydrogen welding process, in particular. Group III refers to high-strength steel with SMYS greater than 360 MPa. This group of steel requires special welding procedures during fabrication. Further, members fabricated with this group of steel should also be investigated for fatigue-related problems, as a part of the routine design check. As seen in the grouping of steel, strength is considered as the most important characteristic to decide the choice of steel for different applications. However, it is also important to note that steel should possess superior low-temperature toughness for the base metal to avoid brittle failure of welded joints. Codes specify the impact test properties based on Charpy impact test results. They also demand that steel should possess good crack tip opening displacement (CTOD) properties. CTOD is one of a family of fracture mechanics tests that measures resistance of a material to the crack growth.

Toughness is another important structural property of steel, which is vital in selection of steel for marine applications. It is described as a measure of resistance to failure in the presence of a crack, notch, or similar stress concentrator. High toughness therefore is preferred for offshore steel. Therefore, this becomes as one of the important requirements for selecting a material for the offshore structural system as there exists a high probability of stress concentration at the joins of members in offshore structural systems. A high toughness material is one where a considerable amount of plastic deformation is required at the crack tip before the crack advances. Conversely, if the application of stress causes the elastic failure of atomic bonds at the crack tip, relatively little energy of deformation is involved, and the result is a brittle fracture. Toughness is expressed in terms of impact and fracture toughness. Impact toughness is the energy measured in joules and commonly related to the Charpy V-notch test, whereas fracture toughness is computed based on CTOD or J-integral test. The latter is useful in prescribing the critical stress intensity factor required for the design.

Steel is also grouped according to the notch toughness characteristics, computed based on the impact tests. They are grouped as class C, class B, and class A. Class C refers to a group of steel for which no impact tests are specified. Applicability of this group of steel is limited to primary structural members involving limited thickness, moderate forming, low restraint, modest stress concentration, and subjected to quasi-static loading only. Few examples are piles, bracings in jacket platforms, and legs, deck beams, and legs. Class B refers to a group of steel that are recommended for members with larger thickness and are subjected to high stress concentration, impact loading, and so on. Class A refers to a group of steel that are recommended for use at subfreezing temperatures. Interestingly, the codal provisions guide the selection of steel as a construction material for ocean structures. Recommendations made by different codes are very prescriptive in terms of type of members, manufacturing process, loading encountered by the members, and so on. Thanks to the constant update on codal provisions from the steel manufacturers, offshore structures are relieved off from one important source of error, which is wrong material specification. This reduces the risk encountered by offshore structures from the material incapability to resist the encountered loads.

Fixed offshore structures used medium-grade structural steel with a yield strength of 350 MPa. Existing codes and standards widely cover these groups of steel through prescriptive documentation. In recent years, there has been an increasing use of higher strength steels for these installations. The primary benefit is the increase in strength-to-weight ratio, which results in savings of cost of materials. Jacket platforms are constructed using steel with a yield strength ranging from 400 to 450 MPa and installed in the North Sea. However, to date, fatigue-sensitive components such as tubular members (joints, in particular) are been fabricated using medium-strength steel only. This is mainly because of the better know-how of their fatigue performance, which is seriously lacking in case of high-strength steel. In the recent times, increased application of high-strength steel is seen in the fabrication of jack-up platforms. Steel with a yield strength ranging from 500 to 800 MPa is used to fabricate the legs, racks and pinions, and spud cans of jack-up platforms. High-strength steel is commonly used for tethers in compliance with offshore structures such as TLPs and for mooring lines in semisubmersible module offshore drilling units. On average, about more than 40% of offshore structures use steel with a yield strength of more than 350 MPa. Table 3.1 gives a brief

TABLE 3.1
High-Strength Steels Used in Offshore Structures

Strength (MPa and Grade)	Process Route	Application Area
350 (X52)	Normalized TMCP	Structures
Structures and pipelines		
450 (X65)	Q&T TMCP	Structures and pipelines
550 (X80)	Q&T TMCP	Structures and mooring pipelines
650	Q&T	Jack-ups and moorings
750	Q&T	Jack-ups and moorings
850	Q&T	Jack-ups and moorings

Note: Q&T, quenching and tempering; TMCP, thermomechanical controlled processing.

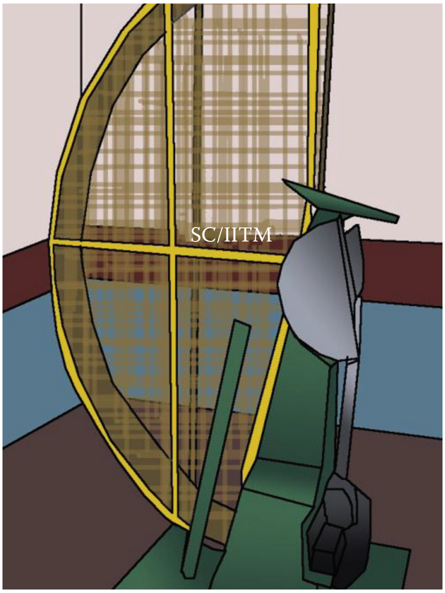


FIGURE 3.4 Schematic view of a Charpy test setup.

summary of steel used in offshore structures; yield strength and grade of steel used along with the location of usage are also mentioned. It can be seen from the table that steel with a higher yield strength is used for jack-up and mooring lines. Different types of failure modes are encountered by the material when they are exposed to offshore structures. For example, buckling, corrosion, creep, fatigue, hydrogen, embrittlement, impact, mechanical overload, stress concentration, cracking, thermal shocks, wear, and yielding are very interestingly a wide variety of failure modes. Selection of material other than steel is instead based on Charpy V-notch test. Figure 3.4 shows a schematic view of the test setup.

3.7.1 Charpy Test

This impact test is a very simple experiment conducted in the laboratory to understand the impact strength of any material. It is a standardized high strain rate test and useful to determine the amount of energy absorbed by the material during fracture. Absorbed energy is given as an index of notch toughness of the material. It is a tool to study the temperature-dependent, ductile–brittle transition. The test gives results only on a comparative scale. The Charpy impact test procedure consists of a simple pendulum of a known mass and length (Kayano et al., 1993). It is dropped to cause an impact on the specimen of the material. The energy absorbed is inferred by comparing the difference in height of the hammer before and after fracture. Notch in the sample affects the test results. Results are influenced by the notch geometry. These tests provide both quantitative and qualitative results on a comparative scale. Quantitative results given by Charpy test indicate the energy required to fracture the material. This can be used to measure the toughness of the material. Strain rate can also be studied at which the material fails. The test gives a ductile–brittle transition temperature, which is defined by the significant change in the energy level required to fracture the material. It also gives some qualitative results, which are useful to derive the ductility of the material indirectly. If the material breaks on a flat plane, then it is considered a brittle material; if the material breaks on a jagged edge or shear lips, then it is considered a ductile material. Usually, fracture will be a combination of flat and jagged edges, which is helpful to determine the percentage of brittleness and ductile failure. The mechanical properties of steel should be based on the tensile test as well as Charpy's V-notch test. International codes recommend these two test results as reference for selecting materials for offshore construction. Charpy's V-notch test results should be obtained with a longitudinal axis parallel to the direction of rolling. Further, steel should be heat treated, which is also one of the prerequisites to use them in offshore applications.

3.7.2 Weldability

Generally, weldability is a very important characteristic, which is a requirement for an offshore structural member. This can be computed by cold-cracking susceptibility and carbon equivalency as given in the following two equations:

$$C_{\text{eq}} = C + \frac{M_n}{6} + \frac{N_i + C_u}{15} + \frac{C_r + M_o + V}{5}\%$$

$$p_{cm} = C + \frac{S_i}{30} + \frac{M_n}{20} + \frac{C_u}{20} + \frac{N_i}{60} + \frac{C_r}{20} + \frac{M_o}{15} + \frac{V}{10} + 5B\%$$

Formability, weldability, and toughness are the important criteria to be checked for recommending steel for offshore drilling units, in particular. In case materials are used in combination of steel, one must also check for the galvanic effects before it is recommended for offshore drilling units.

3.8 ALUMINUM

Aluminum is a phenomenally attractive material for offshore structures. It is widely used in hulls, deckhouses, and hatch covers of commercial ships. It is used in fabrication of ladders, railings, gratings, windows, and doors due to their maintenance-free property. Passenger vessels utilize large quantities of aluminum in superstructure and equipment. High-speed boats, in particular, are constructed using 5xxx alloy of aluminum. Aluminum alloys have strength comparable to that of mild steel, which enables to design the members with an equivalent strength of that of steel while reducing the weight up to about 60%. As the specific gravity of steel is about 2.5 times more than that of aluminum, it results in significant savings in dead weight. In addition, aluminum does not require any protective coatings, whereas steel needs to be protected from corrosion. The yield strength of aluminum alloys of 5xxx series, which are widely used in marine applications, varies from 100 to 200 MPa.

Aluminum is commonly used in pressure vessels in liquid natural gas (LNG) transport ships, where it is insulated against temperature loss or transfer.

Aluminum is also used as one of the alternate materials for construction in the arena of deep-sea mining. Deep-sea mining has limitations in terms of hyperbaric and low-temperature conditions for long-term operations. Aluminum is used for fabricating the subcomponents in the mechanical equipment used in deep-sea applications. In particular, lightweight crawlers that have been recently attempted in deep-sea mining is a candidate of favorable use of aluminum. Suitable aluminum alloys and filler wires are used in such equipment to improve the postweld strength in the heat-affected zone. Corrosion resistance of aluminum is one of the most attractive features, which makes it suitable for deep-sea mining crawler structures. Figure 3.5 shows a typical traction unit, which has aluminum components. Strength, malleability, resistance to corrosion, good conductor of heat, electricity, and capability to be polished to give high-reflective surfaces are the desired characteristics of aluminum. These are also vital for choosing them for deep-sea applications. Temper designations are used to indicate the cold-worked or heat-treated conditions of aluminum alloys. Table 3.2 shows the details of temper designations used in aluminum alloys. It is seen from the table that the structural properties of aluminum are customized based on the manufacturing process to suit to marine applications.

An alloy of aluminum has a unique designation system, which enables us to understand its alloying composition. The designation system consists of four digits, namely, XXXX. The first digit refers to the principal alloying constituent(s); the second digit signifies the variations of initial alloy; the third and fourth digits indicate individual alloy variations. However, the number has no significance, but it is unique. The following designations of aluminum alloy clarify the nomenclature: 1xxx refers to pure aluminum, whose purity is 99% and above; 2xxx indicates aluminum–copper alloys; 3xxx indicates aluminum–manganese alloy; 4xxx refers to aluminum–silicon alloys; 6xxx refers to aluminum–magnesium–silicon alloys; 7xxx refers to aluminum–zinc alloys; 8xxx refers to aluminum with other elements; and 9xxx refers to unused series. As we understand that an alloy is simply a mixture

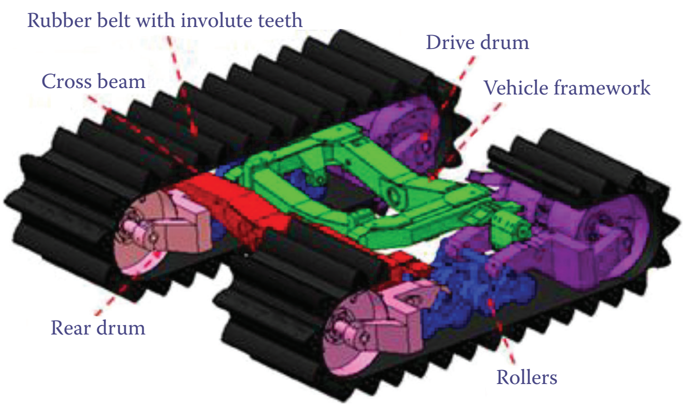


FIGURE 3.5 Typical traction unit for deep-sea mining.

TABLE 3.2
Temper Designations of Aluminum Alloys

Designation	Condition
F	As fabricated
O	Annealed
H_1	Strain hardened only
H_2	Strain hardened and partially annealed
H_3	Strain hardened and thermally stabilized
W	Solution heat treated
T_1	Cooled from an elevated temperature for shaping process and naturally aged
T_2	Cooled from an elevated temperature for shaping process and cold worked and naturally aged
T_3	Solution heat treated, cold worked, and naturally aged
T_4	Solution heat treated and naturally aged
T_5	Cooled from an elevated temperature for shaping process and artificially aged
T_6	Solution heat treated and artificially aged
T_7	Solution heat treated and stabilized
T_8	Solution heat treated, cold worked, and artificially aged
T_9	Solution heat treated, artificially aged, and then cold worked
T_{10}	Cooled from an elevated temperature for shaping process and cold worked and artificially aged

of metals melted together to form a new metal, whose characteristics differ from those of the parent metals. Aluminum alloy is primarily pure aluminum, mixed with different alloying elements that give rise to an entire range of materials. Each of the alloys is designed to maximize a particular characteristic such as strength, ductility, formability, machinability, or electrical conductivity.

3.8.1 Alloying Elements

Commercially pure aluminum is a white, lustrous, lightweight, and corrosion-resistant metal. Aluminum alloys contain the principal alloying ingredients such as manganese, magnesium, chromium, magnesium, and silicon. However, these alloys in which a substantial percentage of copper is used are more susceptible to corrosive action. Among all the aluminum alloys, the non-heat-treatable aluminum–magnesium alloys (5xxx series) are the most suitable materials for marine applications. Magnesium, as the main alloying constituent, lends itself to a reasonable strength for marine applications. Corrosion resistance of these alloys makes them the most suitable materials for shipbuilding as well. The magnesium content of 5xxx alloys significantly influences the mechanical properties such as yield strength, tensile strength, and ductility. One of the principal reasons for the increase in strength is due to the formation of intermetallic particles of aluminum–magnesium, which reinforce the alloy. An increase in magnesium content adds strength to the alloy, but if added beyond 4% (approximately), corrosion resistance of the alloy gradually decreases. This is because intermetallic particles of the alloying elements will precipitate at the grain boundaries at higher percentage of magnesium. These particles are anodic with respect to aluminum and therefore results in electrochemical imbalance in the grains. This leads to intergranular corrosion, causing pitting and weight loss; it may also lead to stress–corrosion cracking.

Heat treatment is carried out to improve the mechanical properties of the alloy by developing the maximum practical concentration of the hardening constituents in solid solution. It involves heating above the critical temperature, holding the constituents at that high temperature for a designated period, and then quenching abruptly. The faster rate of cooling the alloy enables to retain a supersaturated solid solution of alloying constituents without introducing adverse metallurgical or mechanical conditions. Most common quenching media are water, air blast, soap solutions, and hot oil. Precipitation hardening, alternatively referred to as age hardening, is used on aluminum, copper, nickel, magnesium, and some stainless steel alloys to improve their mechanical properties. Aging process is divided into two main categories: natural aging and artificial aging. Heat-treatable alloys change their properties when stored at room temperature after solution heat treatment and quenching. This is known as natural aging. In case of artificial aging, by heating the solution heat-treated material to a temperature above the room temperature and holding it, precipitation accelerates. This improves its strength further compared to that of natural aging. Preheating or homogenizing is carried out to reduce chemical segregation of cast structures and improve their workability. It also reduces brittleness in the cast structure. Annealing aids in workability by softening aluminum and heat-treated alloy structures to release residual stresses. This also helps stabilize the mechanical properties and dimensions of product. Table 3.3 shows aluminum materials with

TABLE 3.3
Aluminum Materials for Deep-Sea Mining

Alloy	Description	Yield Strength (MPa)	Ultimate Tensile Strength (MPa)	Elongation (%)	Bend Angle	Impact Energy (kJ)
AA5083 MIG preferred for deep-sea mining	Base material	245	350	20	125	35
	Multipass weld	180	320	16	135	26
	Single-pass weld	158	265	14	100	22
AA6082 MIG	Base material	285	305	10	60	18
	Weld with ER4043 fillers	175	200	7	65	15
	Weld with ER5183 fillers	170	205	8	140	30
AA5083 TIG	Base material	245	350	20	xx	xx
	Welded material	200	335	20	xx	xx
AA6061 Plasma arc weld	Welded material	155	205	8	xx	xx

Note: xx - not applicable.

metal inert gas (MIG) welding and tungsten inert gas (TIG) welding. MIG welding is a process in which an electric arc is formed between a consumable wire electrode and the metal workpiece. Electrode heats up the workpiece, melts, and joins them. TIG weld uses a nonconsumable tungsten electrode to produce weld. The mechanical properties of different alloys that are useful for deep-sea mining applications are highlighted.

As seen from the Table 3.3, AA5083 MIG welds have shown a joint efficiency of about 90% based on UTS tests. Impact strength, bend, ductility and ultimate strength of the alloy with multipass welds are comparable with that of the base material. Multipass welds have shown a better microstructure properties compared to single-pass weld. It is seen in the literature that AA5083-H116 are recommended for crawler structures in deep-sea mining.

3.9 TITANIUM

Titanium as an element has been in recognition for over 200 years. It gained strategic importance in the past 60 years. In 1938, Dr. Kroll developed a process for manufacturing titanium. Reduction of titanium chlorate, first with calcium and later with sodium and magnesium under inert atmosphere, was the process used. Titanium alloys are commonly available in wrought products; most of them are used in aerospace industries. The melting point of titanium is 1678°C, which is higher than that of steel. The specific gravity of titanium is 4.5, which is about 55% of that of steel. It has an hexagonal close packed (HCP) crystal structure with an atomic weight of 47.88 and an atomic number of 22. Being a light metal with lower density, it has higher strength-to-weight ratio, which is vital for offshore applications. Even though titanium is highly reactive, on exposure to the marine environment, it reacts with atmospheric oxygen to form a protective layer; this makes it corrosion free. With higher melting point and increased strength at higher temperature, its applicability is better than that of aluminum as it cannot be used at higher temperature. Titanium and its alloys can be used up to 540°C. Titanium undergoes allotropic transformation from α-Ti to β-Ti at 882°C; alloying elements influence this transformation. Therefore, a wide variety of microstructure can be produced by heat treatment. Manipulation of these crystallographic variations through alloying additions and thermomechanical processing is the basis for the development of a wide range of titanium alloys. Titanium can readily form an alloy with other elements because it is a transition metal with an incomplete"d" shell; therefore, it forms solid solutions with almost all substitute elements. Titanium alloy results in the formation of solid solutions and compounds with metallic, ionic, and covalent bonding. These alloys have good fatigue resistance and high fracture toughness. Other useful properties of titanium alloys include nonmagnetic, good heat transfer property with low-heat thermal conductivity, low coefficient of thermal expansion (about 9–11 ppm/°C), and nontoxic, which make them biologically compatible.

Titanium is as strong as steel but about 45% lighter. High strength, low density, and corrosion resistance of titanium contribute toward its cost reduction in long-term maintenance even though the initial cost is prohibitively high. Titanium is commonly used in small submersibles. High strength-to-weight ratio is the most attractive features of titanium; good corrosion resistance and relatively high modulus of elasticity are additional benefits. It is used for fabricating surfaces, which cannot be painted such as propellers, special valves, and hot and cold pipes. As titanium requires no corrosion allowance, equipment is designed to satisfy the minimum requirements for mechanical strength and handling. Titanium has an outstanding corrosion resistance even if placed in heavily polluted seawater. Titanium alloys are classified into three groups: alpha, alpha–beta, and beta; the classification depends on its microstructure. Alpha stabilizers such as peritectoid result in solid solutions, whereas beta stabilizers such as isomorphous and eutectoid form either solid solutions or compounds, respectively. Eutectoid stabilizers form compounds with manganese, iron, chromium, and nickel. Depending on the type and amount of impurities or alloying additions, the transformation temperature of α- to β-Ti can be raised or lowered. Addition of alloying elements divides the single temperature for equilibrium transformation into two temperatures: α-transus and β-transus.

3.9.1 Classifications

Titanium alloys are classified based on phase diagrams as type I, type II, and type III. In type I phase diagram, addition element has greater solubility in α than in β. Addition element stabilizes α, that is, $(\alpha+\beta)$ region, is elevated to higher temperatures by increasing the alloying element concentrations. In case of type II phase diagram, addition element has greater solubility in β than in α. It stabilizes β, that is, $\alpha/(\alpha+\beta)$ and $\beta/(\alpha+\beta)$ boundaries, are depressed to lower temperatures with increasing concentration of addition element. Phase diagram includes a line to indicate the temperature at which β begins to undergo a diffusionless transformation to a supersaturated α, that is, α'; this occurs usually on rapid cooling. Type I alloys contain only transition elements to stabilize beta, whereas type II alloys contain both transition and nontransition elements to stabilize alpha. The alpha phase is stabilized by elements with an electron-to-atom ratio less than 4 (e.g., oxygen, carbon, nitrogen, and aluminum). Beta phase is stabilized by elements with an electron-to-atom ratio greater than 4 (e.g., iron, molybdenum, and manganese). Elements with an electron-to-atom ratio equal to four are neutral (e.g., zinc, tin, and silicon).

3.9.2 Effect of Alloying Elements

Aluminum, as an alloy with titanium, increases the tensile strength, creep strength, and elastic modulus. However, if the content of aluminum increases more than 6% by weight, an intermetallic compound called α_2 is formed, which is brittle. Tin, if alloyed with titanium, dissolves in both apha and beta. It is a weak alpha stabilizer, which results in solid solution and hardens the alloy with aluminum, without causing embrittlement. Zirconium, if alloyed with titanium, retards the rate of transformation as it is a weak beta stabilizer. If added more than 6% by weight, it reduces ductility and creep strength. Molybdenum is a strong beta stabilizer and, if alloyed, helps to increase hardness but reduces long-time, high-temperature strength; weldability is also subsequently reduced. Niobium improves high-temperature oxidation resistance, being a strong beta stabilizer. Iron, if alloyed with titanium, reduces creep strength, whereas carbon widens the temperature range between alpha-transus and beta-transus; it is a strong aplha stabilizer.

Pure titanium has low strength and high ductility. Addition of small amounts of elements to their chemical composition increases their strength and decreases ductility. An increase in disruptive failures of stainless steel and copper-based alloys in the marine environment raised a serious concern about safety. Thanks to the manufacturing industry for sharing information about the structural properties, chemical composition and fabrication experience of titanium in the recent times. Due to the availability of information, design engineers are attracted toward its usefulness for structural engineering applications as well. It is also evident that in flowing or static seawater at temperatures up to 130°C, titanium surfaces are immune to corrosion, whereas other metals and alloys corrode significantly. Titanium is immune to crevice corrosion up to at least 70°C in seawater, whereas steel tends to corrode even at 10°C. In recent times, titanium has been available at a very competitive and stable price in the market. One of the encouraging aspects of titanium is availability of the fabrication experience. People with fabrication skills in titanium with respect to the different methodologies of fabrication are available as a technical workforce to handle the construction and fabrication challenges. In the presence of such scenarios, titanium alloys are used for pipeline fittings and systems. Although titanium enables a maintenance-free system, its yield strength can be as high as 400 MPa, which is equivalent to that of steel. Good corrosion resistance and relatively high modulus elasticity are additional benefits of titanium alloys. Table 3.4 shows a comparison of properties that titanium posses with other competitive materials used in offshore construction. With reference to the table, it is seen that titanium alloys are almost resistant to all types of corrosion that are common in the marine environment. Titanium has a very clear edge as a construction material for marine applications. The only demerit is the increase in the initial investment, but by including the cost of

TABLE 3.4
Titanium: A Comparison

Mode of Corrosion	Copper-Based Alloy	Stainless Steel 316	Stainless Steel 6 Mo and Duplex	Titanium Alloys
General corrosion	Resistant/susceptible	Resistant	Resistant	Resistant
Crevice corrosion	Susceptible	Susceptible	Susceptible (>25°C)	Resistant up to 80°C
Pitting corrosion	Susceptible	Susceptible	Resistant	Immune
Stress corrosion	Susceptible	Susceptible (>60°C)	Resistant	Resistant
Corrosion fatigue	Susceptible	Susceptible	Susceptible	Immune
Galvanic corrosion	Susceptible	Susceptible	Resistant	Immune
Microbiological corrosion	Susceptible	Susceptible	Susceptible	Immune
Weld/heat-affected zone (HAZ) corrosion	Susceptible	Susceptible	Susceptible	Resistant
Erosion corrosion	Susceptible	Susceptible	Resistant	Highly resistant

maintenance over a service life of about 15 years, titanium and aluminum will become a more competitive, affordable, and replaceable material in comparison with steel.

Titanium can be hot worked, but the reaction of titanium with atmospheric gases is an important factor in hot working. It absorbs hydrogen above 300°F, oxygen above 1300°F, and nitrogen above 1500°F. Absorption of these gases in larger quantities leads to embrittlement. Hot working is therefore done in an oxidizing atmosphere to avoid hydrogen absorption; oxygen-embrittled layer is removed after hot working. Heat treatment is done on titanium to improve fracture toughness, fatigue strength, and high-temperature strength. It also reduces residual stresses developed during fabrication; this is called stress relieving. Annealing helps to produce an optimal combination of ductility, machinability, and dimensional and structural stability. Solution treating and aging increase its strength.

The first fishing boat, fabricated in all-titanium, was launched in Japan in 1998. Weighing only 4.6 tons, the boat was 12.5 m long, which could travel at 30 knots with improved fuel efficiency. Savings in the operational cost include no necessity for hull painting and easier removal of biofouling. Progressive degradation of glass fiber boats by repeated fouling and cleaning is an ongoing penalty for the Japanese fishing fleet; titanium is used as an alternate material. Titanium is increasingly used in marine applications due to its high strength, high toughness, and phenomenal corrosion resistance. The majority of offshore applications use titanium for submarine ball valves, pumps, heat exchangers, hull material for deep-sea submersibles, water jet propulsion systems, propeller shafts and propellers, exhaust stack liners, navel armors, underwater manipulators, high-strength fasteners, yacht fittings, shipboard cooling and piping systems, and many other components in the ship design.

3.10 COMPOSITES

Composites are materials consisting of two or more constituents. The constituents are combined in such a manner that they keep their individual physical phases and are not soluble in each other; they also do not result in the formation of a new chemical compound. One constituent is called the "reinforcing phase." The one in which the reinforcing phase is embedded is called the "matrix." Composites are classified based on the geometry of the reinforcing phase and types of matrices. Based on the geometry of the reinforcing phase, they are further classified as (1) particulate reinforced, (2) flake reinforced, and (3) fiber-reinforced composites. Fiber-reinforced composites are also grouped into continuous fiber, short fiber, and whiskers. Based on the type of matrices, they are classified as (1) polymer matrix, (2) metal matrix, (3) carbon fiber matrix, (4) fiber-reinforced polymeric composites, and (5) particulate-reinforced metal matrix composites. There are hybrid varieties of composites that use multiple reinforcements and matrices, for example, carbon and fiberglass in epoxy matrix. Glass-reinforced epoxy (GRE) composites are extensively used in the piping system in the offshore environment. They offer good resistance against highly corrosive fluids at various pressure, temperature, adverse soil, and weather conditions. These characteristics make them suitable for many special applications such as oil exploration, desalination, chemical plants, fire mains, dredging, and portable water. Pultruded glass or phenolic gratings are a particular type of GRE being commonly used in such process industries. Worldwide many industries are manufacturing pultruded or compression molded composite grids and gratings, which are commonly used industrial walkways, hand rails, ladders, cable trays, and so on in chemical, pharmaceutical, transportation, and infrastructural sectors.

Performance of a composite product mainly depends on the process of its fabrication. For example, pultruded fiber-reinforced polymer (FRP) grating is an assembly of preshaped FRP pultruded sections, which are joined together by various mechanical means. Pultruded structural profiles provide extremely useful options to offshore designers. Pultruded products, due to their high fiber-to-resin ratio (70:30), help in achieving higher load-bearing capacity. Pultruded gratings have longer span with less deflection compared to that of the molded gratings. This is an added advantage to use them for decks on the topside of offshore platforms. Thus, load-bearing capacity of a pultruded product of a composite can be as good as that of any other material, which is used for structural members in offshore platforms. By comparing the performance of molded gratings with that of the composites, pultruded gratings have an edge over the molded gratings in the offshore applications. They can sustain a longer span with less deflection, which is one of the important criteria for the topside of offshore structures. To fit any modular dimensions of the floor or the plant requirements, pultruded grating panels can easily be cut and modified. In recent times, phenolic gratings are also seen at larger applications in offshore platforms where fire safety is important. The main advantage of phenolic gratings lies not only in their performance during fire but in their ability to retain a significant level of functionality even after fire exposure; one vital characteristic is the low smoke emission.

Composites meet diverse design requirements and exhibit high strength-to-weight ratio compared to other conventional materials used in the construction of offshore platforms. They are proved to be worthy alternatives under high-pressure and corrosive environmental situations. Superior corrosion resistance and resistance to cyclic loads are exclusive advantages of composites. Good resistance to temperature under extremes and resistance to wear and tear make them suitable for offshore production and process lines. Structural properties and mechanical characteristics of composites can be easily altered either by the method of manufacturing or by the method of fabrication. Properties of composites are customized to suit special features such as low thermal conductivity, low thermal coefficient of expansion, and higher axial strength and stiffness. Hence, they find increased applications in offshore installations. Extensive applications are seen in the oil and gas industry since the past two decades. Significant advances in the application of composites are made in the process line layouts used for hydrocarbon handling. High cost to replace steel piping in retrofit applications prompted to use composites, while increased longevity in the new construction is an added advantage. Heavy metal pipelines are replaced with lighter ones made of composites, which also results in cost reduction. Composite pipes are also used for fire-fighting mains, seawater intake systems, cooling towers, draining systems, and sewerage systems.

The cost advantages of composite products are much greater when they are replaced by expensive corrosion-resistant metals such as copper–nickel alloys and titanium. Their resistance to corrosion helps in improving reliability and safety. It also leads to lower life cycle costs, which is an important assessment in the construction management techniques of offshore industry. With the recent advancements in the processing methods and product development, composites have become an attractive

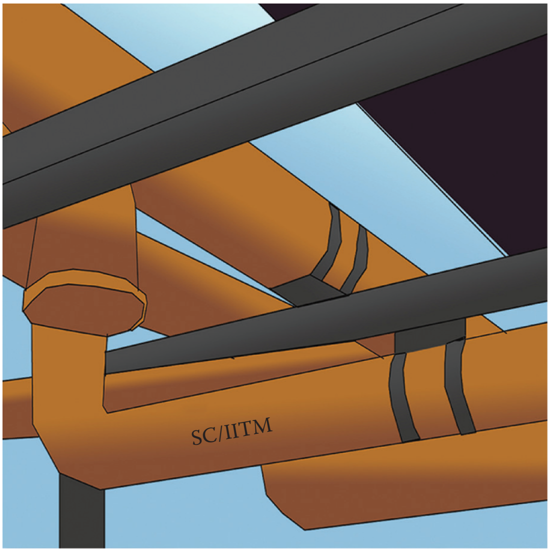


FIGURE 3.6 GRE piping system.

candidate for topside applications, down-hole tubing in subsea, and others. Selection of a suitable resin plays an important role for imparting the durability of composites when exposed to aqueous fluids. Smoke and toxicity resistance, mechanical properties including resistance to shock and impact loads, and resistance under adverse environmental conditions are a few important properties that are investigated before composites are chosen for topside applications in offshore platforms. GRE piping systems are suitable for the offshore environment against highly corrosive fluids at various pressures, temperatures, and adverse soil and weather conditions. GREs are used widely in oil exploration, desalination, chemical plants, fire mains, dredging, and so on. Figure 3.6 shows a typical GRE piping system of a topside installation.

3.10.1 Glass-Reinforced Epoxy

GRE pipes are commonly used in oil transportation where resistance to crude oil, paraffin buildup, and ability to withstand relatively high pressure are required. GRE piping system is also being used in offshore rigs for seawater cooling lines, air vent systems, drilling fluids, firefighting, ballasts, and drinking waterlines in offshore applications. Availability and lightweight modular forms of GRE help to reduce the construction cost. Established oil fields use GRE pipes for high pressure and steam injection lines for the recovery of oil preserves. GRE piping systems are capable of withstanding corrosive effects of water that are expelled under pressure from the fire mains. The effect of rupture-free GRE pipes under such shocks makes the system more reliable. GRE piping systems are therefore extensively used for firefighting systems and recovery systems in oil preservations. Chemical resistance and service temperature of such composites mainly depend on the resins and additives used for product formation and bonding. Figure 3.7 shows a typical application of composite grating on the topside of offshore installation.

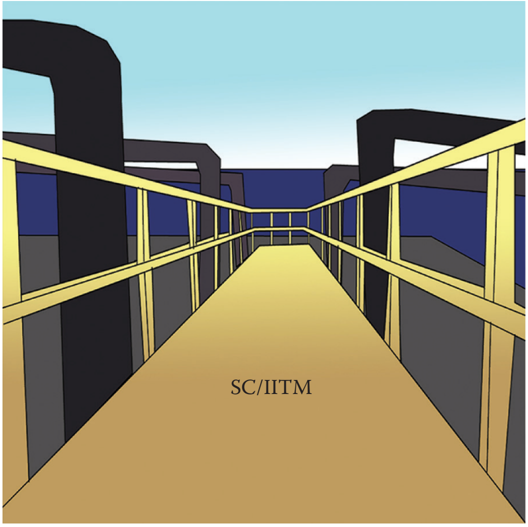


FIGURE 3.7 Topside applications.

3.11 NONFERROUS METALS

There are other nonferrous materials, which are also suitable for offshore construction. Cupronickel alloy, which is mixture of 69% copper and 30% nickel, is a popular alternate for offshore applications. Cupronickel alloy is widely used for condenser applications such as tubes, tube sheets, and manifolds. The term "manifold" refers to a pipe with single inlet and multiple outlets or vice versa. K-Monel alloy, which is a combination of 65% nickel and 30% copper is another alternative. The next competitor is MONEL nickel–copper 400 alloy, which constitutes 66% nickel and 32% copper with rest from other materials excluding aluminum and titanium. Bronze, which is an alloy with 90% copper and 10% zinc, finds increasing applications in the offshore industry. However, a variety of these materials as discussed above make an engineer to select the suitable material that is required for the service life of an offshore application. It is important to note unlike other structures, materials for offshore structure applications are selected not only based on their strength but also based on the desired performance criteria.

3.12 FIBERGLASS

The most prominent nonmetallic material for ocean applications is fiberglass, which is reinforced with plastic (FRP). Small boats and buoys are made of FRP. They are one of the variety of composites, which consist of plastic fibers as reinforcing materials that are bind together; the reinforcing material gives strength to the composite. Composites consist of glass fibers, carbon graphite, nylon, silica, or metals such as steel, aluminum, boron, and tungsten as reinforcing materials. Bonding materials are typically epoxies, polyesters, phenolics, and silicon. The most common is the glass fiber with an epoxy or polyester binder. The strength of FRP depends on the manufacturing process. Fiberglass polyester mat is widely used in the production of small boats and buoys. The major advantage of this material is that it is maintenance free and highly durable under a variety of operating conditions. Fiberglass, owing to its internal damping characteristics, heats up when subjected to fast-changing stress cycles. It reaches its fatigue strength in 10 million cycles, making the ratio of fatigue to tensile strength lower than 0.25; this is an important concern to the offshore engineers and naval architects. In the marine environment, fiberglass loses strength by the absorption of water when immersed over long period; strength reduction is also seen when continuously exposed to ultraviolet rays. Although water absorption reduces compressive strength, exposure to ultraviolet rays causes brittleness, as it gets laminated upon application of heat. High-quality manufacturing methods are necessary, as many resins that are used to manufacture fiberglass are highly flammable. Resins used to manufacture fiberglass composites are carefully chosen to modify the fundamental characteristics of fiberglass to suit the marine environment. For example, tempered glass is one of such varieties. Tempered glass shows substantial promise as a material when used in compression. Because offshore structures are encountered by a combination of variety of loads, the mechanical characteristics of glass needs to be modified. This is done by the choice of resins and adopting appropriate manufacturing methods. However, it is important to note that the nonavailability of large sections of tempered glass without defect is one of the major concerns.

3.13 WOOD

One of the oldest materials used in the marine environment is wood; for many years, it is the only material used for ship building. Currently, wood is extensively used for pilings, docks, and similar applications. In the recent times, wooden laminates are also being used as structural members. Flammability characteristics, nonavailability in large size, and deterioration of strength under continuous exposure to seawater are a few of serious limitations of wood as the choice of construction material. One of the most important drawbacks of wood is its significant change in strength characteristics and grain orientation with respect to the loading direction; it shows a significant difference when the load is applied parallel to the grains with that of the perpendicular direction.

3.14 GLASS-REINFORCED PLASTICS

GRP is essentially recommended for construction of lifeboats. Lifeboats are self-righting, enclosed, motor-propelled, survival crafts, which are used in offshore oil industry for rescue operations. They are manufactured using GRP, which we call them as GRP. One important property what the GRP attains is fire-retardant resins. It is also important to note that survival crafts are required to withstand about 30-m-high kerosene flames and a temperature of about 1200°C, as a part of the safety norms of the safety directorate; GRP passes such stringent fire safety norms.

3.15 BUOYANCY MATERIALS

Materials that have a specific gravity considerably lower than that of water are used as buoyancy materials. Few common applications are small submarines, oil well drill pipes, deep-sea buoys, and so on. The most common buoyancy materials are wood and gasoline whose specific gravities are 0.5 and 0.7, respectively.

Buoyancy materials should possess certain desirable properties: (1) no water absorption and (2) no dis-configuration under compression.

3.15.1 Syntactic Foams

Syntactic foams are composite materials synthesized by filling a metal, a polymer, or a ceramic matrix with hollow particles called micro-balloons. The presence of hollow particles results in lower density and higher strength. Syntactic foams at 600 bar pressure possess 3 times higher strength than that of concrete (80 MPa). When used, similar to reinforced cement concrete structures, they can combine the advantages of strength and buoyancy. It is free from corrosion, and the structure by itself acts as a buoyancy to the system. The system, while in operation, reduces the amplitude of vibration response, and the mining becomes more reliable. Recent studies in the literature shows favorable characteristics of syntactic foams with the composition: epoxy as the base material and glass microspheres as micro-balloons. The tensile strength of the foam depends on the matrix material. Table 3.5 shows typical values for syntactic foam with epoxy resin as the matrix material. Syntactic foams are buoyancy materials that cater to certain special needs in offshore engineering. They are essentially hollow glass spheres dispersed in a plastic matrix. The most efficient syntactic foams use glass spheres of extremely small diameter called micro-balloons with an epoxy resin binder. They have a high compressive and shear strength with low water absorption. They can easily be handled with woodworking tools.

3.16 COATINGS

Coatings are extensively used in the marine environment to protect surfaces against deterioration from salt spray, barnacles, corrosion, pollution, and all other contaminants of the sea. As fouling increases with the increase in water temperature, new plastic coatings are used in the recent times for antifouling protection. Certain epoxy coatings are also used for corrosion protection. Coatings serve as a physical and a chemical barrier and prevent materials from degradation. Protective coatings are applied up to five coats, resulting in a film of about 0.25–0.5 mm thick. Polyurethane coatings are successful in protecting wood in the marine environment. Most common anticorrosive coatings are coal tar epoxy, epoxy, polyurethane, vinyl anticorrosive coating, neoprene, and other rubber coatings.

TABLE 3.5
Properties of Syntactic Foam with Epoxy Resin

Properties	Value
Young's modulus	2.1 GPa
Density	640 kg/m^3
Microsphere density	349 kg/m^3
Resin density	1120 kg/m^3
Hardener density	1050 kg/m^3

3.17 CONCRETE

Concrete is seen as a strong competitor to steel and widely used in marine construction. Various types of cement that are specially manufactured to cater to the marine environment make concrete as the most-preferred choice of offshore designers. Excellent compressive strength and high resistance to seawater attack make concrete as the first choice for marine constriction. Problems of low tensile strength are overcome by reinforcing concrete with steel to form reinforced cement concrete (RCC); prestressing and ferrocement concretes are also desirable. Ferrocement concrete is widely used to construct barges, boats, and so on, which is reinforced with wire mesh to improve tensile strength and stability. Prestressed concrete, which can withstand very high compressive and tensile strength, are the best candidates for pressure vessels for LNG storage. Although concrete performs comparatively better than steel in corrosion resistance, it suffers deterioration during freezing and thawing. Therefore, one has to be careful in treating concrete if it is used at different temperature gradients in the marine environment.

3.18 CONCRETE IN THE MARINE ENVIRONMENT

Marine structures are under corrosive environment; apart from strength, durability is a subject of major concern, especially in the marine environment. Ocean structures are exposed to seawater directly, which results in the simultaneous action of a number of physical and chemical deterioration processes. Concrete, as one of the most preferred construction materials, undergoes complex problems in the marine environment. Corrosion that takes place in the marine environment is not uniform throughout the length of the member. Corrosion, based on its consequences, can be grouped in different zones: atmospheric zone, splash zone (tidal zone), and submerged zone, as shown in Figure 3.8.

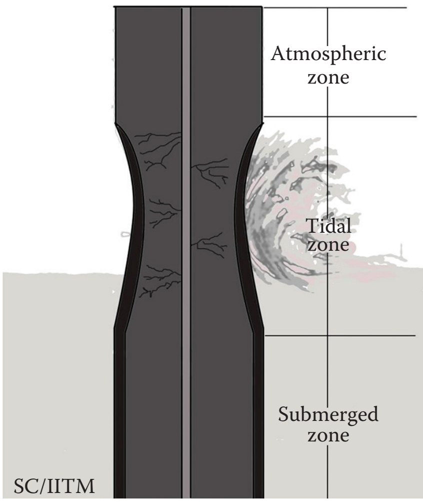


FIGURE 3.8 Corrosion zones on a concrete pile in the marine environment.

The submerged zone is below the surface of the water. The surface of the concrete structure is continuously and constantly exposed to seawater in this zone. The tidal zone is limited by the extent of the tidal actions. The surface of the concrete structure is exposed to seawater in a cyclic manner in this zone. The splash zone is limited by the extent of splash from breaking waves above the tidal zone. The surface of the concrete structure in this zone is randomly exposed to seawater. The atmospheric zone is limited by the extent of spray from breaking waves above the splash zone. The surface of the concrete structure in this zone is randomly exposed to spray from breaking waves. Reinforced concrete structures that are partially or fully submerged in seawater are especially prone to corrosion of reinforcing steel due to a variety of reasons. These include high chloride concentration levels from the seawater, wet/dry cycling of the concrete, high moisture content, and oxygen availability. The tidal zone is characterized by periodical wetting and drying, and possible freeze–thaw actions. The surfaces in the tidal zone are mostly wet with a limited access of oxygen. The extension of the tidal zone varies between 0 and 15 m. The splash zone is characterized by randomly wetting and drying waves, depending on the wave actions. The extension of the splash zone depends on the wave heights and variations in tides. The corrosion rate below the water level is limited by low oxygen availability. Conversely, lower chloride and moisture content limit the corrosion rate above high tide. Corrosion is most severe within the splash and tidal zones where alternate wetting and drying result in high chloride and oxygen content. The atmospheric zone is the uppermost zone layer, whose corrosion rate is 5–10 mills per year; one mill is about (1/1000)th of an inch, which is about 0.025 mm (about 25 microns). Control methods, which are generally employed to retard the corrosion rate in the atmospheric zone, are through external coatings. These coatings are generally epoxy-based resins or chlorinated rubber vinyl or zinc sulfate. The corrosion rate in the splash zone is about 55 mills per year, which is very high. This is mainly due to alternate wetting and drying caused by splash waves. Control methods prevalent are coatings or additional cladding. The submerged zone has the corrosion rate of half of that of the splash zone, which is about 25 mills per year. Control methods are generally cathodic protection or some coatings. As seen above, corrosion results in loss of metal (material from the metal surface). This will lead to loss of desired thickness in the members. The member also substantially loses its strength or degrades from its functional purpose for which it is designed. Therefore, corrosion as a deteriorated process should be addressed very carefully.

3.18.1 Deterioration of Concrete

From long-term studies of Portland cement mortar and concrete exposed to seawater, it has been seen that magnesium ion attack is well established by the presence of white deposits of $Mg(OH)_2$, also called brucite and magnesium silicate hydrate. In seawater, well-cured concrete containing large amounts of slag or pozzolona in cement usually outperforms the reference concrete. This is due to the lower presence of uncombined calcium hydroxide after curing.

There is a potential loss of concrete mass by leaching away calcium from hydrated cement paste due to the carbonic acid attack. Loss of material is associated with the concentrations of carbon dioxide present in seawater. This is also accelerated in the presence of dissolved carbon dioxide. Presence of thaumasite (calcium silicocarbonate), hydrocalumite (calcium carboaluminate hydrate), and aragonite (calcium carbonate) is responsible for deterioration of concrete in seawater. The following chemical reactions explain the deterioration of concrete in seawater:

1. Action of carbon-di-oxide:

$$Ca(OH)_2 + CO_2 + H_2O \rightarrow \underset{\text{Precipitate}}{CaCO_3} + 2\,H_2O$$

Precipitate → Aragonite [Coating]; Precipitate → Calcite

2. Action of Magnesium sulphate

$Mg^{2+} \rightarrow Ca^{2+}$ substitution

$$MgSO_4 + Ca(OH)_2 \rightarrow CaSO_4 + Mg(OH)_2$$

$CaSO_4$ → Soluble [Leaching]; $CaSO_4$ → Solid secondary Gypsum [Expansion]

$Mg(OH)_2$ → Precipitate [Coating]

3. Action of secondary gypsum

$$CaSO_4 + C_3A + 32\,H_2O \rightarrow \underset{\text{Ettringite}}{C_3A.3CaSO_4.32\,H_2O}$$

4. Action of Magnesium chloride

$Mg^{2+} \rightarrow Ca^{2+}$ substitution

$$MgCl_2 + Ca(OH)_2 \rightarrow \underset{\substack{\text{Soluble}\\ \text{[Leaching]}}}{CaCl_2} + \underset{\substack{\text{Precipitate}\\ \text{[Coating]}}}{Mg(OH)_2}$$

5. Action of calcium chloride

$$CaCl_2 + C_3A + 10H_2O \rightarrow \underset{\text{Chloro aluminate}}{C_3A.CaCl_2.10H_2O}$$

$$\downarrow SO_3$$

$$\underset{\text{Ettringite}}{C_3A.3CaSO_4.32H_2O}$$

$$\downarrow CO_2 + SiO_2$$

$$\underset{\text{Thaumasite}}{CaCO_3.CaSO_4.CaSiO_3.15H_2O}$$

The presence of thaumasite, hydrocalumite, and aragonite is reported in cement pastes derived from deteriorated concrete, which is exposed to seawater for a longer time. The major deterioration is observed in the samples having greater thaumisite. Several case studies reported by researchers show that chloride profiles indicate greater damage due to corrosion. This is seen higher on the surface of coastal structures.

3.18.2 Selection of Cement

Sulfate-resisting cement suffers less chemical decomposition in seawater than that of the ordinary Portland cement. However, the issue of which type of cement is most effective in controlling the migration of chloride ions is still debatable. Calculated addition of pozzolona can improve the durability of concrete by removing a part of free lime, reducing permeability, and protecting the reinforcement. Studies have shown that blast furnace slag cement, especially when well cured, resists the action of seawater fairly well. However, blast furnace cement cannot be always the governing cement. Deterioration of concrete is mainly due to physical, chemical, and biological processes. Physical process includes cracking, abrasion, and attack caused by frost and deicing salts. Chemical process arises from acid, sulfate, and alkali attacks. Environmental factors arise from exposure conditions, temperature, humidity, and presence of aggressive elements present in seawater. Other reasons for deterioration of concrete in the marine environment are design and construction defects, poor quality of materials, poor quality of construction, corrosion in rebar, and other technical factors. Figure 3.9 shows a typical spillway, which is corroded but subsequently retrofitted with galvanic cathodic protection.

3.18.3 Inspection Methods

There are a variety of methods by which one can assess the failure of concrete construction. They are (1) visual inspection, (2) by observing the cracking conditions of exposed metal components, (3) conditions of foundation, and (4) by observing the extent of marine growth. After successful inspection, repair methods are advocated in the following sequence:

FIGURE 3.9 Corroded spillway.

- Removal of deteriorated concrete
- Sealing of cracks
- Replacement of concrete
- Surface treatment
  - Vapor permeable coatings
  - Vapor barrier coatings
- Restoration of structures
  - Realkalization
  - Desalination

Various field methods are deployed to assess the *in situ* conditions of concrete in the marine environment. They are (1) Schmidt hammer test, (2) portable adhesion tester, (3) galvanized pulse method, and (4) half-cell potential measurement. Nondestructive testing (NDT), as applicable to marine structures, are discussed in Chapter 5.

3.19 PROTECTING CONCRETE

Concrete is one of the promising construction materials for offshore structures. In the marine environment, concrete also deteriorates. Removal of deteriorated concrete is important while carrying out the repair of ocean structures. Sealing of cracks using different chemical components has also been attempted by various engineers in different capacities, all over the world to improve the performance of concrete members, especially in the sea environment. Replacement of concrete, of course, is a better alternative but as expensive as constructing a new structure. Therefore, rehabilitation of marine structures can be even higher than that of the cost of the principal structure. Alternatively, many practicing engineers attempted surface treatment. This is a cosmetic type of repair carried out to improve the serviceability from a deteriorated condition to a basic acceptable level. Different types of vapor permeable coatings or vapor barrier coatings are applied on the surface of degraded concrete. Such treatments will only protect concrete in a superficial manner but do not help to enhance structural integrity of deteriorated members. Before protection of concrete is attempted, it is important to understand the level of strength degradation. Crystalline technique is one of the interesting and recent advancements in protection of concrete used in the marine environment. As it is commonly felt that concrete is already impervious and has enough strength, it is believed that concrete should be able to withstand any worst environment. However, the fact is that concrete is porous and permeable. Figure 3.10 shows a 5000-time scanned electron microscope (SEM) photograph of concrete, which verifies this statement.

In reality, as concrete is not a homogeneous impervious material, there are a lot of possibilities of pores. This will affect the performance of concrete in the marine environment. Figure 3.11 shows the composition and characteristics of concrete. Concrete composes of coarse aggregates, fine aggregates, sand, and cement. For making it workable, water is added to its limiting water:cement ratio. More water, if added than that required for cement hydration, results in bleeding of water. This action leaves a network of capillaries and pores. As it dries, concrete shrinks and changes volume, which results in the development of micro- and macrocracks. Concrete may be

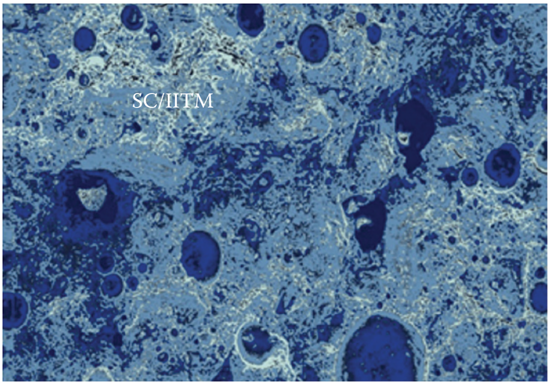


FIGURE 3.10 SEM photograph of concrete.

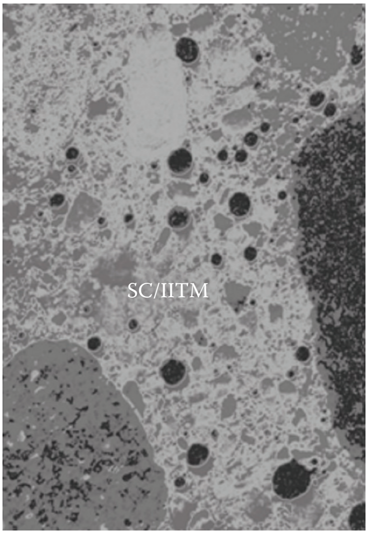


FIGURE 3.11 Composition and characteristics of concrete.

permeable on several and different scale sizes. Figure 3.12 shows the pores of different sizes and their role in the degree of deterioration of concrete. Capillary voids are essentially the residue of water-filled spaces. Water and other aggressive ions can penetrate into concrete and cause durability problems through this primary path.

Concrete may be permeable on several different size scales as well. As seen from the figure, the size scales of entrapped voids vary from 1000 to 10,000 μm. The size of the cracks varies from 100 to 3000 μm, whereas that of the entrained air is in the range of 70–400 μm. If you look at the entrained air, it can vary from 70 to 400 μm. It can also result in microcracks, which are less than 0.1 to 100 μm. Capillary pores of size 0.01 to 1 μm are responsible for making concrete permeable. Microcracks are further caused by stresses induced by loads or by shrinkage around aggregates. Figure 3.13 shows the formation of macrocracks by structural, thermal, drying, and

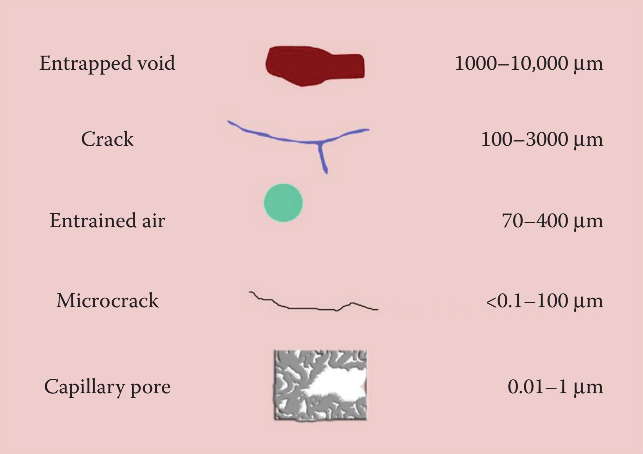


FIGURE 3.12 Pores and their influence on performance.

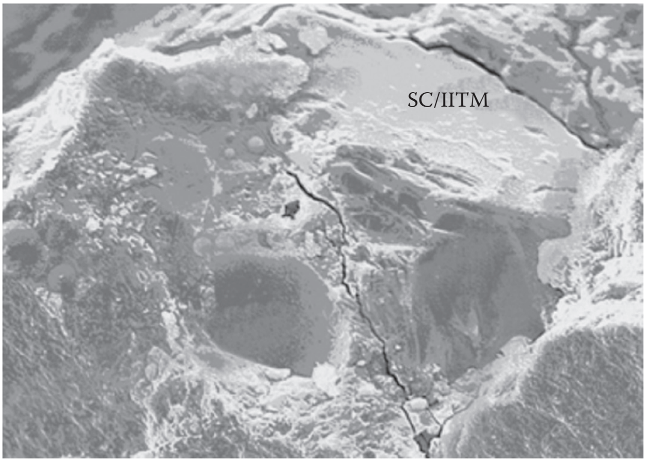


FIGURE 3.13 Formation of macrocracks.

plastic shrinkage, whereas Figure 3.14 shows the deterioration of concrete on marine structures due to the ingression of aggressive chemicals.

3.19.1 Crystalline Technology

Capillary pores present in concrete make it porous and permeable. Capillary voids are essentially the residue of the originally water-filled spaces. When this capillary voids are started attracting water from the moisture content or from the sea environment, it enables aggressive ions to penetrate into concrete, causing serious durability problems. For example, reinforcements corrode when water is entrained in these capillary voids. Concrete is a good performing material, which is widely preferred for

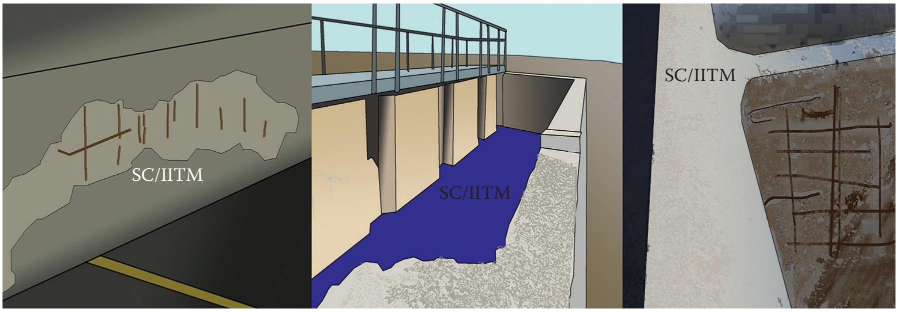


FIGURE 3.14 Deterioration due to the ingression of chemicals.

construction of ocean structures. Crystalline technology is one of the recent advancements in concrete technology, which addresses this problem. Crystalline material is a fine particle, which is to be mixed in concrete to fill up the voids and make it impervious. It is important to note that it is neither a coating nor an ingredient, which is added to concrete when it is being prepared. A reactive component reacts with calcium hydroxide and other by-products, during the hydration process of cement. This results in nonsoluble crystalline formation, which is permanently fixed in the pore structure of concrete. Figure 3.15 shows the by-products of cement hydration, which precipitates into the capillary tracts of fresh concrete. These by-products of cement hydration react with the crystalline reactive chemicals to initiate "crystallization."

Crystalline formation sticks onto the pores of concrete permanently. With the passage of time, they develop into a crystalline structure, as seen in Figure 3.16. Due to the formation of this permanent crystalline structure, capillary pores that were present earlier are completely filled and permanently closed. This is the result of chemical reaction between the chemical adhesive present in the concrete and that of the crystalline material. Figure 3.17 shows the comparison of concrete with and without crystallization.

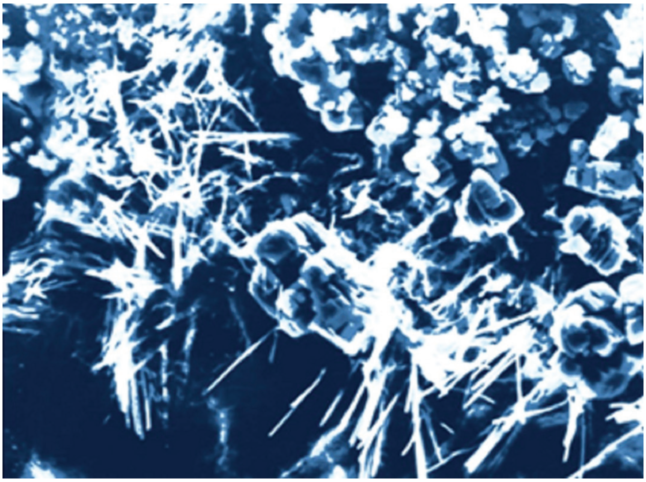


FIGURE 3.15 By-products filling capillary tracts.

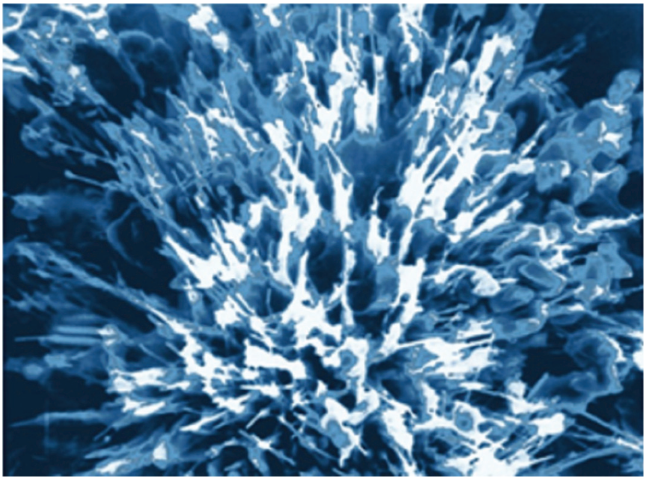
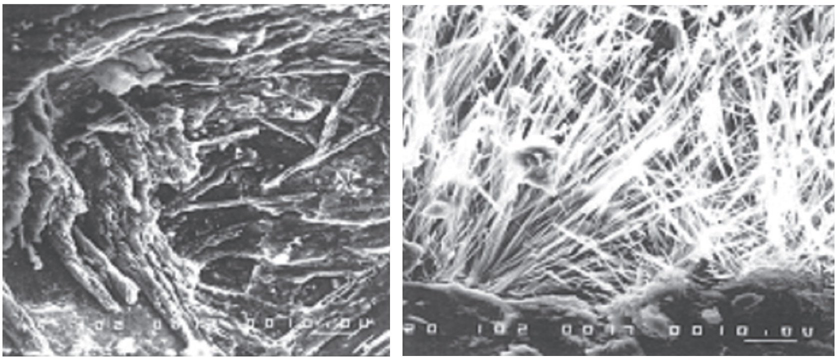


FIGURE 3.16 Crystalline formation.



(a) (b)

FIGURE 3.17 Comparison of crystalline concrete: (a) before; (b) after.

Crystalline treatment ensures permeability; pores and voids present in concrete are gradually filled up with the permanent formation of chemical, making concrete truly impervious. Crystalline formation cannot be punctured or damaged as it is within the concrete. It is therefore better than a coating or a membrane layer. It is capable of withstanding high hydrostatic pressure and also highly resistant to chemicals of pH ranging from 3 to 11 in constant contact and 2 to 12 in periodic contact. Concrete with crystalline treatment can sustain significant thermal variations ranging from –32°F to 265°F. Crystalline-treated concrete is not affected by humidity, ultraviolet light, and variation in oxygen concentration. Performance characteristics of crystalline concrete are very important issues. Crystalline concrete should be tested to accept its performance behavior in the marine environment. Common checks carried out are permeability, chemical resistance, crack sealing, compressive strength, and freeze–thaw durability. Results of the chemical durability tests carried out by Iwate University, Tokyo, Japan, show the satisfactory performance of crystalline concrete compared with that of a protective coating. Studies also showed that there is a significant delay in the corrosion initiation in the embedded rebar. Under normal conditions, corrosion is initiated approximately after 40 years of construction with a concrete cover of 70 mm; such high cover is recommended by many international codes for severe exposure. Crystalline concrete showed a delay in the corrosion initiation by about double of this time, which is a great advantage for marine structures; this is ensured with proper periodic maintenance in parallel. Crystalline concrete is termed as "green product" as it is nontoxic and does not produce fumes. Crystalline products do not contain any volatile organic compounds, which is a common problem with most of the chemical treatments carried out on concrete surface.

Crystalline treatment can be done either by coating or by mixing the product in the green concrete during the preparation of concrete. Coatings have high concentration, which can readily diffuse in the solution of lower density until both of them equalize themselves. Any open pore surface will readily admit such chemical transfer into the substrate. Concrete surface, which needs to be coated, should be cleaned thoroughly to make it free from oil and other foreign matter. Concrete surface must be thoroughly saturated, and the surface should be in a damp condition to apply this treatment. In hot weather, it may be necessary to soak the surface overnight. In cold weather, the surface temperature must be above 33°F for at least 24 h before start applying this treatment. Crystalline material is mixed at a ratio of 5 parts of powder to 2 parts of water by volume and stirred thoroughly to ensure the consistency of the slurry formed by this mixture. Subsequently, it can be applied by either a brush similar to that of painting or using a hopper gun. Moist curing of coating with water is necessary for proper performance of this treatment. It controls evaporation, cures and hardens the cement and coating, initiates crystalline formation after 3–4 h of application. Curing should be done for at least couple of days under both hot and cold weather. This is to ensure that the surface does not get dried up before the crystalline formation happens in the voids of the concrete structure. Alternatively, this can be applied as an admixture, which is commercially available in the readymade form. The admixture can be directly mixed in concrete at the time of batching itself. The dosage rate is about 1% by weight of cement used in the mix design. It is compatible with other admixtures, which are commonly used for slump control, rapid hardening, and so on. Adding crystalline admixture at the batch plant ensures a uniform distribution throughout the concrete and therefore throughout the member. It makes concrete impermeable and reduces the shrinkage cracking. It also increases the compressive strength of concrete as a structural member. Construction cost added to the maintenance as initial investment is significantly reduced, as the service life of the structure is enhanced.

3.20 CORROSION

Corrosion is deterioration of material by chemical interaction with the environment. This term also refers to degradation of plastics, concrete, and wood, but generally refers to metals. Corrosion process produces a new and less desirable material from the original metal, which results in a loss of function of the component or system.

The common product of corrosion is "rust," which is formed on the steel surface. The basic corrosion cell is shown in Figure 3.18. The basic corrosion cell needs three components: an anode, a cathode, and an electrolyte medium.

Electron flow during a corrosion process is shown in Figure 3.19. There will be a measurable direct current (DC) voltage, which can be read in the metallic path between the anode and the cathode. When both the anode and the cathode are electrically bonded, the anode is positively charged and the cathode is negatively charged. Conventional current flows from positive to negative, and thus, current discharges from the anode and is picked up at the cathode through the electrolyte. Current returns from the cathode to the anode through an electrical path. This flow has a detrimental effect on the anode known as "corrosion." It is important to note that corrosion occurs at the anode and not at the cathode. Corrosion is a process in which ions are involved. For corrosion to take place, three basic requirements are necessary: (1) a medium to move, which is water in case of ocean structures as the members are continuously exposed to seawater; (2) oxygen to activate the process, which is

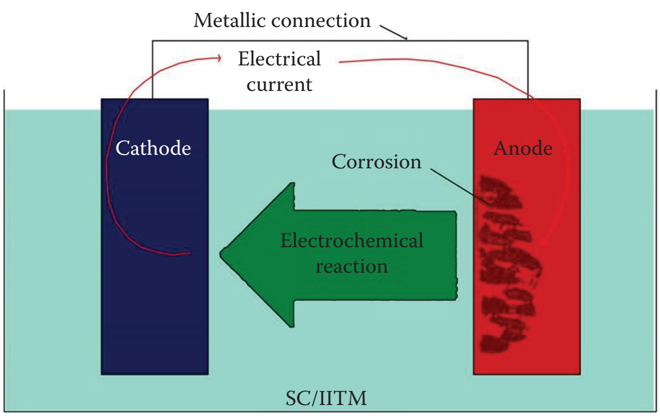


FIGURE 3.18 A basic corrosion cell.

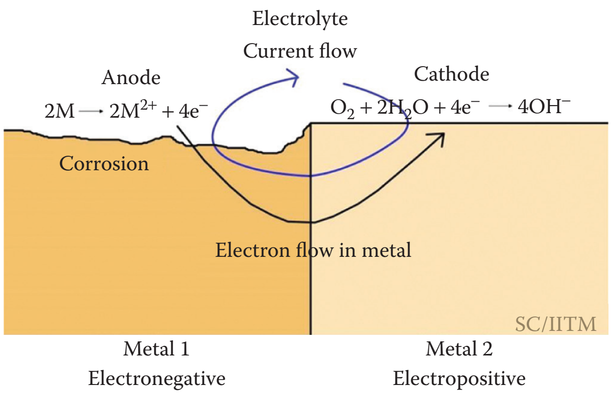


FIGURE 3.19 Electron flow during the corrosion process.

present in abundance; and (3) a metal, which should be willing to give up electrons to start the process. Corrosion process results in formation of a new material, which may react again or could be protective of the original metal. The anode and cathode in a corrosion process may be on two different metals connected together forming a bimetallic couple (galvanic couple), or, as in the case of rusting of steel, they may be formed on the metal surface.

3.20.1 Corrosion in Steel

Steel is the basic material of construction in the offshore industry. Corrosion, to a large extent, is governed by the oxygen content of seawater. The corrosion rate of steel in the marine environment is related to the rate at which a ferrous corrosion product is leached or washed from film of rust. When one of the products of corrosion becomes soluble, the formation of a protective barrier film becomes impossible. The presence of copper and nickel, even in small quantities in the low alloy steel, enhances their corrosion resistance by altering the structure of the barrier film formation. They help to produce a tighter, denser barrier film with less of a tendency to be removed by leaching or spalling. Figure 3.20 shows a typical offshore platform with different corrosion zones marked. The figure shows different regions at which the corrosion takes place in an offshore platform. The top zone is the atmospheric zone where derricks and deck modules are located. This zone experiences the minimum rate of corrosion due to the fact that the members are not in direct contact with water. Hence, leaching or washing of the thin barrier film is at a lower probability.

Atmospheric sea exposure is always present on the top portion of the topside where derrick and deck modules are present. It contains generally precipitated salt, and condensation process takes place in this region; soffit of the deck slab is the most vulnerable candidate for corrosion in this region. The corrosion rate of steel in the marine atmosphere is related to the rate at which the ferrous corrosion product is

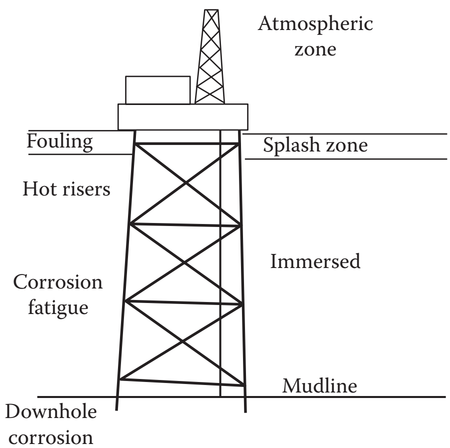


FIGURE 3.20 Offshore jacket platform with various corrosion zones.

leached or washed off from film or rust. A protective barrier film is being created on the top of the member, as a by-product of the corrosion process. If there is a possibility that this barrier film can be leached off or washed off, then the corrosion process can be activated. In case of atmospheric zone, there is a least possibility that this film can be washed off. When one of the products of corrosion becomes soluble, the formation of a protective barrier film is impossible.

Severe corrosion is seen in the splash zone due to continuous wetting and drying because of tidal variations in the sea. It results in pitting corrosion in the tidal area. Due to continuous contact of seawater with a higher lateral force, thin barrier film, even if formed, will be washed away immediately; this expedites corrosion in the splash zone. Rust films in this zone have a little opportunity to become dry as this zone is subjected to alternate wetting and drying continuously. Even though the rust films may be formed, they will be leached off automatically. This is aggravated because of the presence of abundant oxygen content in this region. The rate of corrosion in this splash zone is several times greater than that of the continuous immersion part of the member. It is interesting to note that the same member (say, e.g., jacket legs) passes through different regions of corrosion, resulting in the development of a bimetallic couple. One part of the member becomes anodic and the other cathodic; the presence of an electrolyte activates the corrosion process very fast. Therefore, the rate of corrosion in this region is seen as several times higher than that of the other regions, which is continuously immersed in seawater.

In the tidal zone, corrosion reaches a minimum because of the protective action of oxygen concentration cell currents present in this region. Steel surface in a tidal zone is in contact with highly aerated seawater, and therefore becomes cathodic. As only the anodic part of the member corrodes, the adjacent submerged surface where the oxygen content is less becomes anodic, and therefore, it gets corroded severely. For example, the members that are covered with oxygen-shielding organisms like marine growth may get less oxygen content on the surface, and they become anodic and corrode faster than the members present in the tidal zone. The current flows from the anode, which is submerged surface, to the cathode, which is tidal zone in this areas in the sea environment. This enables sufficient cathodic protection to the members in the tidal zone automatically. This is caused by the differential aeration or formation of marine growth in the regions, which are immersed below.

In the immersed zone where jackets and mud lines are present, corrosion is reduced due to the decrease in oxygen concentration. In this zone, corrosion is principally governed by the rate of diffusion of oxygen through layers of rust and marine organisms. The corrosion rate is not influenced by the seawater temperature and tidal velocity. Kindly note that rate of corrosion is not determined by the temperature gradient in seawater with the increase in water depth. The corrosion rate may go up in the vicinity of the mud line, but further down, it is very less. This is due to the presence of marine organisms, which can generate additional concentration cells and sulfur compound in the vicinity of the mud line. Due to this, they become anodic compared to the remaining part and get corroded. The corrosion rate reduces well below the mud line because of lower availability of dissolved oxygen content. Barrier films, once they are formed in this region, are relatively undisturbed. Therefore, they form a protective coating automatically, and that protects the members in this region.

Various factors influence the corrosion rate of steel members in offshore structures. Considering the effect of current velocity on the corrosion rate, it is understood that any gentle motion will not affect or disturb the formation of the protective barrier on members. However, when the platform motion is larger, the barrier layer formation becomes thinner and is easily broken under higher current velocity. Therefore, current velocity plays an important role in accelerating the corrosion rate; increase in current velocity increases the rate of corrosion. Considering different methods of corrosion protection, the majority of the members of the platform present in the atmospheric and tidal zones can be protected by painting. For those present in the splash zone, special methods such as providing extra steel, Monel wrapping or sheathing is recommended as a corrosion protection measure. In case of the immersion zone, sacrificial anode technique or cathodic protection methods are employed. In the mud line below, no special methods of corrosion protection are advocated as there is a little amount of corrosion.

The effect of water depth on the corrosion rate is also an interesting viewpoint. At a water depth of greater than 1800 m, the temperature drops to less than 4°C in comparison with 24°C at the surface. This results in substantial reduction in the corrosion rate. Metal surfaces are relatively free of marine biofouling below 700 m. Dissolved oxygen drops along the depth but rises again below 820 m. This raise is significant in comparison with the surface concentration. Ocean layers are not homogeneous; various layers are differentiated by different oxygen and salinity contents. Corrosion decreases at greater water depth as the temperature decreases. Fouling and pitting associated with fouling also tend to decrease. Mooring lines, which extend to different zones of corrosion, face a critical problem. Due to part of the mooring line becoming cathodic, corrosion is set at the local level along the length of the mooring. This is called "long-line effect." Mainly due to the typical oxygen cell concentration attack, long length of mooring lines is subjected to different layers of varied oxygen concentration. This alters the rate of corrosion in different segments along the length of the mooring line, which is a long-line effect. Galvanized mooring lines are common candidates of such problems.

3.20.2 Corrosion in Concrete

Concrete, which has embedded steel, has a high degree of protection against corrosion. As concrete is alkaline in nature, it provides barrier protection to steel reinforcement. Presence of chloride in sufficient quantities in the vicinity of steel results in cracking, spalling, and delamination of concrete. Early detection of the corrosion activity can assist to plan corrosion preventive measures. Chloride-induced corrosion is the most serious cause of deterioration in RCC structures. Structural weakening caused by corrosion can reduce its service life by 20 years. The corrosion rate is accelerated in RCC members whenever there is exposure to the source of chloride. Patching of damaged areas does not stop from corroding but spreads to other areas faster. The corrosion mechanism in RCC structures can be easily understood: Steel is in a passive state in concrete. If chlorides reach steel surface by ingression, this passive layer is broken. This initiates the corrosion process. Corrosion current flows from one part of the reinforcement (anode) to another part (cathode). Because of this current flow, steel corrodes at the anode and produces rust. As a result, reinforcing steel develops a tendency to revert to its natural oxide state, which is not capable of withstanding the encountered stresses. Corroded steel can expand 4–5 times of that of its normal volume. This will result in cracking, spalling, and delamination of concrete. This further exposes more steel contact areas for chloride ingression and accelerates corrosion.

3.20.3 Realkalization

Fresh concrete has inherent alkalinity, which provides passive protection to steel. Ingression of carbon dioxide creates carbonated concrete with lower alkalinity, which results in loss of passive protection to rebar. It also accelerates the corrosion of steel reinforcement. Realkalization involves an electrochemical technique of passing sustained low-voltage current between the temporary anodes on the surface of concrete and steel reinforcement. The period of application can vary from 3 to 7 days. Electrolyte covering is done by spraying cellulous fiber, saturated in sodium carbonate solution. Surface nodes, embedded in alkali-rich paste, draw alkali into concrete through rebar. Realkalization takes place in concrete to initiate the formation of natural protective oxide film over rebar.

3.21 CORROSION PREVENTION

There are different ways by which corrosion can be prevented: (1) by conditioning the metal surface; (2) by conditioning the corrosive environment; (3) by controlling the electrochemical reaction, which is responsible for corrosion; (4) by fighting corrosion with corrosion; (5) by coating the metal; and (6) by alloying the metal. The rate of corrosion can be reduced by retarding either the anodic or the cathodic reaction. The principle aim behind any corrosion prevention or protection method is to fight corrosion with corrosion. It means that, to actually reduce corrosion, create an additional member and allow it to corrode. By sacrificing the additional member, corrosion on the existing members of the structure can be prevented. This is called sacrificial anode method. Using another metal to coat an existing metal surface is the common case in zinc or tin coating. It is generally applied on steel as an external coating surface. A protective coating derived from the metal surface itself can also be applied. For example, the metal surface of a member can be coated with aluminum oxide; organic coatings such as resins, plastics, paints, enamel, oil, and greases are also used. Coating a metal can also help reduce the corrosion rate but pose a serious threat to the sea environment. Alternatively, one can also alloy the metal to produce a corrosion-resistant alloy. A classical example is stainless steel in which ordinary steel is alloyed with nickel and chromium. By conditioning the corrosive environment, one can control corrosion. Oxygen is one of the main components required to activate the corrosion process. Removal of oxygen can help to retard the rate of corrosion. Removal can be achieved by adding strong-reducing agents. For example, sulfites can reduce the presence of oxygen content in the sea environment. Removal of oxygen is not advisable in the open environment because of the presence of oxygen in abundance.

3.22 CORROSION PROTECTION

There are many methods to protect offshore structures from corrosion. Atmospheric zone is one where corrosion is not very severe, but marginally high. One can use coatings to protect the members. In the splash zone, where the corrosion rate is very severe, one can use Monel-400 or other metal cladding. Monel-400, an alloy of 18 gauge thick (approximately equal to 1.02 mm) is attached to the tubular member in the splash zone. This is done either by bonding the Monel sheathing on the parent member or by welding. Monel-400 has high modulus of elasticity and will not get damaged under the stress conditions, which are caused by this installation process. However, it is likely to get damaged by tearing or impact forces as the sheathing is too thin. There is a tendency that the sheathing may even peel off or tear off from the parent surface of the material. Alternatively, austenitic stainless steel 304, which is an alloy of chromium–nickel stainless steel, can be wrapped over the surface of the members in the splash zone. The advantages of these applications are high degree of weldability and increased stiffness. Another alternative material, which is also commonly deployed in the splash zone, is copper–nickel alloy of either 70%–30% or even 90%–10% composition. This adds stiffness to the members during installation. One can also use steel wear plates of 6–13 mm thickness. They also add strength and stiffness to the members and improve their resistance against impact loads. This application is more common in the Arctic regions, where the temperature variations can be very large. Splashtron and vulcanized neoprene are the two varieties of rubber products, which can be used as a sheathing layer on the members near the splash zone. Splashtron is an elastomeric rubber sheathing, which is braced to the members. It is highly resistant to corrosion and mechanical abuse. It has very high tearing strength when it is hardened. It adheres to the parent material very strongly and becomes more or less homogeneous in action with that of the parent material. Thickness usually varies from 5 to 13 mm, which is high in comparison with that of the Monel sheathing. Table 3.6 shows the summary of corrosion protection measures in the splash zone.

Use of corrosion inhibitors is also one of the effective methods of reducing corrosion. Corrosion inhibitors are of different types: anodic, cathodic, adsorption, and mixed. Corrosion inhibitors are other alternatives for corrosion protection of members of ocean structures. These are chemical additives, when added to the corrosive aqueous environment, interferes with the chemical reaction and reduces the rate of corrosion. Anodic inhibitors interfere with the reaction that takes place at anodes. They suppress the cathodic reactions that occur on a bimetallic couple. Adsorption-type corrosion inhibitors generally form a film on the surface of the member; they physically block the surface from the corrosive environment. Corrosion inhibitors are commonly deployed in deepwater platforms in the immersion and splash zones. Anodic inhibitors are more popular among all of these three types of corrosion inhibitors.

Another effective method is to control the electrochemical reaction responsible for the corrosion process. This technique is done by passing an anodic or cathodic current inside the metal. Cathodic protection is an important and one of the common methods of corrosion protection in the marine environment. In principle, it

TABLE 3.6
Corrosion Protection in the Splash Zone

	Materials	Uses
1	Monel alloy 400 sheathing	Normally, Monel alloy 400 of 18 guage (1.02 mm) thickness is attached to tubular members in the splash zone either by bonding or by welding. Because of its light gauge, Monel sheathing is vulnerable to tearing on impact. It has a high modulus of elasticity, which gives rigidity for field installation.
2	Austenitic stainless steel type 304(18 Cr, 8 NI) chrome–nickel stainless steel	These materials are also wrapped in the splash zone, which gives necessary corrosion resistance, weldability, and stiffness to quality them from sheathing.
3	Cu–Ni alloy of 70/30 and 90/10 composition	This gives adequate space for installing.
4	Steel wear plates	These are used with a thickness on the order of 6–13 mm in order to add stiffness and strength, thereby providing geater impact resistance. They are used to protect the structure from anticipated erosion due to ice or from high-velocity silt-laden water.
5	Rubber	An elastomer rubber sheathing material called splashtron is generally used to send blasted leg and bracing section. It is highly resistant to mechanical abuse. Neoprene of 5–13 mm thickness is used.
6	Splashtron	Normally used as sheathing material.
7	Vulcanized neoprene	Normally used in layers of protection.

can be applied to any metallic surface that is in contact with the bulk electrolyte. This condition is automatically fulfilled in case of offshore structures as seawater with impurities of sulfites and chemicals acts as the electrolyte. This is advantageous for members buried in soil or immersed in water, and hence cannot be applied in the splash and atmospheric zones; alternate wetting and drying condition is not suitable for this kind of corrosion protection measure. Figure 3.21 shows a schematic view of anodic and cathodic reactions on a metal surface. The figure shows

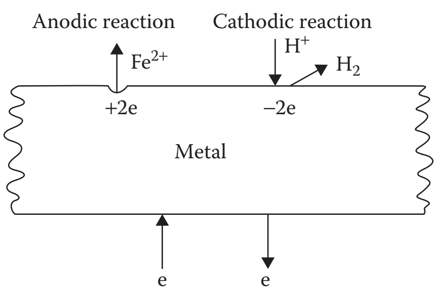


FIGURE 3.21 Cathodic and anodic reactions on the metal surface.

the formation of bimetallic couple in the presence of bulk electrolyte. Anodic reaction releases electron and becomes the positive, whereas cathodic reaction receives electron and becomes negative. Therefore, the anodic part is continuously corroded, but the cathodic part is protection, hence the name cathodic protection. To protect the parent member, one should provide another material as anode, which is capable of forming bimetallic couple with the parent metal; in this case, anode is sacrificed.

Cathodic protection can be achieved in two ways: (1) by using galvanic anodes, termed as sacrificial anode technique, and (2) by impressed current method. Metal to be protected is connected as cathode, whereas an external metal is connected as anode. Anode is ready to release electrons in a chemical reaction when connected to an electric DC current. The external metal, which is provided, may be a galvanic anode, where the current is a result of the potential difference between two metals. It forms a galvanic couple as well; alternatively, current is impressed from an external DC power source on the metal.

The galvanic anode systems employ reactive metals as auxiliary anodes that are directly connected to the metal, which is to be protected. Therefore, the member or the steel surface, which is to be protected, should be made as a cathode. An additional member is introduced to act as a galvanic anode deliberately. The potential difference between the anode and the parent steel, as indicated by their respective positions in the electrochemical series, will make the new material anodic; corrosion is initiated in the presence of electrolyte. The current flows from the anode to the parent metal, which results in corrosion of anode. Thus, the whole surface of steel which is now cathodic, is protected as it is negatively charged. This is termed as "cathodic protection." As a new metal, which is provided, acts as an anode that corrodes, this is also termed as sacrificial anode technique of cathodic protection. Metals that are commonly used as sacrificial anodes are aluminum, zinc, and magnesium. They are used in the form of rods, big blocks, or wires that can be wounded around the members. Big blocks can either be bolted or be welded to the structure.

This system is advantageous as it is very simple to install and requires no external source power. Localized protection is highly effective and immediately available on float-out. Moreover, this has less interaction with neighboring structures. One of the main disadvantages is that the current output available is relatively small. Therefore, monitoring a galvanic system for effective corrosion protection is very difficult under surveys. A monitor system is highly sensitive to record small variation in voltage. The flow of electrons depends on the electrical resistivity of the electrolyte. A change in the structure, say for example, deterioration of coatings, demands more current and hence more sacrificial anodes; therefore, sometimes, this method proves to be expensive.

An alternate method by which one can also use cathodic protection is by passing the impressed current on the metal. Impressed current systems employ either zero or low dissolution anodes. They use an external DC power to impress the current from an external anode onto the cathodic surface. Connections are similar to that of the cathodic protection and commonly applied to metallic storage tanks and RCC ocean structures. This is used as corrosion protection of members in the immersion zone.

Figure 3.22 shows a schematic view of the impressed current method, as applied for a steel pipeline. Anodes are externally connected to the remote pipe. The current flows from the anode to the pipeline through earth or water, which is the bulk electrolyte

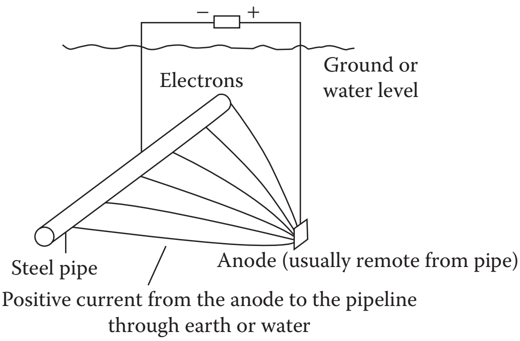


FIGURE 3.22 Impressed current method of cathodic protection.

(medium). Impressed current is passed at the location where the pipeline is laid; as it remains cathodic, it is protected. This method is effective only when the members are fully immersed in the medium; this method requires the contact of bulk electrolyte to activate the process. One of the main advantages of cathodic protection over other forms of anticorrosion treatments is its effectiveness to monitor continuously. This is possible by maintaining a DC circuit. One can record the amount of electronic flow between the anodic and cathodic terminals, which is the index to measure the effectiveness of the treatment. Cathodic protection is commonly applied to members that are surface coated, and there is high probability of this coating being damaged. For example, members in the atmospheric or splash zone have a tendency for coatings to get washed or leached off due to the chemicals present in the sea environment.

The impressed current method has few merits. As it can supply relatively a larger current in comparison with that of galvanic systems, an effective monitoring of control mechanism is highly feasible by impressing the current mechanism. It is able to provide high DC driving voltages and can be used in most types of electrolytes. As it is capable of providing a flexible output, it may accommodate respective changes in the structural members. However, there are some demerits of the system. For example, an intensive care should be taken to minimize the interaction with other structures. As it is uniformly available for larger protection surfaces, interaction between the structural members in elements is also highly feasible. Regular maintenance or monitoring is very important in this kind of protection system.

Cathodic protection is a common phenomenon that is implemented in the design stage of ocean structures. Exterior surfaces of ocean structures are protected by cathodic protection. Examples are pipelines, hull of ships, base of storage tanks, jetties and harbor structures, tubular joints in jacket structures, and foundation piles. Floating offshore platforms and subsea structures are common examples where cathodic protection is very largely deployed in the recent times. In the North Sea, the galvanic protection method against large uncoated platforms is found to be very cost-effective. Because the cost of coating in maintenance is very high, offshore engineers prefer to use galvanic protection techniques in the Gulf of Mexico or the North Sea for majority of the platforms. Galvanic systems are easy to install and are robust systems. As it requires no external power source, it is considered to be one of the main advantages.

Moreover, it provides protection immediately on float-out of the structure; this method of corrosion protection is instantaneous. Cathodic protection is also used to protect the internal surface of large diameter pipelines and ballast tanks in ships. The inner surface of large oil storage tanks are also common candidates of this method of corrosion protection. In a process industry that uses continuous circulation of coolant or water under differential temperature, cathodic protection is the most preferred method to control the rate of corrosion; if not, it can be completely prevented.

The cathodic protection system has certain requirements. This can be applied to members that are in contact with bulk electrolyte. In addition, a galvanic system requires a sacrificial anode, which should be direct welded to the structure; alternatively, a conductor can also connect the anode to the structure. A secured connection with a minimum resistance between the conductor and the structure is to be also ensured. An impressed current system requires inert anodes, which are cluster of anodes connected together often in a backfill. It also requires an external DC power source and electrically with a well-insulated system to ensure minimum resistance and secured connection between the anodes, conductors, and the power source. The source of DC power, which is vital in case of the impressed current method, can be ensured by using rectifiers of by transformer units in conjunction with an existing AC supply; alternatively, one can use either diesel- or gas-driven alternators. In remote areas, power source include thermoelectric generators and solar or wind generators for generating the required DC power for impressed current.

3.23 MATERIALS FOR REPAIR AND REHABILITATION

Materials for repair and rehabilitation of ocean structures are not under the recommendation of international codes. These codes only suggest repair procedures and desirable characteristics of materials for repair. Among several reasons for this limitation, the foremost is that the material choice for repair is case specific. Ocean structures are constructed for a variety of functional requirements, which are very specific to the type of the chosen structural system, as discussed in Chapter 2. Repair of ocean structures is required to be carried out without affecting their functional routine. Furthermore, they cannot be relieved off from the encountered environmental loads during repair. This means that ocean structures need to undergo repair, although they are under the influence of various environmental loads, which is an important challenge. The characteristics of a material chosen for repair should enable speedy construction and attain the desired strength at the earliest possible time. This is because the downtime available for repair of ocean structures is generally for a limited period; constraints may arise from the weather window or functional priorities. Moreover, the repair of ocean structures is not preventive in general but only prescriptive to functional failure. In such cases, special issues related to their survivability under critical load combinations encountered by them are a very critical issue. These types of structures cannot be dismantled or reconstructed but only be repaired.

There are instances where an extensive repair needs to be carried out under water (see, for example, the details of repair of ship dockyard, Pennsylvania, as shown in Figure 3.23). Repair carried out on the dockyard in the recent times involved a new approach of supporting the deck on a new set of piles, although the dockyard was

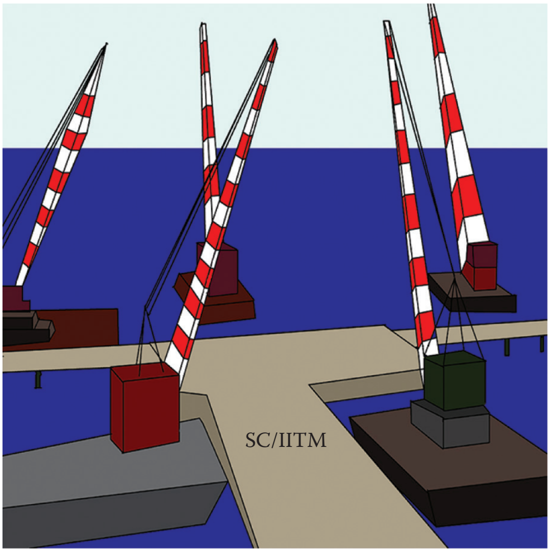


FIGURE 3.23 Ship dockyard, Pennsylvania.

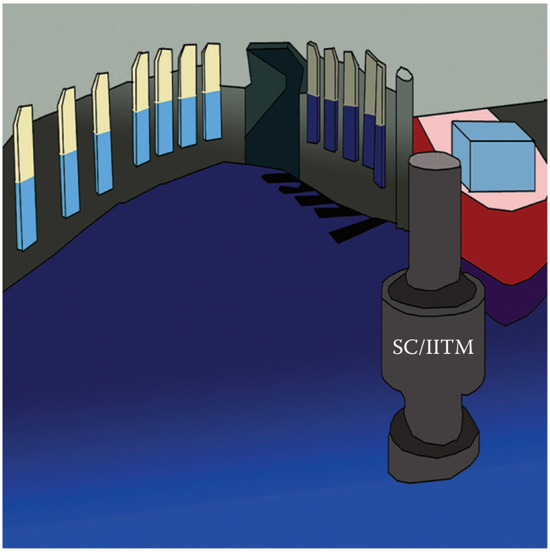


FIGURE 3.24 Upgradation of Cape May, New Jersey.

in service. Repair of the Cape May Ferry berthing Jetty in Cape May, New Jersey, resulted in enhancing the ferry-handling capacity of the Jetty (Figure 3.24). Designed constructed new boardwalks using prestress concrete bulkheads are also equipped with state-of-art fender systems. Details of the repair works carried out in Exelon Power Corporation, Philadelphia, showed that a new set of steel auxiliary piles was installed to replace the deficient piles on the front end (Figure 3.25). From the above

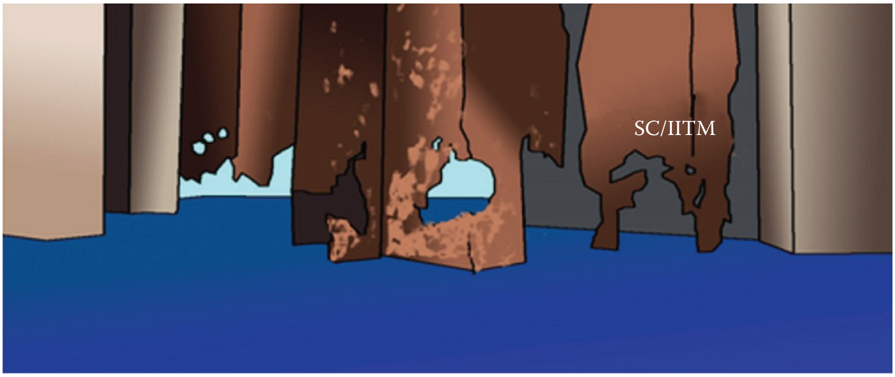


FIGURE 3.25 Schematic view of steel auxiliary piles, replacing damaged piles.

examples, it is clear that repair of ocean structures is a state-of-art procedure due to the update demand on functional characteristics and enhanced load-carrying capacity. Hence, material chosen for repair is not based on the existing design requirements but should also compensate for the degraded performance of materials that are deteriorated. Both the factors, namely, strength and serviceability, are required to be fulfilled.

Before the actual repair can be carried out, the following factors need to be established: (1) existing strength of the structure, (2) magnitude of the proposed repair, (3) cost factor, (4) shutdown time of the service of the structure, and (5) feasibility of the proposed repair work. If all the above factors are included in the study based on which repair methodology is suggested, then the study is termed as "integrity analysis." Repair of ocean structures is full of challenges. Unlike land-based structures, ocean structures need to be repaired in the hostile environment. It requires a set of specialized equipment, chemicals, and construction expertise to carry out such repairs. It also requires state-of-art electronic systems to map underwater conditions before and after repair. These equipment studies include hydrographic survey equipment, side-scan sonar imaging, instruments to measure the ultrasonic thickness of steel members, underwater photography/video, and marine borer assessment.

Repair of ocean structures also poses a set of unique challenges; the foremost is that the structure has to remain in service during repair. Therefore, the load-carrying capacity should not be challenged when repairs are being attempted on ocean structures. Specialized methods and equipment are generally used for two reasons: (1) to minimize the shutdown time of the structure during repair due to limited availability of time for carrying out repair, and (2) to minimize the damage on existing structures during repair. Other factors are as follows: (1) the repair process should also be cost-effective and (2) long-term solution is demanded. It is not because such repairs need to be evaluated under an economic perspective, but for a valid reason that ocean structures cannot be intervened for repair frequently. As preventive maintenance is not a usual practice in many of the ocean structures commissioned around the world, repair processes become more complicated as they are generally requested only on an emergency situation; enough time is not available for detailed studies and verification. Hence, offshore engineers should have a thorough understanding of various repair methodologies and chemicals available to carry out repair on emergency situation. Unlike land-based structures, ocean structures need to be repaired in the hostile condition as structures mostly have less or remote access to land. They require specialized equipment, chemicals, and construction expertise to carry out the repair of ocean structures. They also require state-of-art electronic systems to map under water conditions of the ocean structures. They include hydrographic survey equipment site scanners, and sonar imaging equipment. Underwater videography, photography, and marine borer assessment are the common methods used during repair process. Repair processes are not generally prescribed in the standard literature and are not generally recommended by international codes. This is due to the fact that various chemical admixtures that are generally used for repairs are case specific.

3.24 REPAIR OF CONCRETE STRUCTURES

Before reviewing different methods of repair of concrete structures in general and reinforced concrete structures in particular, it is imperative to understand the process of damage of concrete. Concrete structures deteriorate due to chemical reactions that occur in the marine environment. Loss of strength is mainly associated with degradation of rebar in reinforced concrete structures, which is due to corrosion of steel reinforcement. Figure 3.26 shows a graphical representation of various factors that influence deterioration of concrete along with their significance, expressed in percentage (Gettu, 2015). It is seen from the figure that presence of external chlorides influences to the maximum. Figure 3.27 shows the causes for failure of concrete structures in general but not specific to the failure of ocean structures (Gettu, 2015). It is seen from the figure that the major factor that causes failure arises from improper material specification and even incorrect choice of material amounts close to about half

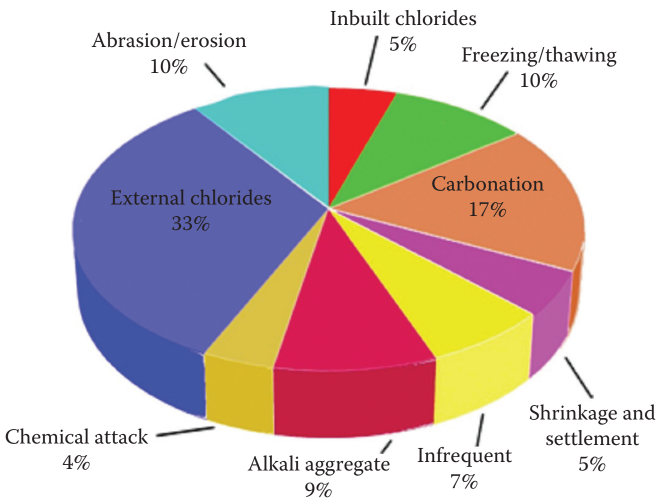


FIGURE 3.26 Factors for deterioration of concrete.

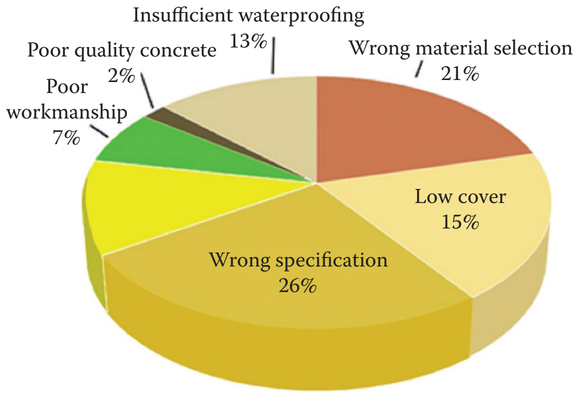


FIGURE 3.27 Causes for failure of concrete structures.

of the total factors that influence the failure of concrete structures. Hence, selection of material, both construction and repair, plays a major role in successful functioning of concrete structures, in general.

3.24.1 Deterioration Due to Chemical Reaction

Leaching and sulfate attack are considered to be serious problems for deterioration of concrete under chemical reactions. Dissolved calcium hydroxide reacts with carbon dioxide to form calcium carbonate. This forms a white powder, which is deposited within concrete core and on its surface as well. Extensive leaching could decrease the strength of concrete and also facilitate the ingression of aggressive agents into the concrete. It further reacts with rebar-embedded concrete and causes corrosion.

Sulfate attack deteriorates the strength of concrete. Sulfates react with calcium hydroxide to form a compound called "gypsum." Gypsum, in turn, reacts with hydrated compounds to form "ettringite," which results in expansion of concrete in manifold volume but initiated from the surface. Once initiated, concrete is exposed to a corrosive marine environment. This further admits the penetration of chlorides under the humid weather conditions, which in turn initiates severe corrosion in steel. In addition, attack by magnesium sulfate is more damaging because magnesium hydroxide, which is formed from the reaction, replaces calcium ions with those of magnesium. It destroys the cementing effect in concrete; further, alkali–silica reaction is one of the important reasons for deterioration of concrete. Alkali–silica reaction occurs in the presence of hydroxides of sodium and potassium that are present in cement. They react with silica aggregates to form "silicate gel," which absorbs water and further expands. Although all pores are filled with water, further expansion causes cracking. Although dehydration of gel leaves the cracks in open condition, this further deteriorates concrete. Figure 3.28 shows the damage of columns and beams of a port structure.

Alkali–carbonate reaction in concrete in the marine environment initiates the loss of bond strength and develops microcracking; this is the consequence of reaction of dolomitic limestone aggregates with alkaline material. The steps involved in the reaction are as follows: in the first step, the release of alkali from cement during hydration increases the concentration of hydroxide ions in the pore solution. In the second step, the initial hydrolysis of siliceous fraction of aggregate, present in the highly alkaline solution, destroys the integrity of aggregates. This results in swelling of alkali silicate gel by inhibition of water. This causes local swelling and increases the internal pressure, which results in the cracking of concrete. Finally, liquefaction of alkali silicate gel takes place due to further inhibition of water. This results in the expulsion of liquid gel through cracks. Figure 3.29 shows the damaged jetty

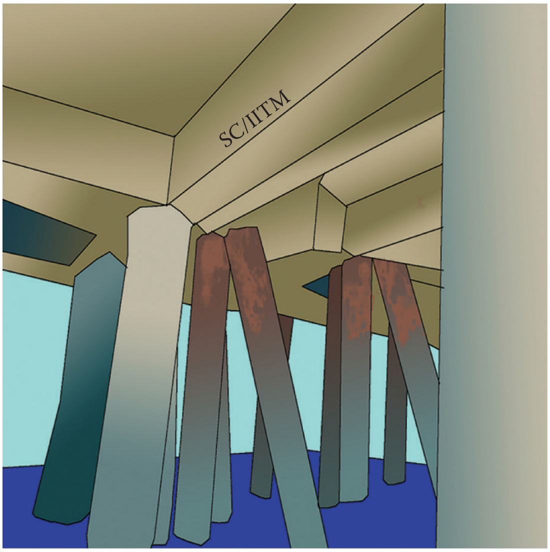


FIGURE 3.28 Damage in columns and beams due to chloride attack.

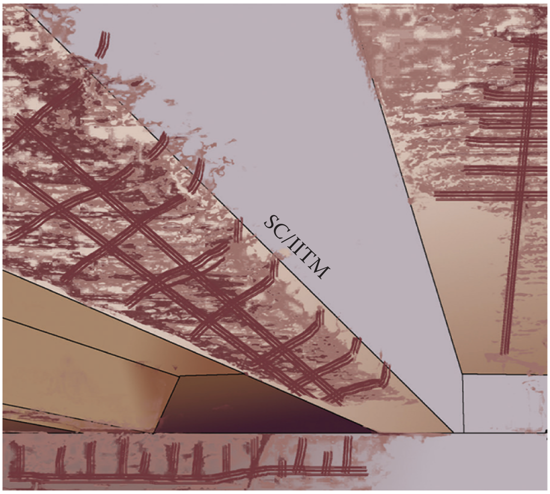


FIGURE 3.29 Damaged concrete jetty in the marine environment.

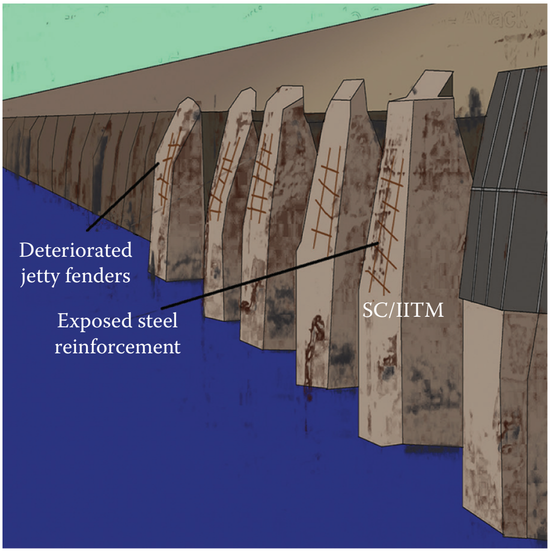


FIGURE 3.30 Damaged fender of a concrete jetty.

in the marine environment. Due to alkali–carbonate reaction, an expanded concrete resulted in spalling of cover. As seen in the figure, reinforcement is extensively corroded. Figure 3.30 shows the deterioration of fenders of the jetty due to chloride attack. Exposure of steel reinforcement is the most serious consequence of chemical attack in concrete, as this initiates a series of failure, followed in order.

3.24.2 Role of Chemical Admixtures in Repair

Admixtures play an important role in inculcating corrosion resistance to the reinforcement embedded in concrete. They influence the performance behavior of concrete under various compositions and their role is case specific. Without compromising on strength, plasticizers help to limit the water:cement ratio. This can yield concrete with low permeability, better contraction, and good quality top layer. In the presence of plasticizers, the top layer of concrete will remain dense and free from bleeding water. Retarding and plasticizing admixtures will help to achieve rapid workability when concreting is done at higher temperature. It will result in quick setting of concrete, which is not desirable as far as ocean structural construction is concerned. Corrosion-inhibiting admixtures increase the corrosion threshold of steel by providing additional resistance to rebar. High-strength superplasticizers improve the resistance to abrasion. Corrosion inhibitors provide the second line of defense to prevent corrosion of steel reinforcement; protection is provided by the alkaline nature of concrete. The most commonly used corrosion inhibitors are nitrite-based compounds. They result in the formation of a protective ferric oxide layer on steel, which protects steel from direct access. Corrosion inhibitors provided resistance at lower water:cement ratio.

3.25 ADVANCED METHODS OF REPAIR

3.25.1 Cathodic Protection

Cathodic protection forms a very major role in corrosion protection of concrete structures. It is one of the effective methods to stop corrosion of reinforcement bar in concrete. This method uses the DC from an external source through the anode that is embedded in concrete cover. When the electrons flow in between the supplemental anode and the rebar, the rebar becomes cathodic and therefore protected. Figure 3.31 shows the cathodic protection to rebar of RCC beam. As seen in the figure, cathodic protection is enabled through the current flow from an external supplement node, which is acting as an anode to the embedded reinforcement. As the supplemented anode is connected to the embedded rebar, which was corroded earlier, now they are protected by cathodic protection. Ribbon anode is also used in RCC beam prior to concrete as seen in the figure. All ribbon anodes are allowed to corrode, whereas the rebar embedded in concrete is protected.

3.25.2 Electrochemical Protection Systems

The electrochemical protection systems (EPS) is an advanced method of protection. Embedded steel remains in the passive state in concrete. In the presence of chlorides, the passive layer formed on the steel surface is attacked. This enables corrosion to progress freely. Therefore, corrosion current flows from one part of steel, which becomes the anode through concrete into another part, which becomes the cathode. This makes the steel corrode and produces rust. It is important to note

FIGURE 3.31 Cathodic protection to a rebar.

that corroded steel can expand 4–5 times that of its normal volume, which cracks concrete. This can result in spalling and delamination of concrete. This, in turn, exposes the steel further and accelerates the corrosion process. Early detection of corrosion, therefore, is a very important stage in planning corrosion prevention measures. One of the possible solutions could be a well-designed corrosion monitoring system. This provides information with respect to the rate at which the rebar is corroded. Early detection of chloride contamination of concrete is also an integral part of the corrosion monitoring system. This system consists of corrosion monitoring units embedded in concrete itself. It detects corrosion by measuring the galvanic current between carbon steel and stainless steel electrodes. It measures the corrosion rate by measuring the reinforcement potential of the embedded steel using the electrochemical polarization technique.

3.25.3 Nanolayered Coatings

Nanostructured coatings enhance surface resistance against corrosion. Nanostructured coatings are a form of gel solution, which are prepared using alkoxide tetra-*n*-butyl orthotitanate (also named as TNT). As the coating is in the form of a gel, it can be easily applied; its advantages are simplicity, homogeneity, and high uniformity of the applied coating. It forms a very thin film on application, and hence termed as nanolayered coating. Ethanol and ethyl acetoacetate are mixed together at room temperature. Tetra-n-butyl orthotitanate (TBT) is added to the solution, and it is stirred well; while stirring, some drops of distilled water are also added. For polymeric reactions to take place, the prepared solution is left for about 6 h. After surface preparation is done using titanium oxide, nano coating is then applied on the surface of steel by the submerging method.

3.26 ADMIXTURES FOR REPAIR

Admixtures are added to concrete to modify the properties of fresh and hardened concrete. There are different types of admixtures, which are available in the commercial market. Although they have different functions, all of them modify the properties of fresh and hardened concrete. Water-reducing admixtures, also called plasticizers, superplasticizers, retarding plasticizers, accelerators, surface retarders, and corrosion inhibitors, are commonly used admixtures. They are added to green concrete when poured into the formwork. A few of the admixtures can also be sprayed on the concrete surface. Water-reducing admixtures are used where improved density and quality of concrete is required. Use of these admixtures in the construction of ocean structures can result in saving of cement without affecting the workability and the strength of concrete. A typical dosage of these admixtures is about 100–300 mL per bag of cement depending on the type of admixtures. Generally, manufacturers advise the recommended dosage depending on the variety and chemical composition of the plasticizers. Typical commercial brands available in the Asian market are Conplast P211 and Conplast P505.

3.26.1 Superplasticizers

Higher range of water-reducing admixtures, also called superplasticizers, are also used in precast construction of offshore structures. Precast elements play a very major role in offshore construction as they are used in large scale. Precast members are mostly preferred due to many reasons: (1) good quality control as they are factory cast, (2) speedy construction as there is no delay time for casting and curing, and (3) repetitive size of members that makes construction easy and well organized. To enable easy compaction of concrete in the heavily reinforced precast elements, superplasticizers are added to concrete. They improve the compaction of concrete in reinforcement dense shuttering. However, they also improve the workability and cohesion between the aggregates. They aid pumping of concrete by reducing friction of flow and dry packing. It results in low porosity and improved resistance to water penetration, which is very important to protect the reinforcement from corrosion. A typical dosage is about 200–500 mL per bag of cement. A few of the commercial brands available in the Asian market are Conplast SP337 and conplast SP430, which are used as superplasticizers for marine structural systems.

3.26.2 Retarding Plasticizers

Retarding plasticizers are used to retain concrete in a fresh state when placing of concrete is delayed. This is a very common problem in offshore construction as it is controlled by the weather window and sea states. These admixtures retard the setting time and help concrete to remain green during the delayed time of placement. Such delayed construction activities in offshore engineering are termed as "cold joints." They are useful in the construction of piles in offshore structures. The usual dosage of retarding plasticizers, added to fresh concrete, is about 10–300 mL per bag of cement. One of the commonly used and commercially available brands of retarding plasticizers in the Asian market is Conplast RP264.

3.26.3 Air-Entraining Agents

Low-permeable concrete is preferred for ocean structures as it improved serviceability. Air-entraining agents, when added as admixtures to fresh concrete, ensure low permeability of concrete. The improved resistance to salt in marine and coastal structures, which is an outcome of low-permeable concrete, is also important for ocean structures. Air-entraining agents improve cohesiveness with harsh aggregates. It may be required as a fundamental requirement for massive foundation; for example, in case of gravity-based structures (GBS) platforms, seawalls, and jetties. A commercially available product in the Asian market is Conplast PA21(S). The normal dosage of this air-entraining agent varies from 10 to 200 mL per bag of cement.

3.26.4 Accelerators and Surface Retarders

Accelerators and surface retarders can be also used as admixtures in the construction of concrete structures in ocean environment. Accelerators are essentially required for concrete placed in cold weather. As we all know, in the ocean environment, the weather window varies drastically. During the construction process, if the weather window varies unsuitably for the construction process, then accelerators are required to be used in concrete to place it in cold weather. They are also required when attempting a quick repair on ocean structures. Coastal structures of strategic importance, for example, jetties used by naval bases, have very less shutdown time available for repairs. One has to therefore undertake the repair of such structures in a quick mode of construction. Accelerators are admixtures; when added to the quick repair process, they reduce the time of hardening of concrete. Concrete attains the desired strength earlier than the scheduled time, which ensures preparedness of the structure to perform the intended function. Surface retarders, when added to concrete, can improve the concrete finish in the facedown of precast members. Accelerated drying due to wind in open sea can be controlled using these kinds of retarders. Therefore, they allow enough moisture to be available for concrete to have a fresh finish. Commercial products available in the Asian market are Conplast-NC, which is an accelerator, and Conplast-SR, which is a retarder. The recommended dosage is about 1–2 L per bag of cement.

3.26.5 Integral Waterproofing Compounds

Integral waterproofing compounds are added as admixtures to fresh concrete to protect it from the penetration of water. Continuous moisture will initiate the corrosion of reinforcement; waterproofing compounds reduce the permeability of concrete and protect the rebar from corrosion. They also improve the workability of concrete and minimize shrinkage cracks. Commercial products available are Conplast X4211C and WP90, WP112. The usual dosage is about 125–200 mL per bag of cement.

3.26.6 Sprayed Concrete Accelerators

Shotcrete and Gunite are highly common in the repair of marine structures. Sprayed concrete accelerators, when added to fresh concrete, accelerate the setting time. This is a very useful admixture for underwater applications. For example, sprayset HBL is a commercial product available and commonly used in marine construction. The usual dosage of sprayed concrete accelerators is about 23 L per bag of cement.

3.26.7 Hyper Plasticizers

Hyper plasticizers are admixtures used in fresh concrete to achieve high early compressive strength and improve workability. They aid pumping of concrete because batching plants cannot be located as close to the construction site of offshore structures. Pumping of large volume of concrete can therefore becomes essential; to aid pumping, hyper plasticizers are added to fresh concrete. It is an inevitable admixture in case of self-compact concrete. Generally, they are used in narrow formwork, for example, in repair of jetties. They also aid placing of concrete in underwater construction very fast. Commercial products of hyper plasticizers are, for example, Structro-100 and Structro-485. The recommended dosage by the manufacture is about 0.25–1.5 L per bag of cement.

3.26.8 Curing Compounds

Surface treatments are also equally important for maintaining the integrity of offshore structures. They are carried out to improve the load-carrying capacity and serviceability. They are required to improve the surface finish of exposed concrete structures. Curing compounds are used as surface treatment. As a spray applied onto the surface, it will retain the moisture in concrete for effective curing. In case of deck slabs of jetties where a large area of concrete is exposed, curing compounds can be advantageous. Commercial products available in the Asian market are, for example, Concrete WB and LP 90. The usual dosage of surface admixtures varies from 200 to 270 mL per square meter area of application.

3.26.9 Grouts and Anchors

Anchors are an integral part of any mooring system, provided in ocean structures. The essential requirements of anchors and grouts are as follows: (1) They should be nonshrinking; (2) they should be free flowing; and (3) preferably the material should be cementitious. This is due to the fact that cementitious products have a good bounding with concrete, which is being repaired. In addition, they should have characteristics similar to those of shrinkage-compensated admixtures and are available in the liquid or plastic state. They should not be a solid material as compaction of grout material in a narrow hole available for the bolts becomes difficult. They should also have high strength as anchor grouts are subjected to high axial pull. It is also required that these kinds of admixtures ensure rapid setting to attain strength within shorter span of time. Epoxy-based resins are used as injected grouts. They are generally preferred for injection grouting of repairs where there are narrow gaps on the surface of concrete. In case of the occurrence of where chemical spillage may occur, epoxy resin-based grouts are more effective. Polyester resin-based chemical anchor grouts are specifically used in case of rock bed anchors, fenders, and fixing of any marine equipment. For example, Lokfix and Conbextra EUW are a few of the commercially available products for anchor grouts.

3.27 SPECIAL REPAIRS OF CONCRETE MEMBERS

Spalling of concrete is a very common problem in marine structures as it is exposed to a severe corrosive environment; presence of salt and continuous moisture is the essential factor. In the splash zone, when rebar corrodes, it expands. This results in external pressure on the cover of concrete, which makes the cover to wither off from the parent member. This is termed as "spalling." The problem associated with such special repairs is to access such kind of repairs. Such repairs are generally restricted on newly placed materials or members undergoing excessive vibration under lateral loads or operation of the equipment. In such special repairs, temporary patching is done rapidly as it is essential to plug the concrete segments from undergoing further damage; blow holes are also filled in parallel. It is required to eliminate such minor irregularities, so that major repairs of the structure can be avoided in the near future. Therefore, the emergency reinstatement of the damaged part of the structure is repaired in a small possible time; this is termed as "temporary patching." Materials to carry out such special repairs are to be carefully chosen. The chosen materials should have a proper structural grade as the parent grade of concrete used in the marine environment is generally very high. The basic grade of concrete recommended for ocean structures is M45. A material chosen for repair should have higher grade of concrete and should be preferably a cementitious material. It is important to understand that the repair attempted in ocean structures is not a cosmetic task but a functional task. Repaired members need to have the same (or even higher, if possible) load-carrying capacity for which they are originally designed. After repair, structural member should remain functional as it was earlier. A few important requirements for special materials used for such repair are as follows: (1) They should be light in weight grade; (2) they are readily available in a prepacked mode; (3) they should remain as anti-washout materials; (4) they should be nonshrinking compounds; (5) they should have high strength; and (6) they should be cementitious materials, preferably. In addition, the chosen material should enable crack sealing completely and should have a rapid setting characteristic. It should also be waterproof.

3.28 PROTECTION OF COASTAL EMBANKMENT

Prior to the selection of materials for repair of coastal structures, it is important to understand the various reasons for strength and functional degradation of coastal structures. Coastal structures are subjected to severe erosion due to natural causes and anthropogenic reasons. Various natural causes are as follows: action of waves and wind, new-shore current, tidal and storm, catastrophic events such as tsunami, slope deterioration along the coastal line, vertical movements that arise due to seabed compaction, and due to instantaneous sea-level rise because of geographical changes. Various anthropogenic reasons are as follows: dredging at the tidal entrance, construction of harbor in the near shore area, construction of other protection structures such as groins and jetties, construction of riverwater regulatory works near the shore area, and also due to hardening of shorelines by construction of seawalls and revetments. It can also result from the construction of sediment-trapping upland dams.

The primary solution for the coastal erosion problem is construction of an embankment; such embankment is termed as "saline embankment." The primary objective is to prevent the tidal ingression into adjoining paddy fields, located along the coastlines. This is a very common problem along the coastline of peninsular India. Many locations where agricultural cultivable lands are located in near proximity of the coastal sector are seriously affected by the tidal ingression. In addition, saline embankments also establish connectivity for saving people in case of flood and cyclone which are unforeseen activities. Saline embankments are constructed along the length of the coastline to protect essentially the hinterland from the coastal erosion. Figure 3.32 shows a typical coastal erosion case.

FIGURE 3.32 A typical coastal erosion case.

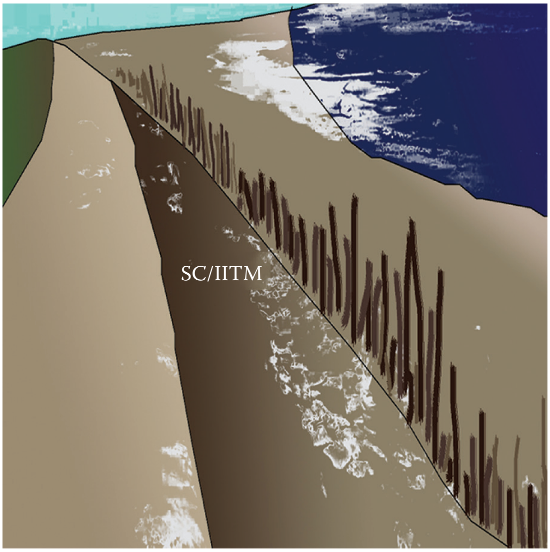


FIGURE 3.33 Temporary arrangement to improve slope stability.

As seen from the figure, a heap of sandbags are placed along the embankment to protect the coastal embankment. Alternatively, as seen in Figure 3.33, temporary wooden poles are provided to improve slope stability of the embankment, which is not an effective and technical solution. In such cases, critical geological characteristics of the site govern the design of embankment and material selection. It is common that such sites have a rich content of montmorillonite clay, which varies from about 7% to 21%. Soil has a very high moisture content. Pore fluid is highly corrosive with enrichment of gypsum, and such regions are also enriched with groundwater potential, which adds to the complication. The retardant embankment soil acts as an impervious layer, creating high pore pressure in the soil column. If the constructed embankment has a very low slope gradient, it will result in instability. Excessive pore pressure warrants a flexible system instead of a rigid system such as rubble-mound embankment. Material selection should also meet the appropriate coastal regulations as applicable. Conventional sandbagging will not be effective for sites where the erosion rate is very high.

Geotube embankment is an alternative solution to address such site-specific problems. Figure 3.34 shows a typical cross section of a geotube embankment. Geotubes are cost-effective and have long service life. They remain noncorrosive in marine environment and are durable with high tensile strength. Apart from enabling speedy construction, they are very effective in dissipating wave energy by using gabion boxes filled with stones. Gabbion boxes are tied together with armour units to maintain the slope stability. Figure 3.35 shows a schematic view of the Geotube

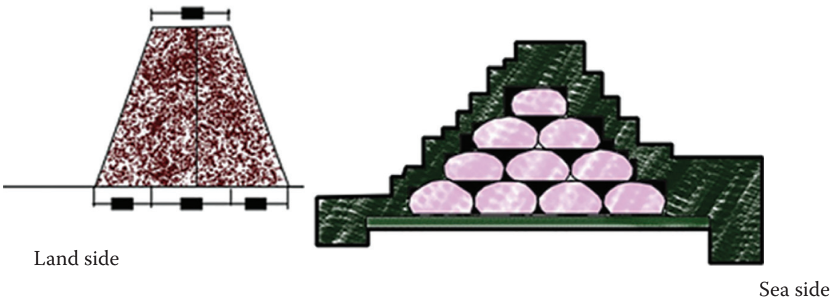


FIGURE 3.34 Cross section of geotube embankment.

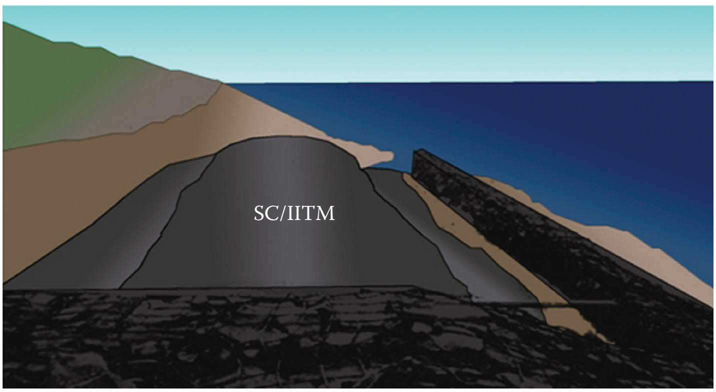


FIGURE 3.35 Geotube embankment with layers of gabion boxes.

embankment, layered with gabion boxes. Table 3.7 shows a technical comparison of geotube embankment with that of a conventional rubble-mound embankment.

Geosynthetic tubes are used as breakwaters to prevent soil being eroded by waves and current. Geosynthetic tubes are used as artificial dunes, reefs, dyes, or groins. SoilTain tubes are one such common application being widely used in practice. SoilTain tubes are geotubes manufactured by HUESKER Synthetic GmbH, Germany. Figure 3.36 shows an embankment constructed with geosynthetic tubes.

TABLE 3.7

Comparison of Geotube and Rubble-Mound Embankments

Description	Geotube Embankment	Rubble-Mound Embankment
Suitability of site with high pore pressure	Suitable as it is a flexible system	No as the system is rigid
Risk of soil erosion to adjacent unprotected area	Minimizes the risk as it dissipates the wave energy effectively	No such mechanism is in place
Design adaptability	Most suitable due to extensive flexibility offered by geotubes	Rubble-mound design is stiff
Eco-friendliness	Remain noncorrosive in the marine environment	Remain noncorrosive in the marine environment
Time of construction	Comparatively faster	Comparatively slower
Inspection and maintenance	Easy to inspect and to carry out periodic repair	No effective procedures are devised and practiced
Geotechnical considerations	Suitable for mud-sliding soil	Not suitable for such sites

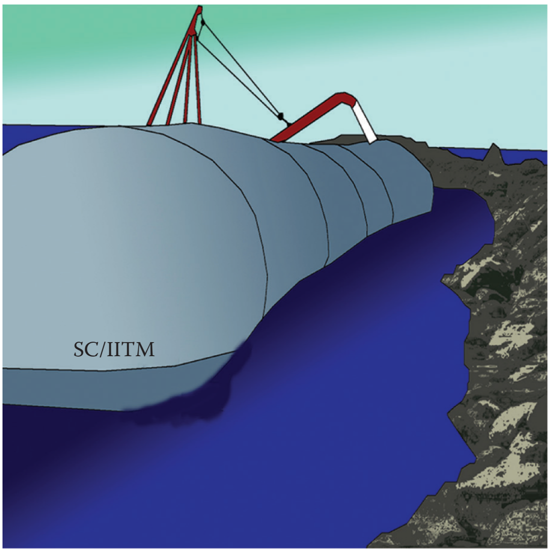


FIGURE 3.36 Embankment with geosynthetic tubes.

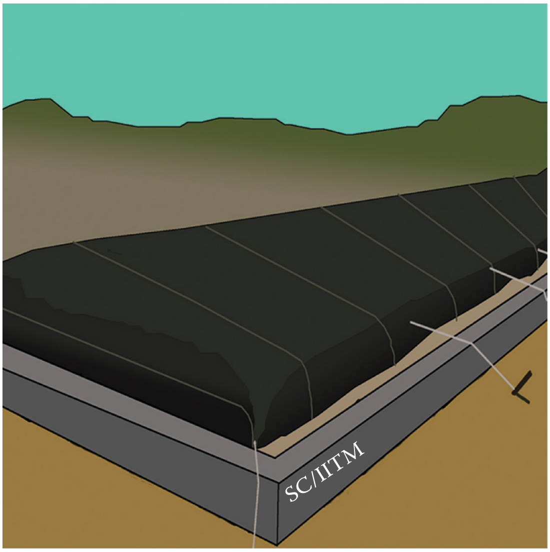


FIGURE 3.37 Dewatering tubes for clearing dredged material.

Dewatering of dredged material is necessary for its effective and compact disposal. This addresses the dredge disposal problems and handling capacity of dredgers. As shown in Figure 3.37, dewatering tubes are used in the recent times for clear-off dredged material. These tubes are made of special geo-textiles, which execute the gravimetric drainage of the sludge. This results in a significant reduction in volume of the dredging material. Slurry is captured inside and water escapes; this process makes it convenient to handle and dispose the slurry and dredge spoil.

3.29 STRUCTURAL ASSESSMENT OF A JETTY FOR ENHANCING LOAD-CARRYING CAPACITY: CASE STUDY

In this section, we will discuss a case study that necessitated a detailed static (and dynamic) analysis of a jetty. Assessment for enhancing load carrying capacity of the deck is carried out as additional loads are expected from the crane with enhanced capacity. The study demands a detailed mathematical modeling of the jetty with the material characteristics of the *in situ* condition. This is one of the classical examples, which integrated experimental and analytical investigations as applied to structural assessment and failure analysis of a marine structure. Assessment of the *in situ* strength of reinforced concrete members is done through NDT and destructive tests as well. Deduced strength of the reinforced concrete members is used in the analytical model for checking the load-carrying capacity with respect to the new crane of double the capacity of the existing ones. It is interesting to note that in such situations, it is not possible to refuse/constrain the enhanced load capacity of the jetty as this is an obligation from a strategic point of view. No conventional repairs can enhance the strength even if it could restore the original strength. Existing reinforcement details of the beam and reinforced concrete panels are modeled using a finite-element software.

3.29.1 Experimental Investigations

Detailed experimental investigations were carried out on the jetty to assess the *in situ* condition of the jetty. Ultrasonic pulse velocity (UPV) values indicate that the integrity of concrete in RCC walls will be considered as "medium." UPV values in the cores indicate that the integrity of concrete may be considered as even "good." Results of the rebound hammer test indicate that the near-surface characteristics of concrete are good. Results of the core tests indicate that the equivalent cube compressive strength of the concrete in RCC walls is in the range of 11–48 MPa. Majority of the core samples showed equivalent cube compressive strengths in the range of 20–27 MPa. Carbonation test revealed that the penetration of CO_2 is negligible in most of the locations. Results of the half-cell potential tests indicate that the probability of corrosion is high in the deck slab and RCC walls as well. Results of the chemical analysis of the powder samples in RCC walls indicate that chloride contents are below the threshold limit of 0.6 kg/m^3. pH values of the concrete powder samples indicate the availability of sufficient alkalinity in the concrete, which is a good sign for strength restoration.

3.29.2 Analytical Investigations

Figure 3.38 shows the details of the analytical model generated for the study. The details of the structural model and the corresponding loading details are shown in the figure. A detailed 3D analysis is carried out to ascertain the critical stresses in the members and conclusions are drawn. Figure 3.39 shows the results of the analytical studies.

Based on the failure analyses carried out, it is seen that the maximum stresses in the deck plate are within the permissible limits. The actual *in situ* strength that is computed based on the equivalent cube strength of RCC samples obtained from destructive tests shows 20–27 MPa; these are used in the analysis. Stresses at corners are more than the permissible values due to the uplifting of corners of the deck slab resulting from torsion. However, it is seen that there is a possibility of redistribution of these stresses as the structure has very high degree of indeterminacy. von Mises stress values, indicating the yield criterion for deck slab, also show values closer to principal stresses. Shear stress yield criterion, shown by Tresca stresses, is not a governing criterion for the current problem under investigation as the deck slab is governed predominantly by bending and not by shear. Figure 3.40 shows a detailed insight of stress exceedance nodes. It is seen that the nodes where the stress exceedance levels are seen in the analysis are not in the vicinity of the crane rails.

Based on the above study, it is seen that strength assessment is done based on the NDT/destructive tests on various concrete members of the jetty. In addition to

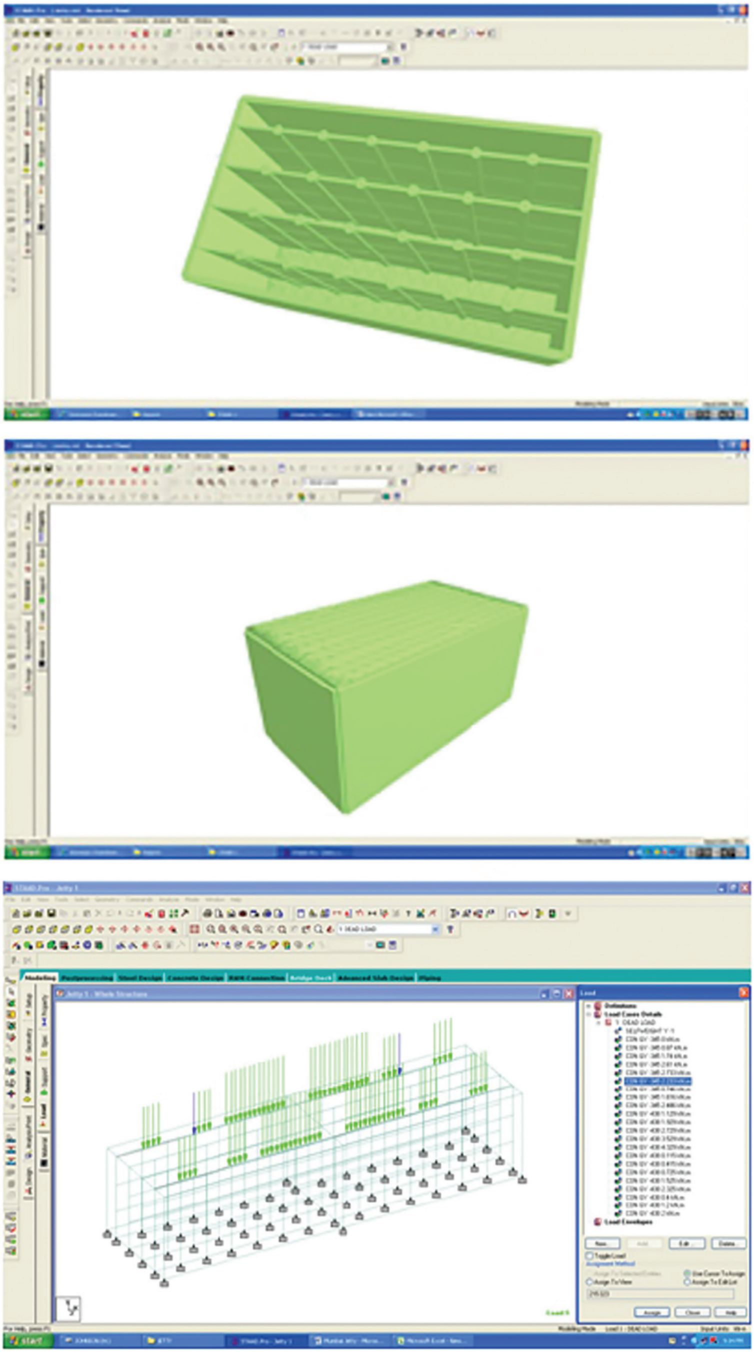


FIGURE 3.38 Details of an analytical model.

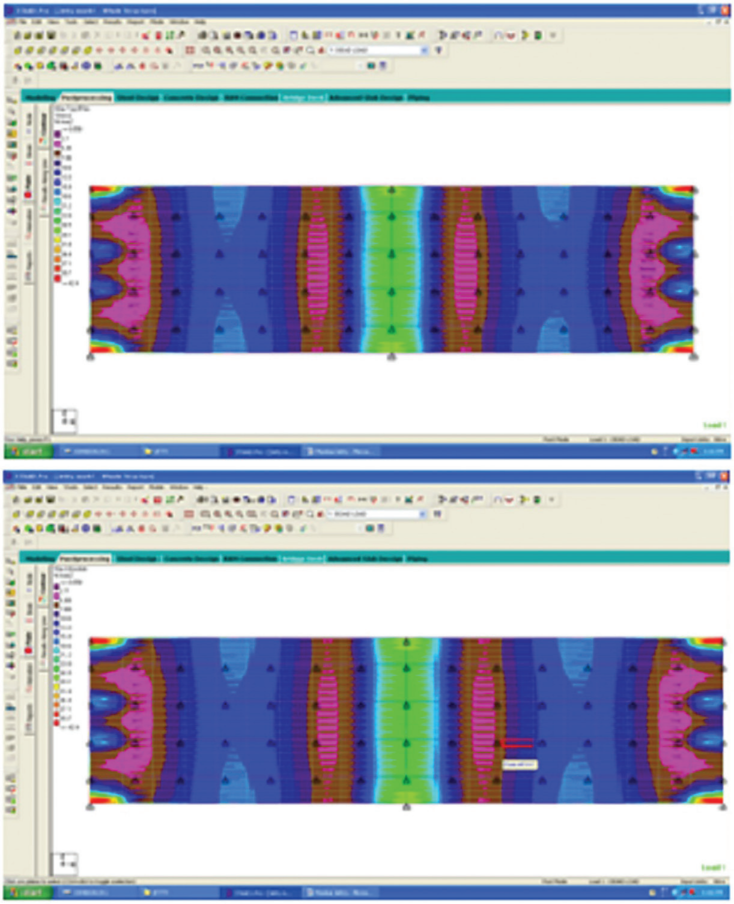


FIGURE 3.39 Results of analytical studies.

the dead load of the structure, wheel loads are also applied onto the crane rails for the enhanced crane capacity. It is also seen that the stress levels in the deck slab are within the permissible limits; however, stresses at corners are more than the permissible values. These stresses will get redistributed as the structure is highly indeterminate. This is probably imposed due to the rigid connection of the deck with the RCC walls. Principal stresses in the deck at a few critical nodes exceed desirable values. Von Mises stress values, indicating the yield criterion for deck slab, also show values closer to principal stresses; these nodes are not seen in the vicinity of crane rails. Shear stress yield criterion, shown by Tresca stresses, is not a governing criterion for the current problem under investigation due to the fact that the deck slab is governed predominantly by bending and not by shear. RCC walls, tie beams, and the deck beam do not show undesirable stress values under the considered loads and load combinations.

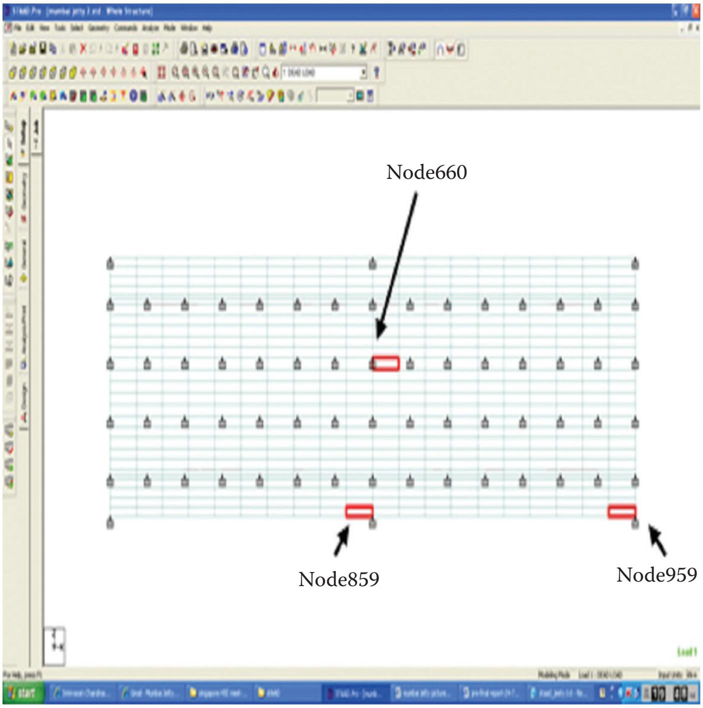


FIGURE 3.40 Stress exceedance levels in the deck slab.

3.30 REPAIR OF OCEAN STRUCTURES USING CHEMICAL ADMIXTURES

In this section, we will discuss a few recent advancements in the repair methodologies of ocean structures.

3.30.1 Electrochemical Protection System

This is a recent development for cathodic protection of reinforced concrete structures and steel structures. The RECON control system, developed by SAVCOR Group Ltd, Australia, is one of the advanced corrosion control and monitoring system. It consists of a full-monitoring and corrosion control using EPSs. Constant current and potentiostatic mode of control allow an automatic adjustment of the circuit current based on selected values of multiple reference electrodes. EPS can also be employed for corrosion protection of new RCC structures. Chloride-induced corrosion is one of the major causes of concrete deterioration in RCC structures in the marine environment. EPS is one of the recent methods that has been found to be effective in addressing such problems. It has a proactive approach to address such durability problems. EPS components of a fresh construction include activated titanium anodes, monitoring probes and sensors, and high durability anode and steel connections.

It is important to understand the cause of corrosion before addressing strength degradation of materials and members due to corrosion. Rebars in RCC are initially protected from corrosion by a passive layer, which is formed due to high alkaline nature of concrete. Corrosion occurs when this passive layer is destroyed or perforated. Many studies are carried out to assess the condition of concrete in the marine environment using visual and delamination surveys, electric continuity testing of rebars, half-cell potential and resistance mapping of RCC, corrosion rate measurement of steel, resistivity testing, measure of cover to rebar, alkali–aggregate reaction tests, and predictive deterioration modeling. It is seen that restoration of alkalinity in concrete is the most effective method of rehabilitation of concrete in the marine environment. New concrete has natural inherent alkalinity, which offers passive protection to rebar. Ingress of carbon dioxide creates a carbonated concrete of lower alkalinity, resulting in the loss of passive layer; this also accelerates the corrosion of rebar. Realkalization is one of the nondestructive methods of concrete rehabilitation. This involves an electrochemical technique of passing a sustained low-voltage current between the temporary anodes on the surface of concrete over a short period of time (3–15 days). Paste of cellulous fibers, saturated in sodium carbonate solution, is used as an electrolyte covering the concrete surface. As surface anodes are embedded in this alkali-rich paste, alkali is drawn into the concrete and reaches rebars, which in turn realkalize the concrete. A natural protective (passive layer) film is then formed, which offers protection against corrosion of rebar.

3.30.2 Methodology of Realkalization

Prior to realkalization, the following tests are carried out to ascertain the nature of treatment: (1) visual inspection, (2) depth of carbonation, (3) chloride analysis, (4) cover to rebars, (5) delamination survey, and (6) alkali–aggregate reaction tests. Rebar connection is installed in the concrete by connecting cables (black in color) to the rebar. These cables are extended to a transformer rectifier circuit. Continuity of the rebar is to be checked and established, which is vital. Anode connections are then made using red cables, which are subsequently extended to the transformer rectifier unit. A reservoir is used to house the anode mesh and alkaline electrolyte. The common practice is to use a cellulous fiber that can be sprayed onto the concrete surface. Wooden battens are fixed to the concrete surface, which act as spacers between the concrete and the anode mesh. The anode mesh is fixed to the battens and the cellulous fiber is sprayed onto the mesh. A regular wetting of the fiber with an alkaline electrolyte is to be ensured during the repair period. Cables from the rebars (cathode) and mesh (anode) are connected to negative and positive poles of the transformer rectifier unit to supply the design current density. During the test period, if required, the concrete surface can be realkalized after switching off the power. The whole arrangement is monitored during the test period by means of current and voltage readings; pH levels of concrete are also periodically tested. Cables are disconnected after the test period and the concrete is cleaned with water.

4 Offshore Structures
Construction Methods and Equipment

4.1 INTRODUCTION

The offshore oil industry has a history since World War II. In the Gulf of Mexico, an offshore structure was placed in 1945 at 6 m water depth by drilling and an exploration well; a platform was installed in 1976 at 200 m water depth offshore California; and in 1978 at 300 m water depth in the Gulf Mexico. Planning of an offshore structure is complex due to a variety of parameters involved in deciding the type of structure that is to be commissioned. Apart from the water depth and sea state suitability, other factors such as functional aspects, geometric form, and construction and installation methods are also equally important. Figure 4.1 shows the preferred range of frequency bands within which offshore platforms are operable.

Offshore structures are deployed in different sea states as seen in the figure. The fundamental period of compliant-type platforms vary from 40 to 60 s, which is shown in the low-frequency zone, whereas that of fixed platforms varies from 3 to 5 s. A typical sea spectrum is also superimposed in the same figure to illustrate the basic idea of design. It is explicit that periods of offshore structures are chosen to remain far away from the maximum energy content of the sea spectrum. Apart from the choice of a structural form for offshore platforms, the riser system for deepwater also poses challenges during their choice of the support system, material selection, and so on.

4.2 DEEPWATER RISERS

Figure 4.2 shows a schematic view of deepwater riser systems. Riser is a structural member, which connects the subsea to the topside. Figure 4.3 shows the cross section of a drilling riser. Drill string is located in the core, whereas auxiliary lines are circumscribed around it. In addition, some parallel lines are interconnected to kill or choke the wells as and when required. They are called as kill risers, which are about 500 mm in diameter. There are different types of risers, namely, drilling risers, production risers, export risers, and water/gas injection risers.

Figure 4.4 shows a schematic view of a drilling riser. Drilling risers are typically top tension risers (TTRs), whereas production risers are further classified as flexible, steel catenary, and hybrid risers. Hybrid risers can be either single or grouped.

Various factors that govern the design of risers are as follows: the floater type and its motion characteristics, water depth, environmental conditions, and operational pressure and temperature. These factors initially drive the design of the risers in deepwaters.

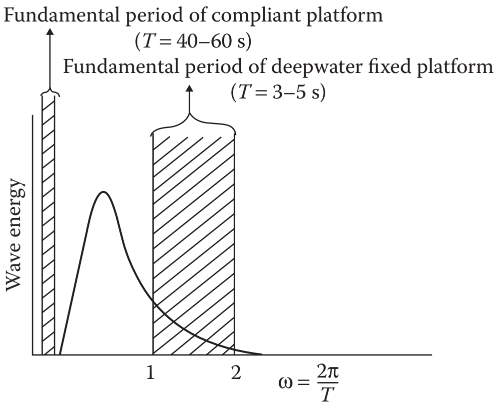


FIGURE 4.1 Operational frequency band for offshore platforms.

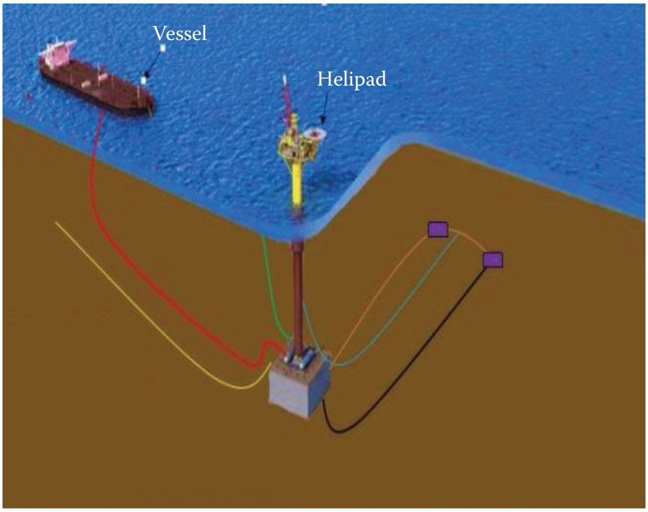


FIGURE 4.2 Deepwater developments.

4.2.1 Top Tension Risers

TTRs are provided with surface trees under high wellbore pressure. They are successively deployed in various operational platforms in the Gulf of Mexico at various water depths varying from 300 to 2500 m. As the name suggests, TTRs require a tensioning system to impart tension on the topside of the risers. A mixing string is provided along with TTRs to compensate the induced tension. Figure 4.5 shows a schematic view of a TTR that is deployed for production drilling. There are other alternate tensioning systems, which are commonly deployed for TTRs in production risers of deepwater drilling systems. They are (1) air-can system or (2) hydro-pneumatic tensioning system. Air-can systems are commonly seen in single point anchor reservoir (SPAR) production risers, whereas hydro-pneumatic tensioning systems are common

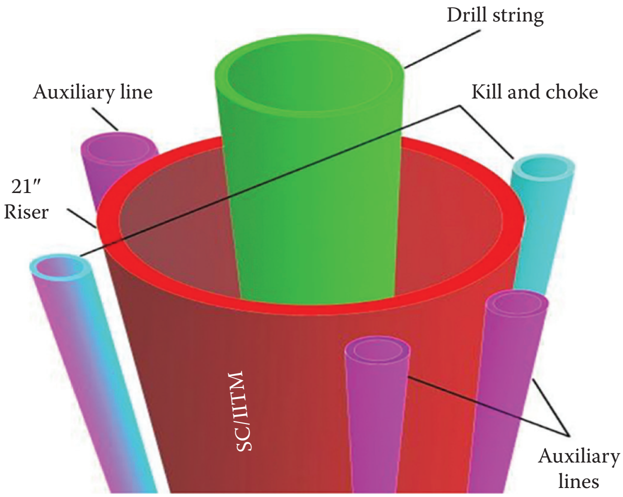


FIGURE 4.3 A typical cross section of a riser.

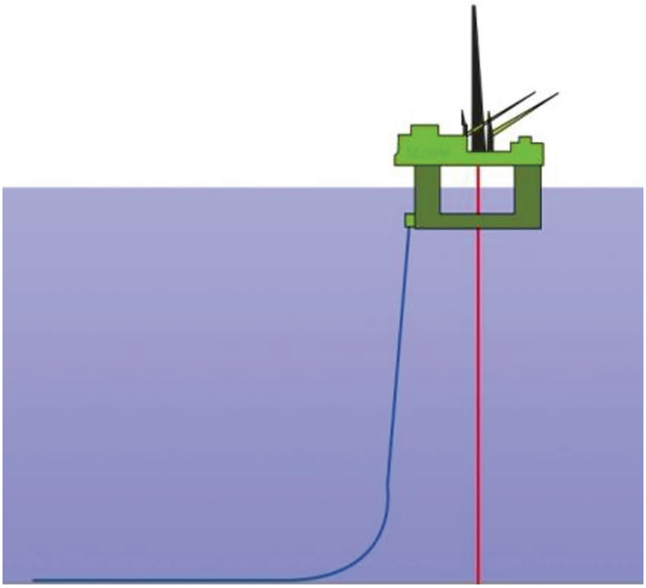


FIGURE 4.4 A typical drilling riser.

in tension leg platforms (TLPs), drill ships, and spar as well. The hydro-pneumatic tensioning system used in SPAR drilling risers consists of a tensioning joint and a tension ring. Piston rods are attached circumferentially to the casing of the tension ring. This is subsequently connected to the hydraulic cylinders at the bottom, which is housed on the cellar deck. This arrangement results in Ram-style (Ram-style means short stroke production riser) connection in the hydro-pneumatic tensioning system.

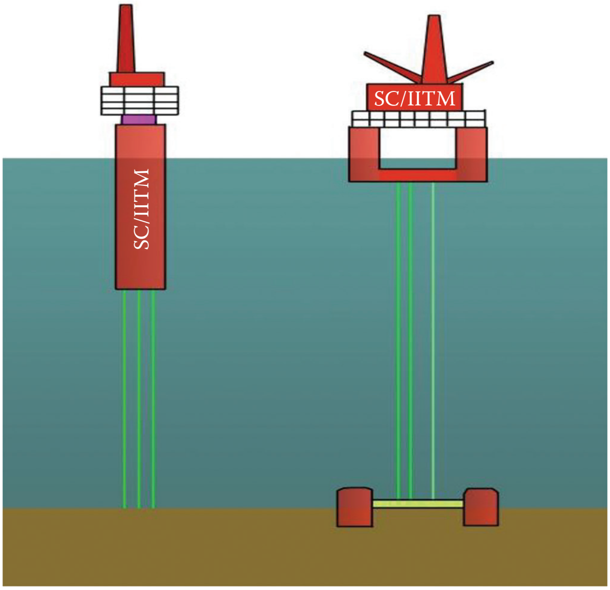


FIGURE 4.5 Top tension risers.

Alternatively, TLPs use a tensioning style, which is slightly different from that of the Ram-style (Chandrasekaran et al., 2013). The tree deck on the top consists of hydraulic cylinders, which are mounted at different angles. These cylinders are supported by accumulators, which are subsequently connected to a load ring. The load ring is terminated to a TLP production riser. Considering the pressure involved in drilling risers, low-pressure drilling risers are typically used for semisubmersibles, drill ships, and jack-up drills. They are deployed at a water depth of about 3000 m. The governing factor for low-pressure drilling risers is that the buoyancy may cost about 50% of the overall riser, which makes the riser up to 98% buoyant. Different couplings that are commonly used in low-pressure risers are flange, dog, and split ring couplings.

4.2.2 Steel Catenary Risers

Figure 4.6 shows a schematic view of steel catenary riser (SCR). This is generally connected to a TLP where there is a tapered stress joint. Usually, SCRs are connected to the topside of the production platform, which is subsequently connected to the drilling units. Vortex-induced vibration (VIV) will be normally more in deepwater risers due to large displacements under wave loads (Chandrasekaran and Pannerselvam, 2009). VIV streaks are provided in these regions at various segments of the riser, as shown in the figure. SCR essentially consists of a steel, an insulated pipe, or a pipe-in-pipe system. They are successfully deployed at water depths varying from 500 to 2500 m and commonly used in TLPs and spar platforms. Although SCRs show low material cost for large diameters, they can sustain high temperature and pressure. The possibility of internal inspection is one of the

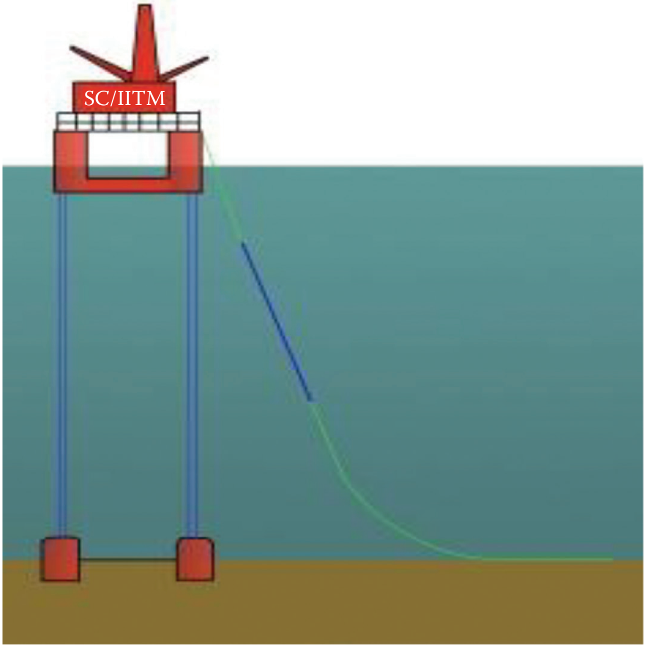


FIGURE 4.6 Steel catenary risers.

most attractive features of SCRs. Pipe-in-pipe is one of the common systems of SCRs, which is found very common in the Gulf of Mexico.

Responses that arise from large vessel motions, large bending at the touchdown zone resulting in local buckling, high fatigue at the touchdown zone, and soil–riser interaction uncertainties are a few of the design challenges of SCRs (Chaudhary and Dover, 1985). These design challenges are addressed by varying the weight along the riser. Weight distribution is generally carried out by varying the density of coatings or even the steel wall thickness of the riser. The coatings are generally industrially qualified materials such as poly ethelene (PE), poly propelene (PP), poly urathene (PU), or rubber coatings, which are commonly used. These coatings and steel wall thickness of the risers vary the weight along the riser length profile. Hence, they are known as "weight-distributed SCRs." Figure 4.7 shows a typical SCR with distributed weight. In weight-distributed risers, different kinds of weight distribution occur along the profile of the riser. The red color in the figure shows the normal riser, which extends from the initial point on the topside to the touchdown zone on the seafloor. Heavy weights (or light weights), as shown in blue or green color, are attached to this riser at different locations.

4.3 FLEXIBLE RISERS

Flexible riser has a hybrid combination of different materials. Typical details of the cross section are shown in Figure 4.8. The majority of offshore platforms use flexible

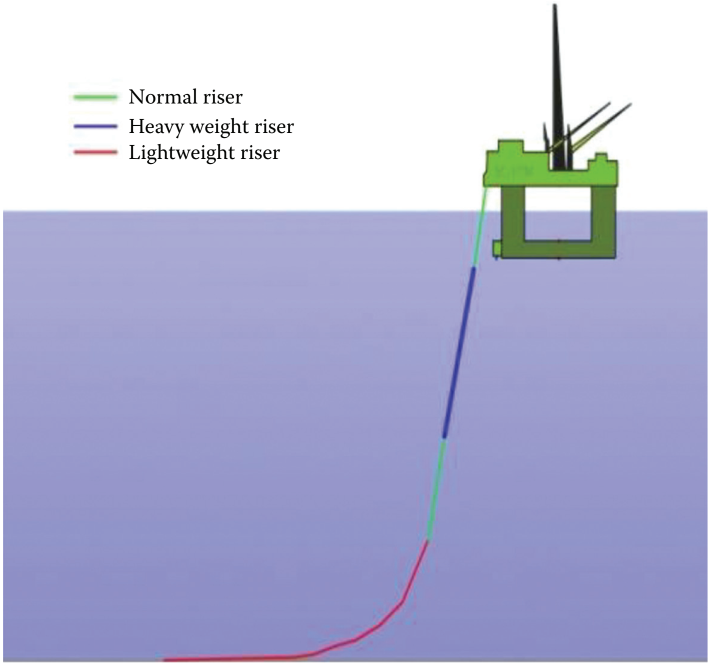


FIGURE 4.7 Weight-distributed SCR.

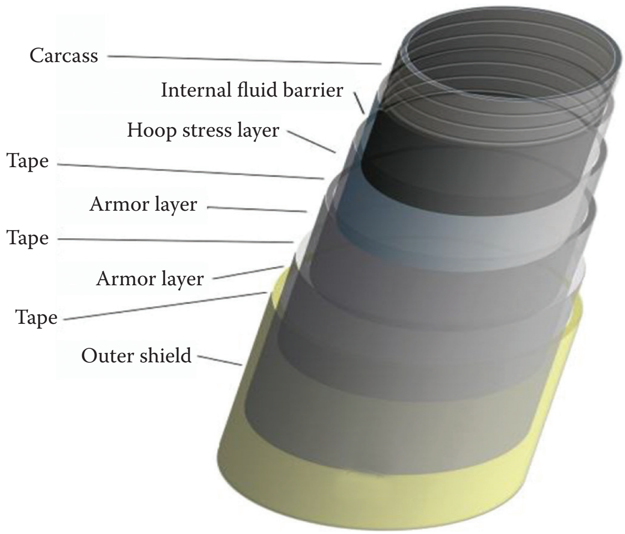


FIGURE 4.8 Flexible risers.

risers as they are capable of withstanding dynamic loads that arise from waves. These risers are deployed in deepwaters up to a depth of 2000 m. They are also commonly used in the fleet of large installation vessel in large volumes and numbers. Apart from the fact that they are very easy to install, highly flexibility, suitability to resist dynamic loads, intensive corrosion resistance, and high reusability make them highly suitable for deepwater offshore applications. As seen in the figure, the inner core is a carcass material, which is surrounded by a couple of hoop stress layers. The carcass layer is subjected to an internal fluid pressure caused along its circumference, which induces a high hoop stress; this is counteracted by the hoop stress layer. It is further reinforced by a circumferential layer, which is subsequently covered by a flexible tape. To protect the riser from any external damage, the exterior surface of the risers is covered by two armor layers. The armor layers are further coated and covered by a tape, which acts as an outer shield. Flexible risers are thoroughly protected and manufactured to become corrosion resistant with high degree of reusability. One of the major advantages of flexible risers is that they can be used for large installation fleet vessels even at greater water depths (>2000 m).

Flexible risers are easy to install, highly flexible, and robust for dynamic applications. They possess intensive corrosion-resistant properties and high degree of reusability. Generally, the concept of manufacturing flexible risers is that concentric pipes are connected to form pipe-in-pipe systems. They are heat treated and developed for a specific use.

4.3.1 Flexible Riser Configurations

There are different kinds of configurations of flexible risers, which need to be known before selecting the method of laying of these risers. Figure 4.9 shows different configurations of flexible risers. These are specific kinds of geometric shapes of risers, which are used for flexible risers in deepwater layouts. Figure 4.10 shows the envelope of different risers deployed in varying water depths. With reference to the figure, it is seen that the riser diameter increases in the case of SCR and at a greater water depth. The current practice in the riser design is to limit the riser diameter to a maximum of 450 mm. Larger diameter risers are usually designed as bottom-weighted risers, which are seen as alternate solutions for deepwater risers.

4.4 FREESTANDING TOWER AND HYBRID RISERS

Freestanding tower risers are alternatives for SCRs. Figure 4.11 shows a schematic view of a freestanding tower riser. A typical riser was first used in the Placid's riser system in Gulf of Mexico at a water depth of about 470 m. The riser was deployed in the semisubmersibles in Girrasol. These type of risers are highly suitable for drill ships and semisubmersibles. Although their installation and flow assurance are more comfortable, they are more adaptive to deepwater installations. Negligible subsea footprint and hang-off area are main advantages of these type of risers. Once these risers are installed, vessels can load lower than those of the flexible risers and SCRs, although they also have controlled onshore fabrication. In addition, these risers have lower in-plane stresses as they are supported by the tower system. Integral and nonintegral buoyancy

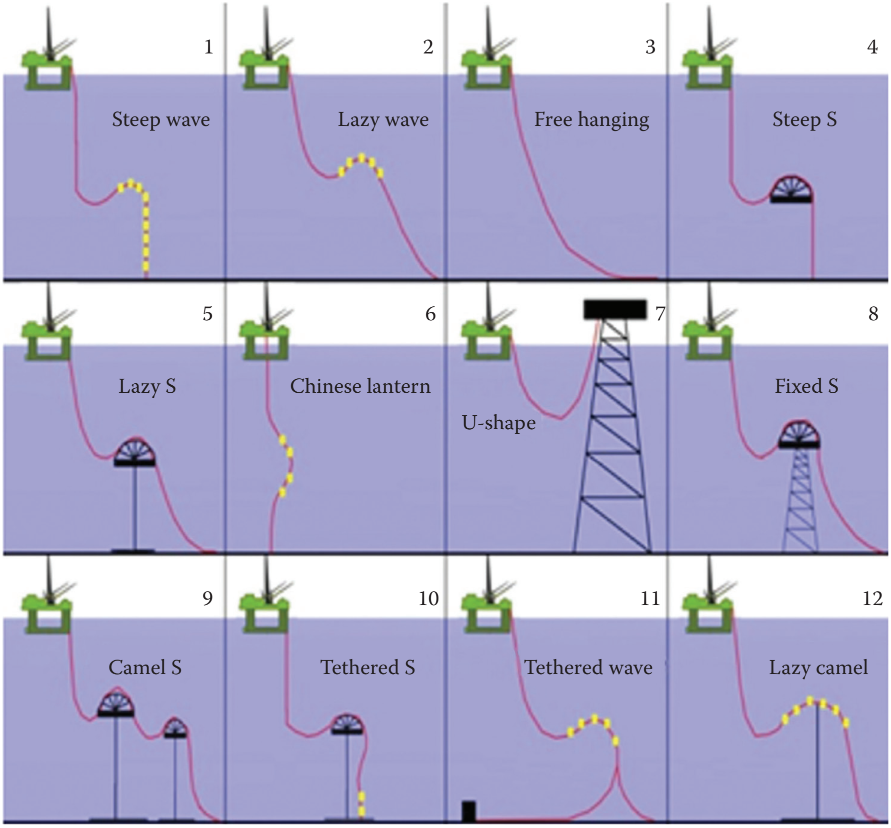


FIGURE 4.9 Different configurations of flexible risers: (1–6) standard flexible riser configurations; (7–12) alternative flexible riser configurations.

types are the different types of freestanding tower risers. They are directly connected to floating, production, storage, and off-loading (FPSOs) and FPS. Generally, they use lightweight materials, which make the riser-laying and riser support system simple in design and installation. Freestanding hybrid risers are also commonly used for deepwater applications. Figure 4.12 shows a schematic view of a freestanding hybrid riser. They are well suitable for drill ships and semisubmersibles. They are very useful for supporting buoyancy tanks. They are connected to FPS directly.

4.5 SINGLE-LINE OFFSET RISERS

Single-line offset risers (SLORs) are primarily deployed for deepwaters up to a depth of 3000 m, which use a freestanding steel section supported by an air can near the surface. The vertical section is connected to the production vessel by a short flexible jumper. The SLOR can be either a single pipe or a pipe in pipe, which is common in gas-lifting systems. The salient advantages of SLOR are the ability to resist dynamic loads, the capability to withstand thermal requirements, and field

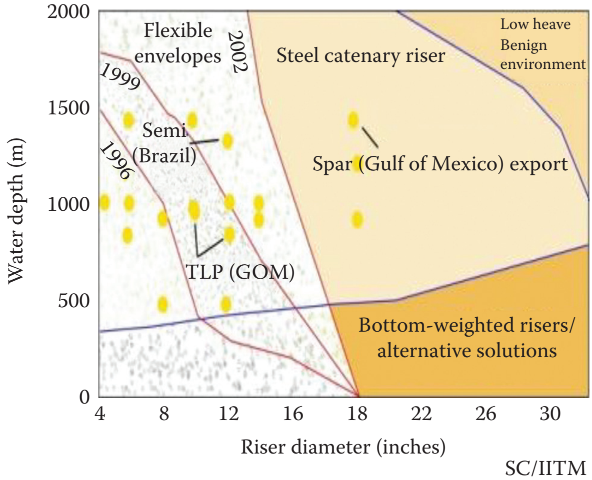


FIGURE 4.10 Riser envelopes.

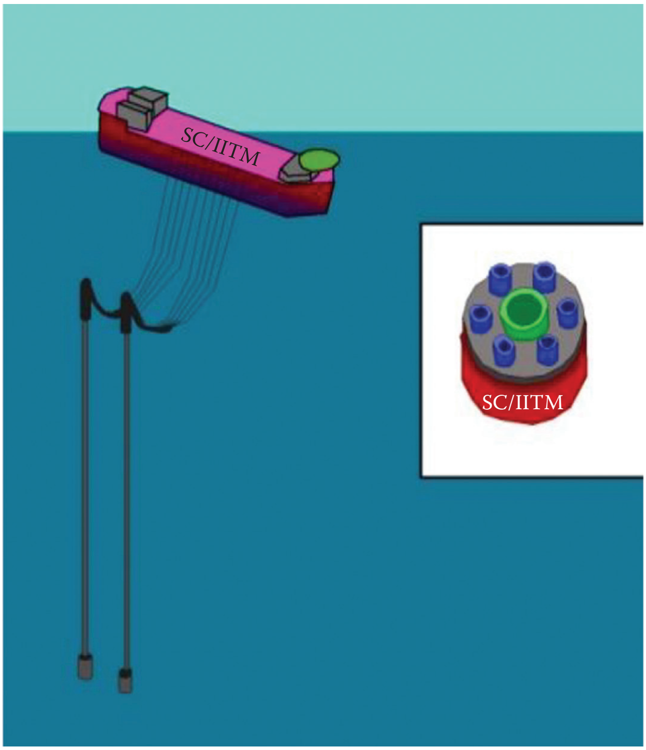


FIGURE 4.11 Freestanding tower riser.

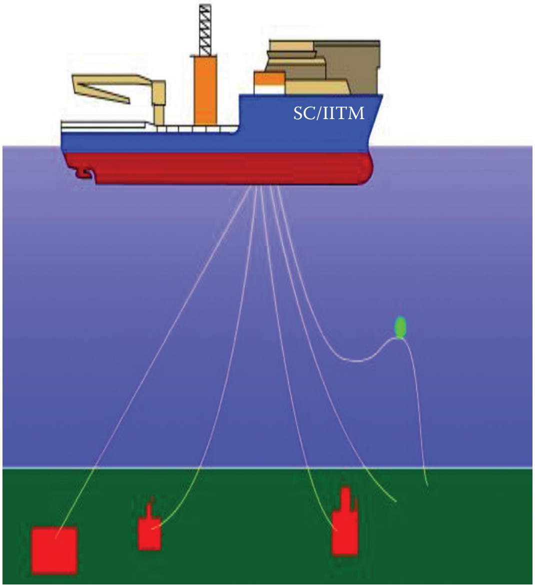


FIGURE 4.12 Freestanding hybrid riser.

development flexibility. They have low fatigue sensitivity and hence can be used with vessels that exhibit large dynamic motion. The common diameters of risers are 450 mm with a single jumper and 750 mm with manifold jumpers. They are primarily used in production and export lines. Figure 4.13 shows a schematic view of grouped SLORs, which are deployed for deepwater applications. The key features that make

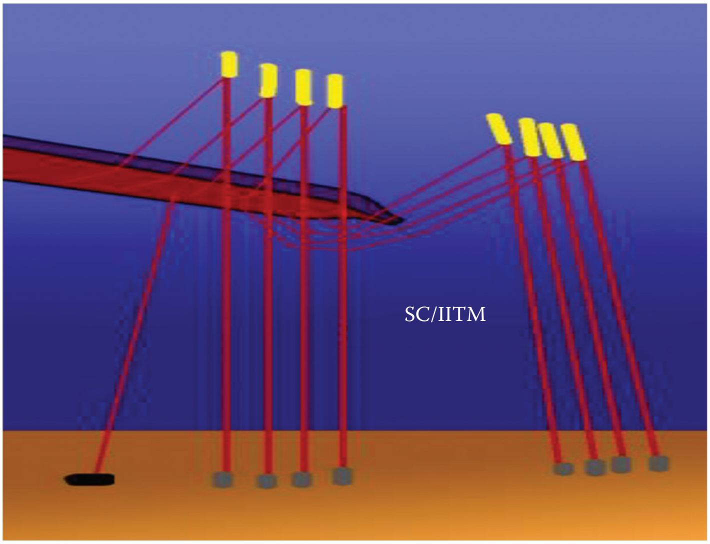


FIGURE 4.13 Grouped SLOR.

SLORs highly suitable for deepwater systems are as follows: flexible configuration, small subsea footprint and less hang-off area, and controlled onshore fabrication. These are well suitable for drill ships and semisubmersibles.

4.6 SPOOLABLE RISERS

In the recent times, spoolable production risers with composite materials are seen as a new alternative type of risers. These risers are deployed at 3000 m water depth with diameters varying from 100 to 400 mm. These risers are highly flexible and hence can be spooled off while transporting them. Figure 4.14 shows a schematic view of a spoolable production riser. These risers are lighter than steel as they are manufactured using composites, but have high strength and good fatigue properties, making them suitable for deepwater applications. Different types of spoolable composite risers are as follows: flexibles with composite armor, which have a 30% weight reduction; new, all composites unbonded flexible risers; bonded composite risers; large-diameter risers that are useful for export risers; and risers, as suspended flow lines.

4.7 FACTORS INFLUENCING THE DESIGN OF OCEAN STRUCTURES

In order to understand the construction practices and learn the choice of appropriate construction methods, a detailed idea on the basic design parameters is useful. The design methodology of ocean structures differs with the type of offshore structures. Although vertical deformation will be less in the case of fixed structures such as jacket platforms and gravity-based structures (GBS), compliant structures are more flexible as they are displaced more under wave action. Floating structures are generally influenced by disturbing factors such as waves and wind, whereas their restoring

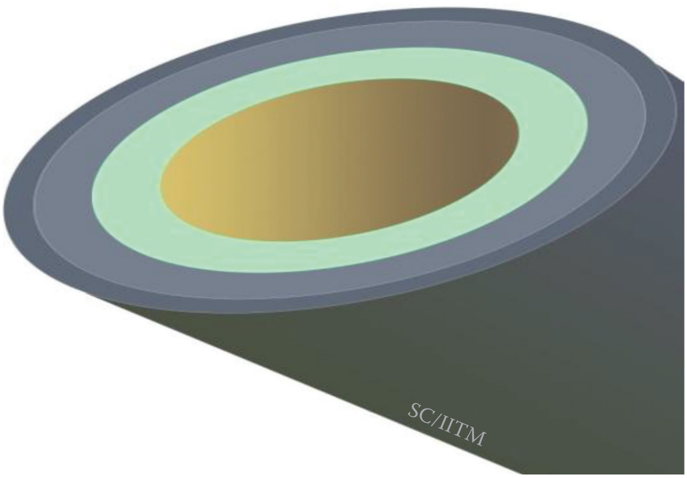


FIGURE 4.14 Spoolable production riser.

force is by usually the variable buoyancy. Hence, the construction methods and practices are also dependent on the type of offshore structures and their governing design criteria.

4.7.1 Designing for Wave Height

Offshore structural members are designed for significant wave height (H_s). It is defined as a representative wave height, which is the average of the highest one-third of the waves in a given wave record. It is given by

$$H_s = H_{1/3} = \frac{3}{N}\sum_{i=1}^{\frac{N}{3}} H_i \tag{4.1}$$

Alternatively, the significant wave height can also be obtained from the spectrum of wave energy as follows:

$$H_s = 4\sqrt{m_0} \tag{4.2}$$

where m_0 is the zeroth moment of the wave spectrum.

From visual observations of marine experts, the significant wave height is given by

$$H_s = 0.775H_v + 7.0 \tag{4.3}$$

where H_v is the wave height by visual observation (given in ft).

The maximum wave height is given by

$$H_{max} = \left[\ln(N) + \frac{0.2886}{\sqrt{\ln(N)}}\right] H_{rms} \tag{4.4}$$

It is important to note that the duration of the maximum wave height in a given wave record is a short time period. Hence, it is a desirable practice to account for the design wave height. From the short-term record, extrapolate the waves for 100 years to determine $H_{1/3}$ and compute the maximum wave height as follows:

$$H_{max} = (\text{factor}) \times H_{1/3} \tag{4.5}$$

where the multiplying factor is advised by several researchers.

4.8 STRUCTURAL FORM OF MEMBERS

A general geometric layout of structural members is arrived based on the Front-End Engineering Design report. It is always customary to carry out a detailed feasibility study on the existing offshore structures of a similar type to arrive at the structural form that is appropriate for the selected site conditions. In the case of selection of structural geometry of offshore structures, geometric form is derived based on several factors: functional requirements, operational conveniences, and construction difficulties such as towing and method of launching. It is therefore imperative to realize that construction methods and practice play an important role to even select the desirable structural form of the offshore structure. Hence, the geometric form of an offshore structure is not only guided by the water depth and site conditions but also dominated by other factors and operational conveniences. Operational conveniences refer to the construction difficulties and installation problems. Therefore, ocean structures, by and large, are function-based form, which is unusual in the case of other land-based structures.

4.8.1 Orientation Layout of Offshore Platforms

Orientation of the platform is another important aspect in the design. Preferred orientation is that members are oriented to have less projected area to the encountered waves. This reduces the response on the members. The predominant wave direction for the chosen site is known to the designer based on which the platform orientation is finally decided. A few data are essential to design the offshore structures. Land topographical surveys are necessary to check whether a sufficient area of the chosen site complies with the hydrographical survey of the proposed location. Hydrographic charts are used to collect information regarding silting at the site. A wind rose diagram showing the information on wind velocities, duration, and predominant direction is used. Cyclonic tracking data showing the details of the past cyclonic storm in terms of wind velocities, direction, peak velocity, and duration are referred. The details of studies required for the preliminary design of an offshore platform are as follows:

1. Oceanographic data
   a. General tide data including the following:
      i. Lowest water level recorded
      ii. Mean low water spring
      iii. Mean low water neaps
      iv. Mean sea level
      v. Mean high water neaps
      vi. Mean high water spring
      vii. Highest water level recorded
   b. Tide table showing any abnormal variation
   c. Wave data including the following:
      i. Maximum wave heights and conditions prevailing at that time
      ii. Significant wave height, period, and wave length
      iii. Wave roses of observed waves
      iv. Storm surge details
      v. Long-period wave data if available
   d. Current including the following:
      i. Seabed characteristics showing seabed levels, composition, bed slope, and so on
   e. Temperature round the year
   f. Rainfall average in the past 10 years
   g. Humidity record round the year
2. Seismicity level and values of acceleration

3. Structural data of existing similar structures, preferably in the near vicinity
4. Soil investigation report

4.8.2 Steps in Structural Design

The following steps are identified as necessary to carry out the structural design of offshore structures. These are also based on various factors such as selection of structural form, construction method to suit the site conditions, and construction practice to suit the chosen structural form (Burrows et al., 1992).

Step 1: Estimate the wave height, wave period, and current distribution along the water depth.
Step 2: Establish an appropriate wave theory suitable for the chosen site location of the platform.
Step 3: Estimate the water particle kinematics (water particle velocity and acceleration) in both horizontal and vertical directions.
Step 4: Choose appropriate values of drag and inertia coefficients.
Step 5: Establish the marine growth and account for the same in the design.
Step 6: Check the wave kinematics factor with respect to current (Doppler effect).
Step 7: Determine the current blockage factor and make suitable modifications in the force estimates.
Step 8: Check for the appropriate modifications deemed necessary in the Morison equation.

4.9 CONSTRUCTION TECHNIQUES

Construction methods and practice govern the choice of structural form of offshore structures, in addition to the critical loads encountered by them. Figure 4.15 shows the construction sequence of the gravity-based platforms. For example, starting

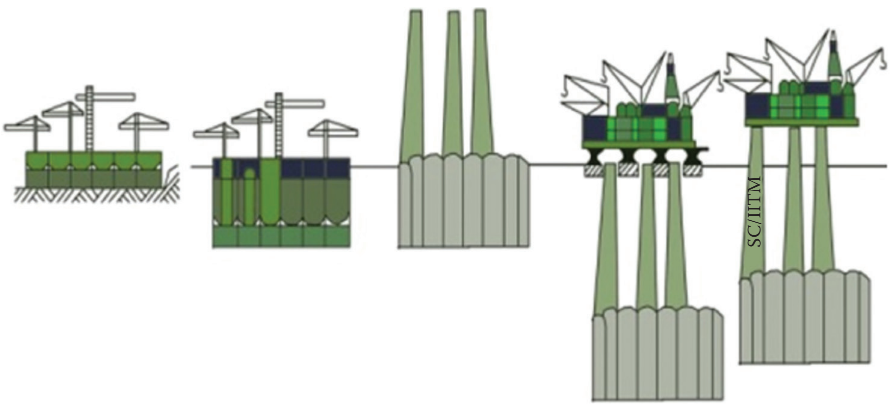


FIGURE 4.15 Construction sequence of gravity platforms.

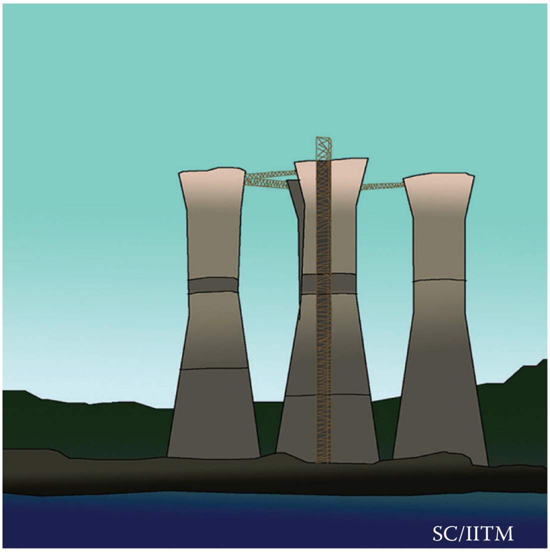


FIGURE 4.16 Towing the tower to the installation site.

from the left, the construction of skirts and concrete raft in dry dock is completed as the first step. Then it is towed to specific site while construction of upper domes is carried out. Subsequently, towers at the deepwater site are constructed and deck mating is carried out. It is important to note that the construction methodology generally dominates the cost involvement in offshore installation. The sequence shown above is preferably followed for gravity type of the platforms in order to reduce any additional cost involvement. Figure 4.16 shows the towing of the tower to the installation site with tugboats.

A sequence of operations is carried out during installation of the offshore structures. Figure 4.17 shows the sequence of operations during installation of a jacket platform. The first operation is termed as "load out" where the template structure is kept on the barge. It is then towed down to the site where it is launched on rollers. Once it is launched, it floats; this is subsequently upended to a vertical position using a special crane barge. This operation is termed as "vertical positioning." The next step is to drive the pile on the location of the template. Jacket is then installed on the said location. Skirt piles are driven to ensure the stability of the jacket against lateral loads. Subsequently, the topside is fixed to the jacket, which is termed as "deck mating."

Figure 4.18 shows the dry towing of a spar on the barge, whereas Figure 4.19 shows a schematic view of deck mating of a jacket platform. Figure 4.20 shows the dry towing of a semisubmersible to the site for exploratory drilling. Alternatively, wet towing is also common in offshore installations in order to reduce the crane load. Figure 4.21 shows the wet towing of TLP; towing takes place directly by the floatation principle. In the case of wet towing, time delay is significant as it cannot take

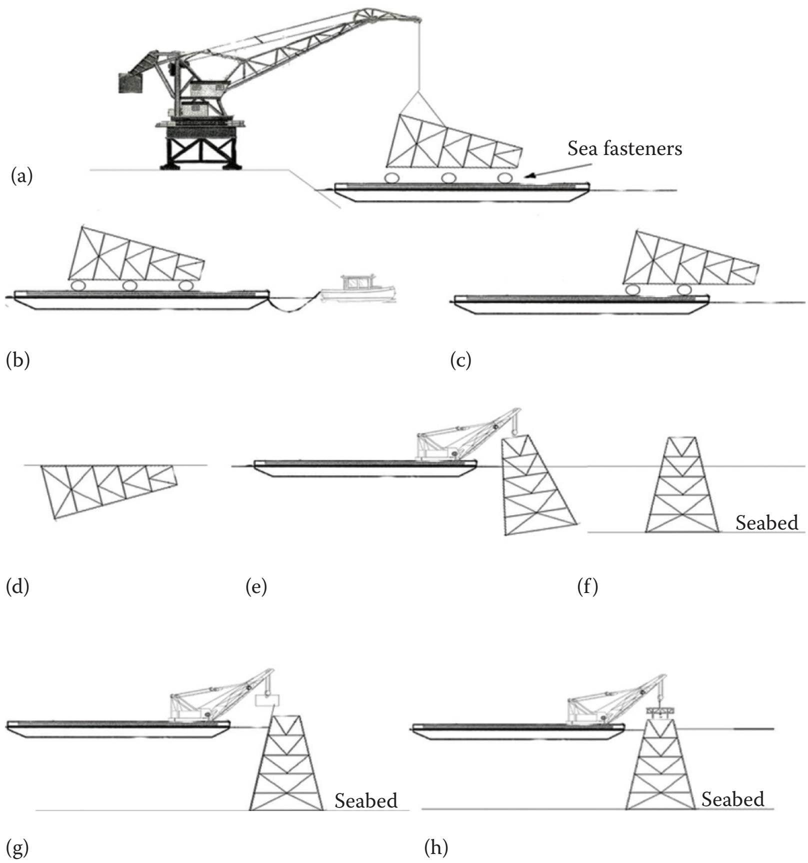


FIGURE 4.17 Sequence of operations during installation of a jacket platform: (a) load-out; (b) towing; (c) launching; (d) floating; (e) upending; (f) vertical position; (g) piling; (h) deck mating.

place under all sea states. It is also important that no damage occurs to the platform by the impact caused by tugboats.

In dry-tow operation, once the platform is towed to the desired location, it is upended from the barge or semisubmersible. Figure 4.22 shows the upending of truss spar platform from a semisubmersible. During upending, the barge or the vessel motion is controlled using lateral control lines connected to tugboats. Large-capacity floating cranes are deployed to upend the platform. Semisubmersible is housed with the special type of crane, which can hold it vertically upside down. Under this holing position, ballast hoses continue with the ballasting operation to make the upended SPAR erect.

Subsequently, deck mating is carried out as shown in Figure 4.23 by deploying a floating crane.

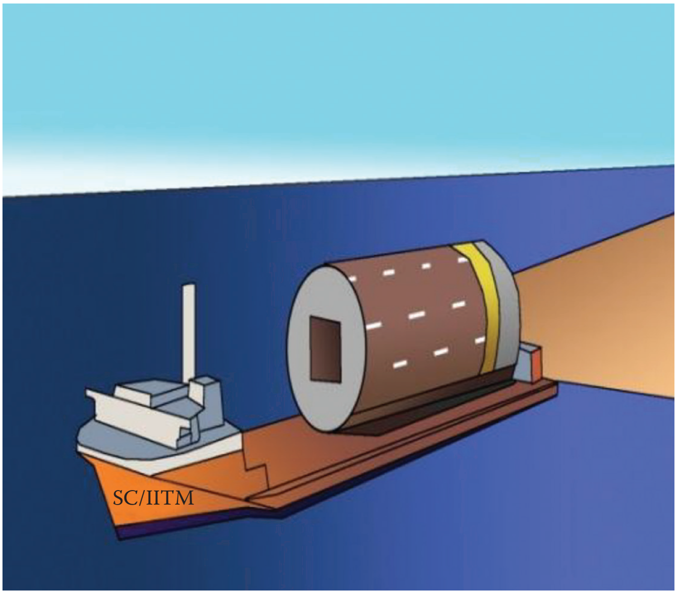


FIGURE 4.18 Dry towing of a spar.

FIGURE 4.19 Deck mating of a jacket platform.

4.10 CONSTRUCTION EQUIPMENT

Most offshore structures are piled structures. Figure 4.24 shows a schematic view of a jacket platform, with the basic components, which are handled as modules during the construction process. The sequence of construction of a piled structure is discussed next.

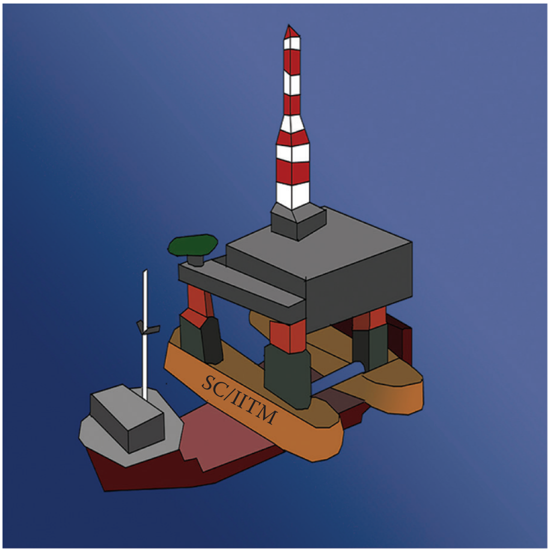


FIGURE 4.20 Dry towing of a semisubmersible.

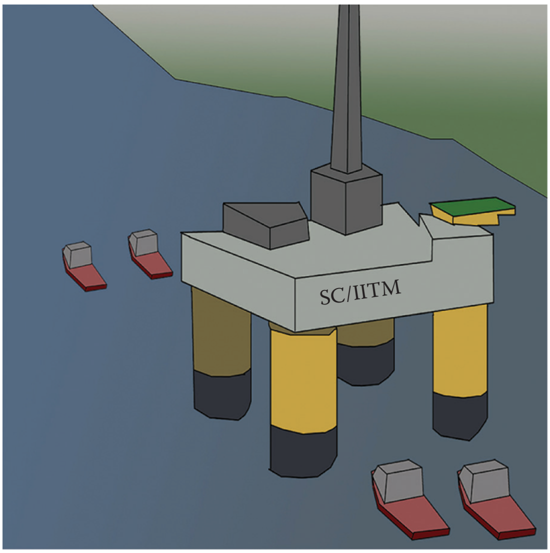


FIGURE 4.21 Wet towing of a tension leg platform.

A variety of cranes are deployed during the construction and installation processes. Offshore construction requires large self-contained crane vessels. Their cost is about 5–10 times of that of the operational cost in the onshore works. Hence, the basic idea of offshore construction methodology should be to maximize the fabrication work onshore and to minimize the construction activity at

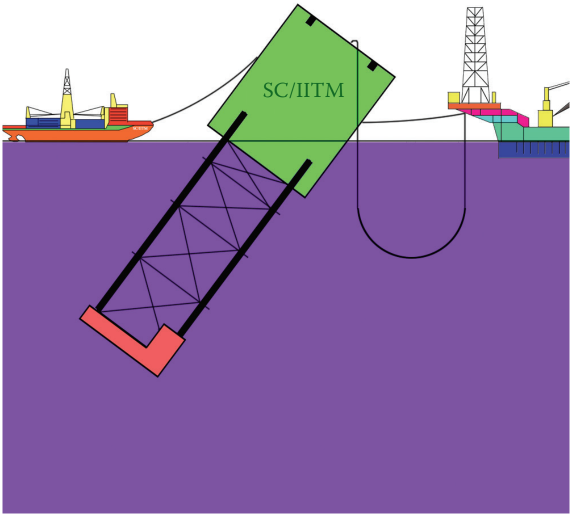


FIGURE 4.22 Upending of a truss spar during dry towing.

FIGURE 4.23 Deck mating of a spar.

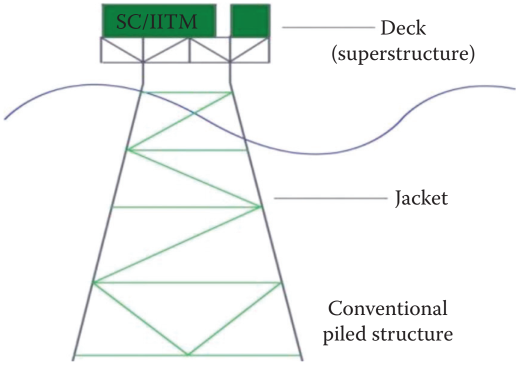


FIGURE 4.24 Offshore jacket platform.

site. Therefore, the entire planning, layout, and design of the facilities are completely based on this specific principle. Figure 4.25 shows a special kind of crane vessel, which is used in the installation of template-type offshore structure. Considering the dry weight of the platform modules and their hold-down time during upending and erection, heavy-duty cranes are also commonly deployed. Figure 4.26 shows a typical heavy-duty crane used in offshore installation.

Even the barges used for dry transportation are equipped with special cranes to facilitate the upending and erection operations. Figure 4.27 shows a crane barge equipped with a heavy-duty crane.

Table 4.1 shows the details of a variety of cranes that are commonly used in offshore construction. Their hold-down capacity, manufacturer, and details of the type of vessel they are equipped with are also indicated.

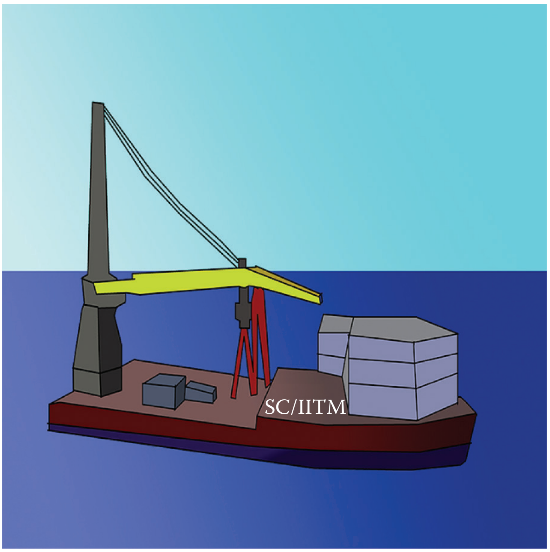


FIGURE 4.25 A special crane for installation of template structures.

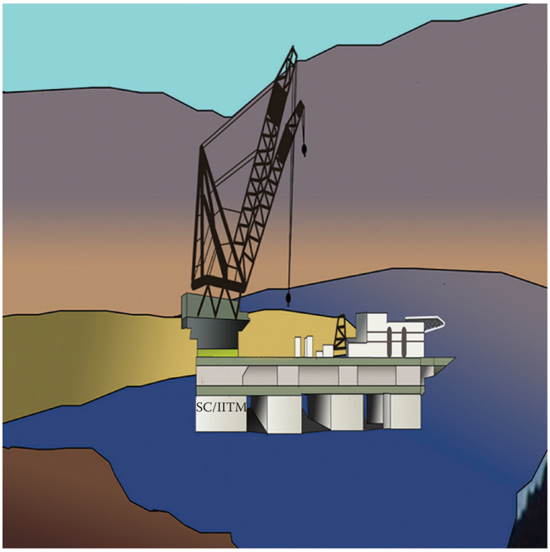


FIGURE 4.26 Heavy-duty crane SSCV Thialf.

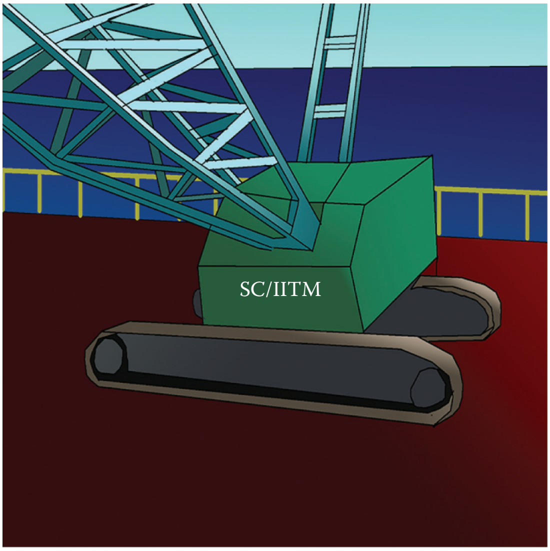


FIGURE 4.27 Crane barge used in upending and erection.

4.10.1 Infrastructure Required Onshore and Offshore

An overview of the infrastructure required onshore to make a platform executable offshore is discussed. First, adequate land and waterfront with sufficient draft to load out on the barges are primarily required. Availability of land with very large waterfront enables fabrication on the yard, which is very close to the waterfront. This

TABLE 4.1
Details of Crane Vessels

Crane Vessel	Capacity (T)	Type of the Vessel	Company
Thialf	14,200	Semisubmersibles	Heerima Marine Contractors
SPIPEM 7000	14,000	Semisubmersibles	Saipem
Svanen	8,700	Catamaran	Ballast Nedam
Hermod	8,100	Semisubmersibles	Heerima Marine Contractors
Lan Jing	7,500	Monohull	CNOOC
Balder	6,945	Semisubmersibles	Heerima Marine Contractors
Seven Borealis	5,000	Monohull	Subsea 7
OlegStrasnov	5,000	Monohull	Seaway Heavy Lifting
PJW4000	4,200	Monohull	Swiber Offshore
Aegir	4,000	Monohull	Heerima Marine Contractors
DB 50	4,400	Monohull	J. Ray McDermott
Rambiz	3,300	Catamaran	Scaldis

reduces complexities that arise during load out sequence of the construction operation of offshore platforms. Mobile cranes are generally used up to a capacity of 1000 metric tons with 4 m boom length. Fabrication yards are equipped with skid-track arrangement or a compacted ground for load-out facility. Only in such cases, load-out operation can become simple. For example, on the skid-track, fabricated jacket structure can be skid to load out on the barges. In addition, the facility for transportation of material and equipment from the source is also required onshore to have adequate fabrication convenience for offshore structures. Further, heavy-lift crane barges along with accommodation facilities, external power source of industrial type (e.g., a three-phase supply), power hammers, and diving support are also required on the crane barges during installation of jacket structures. Cargo barges, tugboats, and survey vessels are required as the ancillary support system during installation.

Construction methodology of a template structure is governed by many factors as it primarily depends on the type of structure being installed. It also depends on the size of the members, overall dimension of the structure, and weight of each module. Selection of the type of crane vessel depends on the above factors, and hence, such analysis is essential. In addition, availability of infrastructure at site also plays a critical role in deciding the construction methodology for the template structures. For example, the facility of equipment available at the fabrication yard decides the maximum permissible weight of each module and therefore its size. The details of the infrastructure required offshore are as follows: heavy-lift crane barge along with accommodation, power, hammers, diving support, and so on; cargo barge(s); tugboats; and large vessels for dry towing.

4.11 ALTERNATIVES FOR LOAD-OUT OPERATIONS

There are different alternatives available for load-out operation in jacket structures. For example, hydraulic transporters can be used for skidding the template structure instead of load out (DOE-OG, 1985). Specialized cranes are generally used for lifting heavy loads; largest crane vessels are often catamaran or semisubmersibles with increased stability (Gerwick Jr, 1986; Graff, 1981). Catamaran is a geometry-stabilized ship, usually multihulled and engine powered, which are generally used for installation as crane barges. Alternatively, jacket installation can also be done by either lifting or launching, whereas deck installation can be carried out by either lifting or float-over (Hagiwara, 1984). Figure 4.28 shows a series of steps involved in jacket fabrication and launching. Jacket fabrication requires a large area of the fabrication yard, which is done closer to the launch track. The launch track is kept on both sides of the quay wall, and the launch truss is fabricated as seen in the second figure. Once the fabrication is completed and the launch track is ready, fabricated launch trusses are brought to the launch track. The launch truss is rolled up before the side panels are fabricated. Subsequently, the side panels are also rolled up to the required alignment as shown in the figure.

On completion of fabrication of the jacket on land, it is now ready for load out on a barge. A typical derrick barge Derrick Barge is a single revolving crane with a lifting capacity of 600–3500 tons. Daily rate or hire charges of these kinds of barges are phenomenally high, and therefore, their effective usage should be planned ahead. Apart from the effective utilization of the hired time, it also controls the overall installation period. Alternatively, twin-crane semisubmersibles are also deployed for jacket launching. For example, SAIPEM 7000, DB102, HERMOD, and BALDER are the different types of twin-crane semisubmersible vessels, which are commonly used for launching jacket structures. A few

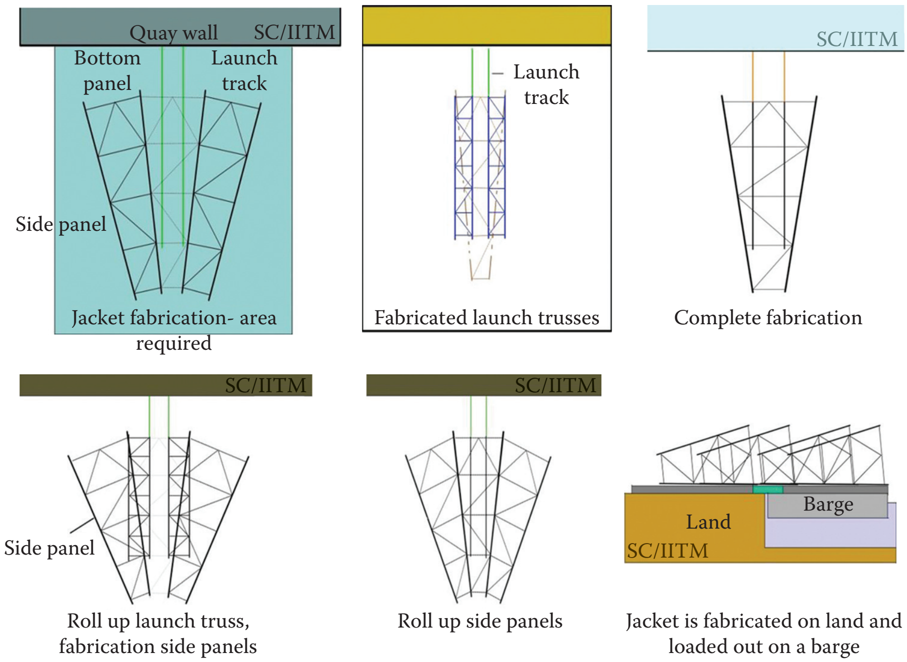


FIGURE 4.28 Jacket fabrication and launching.

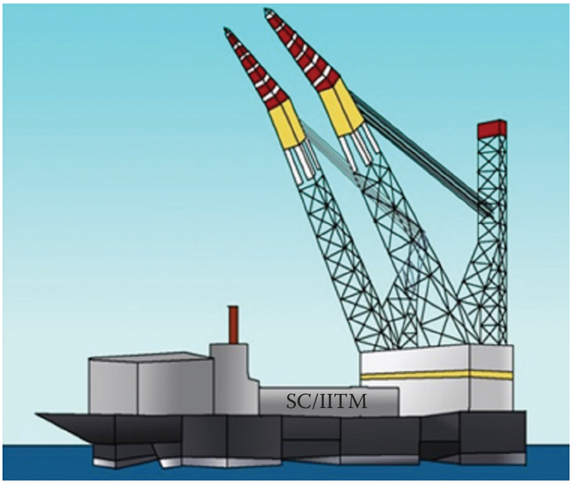


FIGURE 4.29 Crane barge used for jacket launching.

of the crane barges also have an additional capacity of laying pipelines as well. Figure 4.29 shows a schematic view of SAIPEM 7000 crane barge used in launching jacket structures.

4.11.1 Installation of Jackets

At site, once the jacket is moved on the launch track, barge is brought to the site and loaded. Subsequently, the jacket is tipped over a rocker arm and made to dive. Figure 4.30 shows the launching and free floatation. Once it is made to dive, separation takes place; jacket template gets disconnected from a rocker arm of the barge, as seen in Figure 4.31. Free floatation of the jacket is shown in Figure 4.32. Once it gets separated, it starts free floating as the design enables

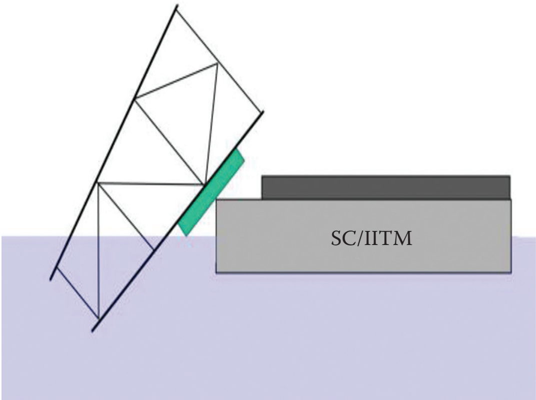


FIGURE 4.30 Launching and free floatation (diving).

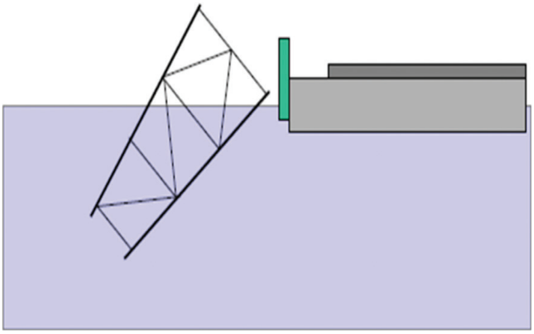


FIGURE 4.31 Separation of jacket.

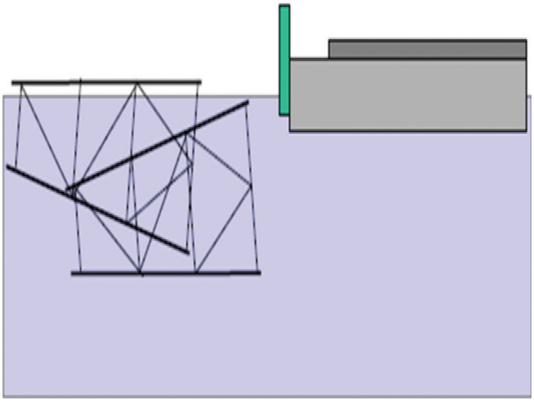


FIGURE 4.32 Free floatation of the jacket.

sufficient buoyancy for free floatation. Once it starts free floating, a special kind of crane or a semisubmersible, say for example, SAIPEM 7000, is used to set it in the vertical position; this is termed as "upending operation." Upending is related to making the offshore jacket in a vertical position. Subsequently, the jacket is now lifted up and made erect, as seen in Figure 4.33; this is termed as "setdown operation."

After the setdown operation is complete, piling is carried out to hold down the jacket structure in place. A series of steps are involved in piling operation. The first step is to insert the lead section, which has to be piled inside the seafloor into the template area, as shown in Figure 4.34. As many sections are required, they are added, as shown in Figure 4.35, to complete the piling operation. Once it is completed, one needs to drive a pile of the specific legs into the seafloor by hammering, as shown in Figure 4.36. A special kind of crane barges with the capacity of pile driving is deployed to perform this operation.

Once the pile is driven, the operation is terminated with grouting. Figure 4.37 shows a schematic view of conventional grouting. The outer layer indicates the

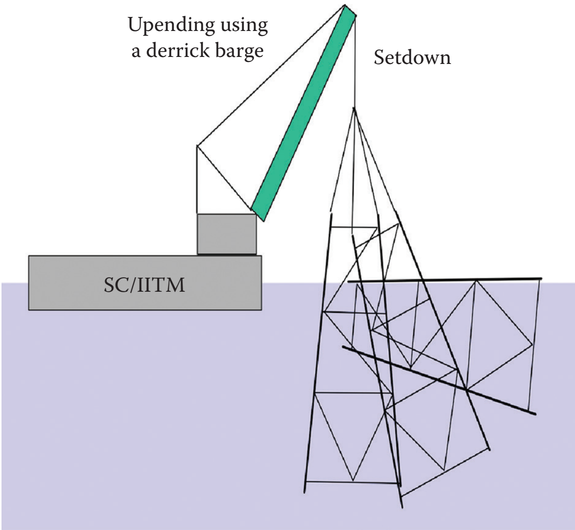


FIGURE 4.33 Upending and setdown.

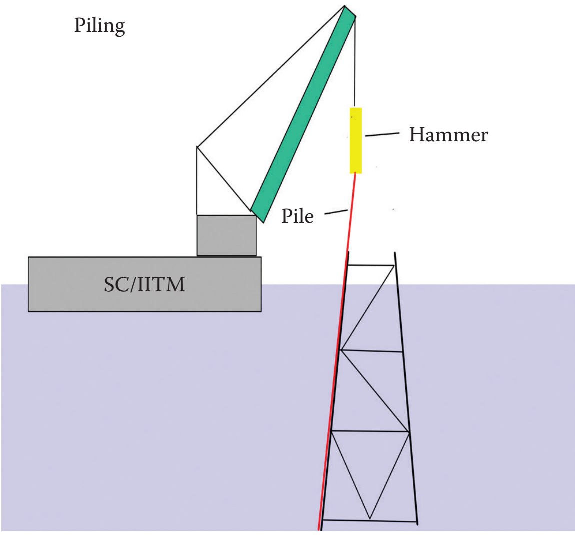


FIGURE 4.34 Piling: Insert lead section.

jacket leg or the pile sleeve. The grout line is run all along the periphery of the pile. To avoid leaking of the grout and ensure stable grouting, a grout packer is used at the terminal end, as shown in the figure. An inflated line collects the accumulated water at the grout after which the grout is well packed and sealed. Alternatively, a pressure grouting system is also common in jacket installation. Figure 4.38 shows a schematic view of pressure grouting for the main legs of the jacket. In this case,

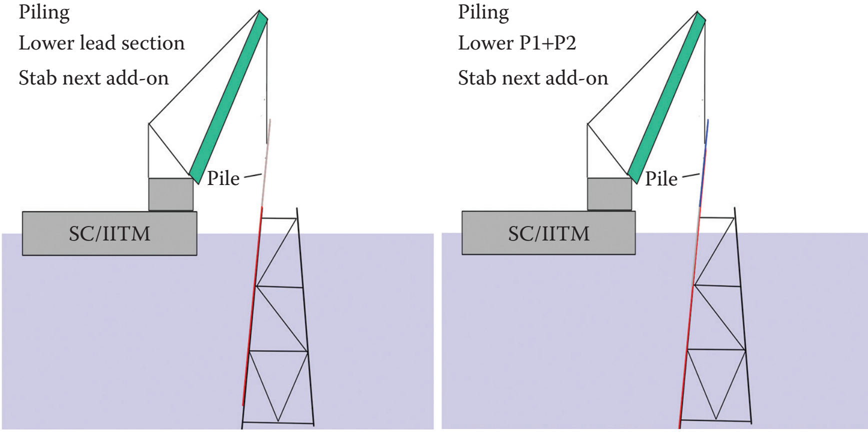


FIGURE 4.35 Piling of successive sections.

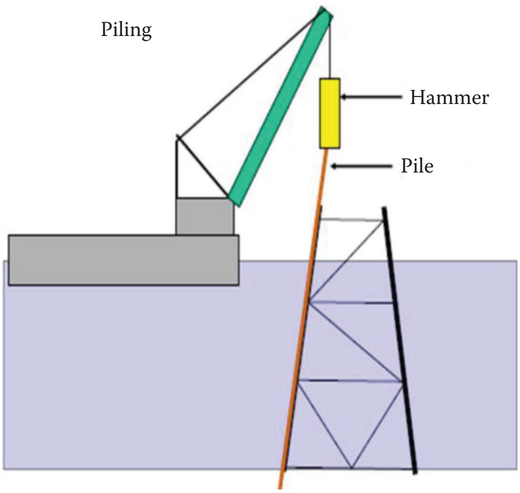


FIGURE 4.36 Pile driving by hammering.

a crown shim is placed on the top and the grout line is connected to the top. As the grout flows during the operation, an inflation line releases the accumulated water and releases the pressure.

Various steps involved in pressure grouting for the main legs are shown in Figure 4.39. Other legs are grouted by the conventional method, as explained earlier. Conventional grouting operation is in contrast to that of pressure grouting. First, the inflation packer is fixed before the grout is pumped; the grout displaces water in the upward direction. Apart from this, a crown shim is provided for the main legs only.

Once the jacket legs are grouted and set, the jacket is now ready to receive the deck. The deck can be installed using two methods: (1) module installation and

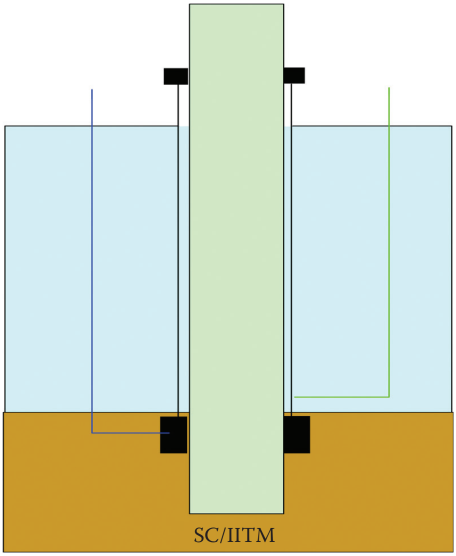


FIGURE 4.37 Conventional grouting.

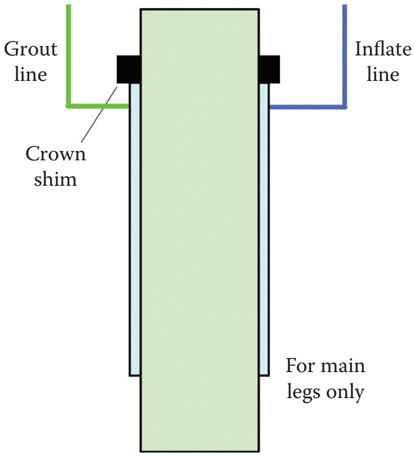


FIGURE 4.38 Pressure grouting.

(2) float-over. In the module installation, various modules such as living quarters, process module, and operation module of the deck can be installed as shown in Figure 4.40. Alternatively, the prefabricated deck module can be installed in total by the float-over method.

In the float-over method of installation, a complete prefabricated deck is transported to the site using a barge. It is then placed in position over the jacket, as seen in Figure 4.41.

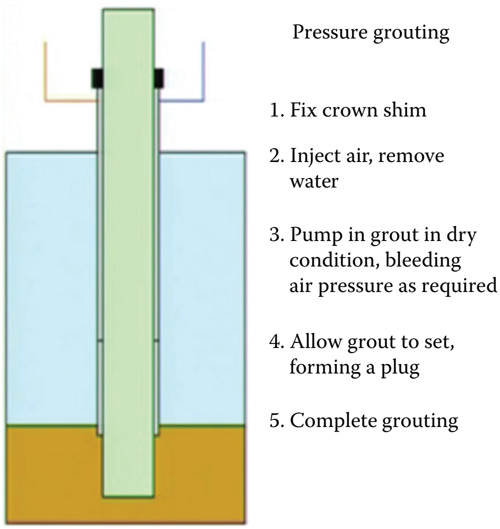


FIGURE 4.39 Steps in pressure grouting.

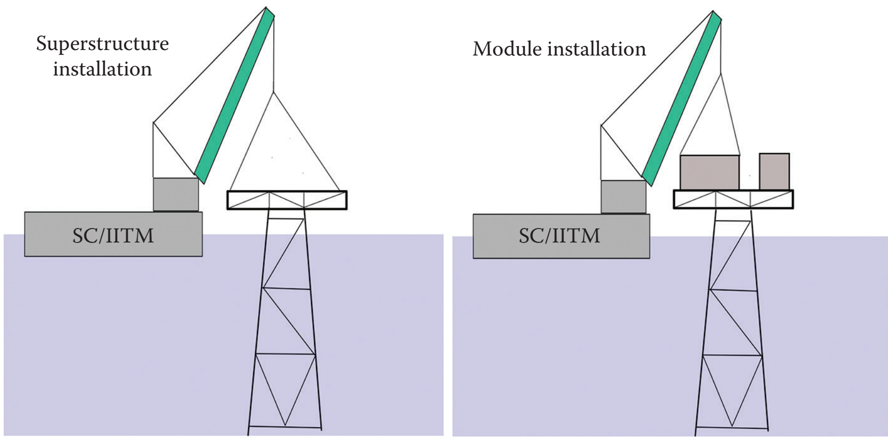


FIGURE 4.40 Superstructure installation in modules.

The float-over method of installation of deck is a successful alternative for installing the deck because module installation requires special cranes for lifting. This also restricts the size of each module as the constraint may be imposed by the capacity of the cranes used for lifting and installation. Apart from increasing the cost, this method is expensive in terms of cost and operational safety. However, the float-over method enables the fabrication of the deck in the yard under good quality control. Expensive derrick barges are not required, which also minimize the hookup time. In the offshore installation, higher hookup time increases the

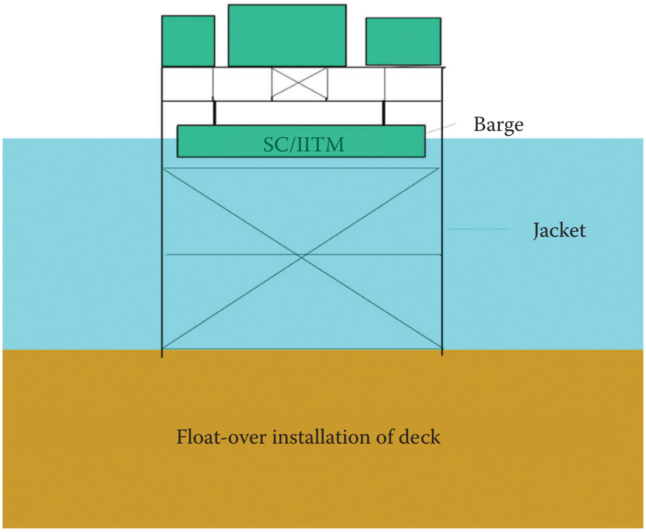


FIGURE 4.41 Installation of deck by float-over method.

accident possibility. However, the float-over method of deck installation is suitable only for calm sea state.

4.12 SUBMARINE PIPELINES

Submarine pipelines have different levels of application. They are used in hydrocarbon transportation and effluent disposal. Nowadays increasingly, they are used for seawater intake and disposal systems. Prior to knowing the construction methods of submarine pipelines, it is imperative to know the various factors that govern their design. The design of submarine pipelines is influenced by several factors, namely, internal pressure in the pipeline, external pressure exerted by hydrodynamic forces on the pipeline, and level at which the pipeline is installed (Bai, 2001). Although the level of installation influences the stability under storm loading, stresses developed during the installation process are also considered in the design. However, it is very interesting to note that the method of laying of pipelines even influences the design to a larger extent. Among the different methods of laying pipelines, the foremost method is shore-pull technique. Figure 4.42 shows a schematic view of the shore-pull method. As the name suggests, pipeline assembly is made to lay on intermediate supports as seen in the figure. Floats are attached to the pipelines, which enable them to float. Although one end of the pipe is placed on land, the other end is kept on the barge, which can be pulled. During pulling, floats attached in between the pipe length act as intermediate supports, which facilitate the operation.

Alternatively, one can also use S-lay method for laying of pipes. Figure 4.43 shows a schematic view of the S-lay method. Field joints, field coating, and tensioners are connected to the pipeline, which is laid on a barge; this is termed as “lay barge.” It is

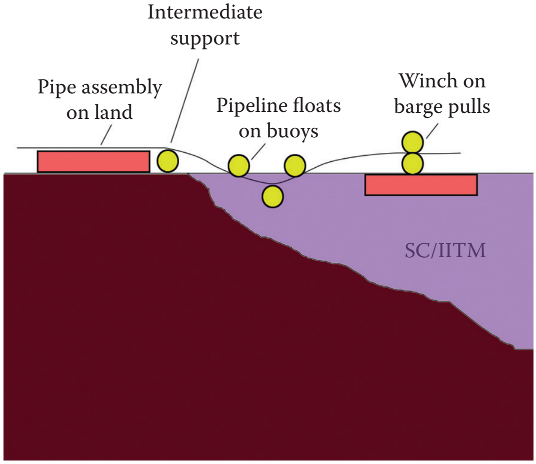


FIGURE 4.42 Shore-pull method of laying pipelines.

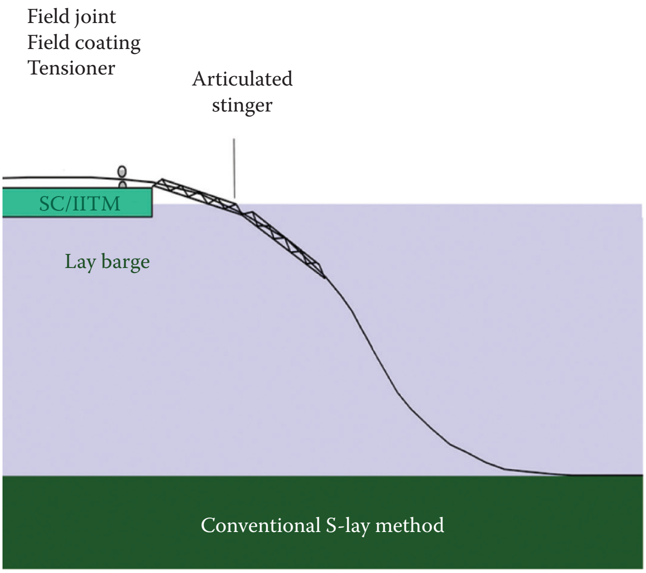


FIGURE 4.43 Pipeline laying: S-lay method.

subsequently articulated in a stinger and the pipe is laid. Because the profile of the pipe, during laying, forms a shape similar to S, this method is termed as the "S-lay method."

Different processes take place in the laying of the pipeline once the pipe is laid in the lay barge. Figure 4.44 shows these stages of the process. The joints of the pipeline are welded and inspected using X-ray diffraction. Subsequently, field coating is

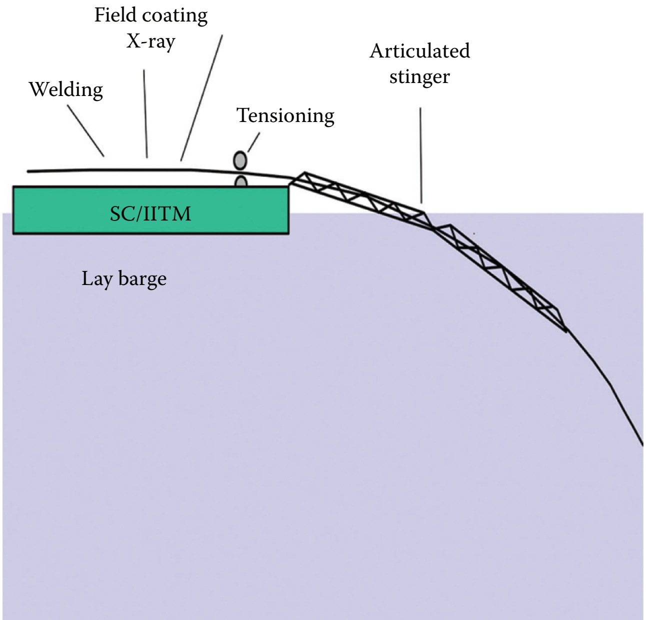


FIGURE 4.44 Stages of pipeline laying in lay barge method.

applied on the entire pipeline. Anticorrosive coatings such as fusion-bonded epoxy or a multilayer polyolefin system are commonly used. Cathodic protection, in the form of sacrificial anodes, is also normally installed to ensure corrosion control. As the pipelines are welded together in either single- or double-joint segments on the offshore lay vessel, coating should be applied at the weld locations; these coatings are termed as "field-joint coatings." Typical filed joint coatings are in the liquid state such as epoxies, urethanes, epoxy/urethanes, or heat-shrinkable sleeves. Field joint coating is a time-consuming process as there could be large number of joints. This can delay the pipe-laying process, which is otherwise an expensive schedule in offshore construction (Hitchings et al., 1976; Paik and Thayamballi, 2007). If one wishes to eliminate the field joint coatings, then it is necessary to upgrade the pipeline's cathodic protection system to account for the additional uncoated surface area. Finally, tensioning system is applied before it is drifted using the articulated stingers, as seen in the figure. Figure 4.45 shows the conventional slay method, which is articulated by the stringers.

4.13 PHYSICAL AND ENVIRONMENTAL ASPECTS OF OFFSHORE CONSTRUCTION

A few important physical and environmental aspects of offshore construction are now discussed. The ocean environment generally dominates the method of construction, choice of equipment, support systems, and procedures to be deployed for construction and installation of offshore structures (Mather, 2000; Matsuishi and

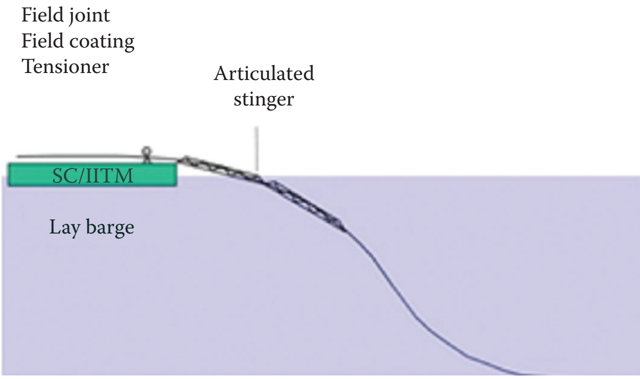


FIGURE 4.45 S-lay method guided by articulated stingers.

Endo, 1968). The most important factor in offshore construction is to cater to the influence of the environment on the construction process (Schwartz, 2005). It is therefore important to understand the interaction of the environment with that of the structure. Uniqueness in the design of offshore structure, which is also governed by the environmental process, conforms the constructability. The influences caused by the ocean environment on the construction process are as follows:

- Distance from onshore
- Depth of water at which installation has to take place
- Temperature
- Seawater and sea–air interface chemistry
- Currents
- Waves and swells
- Winds and storms
- Rain, snow, fog, whiteout, and spray
- Atmospheric icing and lightening
- Sea ice and ice bergs
- Seismicity, sea quakes, and tsunamis

4.13.1 Geotechnical Aspects

The seafloor is highly complex due to its geological history; the effect is more predominant in shallow waters and continental shelves. The slope of the seabed, which is normally seen as about 4°, can even be altered to as steep as 30°, which can cause serious challenges during construction of foundation. Ice age has a significant effect on the shelf areas. For example, when the Wisconsin ice age was at its peak about 20,000 years ago, the seawater was withdrawn; the water level lowered to about 100 m. In such cases, shelves will be exposed to steep contour of seabed. On the coastal shelves, land erosion also takes place. Rivers became steeper with increased velocity of flow. When ocean water-level rises, velocity reduces and finer sediments are formed. This results in large deposits on shelves. During ice age, glaciers extend far inside and are carved as deep trenches. Formation of Norwegian trench, Cook Inlet, and the Strait of San Juan de Fuca are examples of such formations. With the present global warming, the sea level is rising, which results in flooding hinterland in coastal areas, change in natural drainage pattern, and creation of additional shoreline features. Geotechnical investigations on the seafloor are essential to decide the construction methodology of ocean structures. Modern methods such as electric receptivity method are available to improve sampling techniques to assess the geotechnical soil properties. However, in many places, *in situ* strength is seen to be greater than that indicated by the conventional test results. However, this may result in conservative design of foundations but poses serious difficulties in the construction processes. An offshore geotechnical engineer needs to correctly interpret these reports through appropriate logs of boreholes. These interpretations will govern the selection of equipment and construction/installation methods and procedures. Most importantly, they will also govern the cost. If the geotechnical reports are not carefully interpreted, then this may result in serious cost overruns and delays. In addition, many structures in the ocean extend over substantial areas. In these areas, there can be significant variations in soil properties. Increase in cost toward more boreholes in larger extent of areas limits the number of boreholes but causes serious problems as the true situation may not get interpreted correctly. Alternately, the study of site geology will alert the construction engineer about the wide variety of soil properties. Few detailed geological aspects of seabed, namely, dense sand, calcareous sand, boulders on and near the seafloor, glacial till, overconsolidated silts, subsea permafrost and clathrates, weak arctic silts and clays, ice scour and pingos, methane gas, and mud and clay pose serious problems to the constructor. In addition, coral and similar biogenic soil, cemented soil, unconsolidated sand, underwater sand dunes, rock outcrops, cobbles, and deep gravels deposits also pose serious problems. Seafloor oozing, seafloor instability, slumping, and presence of turbidity currents govern the method of pile driving. There are few problems caused by mud and clay as underwater slopes are predominantly governed by their presence. Pile driving can become a challenging task as the presence of mud and clay can result in short-term bearing strength. Although dredging can also become difficult in such situations, penetration of piles and consolidation of clay can delay the construction process and increase the cost.

4.14 CONSTRAINTS ON OFFSHORE CONSTRUCTION AND INSTALLATION

The foremost constraint is the emphasis laid to protect the natural and built environments. Unfortunately, interaction issues with man-made structures in ocean and natural ecology are realized only after plans are made to commission the offshore project. This results in amelioration instead of integration. Current legislations are made more stringent due to the above negligent pattern followed in offshore projects. These implications are even causing serious delays during the construction of ongoing offshore projects. The nature of work in offshore construction is inherently dangerous with high probability of risk. The cost of operations of specialized equipment is higher, and the cost of technical manpower per hour is even higher. One of the major sources of cost control in offshore construction projects is to choose the equipment and its operational window, if it is hired. An offshore construction engineer must understand thoroughly the capabilities and limitations of equipment he or she uses while keeping in mind his or her operational safety. Proper training is to be imparted to detect early signs of problems before they become catastrophic.

4.15 SELECTION OF CONSTRUCTION EQUIPMENT FOR OFFSHORE PROJECTS

A variety of equipment used in offshore construction projects are as follows: barges such as crane barges, offshore derrick barges, semisubmersibles, jack-up barges, launch barges, and pipeline-laying barges. Vessel barges and pipe-laying barges are chosen to have a high degree of maneuvrability as well. Offshore dredging equipment is also used in such projects. In addition, supply boats, anchor-handling boats, towboats, drilling vessels, and crew boats are also commonly deployed. Various parameters are to be considered before the choice of equipment is exercised in offshore projects. They are motion response, buoyancy, draft and freeboard, stability, and operational safety. The operator of the construction equipment should have a clear understanding about various warning signals that are extended by these equipment during operation. Equipment should be selected that has auto-shutdown facilities in case of any emergency situation. The ability to perform even during unfavorable weather is a desirable characteristic, whereas safety is the highest priority during operations.

4.16 DREDGING

Dredging is an excavation carried out under water, in shallow sea, or in freshwater areas. It is essentially done to collect the bottom sediments and dispose them at different locations, and also to keep waterways navigable and to replenish sand on some public beaches, where sand is lost because of coastal erosion. Dredge is a device used for scraping or sucking the loose materials from the seabed, whereas dredger is a ship or vessel, which is equipped with a dredge. Dredging can be useful in many ways. To create a new harbor, berth, or waterway, dredging is mandatory. In addition, dredging is also done to deepen the existing facilities for allowing access for larger ships, which requires deeper draft. Dredging facilitates the construction of underwater foundation works for bridge piers. It is vital to maintain the navigable waterways, which became silted due to sediment deposits in due course of time. It is also done for mineral exploration on the seabed. Seabed mining is also an important part where dredging has a greater application. It is also considered as an important remedy for deoxygenated waterbodies, which are also termed as "eutrophied waterbodies." To reclaim areas, which are affected by chemical spills, dredging is carried out. Dredging operation is not free from environmental impacts. It disturbs the aquatic ecosystem. Dredge spoils, which are the loosened materials collected from the seabed, may contain toxic chemicals. This may affect the ecological system of the area where they are disposed.

Release of toxic chemicals, including heavy metals from the bottom sediments into water columns, can induce short-term turbidity in seawater.

There are a few important facts associated with dredging. Dredging is a specialized activity, which has very high initial cost in terms of plants and equipment involved in the dredging process. It can be carried out whenever there is sufficient water depth for a dredger to operate. Dredgers are available in a variety of shapes, sizes, and capacities, which we will see one by one in this lecture. They can be moved or transported from one place to another by road transport. Most of the time, they are equipped with self-propulsion systems by which they can be sailed off from one location to another. Dredging is a multidisciplinary task, involving mechanical, electrical, marine, naval architecture, electronics, and civil engineering professionals together.

4.16.1 Dredging Operations

It is a four-stage process. The *in situ* material must be first loosened from its natural state. It should be then moved from its own position to the water surface. The above two stages are called as "dredging operations." The next stage is to transport the dredged material, whereas the final stage is the relocation phase. Transportation of a dredged material can be placed in a barge or a hopper, which can be subsequently sent through the pipeline. Dredgers are expensive to purchase and hire. The operational cost is very high, and hence, successful dredging depends on the effective production rate. Proper planning is required to make the work practical and economically feasible. Site conditions are to be assessed to choose the appropriate type of dredgers and the suitable method of dredging. The important factor that makes the planning difficult is that assessment and surveying of the area under water, where dredging is to be carried out. This is not only difficult but not very efficient. Therefore, in the whole process of planning of dredging selection equipment, there can be a wrong selection of equipment or a wrong method of dredging, which can decrease the rate of production. Therefore, the cost of dredging surely and strongly depends on how you assess the type of plant and equipment that is required for dredging the specific material in that site.

4.16.2 Types of Dredging

Dredging is the process associated with collecting materials from the sea or riverbed. It involves various stages of bringing up, fishing up, or clearing away from the bottom of the sea or river. Capital dredging is carried out for creating new harbor basins and canals or deepening of existing waterways and approach channels. They are required for keeping the existing watercourses and harbor basins at the required operational depth. Mineral dredging is done for extracting minerals that have economic values from underwater deposits. It is carried out to mine gold, tin, mineral sand (such as ilmenite, rutile, and zicron), and phosphates. Remedial dredging is carried out for removing polluted sediments from the harbor basins, rivers, and so on. Environmental remedial dredging is a special type of dredging that is carried out to remove polluted sediments that are hazardous to public health. Establishing horizontal and vertical control in dredging requires special equipment, techniques, and skills. Geotechnical investigations are required to be carried out to determine the nature of the material to be dredged, which is very vital, because this will determine the type, method, and selection of an appropriate dredger.

4.17 DREDGERS

There are different types of dredgers. Broadly, they are classified as mechanical and hydraulic dredgers. Unfortunately, all kinds of dredgers do not fall in these two categories. Third category refers to special types as there are some special kinds of dredging required to be carried out for disposable cases.

4.17.1 Mechanical Dredgers

They essentially consist of a grab or a bucket, which is used to collect the loosened, *in situ* materials. The collected materials are then raised from the seabed (or riverbed, as the case may be) and transported. Different types of mechanical dredgers are (1) bucket dredger, (2) grab dredger, (3) backhoe dredger, (4) backhoe and dipper dredger, and (5) suction dredger. Figure 4.46 shows a schematic view of a bucket dredger. It consists of a conveyor belt fitted with a series of buckets that are tilted

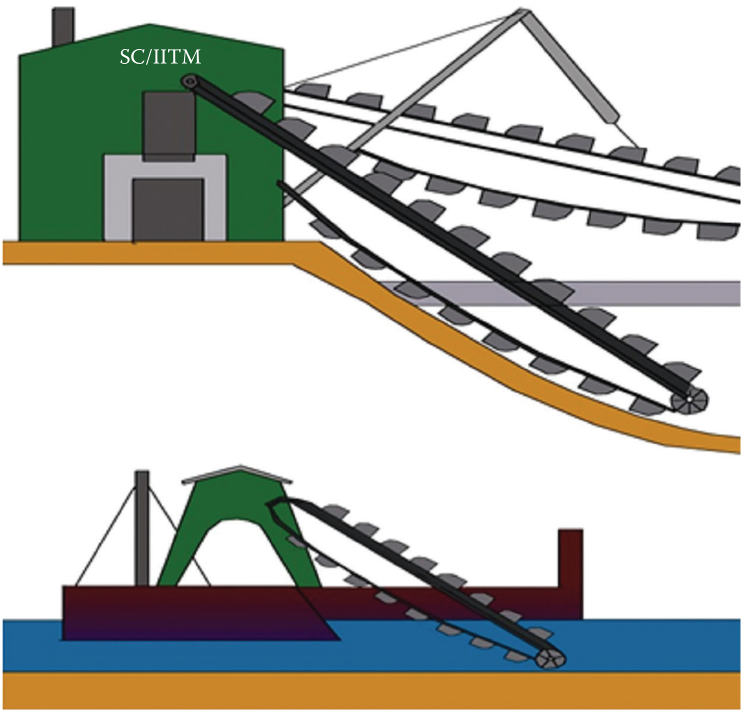


FIGURE 4.46 Mechanical dredgers.

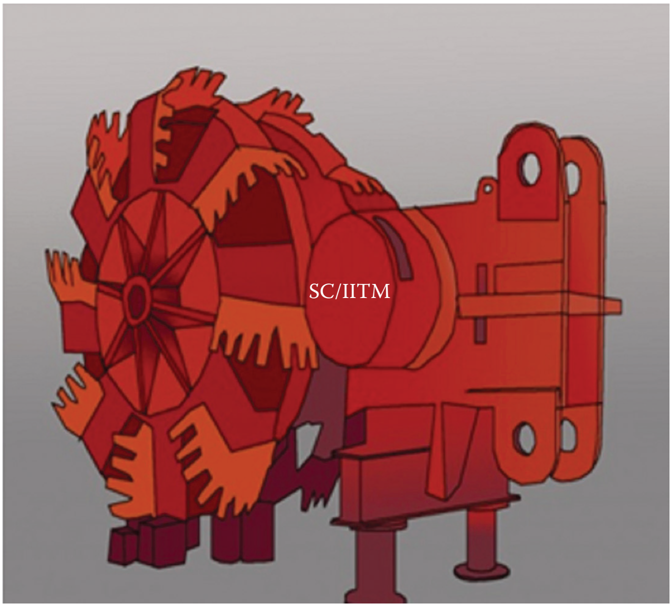


FIGURE 4.47 Cutter equipped in the bucket dredger.

to collect the dredged material. It also has a cutter at its tip to loosen the material. The cutter excavates the material and loads into the bucket; the bucket carries it on rotation to the hopper. From the hopper, it can be further transported to a vessel or barge for disposal. The tip is always equipped with the cutter, which is helpful in loosening the hardened material. Figure 4.47 shows a schematic view of a typical cutter equipped in a bucket dredger.

In alternative models, buckets are also equipped with cutters. The buckets open at their bottom to cut/collect the dredge material. The rate of dredging in such models is comparatively less, but the volume of dredging is higher. In such dredging operations, it is necessary that the material to be dredged should be loosened first. The bucket ladder type is one of the oldest types of dredgers. It consists of rectangular pontoons with a central well in which a ladder is suspended. The ladder supports the endless chain of buckets. Each bucket is equipped with a cutting edge. Flat ended side of the buckets are called as tumblers. They are rotated around this flat bottom.

When the bucket reaches the bottom of the ladder or winch, it scoops up the material. This is carried in the bucket to the top of the ladder. At the highest point of the chain, the bucket overturns and the contents are discharged. Dredged material is dropped inside a drop chute, which is connected to a barge that is collecting the dredge spoil. Each bucket, after discharging, returns empty to refill again. The bucket dredger is graded by the bucket capacity. Usually, the bucket capacity is 100–1000 L. During dredging, a small quantity of water is added to the dredged material so that the rate of production is increased. The bucket ladder dredge is held in position by the mooring system or anchors. The maximum weekly output of the bucket dredger is 10,000–100,000 m^3, whereas the maximum dredging depth is up to 20 m.

4.17.1.1 Grab Dredger

Grab dredgers are equipped on a larger barge, as shown in Figure 4.48. The barge is moored or anchored to the site; alternatively, they can also rest on spud holes. A dredge tool, in this type of mechanical dredger, is grab. Grab consists of two half shelves operated by the hydraulic principle. Dredged material is loaded in barges. Grab dredgers are further classified as open grab, closed grab, and watertight grab. They are mainly used in harbors. Dredged material is discharged in barges as seen in the figure. Grab dredgers are also called "clamshells." They are normally self-propelled, which also house a revolving crane. The size of these dredgers is expressed in terms of the hopper capacity, which varies from 100 to 2500 m^3. The production rate of grab dredgers depends on the crane, grab size, and water depth. They are position restrained by anchors during operation. They perform better in consolidated silt, clay, and loose sand. Due to their compactness, they are effective to operate closer to quay walls and corners of docks.

4.17.1.2 Backhoe Dredger

Figure 4.49 shows a schematic view of a backhoe dredger. Backhoe dredgers are generally mounted on excavator units; while in operation, they rest on spuds. Material is excavated using a bucket. The excavated material is loaded in barges or placed onshore. The older type of this dredger used a dipper or a face shovel for excavation. Figure 4.50 shows a backhoe dredger operating on the riverbank or coastal side.

Backhoe dredgers have an extended boom and stick, which is used as a shovel attachment. At the bottom, buckets of various sizes are attached to the extended boom. These buckets actually collect the dredge spoil from the seabed. The bucket capacity varies from 0.5 to 13 m^3. A standing hydraulic excavator is also available

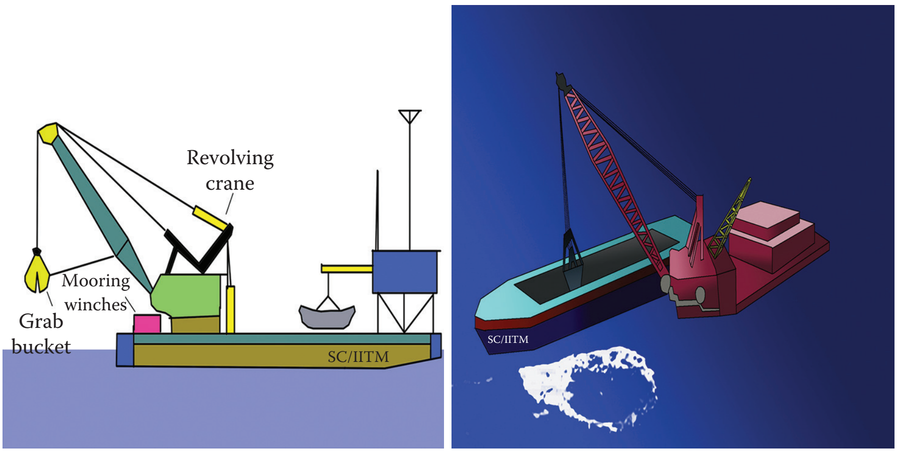


FIGURE 4.48 Grab dredgers.

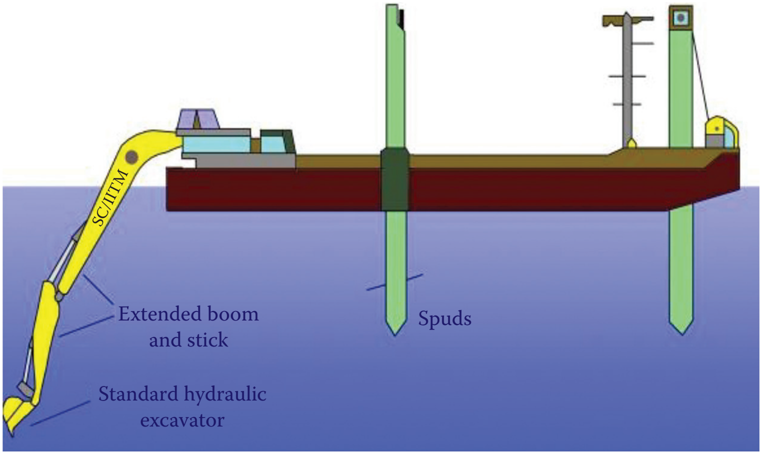


FIGURE 4.49 Backhoe dredger in operation.

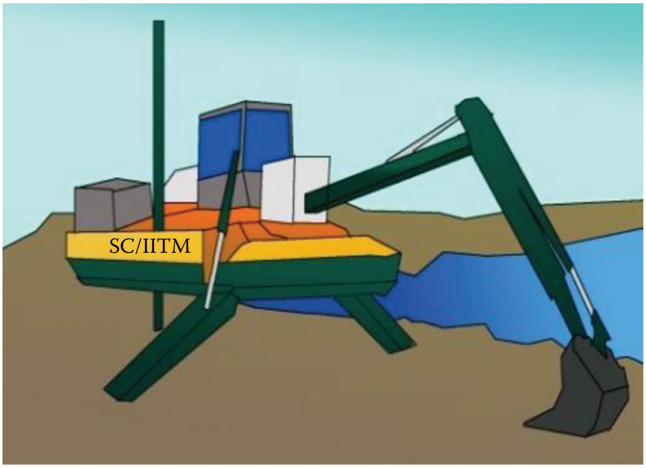


FIGURE 4.50 Backhoe dredger operating on a riverbank.

on the dredger, which is helpful in loosening the dredge spoils. The modern version has an extensive, special kind of a hydraulic excavator, which is attached and housed in the same barge where the dredge is equipped.

4.17.1.3 Clamshell Dredger

Figure 4.51 shows a schematic view of a clamshell dredger. It consists of a clamshell bucket, which does the dredging operation. It is also used to transport the dredge spoil into a barge. It picks up the dredge spoil. As supported by a rotary crane, it discharges the dredge spoil from the site to that of barge located nearby. These dredgers are self-propelled for maneuvring but rest on spud cans while in operation.

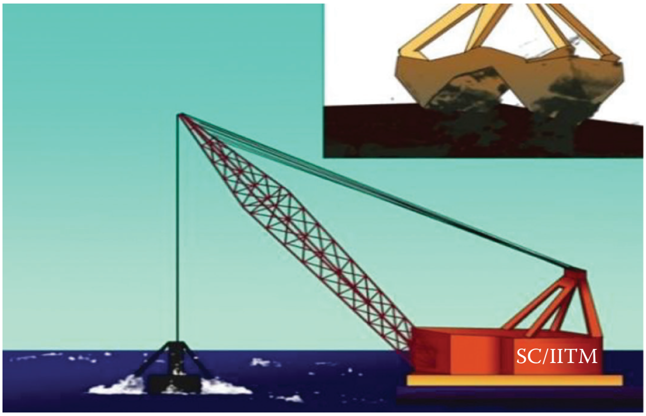


FIGURE 4.51 Clamshell dredger.

4.17.1.4 Bucket Dredger

The bucket dredger, as shown in Figure 4.52, has different parts of important machineries, which are operating with dredgers. This is equipped with a self-propelling engine, which can be supported by the spud cans while in operation. The bucket dredger has a very long boom and winch, based on which the whole operation of cycling or the swinging radius is controlled. Hoisting, swinging machinery, and the control house are located in the front side of the dredger to improve its operational stability. The dredger also has a dump scow in parallel where the dredge spoil can be deposited. This is further transported to the deposit site as required.

The excavated material (dredging spoil) is placed in scows (hopper barges). It is later towed to disposal areas. The bucket dredges range in capacity of 1–10 m^3. The effective working depth is limited to about 30 m. The bucket dredger is not self-propelled but can move itself over a limited area during the dredging process by the manipulation of spuds and anchors. A schematic view of a typical bucket while in operation is shown in Figure 4.53.

There are advantages and disadvantages of bucket dredgers. They only require a very small operating crew. They can even dredge rocks when broken to pieces by blasting using heavy versatile machine, which is very commonly used in harbors. Disadvantages of this kind of dredgers are that they are very slow moving and relatively low output. As they are not self-propelled, moving these dredgers is not simple. It demands a dredge spoil site to be located along the side of the dredger; otherwise, it requires barges for collecting the spoil deposit from the dredgers.

4.17.1.5 Dipper Dredger

It is equipped with a shovel and a dipper. Figure 4.54 shows a schematic view of a dipper dredger. This kind of dredgers can rest on spuds while in operation. As seen in the figure, it is basically a barge-mounted power shovel. The shovel is connected to A-frame, which is supported by the hoist; the hoist is controlled by the vessel, which is called the “dredger equipment.” It is also equipped with a power-driven ladder structure

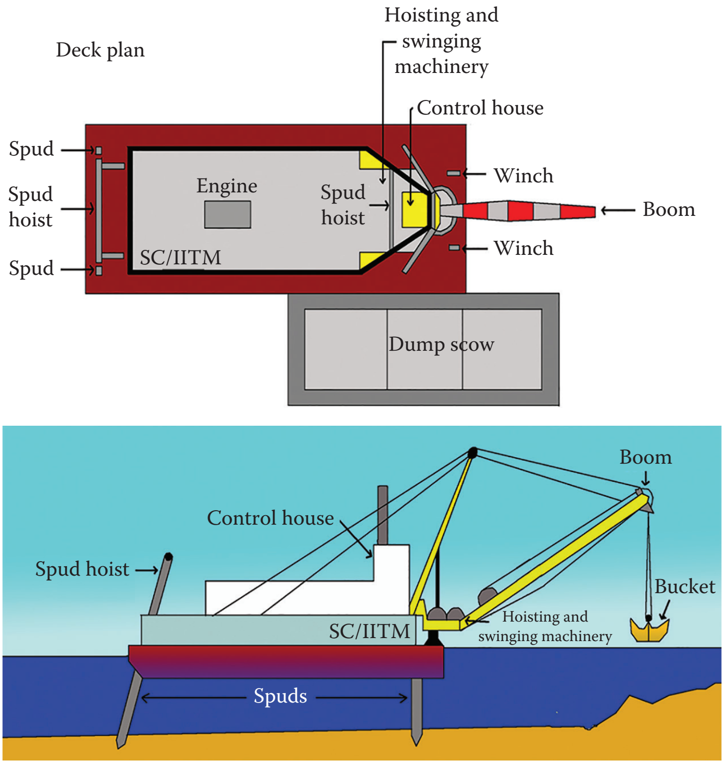


FIGURE 4.52 Bucket dredger.

and operates from a barge-type hull. A bucket is attached to the ladder, which collects the dredged material. Power is applied directly to the cutting edge of the bucket to perform dredging. To increase the digging power, the dredge barge is moored on powered spuds. This transfers the weight of the forward section of the dredge to the bottom. Dipper dredgers have a bucket capacity varying from 5 to 8 m^3 and can dredge up to a depth of 15 m. They are not self-propelled but can move themselves during the dredging process by manipulation of the spuds and the dipper arm. They can remove materials consisting of clay, hard packed sand, stone, or even blasted rocks. They are used for removing old piers, breakwaters, foundation piles, and so on. They require less room to maneuver in the work area than most other types of dredgers. Excavation is precisely controlled by these dredgers, and therefore, there is little danger of undermining of foundations when dredging is done closer to piers and docks.

The dipper capacity limits the operation volume of the dredger as the volume of excess water in the barges is to be unloaded. Dipper-dredged material can be placed in the shallow waters of eroding beaches to assist in beach nourishment. It is difficult to retain soft, semisuspended, fine-grained materials in the buckets of dipper dredges. Scow-type barges are required to move the material to a disposal area. Production is relatively lower compared to the production of cutterhead and dustpan dredges. It is not recommended for dredging contaminated sediments.

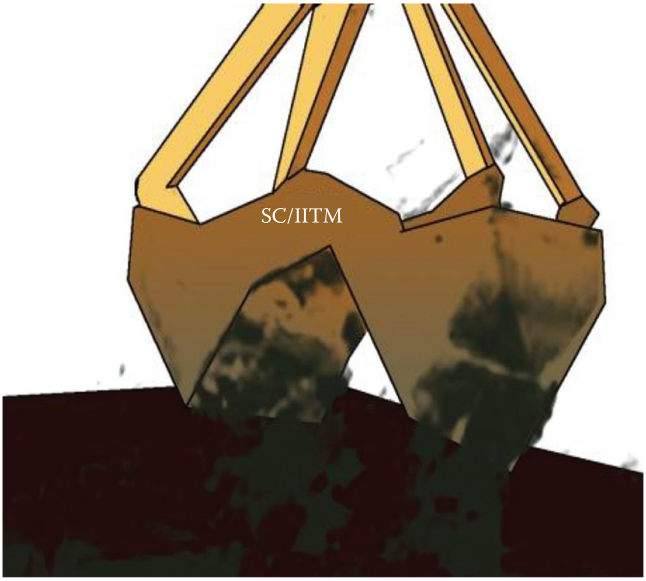


FIGURE 4.53 Details of bucket in operation.

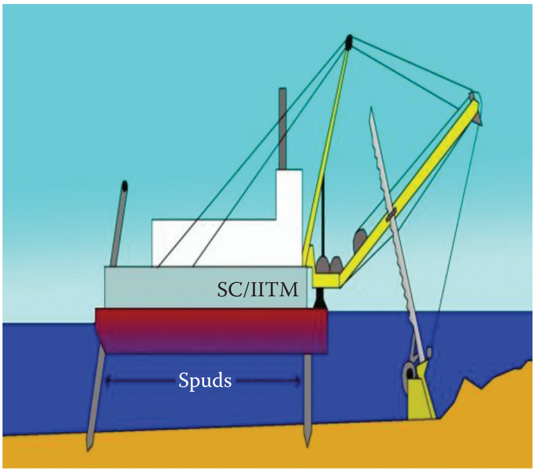


FIGURE 4.54 Dipper dredger.

4.17.1.6 Ladder Dredger

It is another type of dredger, having a series of buckets used for dredging. Figure 4.55 shows a schematic view of a ladder dredger. It dredges at a fixed rate by collecting the dredged material in the bucket; the bottom of the bucket is termed as "tumbler." A ladder connects the chain of buckets, which are cutting, collecting, and transporting. It has a series of buckets on an endless chain that roll over a drum attached to a long frame; this is called "as ladder." One end of the ladder is lowered, which brings

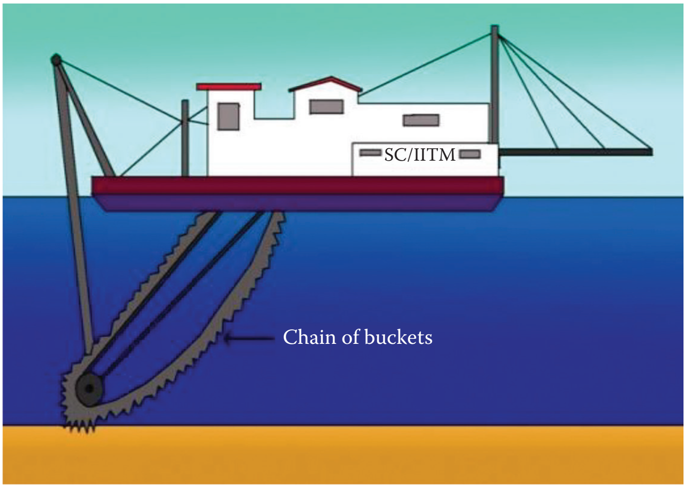


FIGURE 4.55 Ladder dredger.

the buckets into contact with the material that is to be dredged. The buckets dump, by gravity, near the upper end of their travel. The machinery on the hull controls the dredging operation.

The capacity and depth depend on the operational criteria of these kinds of dredgers. They are found suitable up to a depth of about 45 m. The digging volume varies from a maximum of 75 m^3 per hour under the average operational condition. They can be used for mining, but not extensively.

4.17.2 Hydraulic Dredgers

Figure 4.56 shows a schematic top view of a hydraulic dredger. One of the main features of all types of hydraulic dredgers is that loosened material from *in situ* position is removed by a suction pipe. There are no buckets or hoppers used to collect this material. Centrifugal pumps are deployed for the process. Initial dredge material is loosened using water jets. These dredgers are best suitable for dredging fine materials. There are different types of hydraulic dredgers: plain suction type, cutterhead type, side cast type, hopper type, and dustpan type.

4.17.2.1 Plain Suction Type

They operate by sucking through a long tube. A stream of water is caused to flow through the suction pipe by means of a centrifugal pump. Loose materials are sucked up into and further carried along with the stream and discharged from the end of the conveyance pipe. It has powerful suction pipe with a wide suction mouth piece of diameter about 300 mm. The suction inlet air speed is greater than about 200 mph. Mouth piece has a nozzle, which can be rotated to open the suction mouth. Any bigger object, if picked up (that cannot go through the tube), is dropped back. The end of the tube is toothed, which enables it to cut the earth

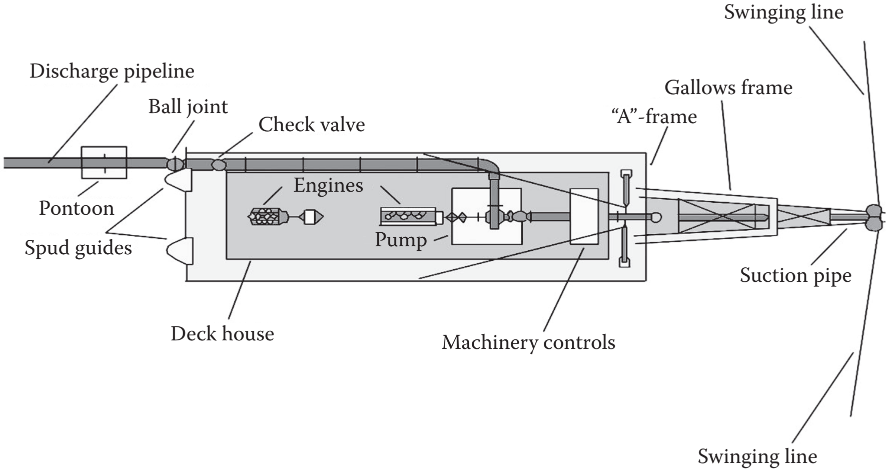


FIGURE 4.56 Hydraulic dredgers: plain suction type.

when excavating. The earth is loosened first with a powerful water jet and then excavated. Excavating with a suction excavator is termed as "vacuum excavation," whereas hydro excavation uses water jets.

These types of dredgers are equipped with pumps, which are used for applying pneumatic pressure during suction. The extended boom line, which has a swinging line, can also move laterally and peripherally along the dredgers. It is attached to the gallows frame as seen in the figure. This frame supports the suction line and swinging line. The stability of operation is assessed and controlled by the gallows frame, which is subsequently attached to the A-frame of the dredger. Machinery controls and the hub are supported by the A-frame, which is used to control the operation of the suction and movement of swinging line, in both lateral and longitudinal directions with respect to the dredger alignment. Hydraulic dredgers have a self-propelled engine, usually located on the rear side of the dredger to facilitate hinder-free operation. Discharge pipeline collects the material and pushes off the collected dredged material to the coast. During operation, the dredgers rest on spud guides, and the whole assembly of the mechanical equipment is kept on deck housing. Dredge spoil is collected on the deck by sucking it from the seabed and then discharged. Three different stages of operation of plain suction type are collection of waste, sucking with the help of pump, and then discharging it either to a barge or to the shore, as the case may be. There are some advantages and disadvantages of the plain suction-type dredgers. The merits are as follows: (1) They are used to excavate loose sand, (2) they do not need for any barges or for towing barges, (3) they are capable of operating continuously even in darkness or fog weather, and (4) they require relatively a very small crew. One of the main disadvantages is that it is limited to removal of loose material only. Discharge pipeline is generally seen as a major obstruction for the passing traffic when dredging is going on.

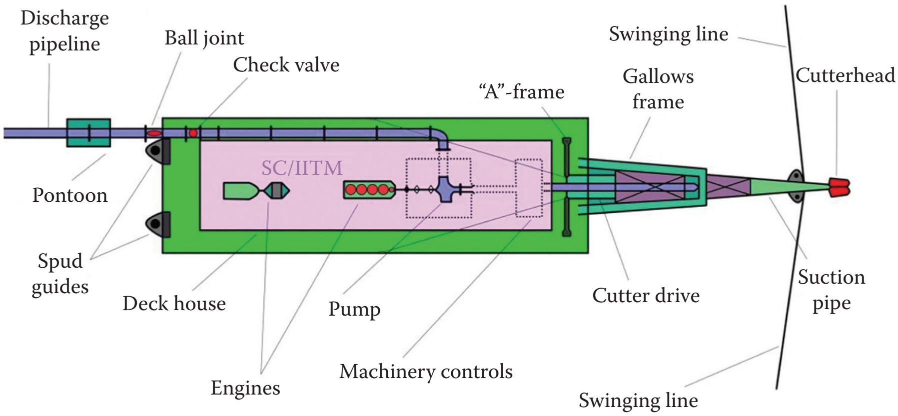


FIGURE 4.57 Hydraulic dredger: cutterhead type.

4.17.2.2 Cutterhead Type

This type works on a similar principle to the plain suction type, except that it employs the cutter at the end of the intake pipe to loosen the material. Figure 4.57 shows a schematic view of a cutterhead-type hydraulic dredger. These types of dredgers are more powerful as they are equipped with heavy-duty cutters and are capable of excavating even hard rock without blasting. The cutter consists of a rotating basket frame of spiral knives surrounding the suction nozzle.

Although dredging hard rocks and soil can be seen as one of the major advantages, discharge pipeline is a major obstruction to the passing traffic when dredging is taking place.

4.17.2.3 Dustpan Type

The dustpan type uses suction heads that are horizontally spread instead of a single suction nozzle. Loosening of material to be dredged is done by high-velocity water jets. Figure 4.58 shows a schematic view of a dustpan-type hydraulic dredger. As seen in the

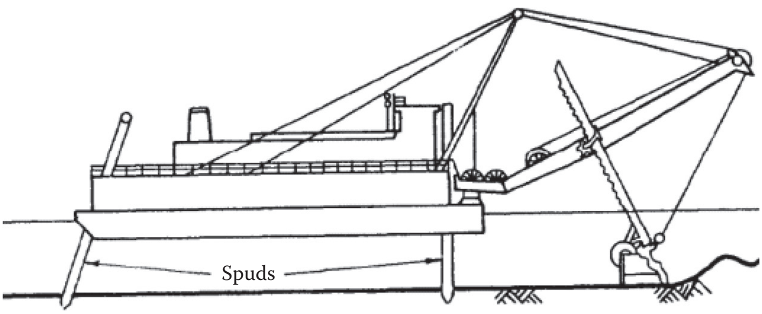


FIGURE 4.58 Hydraulic dredger: dustpan type.

figure, the tip where dredging takes place has a special type of shovel, termed as "dust-pan" and hence the name. There are many merits and demerits of the dust pan type. It is mainly used for cutting channels in loose material. It operates offline, running parallel to the direction of travel of dredge. High percentage of solids in dredge spoil is seen. High mobility is one of the main advantages. One of the main disadvantages is that it requires a nearby spoil site as the dredge spoil is not discharged through any discharge lines. Further, dredge spoil contains high percentage of toxic chemicals, which is another point of worry.

4.17.2.4 Hopper Type

These dredges are simply ships with storage areas connected to the intake pipe. Dredged material is pumped into the storage area. Once the sediment settles, excess water is released back into the ocean and sediments are discharged. Figure 4.59 shows a schematic view of a hopper dredger. They are self-propelled sea-going ships of 50–170 m long with the molded hull and lines of ocean vessels. They are equipped with propulsion machinery (for moving), sediment containers (hoppers), dredge pumps, and other special equipment, which are required to remove the material from the ocean bed. Hopper dredges have adequate power for propulsion at free-running speed and also for dredging against strong currents. They possess excellent maneuverability for safe work in rough, open seas. Dredged material is raised by dredge pumps through the drag arms, as seen in the figure. Hopper dredgers are classified based on the hopper capacity. Larger clause dredgers have a hopper capacity of about 4500 m^3; medium clause dredgers have a hopper capacity of 150–4500 m^3, and smaller ones have a capacity of less than 500 m^3. Hopper dredgers can travel at a ground speed of about 5 km/h. They can dredge in depths varying from 3 to 25 m. Figure 4.60 shows a schematic view of a self-propelled hopper dredge.

It is the only type of dredger that can work effectively, safely, and economically in rough, open water. As it is self-powered, it can move quickly and economically to the dredging site. Its operation does not interfere with the passage of traffic;

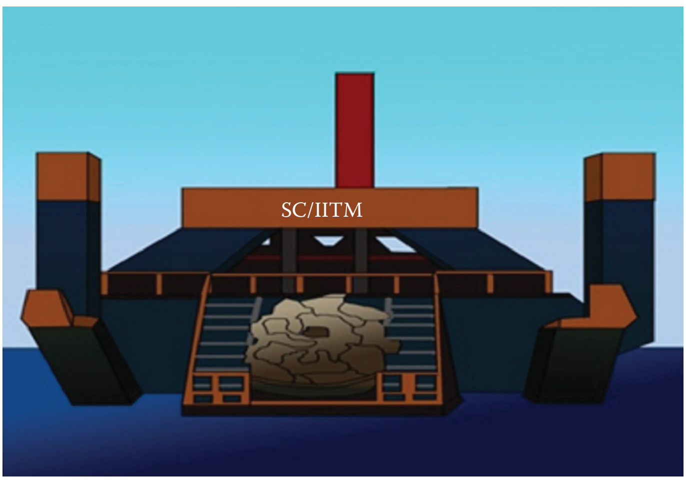


FIGURE 4.59 Hydraulic dredger: hopper dredger.

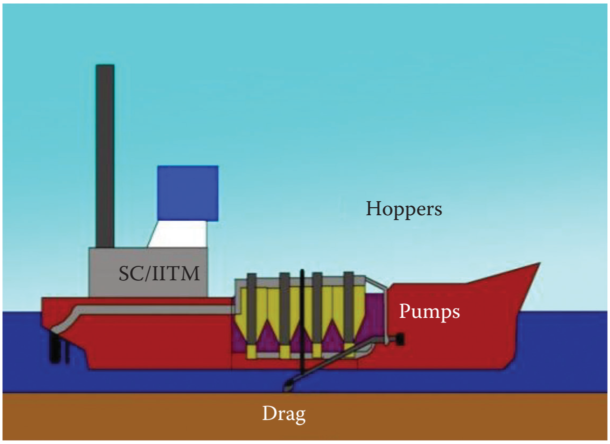


FIGURE 4.60 Self-propelled hopper dredger.

it does not discharge dredge spoil through pipeline. It is most commonly used in the improvement of navigation channel. Hopper dredger may be one of the most economical types of dredgers to use where disposal areas are not available within the economic pumping distances of the dredger. Its deep draft precludes its use in shallow waters, including barge channels. It cannot dredge continuously; normal operation involves loading, transporting material to the dump site, unloading, and returning to the dredging site. It excavates with less precision than other types of dredgers. Side banks of hard-packed sand cannot be dredged by hopper dredgers. This also cannot dredge effectively around piers and other structures. Hopper dredgers are not suitable for site with consolidated clay material.

4.18 OTHER TYPES OF DREDGERS

Other types of dredgers that are smaller in size, capacity, and operational rate are grouped under this category. They are jet-lift, airlift, auger suction, pneumatic, amphibious, and water injection dredgers.

4.18.1 Jet-Lift and Airlift Dredgers

Jet-lift dredgers use water at high pressure to draw the dredge spoil into the delivery pipe. For effective dredging, it is necessary to loosen the dredge spoil. Subsequently, the dredge spoil is sucked or transported through a pneumatic system. Jet-lift dredgers have no moving parts. Therefore, blockage by wires and rock debris is minimum during their operation. They can be easily maneuvred on spud cans as they are very smaller. Airlift dredgers are similar to jet-lift dredgers except that they use air at high pressure to suck the dredge material, instead of water. Hard material cannot be dredged as this will cause damage to the air suction system. Airlift dredgers use hydrostatic pressure to raise material from the bottom into a piston-activated cylinder. Once the cylinder is full of sediments, compressed air pushes the material through a pipe to a temporary barge or a disposal site directly. These types of dredgers are suitable for removing contaminated sediments, because very little water gets mixed up with the dredged sediments during dredging.

4.18.2 Auger Suction Dredger

They operate on the similar principle as that of cutter suction dredgers. In this type, a rotary mechanical cutting tool is used, whereas in the cutter suction type, the cutting tool remains stationary. An Archimedean screw is placed at right angles to the suction pipe. The screw releases the material, which is fed to the suction pipe when it enters the pipe. Auger suction dredgers advance into the cutting face by hauling operation. They are used for very high precision cutting and dredging.

4.18.3 Pneumatic Dredger

This works on the evacuator principle. It contains a chamber, which is opened to draw the material and water inside the chamber. The material sucked into the chamber is then pumped out with the inlet of the chamber in a closed position. The process includes sucking and evacuation for one cycle, which makes the dredging action intermittent.

4.18.4 Amphibious Dredger

These can remain operational while afloat or elevated clear of the water surface. They are fitted with grabs, shovels, and buckets to collect the dredge spoil. They are useful in site locations where the soil is not suitable for operation of other heavy-duty dredgers.

4.18.5 Water Injection Dredger

This type of dredger has a pneumatic water jet, which is supplied with a self-propulsion system. It is composed of a long ladder and a series of buckets as seen in Figure 4.61. The water jet is continuously poured to loosen the material, which is subsequently sucked up using a grab or a bucket. A ladder hoist wire supports the ladder that hosts the bucket, which is a hoisting wire.

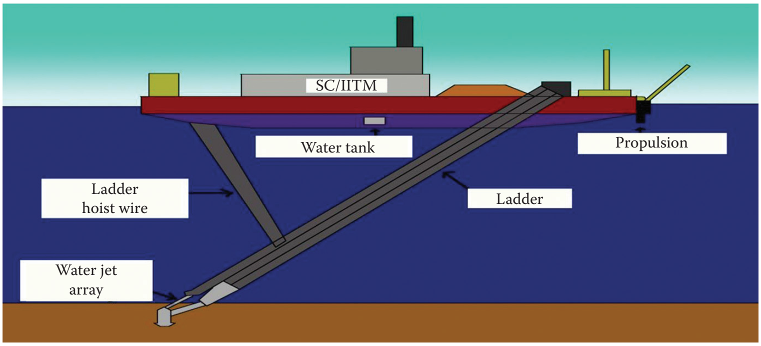


FIGURE 4.61 Water injection dredger.

4.19 DREDGER AUXILIARIES

All dredgers are equipped with a series of auxiliaries. They are sophisticated, electronically controlled data logging systems. They are helpful in positioning a dredger, loading, recording, and station keeping. The position of dredgers, while in operation, is shown in the visual display unit. Any adjustment of the trailer, drag head, cutterhead, backhoe stick, and bucket can be easily made with the help of the control system. This enables to improve precision dredging. When they are in operation, even the depth of cutting, current dredged depth, and slopes are also controlled according to the desired values. As the dredging operation continues, they help in improving the efficiency and avoid overdredging. They also help in improving the precisions of dumping where strict environmental controls do exist.

4.20 DREDGING EQUIPMENT AND SPECIFICATIONS

A few dredging equipment and their specifications are discussed for the benefit of the practicing engineers.

4.20.1 Aquarius

Figure 4.62 shows Aquarius, which is owned by Dredging Corporation of India (DCI). It is a high-powered cutter suction dredger, which is self-propelled. Built

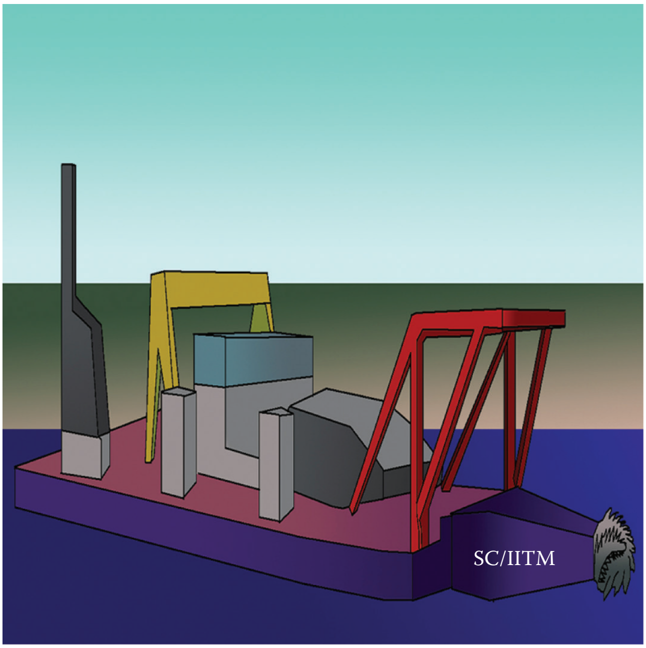


FIGURE 4.62 Aquarius: cutter suction dredger.

by DCI in 1977, its overall dimensions are 107 × 19.66 × 7.6 m for an operational draft of 4.85 m.

It has a gross weight of about 2900 tons and is capable of dredging up to 25 m depth. The maximum pumping distance of the dredge spoil is 6 km. Auxiliary equipment of the dredger are an 800 m-long floating pipe to transport the dredge spoil and a 1.2 km longshore pipeline to transport to the disposal area. This dredger is self-propelled with a sailing speed of 11.5 knots (1 knot is about 1.85 km/h). The dredging capacity of Aquarius is about 5–10 million m^3 per year. This is found suitable for large capital dredging and mainly used for reclamation of low-lying areas with the dredged material. It is equipped with crane barge type 3 and control systems of CAT DV 1740. Figure 4.63 shows a dredger with suitable arrangements for loosening of materials.

4.20.2 DCI Dredge BH-I

DCI Dredge BH-I is a backhoe dredger used for sideline dredging from the pontoon. Figure 4.64 shows a dredger in operation along the coastline. DCI Dredge XVI, which operates along the coastline, is helpful in maintaining the navigation channels of ports and harbors by clearing the accretion. It is a trailer suction dredger with a hopper capacity of 7400 m^3 and can dredge up to a maximum depth of 25 m. This dredger has a shore pumping facility. Although the dredger is heading toward the dumping ground, sediments are collected and the accumulated water is drained off from the dredger. Figure 4.65 shows a typical rotary cutting edge of the dredger. Figure 4.66 shows the backhoe dredger DCI Dredge BH-I in action.

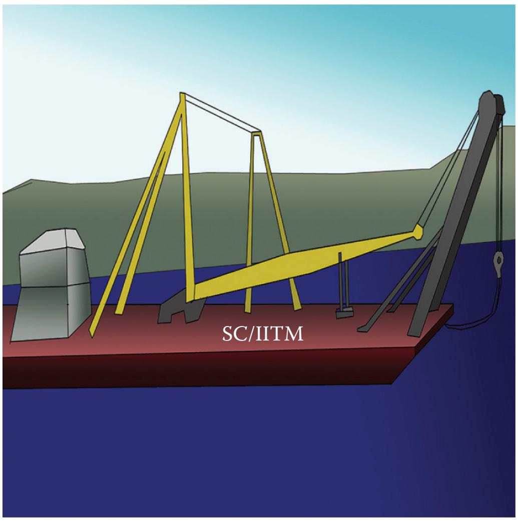


FIGURE 4.63 Dredger with arrangements for loosening of material.

FIGURE 4.64 Dredger in operation: sideline dredging from the pontoon.

FIGURE 4.65 Rotary cutting edge of a dredger.

4.20.3 DCI Dredge XVIII

Figure 4.67 shows the dredger DCI Dredge XVIII in action, transporting the dredge spoil to the coast using a longshore pipeline. It is a cutter suction dredger with a maximum draft of 2.5 m. It is capable of dredging up to 20 m depth, with a maximum pumping distance of the dredge spoil to about 3 km.

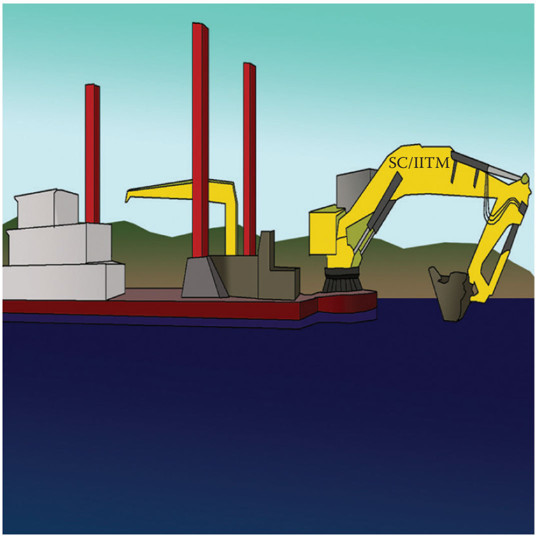


FIGURE 4.66 DCI Dredge BH-I: backhoe dredger.

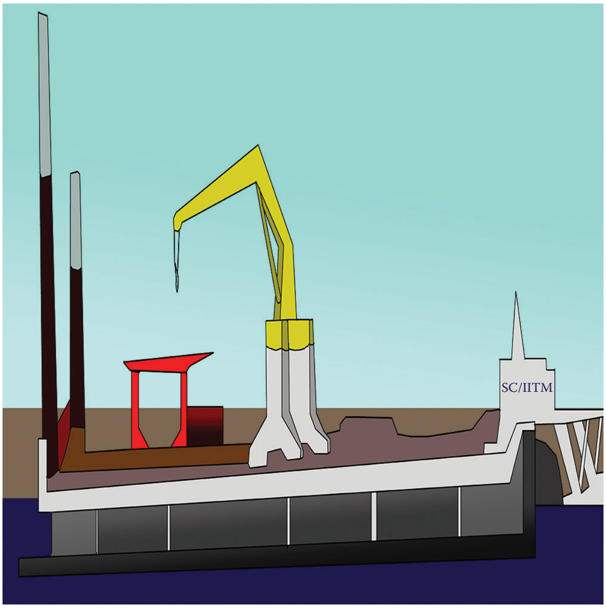


FIGURE 4.67 DCI Dredge XVIII.

4.21 DREDGING APPLICATIONS

Apart from an essential application for safe navigation, dredging has also other important applications. It is vital for environmental protection and beach nourishment for promoting tourism. Flood control and irrigation are seen as additional applications of dredging. Dredging is essential for any port development and maintenance of the navigation channel. It is vital to maintain a navigable path to promote national and international trade. It is also useful to improve water resources by constructing desalination plants. It is carried out at periodic intervals to upkeep the nation's waterways and channels sufficiently deep and safe for shipping. It is vital in beach nourishment and land reclamation projects. Dredge spoil, which is disposed of through the pipeline, is one of the main disadvantages in many dredging processes.

Dredging projects are not cheap; they are one of the most expensive projects being executed. Recent advancements in dredgers make them a state-of-art technology. They are equipped with microchip technology and remote sensing satellites to control dredging alignment. They are custom built to withstand sea roughness. They are also equipped with latest communication devices, instrumentation, and computerized dredge control systems to ensure more accuracy in dredging.

4.22 UNCERTAINTIES

Construction methods of offshore structures govern the design process as well. For example, installation spread restrictions, fabrication yard limitations, load-out procedure, transportation requirements, and geometric restrictions imposed by the choice of lay-barges can also govern the design process. In terms of the details of piles of a template structure, their preliminary member size, diameter, penetration depth, and number of piles govern the construction window timings and the cost of construction as well. It is important to know that a check design of the template structure is carried out by carrying our preservice analysis. This includes fabrication details, load-out procedure proposed in the design, transportation limitations, installation methods (e.g., which of the methods, namely, launch, lift, and upend will be practiced in the current project), and material availability including special material required for grout and any site-specific problems. Although the member dimensions are governed by different load conditions, they are also checked for other loading cases which occur during the construction process such as installation loads, accidental loads, and impact loads due to drop-off.

4.23 SAFETY AND RELIABILITY ISSUES DURING CONSTRUCTION AND INSTALLATION

Offshore construction is a very serious process not because it is very expensive but it requires a lot of engineering inputs during construction. It is therefore termed as "engineered construction." Although the process is governed by several technical factors, it is imperative to understand that the safety is placed on top priority during the whole process. Complications arise due to unexpected and unforeseen situations during the process in almost every offshore construction project. It is interesting to know how reliable the whole process is and therefore to realize the risk involved. Therefore, to ensure safeguard assessments in engineering perspective, multilevel checks are carried out at various stages by various agencies, including the owner, certification agency, marine insurance agency, marine warranty surveyor, and installation agency. A thorough execution of checks is done, including the design, analysis, construction, and installation methodologies in detail. This is required as safety and reliability issues are present in every level of the construction process, which are briefly highlighted in Sections 4.23.1 through 4.23.3.

4.23.1 Engineering

Although the engineer-in-charge of the owner reviews and approves all engineering deliverables, the certification agency reviews only a few selected items and recommends changes if any in terms of safety and environmental management issues. Marine insurance is mandatory for all operations carried out after the completion of fabrication in the yard. Therefore, a marine warranty surveyor reviews and approves all designs and construction procedures related to the relevant operations in detail. In parallel, the installation agency reviews the procedure for installation and commissioning and gives its acceptance of all items related to the project.

4.23.2 Fabrication

Extensive nondestructive examination and documentation of fabrication checks are carried out independently by the engineer-in-charge of the owner. The third-party inspector, the reviewer of the certification agency, and the monitoring agency also inspect the fabrication work and submit their approval. Any deviation from the approved drawings and construction procedures is thoroughly reviewed in terms of the following: reasons for the proposed deviations, subsequent changes in the construction methods due to the proposed changes, and economic consequences due to the changes in terms of extended time window, additional labor cost, equipment hiring, and so on. They are reviewed and subsequently approved by both the engineer-in-charge of the owner and the certifying agency. It is expected that practically all errors, mistakes, or anomalies would be captured and rectified during these review process.

4.23.3 Installation

The engineer-in-charge of the owner reviews the installation process in detail, which needs to be also approved by the installation contractor and the warranty surveyor. Each stage in the construction process should be reviewed in detail and subsequently approved by the warranty surveyor. This step is very important due to the fact that offshore construction projects have very high degree of risk involved due to various uncertainties. Commencement of any operation is approved only after a review of 72 h weather forecast. Every operation and critical equipment should have a standby/backup in order to avoid any delays due to malfunction. Notwithstanding these safeguards, mishaps or accidents may occur. Offshore accidents occurred in the recent times leave a trace of economic and personnel loss resulted from such offshore accidents. However, offshore construction projects are one of the most thoroughly engineered processes, which are reviewed with very stringent norms.

4.24 UNCERTAINTIES IN THE CONSTRUCTION PROCESS

With all the stringent norms adapted in practice, still uncertainties exist in the offshore construction process at different stages.

4.24.1 Fabrication

The primary uncertainty arises from the material availability. There can be nonavailability of requisite materials at any required time in the sequence of fabrication, which is not uncommon. Materials may be available, but at a specific sequence of operation, a desired quantity of any specific material of particular grade of marine steel may not be available. This may occur due to many factors such as cost factor, delay in procurement, and transportation delay. Therefore, in offshore construction projects, one is supposed (and compelled) to have a substitution with an equivalent or a superior material.

Inadequacy in detailing in design drawings may also arise. For example, the design and drawings may not have all details as required by the fabrication contractor; there can be a few inadequacies. In most of offshore construction projects, such issues are generally caught during the review and corrected at the review stage. Alternatively, the fabricator will incorporate the changes in the shop drawings himself or herself after getting them approved from the respective agencies before he or she completes the fabrication. It is important to note that any such deviations made without a thorough review by different independent agencies lead to high degree of risk, compromising safety. It is always a healthy practice to rectify such fabrication errors by capturing and correcting them during the inspection and review stages.

4.24.2 Load-Out

Uncertainties are also associated during the load-out stage of construction. Any deviations in methodology, which were proposed in the initial stages, should be thoroughly reviewed before admittance. All necessary strength checks are then thoroughly reviewed. A revised analysis is carried out to ensure the structural adequacy during load-out operation that arises from the modified procedure. In simple terms, any changes, even if they are minor, cannot be executed until they are reviewed by independent agencies at different levels before final approval. Equipment failure during transfer to the barge with the rising or falling tides should always have a backup to ensure safety. Such potential scenarios are analyzed for a certain structural adequacy during the engineering stage by the designers.

4.24.3 Transportation

Uncertainties during the transportation stage arise due to the prevalence of rough weather or cyclone. In such cases, both the barge and the structural system should be analyzed for their survivability under such rough weather combinations, which are likely to occur during the installation stage. For example, the maximum sea state, which is expected during the transit through various sea states, seasons, and locations, should be examined thoroughly and reviewed. The barge should be inspected and certified for sea worthiness. Further inspection should be done before mobilizing the barge for transportation of the platform. There can be a possibility that damage can occur during transportation because of collision. One should take care of safe operation during collision and during grounding as well.

4.24.4 Installation

During installation, once the load-out is complete and the transportation is in progress, there is a possibility of rough weather or cyclone. Launching or sinking of the jacket after launch is an important failure stage of the buoyancy element. This can be controlled by checking the damaged compartment scenario, which is the part of the routine design check generally done for offshore structures. Unpiled stability of the jacket and mud mat design can also result in jacket toppling during installation. This is highly possible in the case of soft soil locations. Another problem that may arise is tilting of jacket during installation. This can be handled by lifting the jacket and rectifying it during piling. It is one of the standard features, which generally occurs in almost all construction projects of jacket platforms. There can also be problems associated with the slopping seabed. Generally caught during the surveys, it is taken care of in the design by proposing stepped mud mats.

Uncertainties can also continue during the pile-driving stage. Drivability may result in premature refusal, which depends on the soil strata. Alternatively, it may also arise due to the improper hammer performance. Any inaccuracies with respect to the soil conditions and its geotechnical properties may also lead to overdriving of the pile. Such problems are taken care by mobilizing a higher size hammer in anticipation. There can be remedial equipment, which acts as standby during the pile-driving operation.

There can be problems that arise during the grouting stage. There can be inadequacy of grout strength, which is detected from the test conducted *in situ*. In such cases, a better designer should always propose a conservative design to cater to this scenario. Remedial action is not possible due to the fact that this is very expensive and will cause a serious time delay. It is important to note that any delay in the offshore construction happening while the installation is progress in open sea will challenge the safety seriously. Hence, it is a common practice to initiate a conservative design for grout inadequacy. Grouting can also have problems associated with shear keys, which are not available in sleeves. In such cases, increased grout strength using special mixtures can help to counteract such problems.

4.24.5 Topside Installation

There are specific uncertainties associated with topside installations; the major problem arises while lifting the template. The balance in the structure is disturbed if the proposed design does not match the lifting arrangement. In addition, such problems could arise from inadequacy in transferring the lifting loads, which is an oversight in the design. Generally, such errors are captured in the review stage itself. Further, there can be problems associated with the sling length tolerances. This may result in loss of hookup load, which may cause a gentle impact on the structure while lifting. This is one of the routine checks in the design, which is addressed.

4.24.6 Human Factors

There are serious human factors, which contribute to a higher level of uncertainties. Human errors are considered and seen as one of the major causes for many accidents and mishaps in the recent times. This can arise due to inadequacies in the knowledge of construction engineering, training, and experience imparted to the engineer. It can also arise from the lack of application of the knowledge, at the time when they are in demand. There can be a serious nonconformance to safety practices advised by the warranty surveyors. It is very important to impart sufficient training to offshore engineers and promote a strong culture of capacity building in them. A detailed knowledge of design, equipment specifications, emergency shutdown procedures, wave climatology, and weather impact are necessary for an offshore engineer before he or she undertakes any offshore installation.

4.25 SEABED ANCHORS

A permanent anchor is termed as "mooring" and is position restrained. A sea-going vessel may not host a permanent anchor but hire a service to move or maintain it. A temporary anchor is usually carried onboard by the vessel and hoisted aboard whenever the vessel is under way. Anchors are used in water depths exceeding 1500 m. It resists the moment caused by the lateral forces acting on the vessel. There are two ways to resist the lateral loads: (1) by adding heavy mass to resist the lateral load and (2) by hooking it to the seabed.

The purposes of the anchors are as follows: (1) to limit the vertical and lateral movements of the floating and submerged structures; (2) to hold down in position the observer buoys that are used to monitor wave kinematics and water properties such as salinity, density, temperature, or any other aspects at the air–sea interface; (3) to hold down large buoys that are used for data collections and other navigational purposes; and (4) to restrain the movement of oil and gas pipelines laid on the seafloor.

4.25.1 Loads on Anchors

Forces generated on anchors do not essentially arise from wind and ocean currents but from the heave motion of the vessel caused by waves. The vertical movement of the sea waves induces the maximum loads on anchors. Resistance offered by an anchor is essentially a combination of two factors: (1) its geometric shape and (2) the technique used to anchor it to the seabed. Aweigh refers to the anchor when it is hanging on the rope and not while it rests on its bottom. Therefore, an anchor is described as aweigh when it is plugged off from the bottom and hauled up onboard.

4.25.2 Temporary Anchors

4.25.2.1 Fluke-Style Anchor

It consists of a central bar called the shank, an armature with two large flat surfaces called flukes, to grip the bottom, which also has a sharp point for penetration. Figure 4.68 shows a schematic view of a fluke-style anchor. The armature is attached to the shank at the "crown," as seen in the figure. The stock at the crown is hinged so that the flukes can move or rotate about the hinge. Thus, it can be positioned and used for anchoring the object to which it is connected. Vessels or members are connected to the shank by means of a steel chain or a wire.

4.25.2.2 Plough Anchor

This resembles the traditional agricultural plough. Figure 4.69 shows a schematic view of a plough anchor. It comprises a hinged shank, allowing the anchor to turn with the change in direction without breaking out. Delta-type anchors use unhinged shanks; a plough is attached to the shank at specific angles to develop slightly superior performance. It is seen from the literature that anchors with hinged shanks are

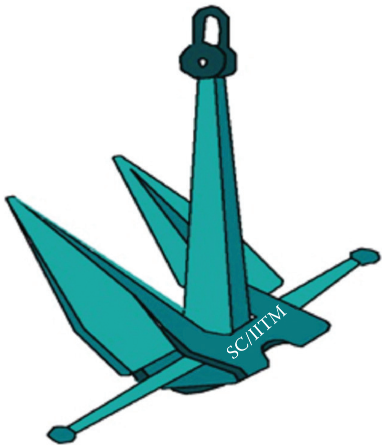


FIGURE 4.68 Fluke anchor.

FIGURE 4.69 Plough anchor.

susceptible to frequent pullout but no structural failure. The hinged point becomes the weakest connection and thus results in failure. However, delta-type anchors show a structural failure as they are rigidly connected to the shank. As there is no hinge connection, there is a higher probability of breaking of the anchor in total. Their holding capacity is better than that of the hinged shanks.

The delta type uses an unhinged shank; sometimes, there are also unhinged shanks and a plough with a specific angle. As seen from Figure 4.69, the plough is a very massive element, which actually holds down the shank at the specific location. It cannot be pulled off so easily when it is placed in position. Due to its tip-weight, the plough is heavier than the average resistance developed by the anchor. These anchors take a longer pull to set thoroughly.

4.25.2.3 Bruce and Claw Anchors

Bruce and Claw-type anchors consist of claws that set in most of the seabed. Figure 4.70 shows a schematic view of a Bruce and Claw anchor. The geometric arrangement of the claws is such that it becomes one of the convenient ways of holding down the object in position. They do not break with tides or change in wind directions; instead, they slow down in the bottom to align with the lateral forces. They slowly turn so that they get set at the sea bottom as the force gently acts upon these anchors. Claw types have difficulty in penetrating because the sharp edges are not as same as that of a plough type. Due to this fact, they have certain difficulties in penetrating in the seabed. However, once fixed, it holds rigidly in position. In particular, the presence of large and intensive vegetation does not cause any problems to these anchors. They offer low holding power due to the absence of sharp edges in the claws. Their holding power is less in comparison with their weight; they are generally oversized.

4.25.3 Permanent Anchors

They are used to hold down the vessel to a permanent position. This may be required during exploration and production.

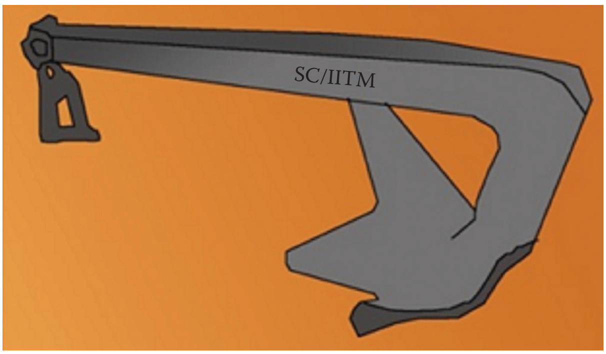


FIGURE 4.70 Bruce and Claw anchor.

4.25.3.1 Mushroom Anchor

Mushroom anchors are one common type of permanent anchors, which are generally carried onboard. Figure 4.71 shows a schematic view of a mushroom anchor, stowed aside of a vessel. It is shaped like an inverted umbrella or a mushroom. While in anchoring position, the head is buried into the silt to hold the vessel in a permanent position. It is suitable for a seabed composed of silt or fine sand. A counterweight is provided at the other end of the shank to lay it down, before it gets buried. A mushroom anchor will normally sink into the silt at the point where it has displaced its own weight into the bottom material. The holding power of this anchor is about twice its weight unless it becomes buried. These anchors are used to permanently anchor a vessel in a specific location.

4.25.3.2 Deadweight Anchor

Deadweight anchors are used where mushroom anchors are not suitable. There are several advantages of these kinds of anchors. Even when they are dragged, they continue to provide their original holding force. One of the important demerits is that these anchors are heavier—about 10 times heavier than that of the mushroom type. As these anchors develop anchorage purely based on their self-weight, they are heavy in mass. The type of soil does not influence the holding operation of these anchors and hence suitable for almost all types of seabed, whereas mushroom anchors have limitations. These anchors hold a large block of concrete or stone at the end of the chain. The holding power is equal to its weight under water, considering buoyancy into account.

4.25.3.3 Suction-Embedded Anchor

A suction caisson anchor is a large diameter cylinder, constructed either in steel or in concrete with an open-ended bottom and closed top. Mooring loads are applied by an anchor line attached to the side of the caisson. The length-to-diameter ratio of the caisson is typically 6 or even less. Once installed, the caisson acts like a short,

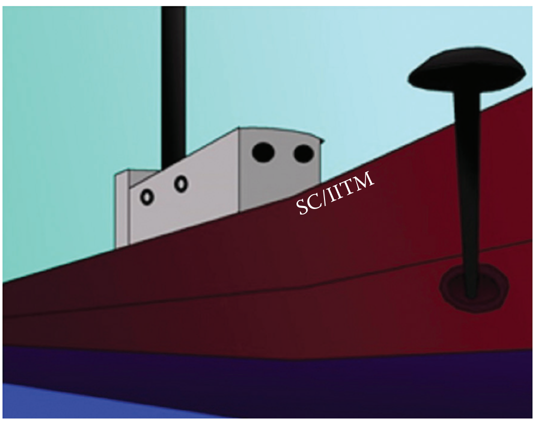


FIGURE 4.71 Mushroom anchors.

rigid pile and is capable of resisting both lateral and axial loads. The suction caisson is installed by applying underpressure ("suction") to its interior after it is allowed to penetrate under its own weight. Because the caisson's interior is sealed from the seafloor by the soil, vertical loading creates an internal drawdown pressure, which in turn mobilizes the end bearing resistance of the soil at the caisson tip. They are best suitable for soft, cohesive-type soil. They are generally used in deep- and ultra-deepwaters. They are installed in water depth varying from 40 to 2500 m. These kinds of anchors have a diameter varying from 3.5 to 7 m. Their penetration is up to a maximum of 20–25 m. They are commonly used to anchor floating exploration and production platforms (FPSO).

4.25.4 Anchoring

While discussing anchoring, a few factors are considered to select the type of anchoring. Most importantly, one needs to check whether the anchorage is protected. A good anchorage should offer protection from both the present and expected weather conditions in the offshore site. One should subsequently check whether the anchorage system is completely protected from the lateral forces. It is very important to check whether the seabed has a good holding ground. Charts indicating the nature of seabed can be studied to understand the ground conditions in detail. Samples can also be collected from the seabed to examine their geotechnical characteristics. Most of the anchors are well suitable for sandy-mud, mud, and clay soil or a firm sand, whereas loose sand and soft mud are not desirable seabeds for holding the anchors. One should also examine the nature of the seabed as the selection of anchor depends on these data. The depth at which anchoring is expected to take place, tidal range in the offshore site, current velocity and direction, and their variabilities also need to be examined before anchoring.

In addition, different holding conditions at which anchorage is to be done need to be examined. Irrespective of the anchor type, the optimum holding capacity depends on several conditions. The foremost condition is that whether the anchors have sharp edges to hold down. Examining their geometric symmetry while buried plays a significant role in choosing anchors for offshore applications. Asymmetric holding of the anchor may result in unnecessary lateral displacement of the vessel during high- or low-tide conditions, which should be avoided. One should also check whether the chosen anchor is capable of completely penetrating into the soil at the chosen offshore site.

The second condition is that if the fluke and crank shafts are hinged together, the articulation must be kept in open position. This is to be ensured to satisfy the condition that the angle of rotation of the fluke can be adjusted by itself depending on the lateral forces acting on the anchor. Third, one should examine whether the chosen anchor is that the tension on the pulling eye of the shank is more or less parallel to the seabed. It is not a good anchoring to hold down the anchor so that its shank remains almost perpendicular to the seabed. This may result in a very less holding power, and the anchor may be pulled off from the seabed even under less lateral forces. Ideally, the shank is almost parallel or closed to around 10°–15° to the horizontal of the seabed so that the anchor generates enough holding capacity,

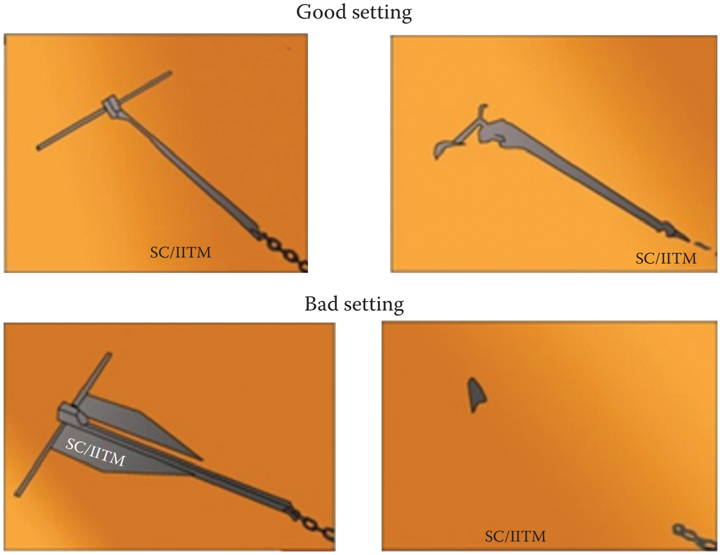


FIGURE 4.72 Holding conditions.

when it is being penetrated in the sea bottom. Figure 4.72 shows a schematic view of holding conditions of the anchors. In case the anchorage is affected by the tide, one should keep in mind that the swing range will be larger at low tides in comparison with that of the higher ones. Therefore, it is important to understand that larger swing range is required, which should be free from obstacles and hazards; other vessels in the anchorage may also have a swing range, which can overlap that of the present vessel.

4.25.5 Requirements of Anchors

They should provide enough holding power and have a minimum size and weight for easy handling. Predominant forces influencing the design of anchors are the nature of sea bottom (clay or sand), the bottom slope direction, and the intensity of the mooring line tension. The length of the cable also plays an important role in the design of anchors. If it is too short, it will result in intermittent submergence or pullout of the anchor. If it is too long, it will permit excessive movement, which results in possible kinking and fouling. This weakens the cable and initiates the failure by breaking. The holding power of an anchor refers to the pulling force that the anchor should resist. It depends on the following factors: (1) the depth of embedment, (2) the submerged weight of the anchor, (3) the angle the cable makes with that of the sea bottom, and (4) soil properties. The angle subtended by the anchor with respect to the seabed and the anchor shank is an important parameter, which will decide the holding power of an anchor. Anchors are rated based on their capability index, which is termed as the holding power of an anchor. If the weight of the anchor in air is known, the capability index is the ratio of holding power to

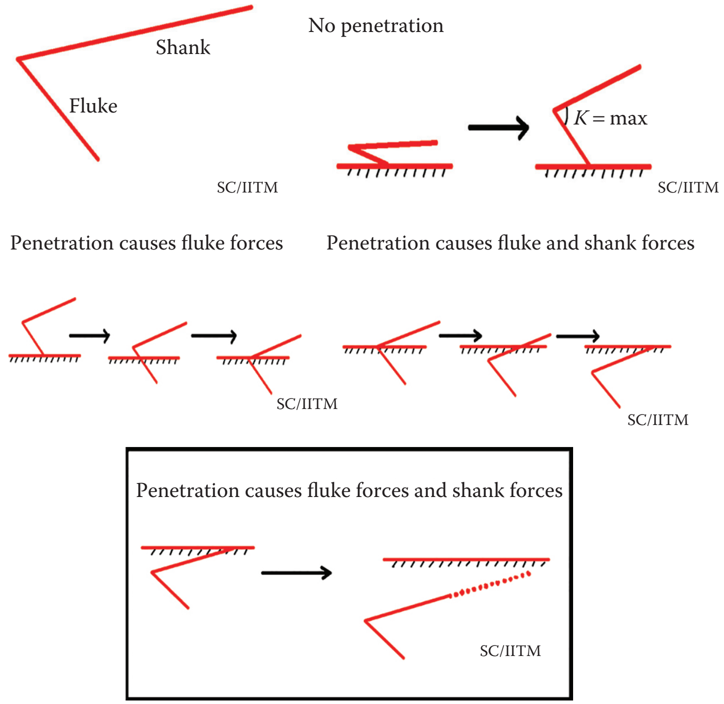


FIGURE 4.73 Forces on anchors.

the weight. Figure 4.73 shows various forces that act on anchors and their response under lateral forces.

4.25.6 Commercial Anchors

There are different kinds of commercial anchors available in different names. Figure 4.74 shows a schematic view of commercial anchors. Commercial stockless anchors are similar to the fluke and shank anchors. There is a hinged connection to which a chain or a wire can be connected. One of the main differences between the standard navy stockless and the Mark 2 stockless is the presence of holes in the flukes. These holes enable them to be connected together along with the shank and offer more resistance. Other anchors have flukes with sharp or blend edges. They can be suitable for different kinds of soil.

4.26 FENDERS

Fendering systems provide protection to both the wharf and the berthing vessel by absorbing the berthing energy imposed by the vessel. They reduce the forces, both on the wharf structure and on the vessel. Provision of fenders results in reduction of cost in wharf structures and provides satisfaction to the ship owners. Various factors that affect the fender design include material selection, geometry, size, positioning, and method

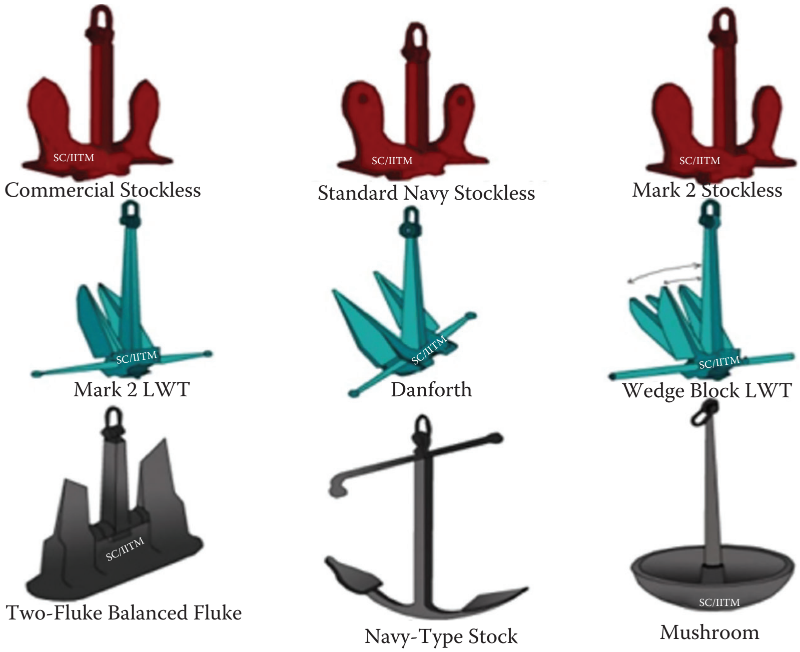


FIGURE 4.74 Commercial anchors.

of attachment to the wharf structure. Selection of the fender system depends on the size and shape of the vessel to be berthed, structural design and limitation of the pier or dolphin, berthing velocity of the vessel, probable berthing angle, whether berthing is assisted by the tugboats, wind speed in the site, tide range and current velocity, and the type of cargo and loading methods. Fenders essentially use rubber as the primary material whose basic characteristics are modified to improve their strength and resistance to environmental conditions. Natural rubber and styrene-based rubber compounds are used to manufacture fenders; material requirements for fender should qualify international regulations. Several manufacturers are competent in fenders; say, for example, DockGuard, Fentek, and so on. Their high-quality assurance and technical support systems enable offshore engineers to obtain ready made solution for fender design and installation. Fenders are of different types: leg fenders, cone fenders, cell fenders, arch fenders, cylindrical fenders, extruded fenders, sliding fenders, fender pads, and polyrub fenders. A few of them are discussed in Sections 4.26.1 through 4.26.7.

4.26.1 Leg Fenders

Leg fenders are molded in a rhomboid shape for optimum energy absorption. Twin notches on the side of the fender ensure uniform compression to absorb energy. These can be mounted vertically or horizontally on any quay wall. Leg fenders are compressible up to about 60% of their height. They vary in dimensions as 750–3000 mm in length and 150–1600 mm in height. They are best suitable for locations where there is limited space for installation. They can be used on faces of piers, columns, and steel plates.

4.26.2 Cone Fenders

Cone fenders are molded in a conical shape to provide good absorption with low reaction. Figure 4.75 shows a cone fender fixed to a berthing jetty. The conical design and circular mounting base make these types of fenders as extremely stable where angular performance is required. These fenders replace large leg fenders, as their performance is better than the latter. They are ideally suitable for a wide range of berthing applications, which include liquid natural gas (LNG) and oil terminals, offshore platforms, bulk handling terminals, container berths, and cruise terminals. The conical shape and the deflection mode allow the angular berthing up to 10° with no reduction in energy absorption. This geometry provides superior stability in resisting both vertical and horizontal shear forces. Fender buckling maximizes the energy absorption with a minimum reaction force, enabling overload protection. The conical shape assists in self-centering of the fender element during compression and thus enhances stability.

4.26.3 Cell Fenders

Cell fenders are molded in a cylindrical shape to provide good energy capability. They are suitable for locations offshore compliant platforms and floating offshore jetties where berthing movements are present. They are ideally suitable for applications that are under circular motion and extreme weather conditions where heavy angular

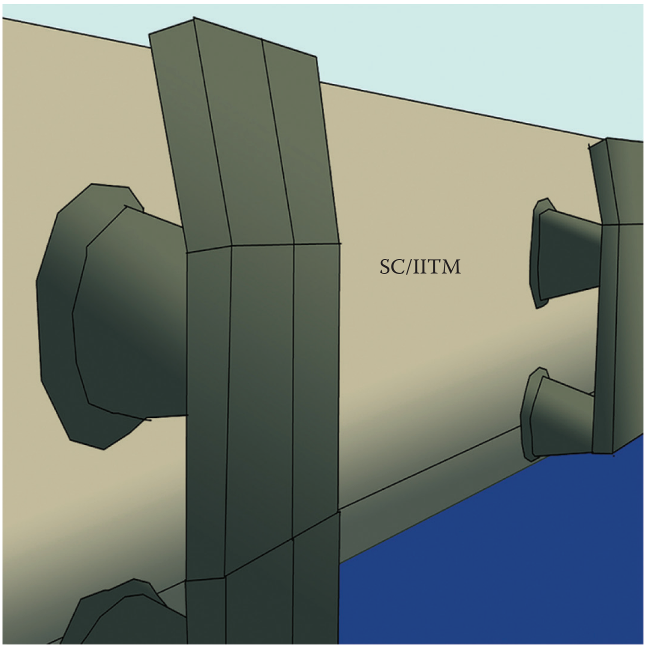


FIGURE 4.75 Cone fender fixed to a berthing jetty.

berthing may be required. With a higher reaction than cone and leg fenders, cell fenders are used on quay structures rather than dolphin berths.

4.26.4 Arch Fenders

Arch fenders are used where the height of fenders is minimized. Steel plates are incorporated in the base of the fender during vulcanization process. This makes them effective and easy to install to the wharf structures. Arch fenders provide increased resistance against shear force in horizontal mounting. The front end of the fender is fitted with ultra-high-molecular-weight polyethylene protector pads, which provide low frictional resistance and increased protection against abrasion. They are ultraviolet (UV) resistant and not subjected to decay by marine organisms. Support chains are not required even when fitted with steel front panels. Arch fenders offer a good protection to the corner of the wharfs. The front phase of the fender has a high friction factor, which makes it ideal for smaller vessels where friction is not an issue. They are also suitable for workboats, barges, and tugboats. Figure 4.76 shows an arch fender installed on a berthing jetty.

4.26.5 Cylindrical Fenders

Cylindrical fenders can be mounted horizontally, vertically, or even diagonally. They are also available with precurved geometry to protect the corners of wharfs. These fenders can be attached to the wharfs by many ways using mounting bars, chains, ropes, or even brackets. They are highly versatile, cost effective, and available in a wide range of diameters and lengths. Smaller diameter fenders are produced by extrusion process, whereas larger diameters are produced by wrapping and curing

FIGURE 4.76 Arch fender.

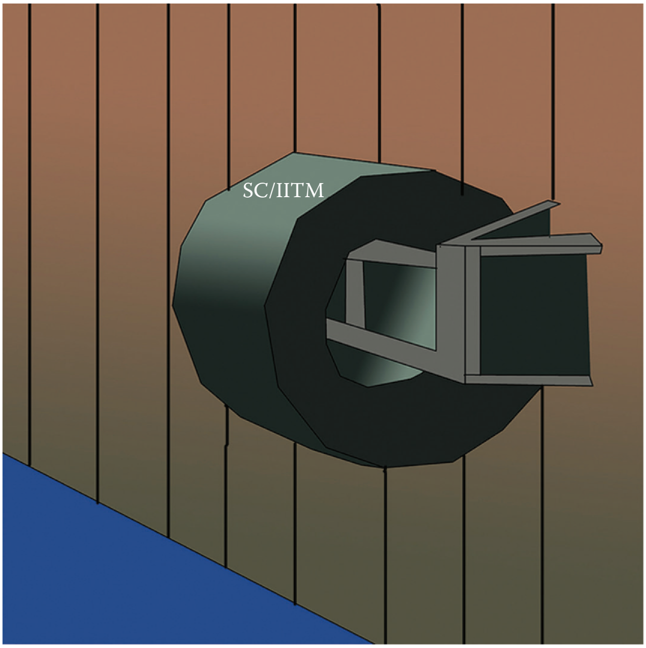


FIGURE 4.77 Cylindrical fender.

under pressure. Their simplicity in fixing and replacement makes them cost effective. Installation and maintenance are easy and cost effective. Figure 4.77 shows a cylindrical fender installed.

4.26.6 Extruded Fenders

The name "extrusion" refers to the manufacturing process. Uncured rubber is forced through a die to produce the required profile; the length of the rubber is cut and vulcanized to any customized size. These fenders are available in user-defined sizes as the extrusion process simplifies the dimension constraints. These fenders are commonly used in harbor installations to house vessels with small displacements, tugboats, and workboats. They are also available in different shapes: D fender, rectangular fender, cylindrical fender, and solid fender. The fixing details of these fenders are specifically designed to accommodate the fender shape, size, and mounting arrangements. These fenders are manufactured with different rubber compounds, namely, ethylene propylene diene monomer (EPDM) and styrene-butadiene rubber (SBR). These rubbers have high resistance to ozone degradation, UV radiation, and waterborne oil pollution. As the fender height and length can be matched to any application, they have a wide range of applications. They are deployed most commonly in cargo and fishing ports. Figure 4.78 shows an extruded fender.

FIGURE 4.78 Extruded fender.

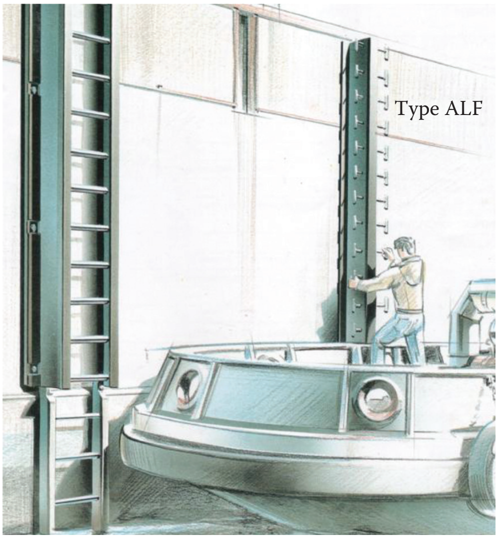


FIGURE 4.79 Ladder fender.

4.26.7 Ladder Fenders

They integrate the function of a ladder and a fender in a single unit. They are extremely robust and corrosion proof but remain flexible to provide protection to both the vessel and the wharfs. They are very useful to inspect the vessel damage and the wharf maintenance systems as they provide accessibility through the ladder. These fenders are composed of rubber-enhanced rungs that are fixed between the chains and enclosed in an elastomer. Figure 4.79 shows a schematic view of a ladder fender.

5 Structural Health Monitoring

5.1 INTRODUCTION

Ocean structures are assigned with special operations that encompass all tasks associated with coastal protection, docking a vessel in a dry dock, berthing of vessels of commercial and naval defense systems, housing marine police stations for coast guard, drilling, exploration, production platforms, and so on. Safe upkeep of these structures ensures the stability of the structure throughout their service life. Four activities that are vital to maintain ocean structures are (1) condition assessment, (2) maintenance, (3) control inspection, and (4) safe operations. Preventive maintenance of such structures is vital as they are of strategic importance. Repair and rehabilitation of ocean structures requires specialized equipment, construction chemicals, state-of-the-art electronic systems to map the existing underwater conditions, and electronic surveillance including hydrographic survey equipment, side scan, sonar imaging, underwater videography or photography, and marine borer assessment. The repair and rehabilitation of ocean structures is therefore a multidisciplinary task, which needs to be carried out with a lot of research ideologies and construction expertise. Data on structural life assessment and failure analyses of ocean structures highlighting the special issues on research interest and construction expertise are discussed in this chapter. Detailed methodologies of structural assessment and repair including the selection of a variety of chemical admixtures used and the construction techniques adopted for the repair are also discussed.

5.2 CONDITION AND DAMAGE ASSESSMENT

Condition and damage assessment is an interdisciplinary research. It demands a clear understanding of the *constructed* and *manufactured* engineering systems. Several factors that influence the damage assessment are size, cost, variations in material properties under different exposure conditions, uncertainties in system identification, and assessing conditions of the system under operations (Aktan et al., 1998a,b; Coppolino and Rubin, 1980; Katbas and Aktan, 2002). As ocean structures cannot be intervened frequently for repairs, preventive maintenance becomes vital to maintain such structural facilities. Factors that influence repair and rehabilitation of marine structures are (1) condition assessment, (2) maintenance, (3) control inspection, and (4) identification of safe operation limits (Chandrasekaran, 2013a,b,c; Chandrasekaran and Saha, 2011). Preventive maintenance of such structures is vital as they are of strategic importance, which precludes a thorough condition assessment of the structure through periodic inspection (Chandrasekaran and Parameswara Pandian, 2011; Chandrasekaran et al., 2011c; Saravanan et al., 2011). Structural intervention for repair (or even for a periodic inspection) is governed by the factors listed as follows (Chandrasekaran et al., 2010a,b,c):

1. Ocean structures remain in service during repair as the shutdown is unacceptable due to emergency demands that may arise any time.
2. Repair methods proposed should be cost-effective and impart long-term solutions as these should not be intervened at frequent intervals.
3. Repair procedures are generally requested only on an emergency. Therefore, immediate corrective measures should be invoked. However, repair procedures should be capable of substituting the structure within a short period of downtime.
4. Detailed analytical studies and verifications cannot be carried out initially due to lack of available data with respect to material strength, extent of marine growth, and so on. This amounts to a decisive point of initiating repair procedures based purely on preliminary inspection or assessment.

All the earlier factors are very critical and are particularly applicable to the structural retrofit of ocean structures (Chandrasekaran et al., 2012; Chandrasekaran and Madhuri, 2012a,b). This should compel the competent authority (the client who is requesting structural repair; for example, a dock manager or a maintenance engineer) to heavily depend on the technical expertise of the experienced professionals (Chandrasekaran et al., 2013). Visual inspections that are used for routine condition assessment pose serious limitations, but recent facts show that they even lack desired reliability (Chandrasekaran and Roy, 2006; Fraldi et al., 2009; Chandrasekaran, 2007, 2008b,c). It is a common practice to carry out repairs on concrete structures when visible damage such as cracks, spalls, chemical deterioration, and corrosion are noticed. Conventional methods of repairs or even use of chemical admixtures without proper know-how may affect the strength and serviceability of ocean structures apart from the fact that such repairs are unduly expensive. It is very important to note that the repair of ocean structures are not cosmetic, which may arise from visible damage; cause for such damage needs to be examined. Unlike onshore structures, offshore and coastal structures are subjected to a critical combination of hydrodynamic loads and impact loads; material degradation, for example, due to corrosion should therefore be seen as an index of loss of strength. This can actually challenge the operability of the structure.

Inspections help to identify signs of damage such as cracks, spalls, and corrosion of rebar when they become visible, but correlating them to assess the condition of reliability and subsequently deciding on the method of repair are very difficult (Chandrasekaran et al., 2003a,b,c, 2010a,b,c; Fraldi et al., 2009). Understanding the causes of such damage can only lead to engineering solutions. Given that the construction industry is flooded with chemical admixtures for instantaneous repair and restoration, one is often confused about the necessity of detailed investigations before the repair methodology is decided. As the agencies that offer such immediate solutions also extend an attractive warranty, the competent authority is compelled to believe the solution provided and the extent of reliability of repair (Chandrasekaran et al., 2003a,b,c). This procedure may be an advantage because it provides instantaneous solutions to the perceived problem without expensive expert opinion, which may be suitable for structures of nonstrategic importance. However, it is not suitable for ocean structures because both strength and durability are required to be addressed without compromise.

5.3 DETERMINING DAMAGE INDICES

Global condition assessment of the existing structure is necessary before proposing a repair methodology. Many practicing engineers consider damage as a change in the material properties within the structure (Chandrasekaran et al., 2005a,b, 2006a,b). Nondestructive testing (NDT) also helps in successfully characterizing the *in situ* properties of materials, which are required for a detailed analysis later. Although NDT is useful in condition evaluation locally, global damage indices should include the damage parameters that influence the overall strength and durability of the structure (excessive deflection, crack propagation pattern, probable in-elastic deformation, etc.). It is very important to note that structural assessment is not carried out to extend the service life of the ocean structure but to certify the usability of the structure under the deteriorated conditions. But unfortunately, maintenance engineers feel relieved of overloading of the ocean structures based on the results of the structural assessment. It is vital to realize that such results can also be erroneous due to serious limitations in the mathematical models used to predict the condition assessment. There are a few popular methods: (1) heuristic method, (2) modal analysis method, (3) numerical analysis method, and (4) geometric analysis method, which are widely used to define the structural damage indices, at various stages of structural assessment (Chandrasekaran et al., 2008a, 2010a,b,c; Giorgio et al., 2008a,b,c,d; Mazars, 1986; Zhang and Aktan, 1995). The *heuristic method*, which is preliminary and based on visual signs and inspection reports, can be used to derive the structural condition as a primary index. This is similar to a first information report (FIR) of a crime investigation; comparisons are made to justify the responsibility of the engineer-in-charge to carry out visual inspection with utmost care. *Modal analysis* is based on the detailed mathematical modeling and dynamic analysis. Sectional properties such as stiffness and modified modulus of elasticity are critical inputs of the analysis. In the case of push-over analysis, which determines the capacity of the structure under deteriorated conditions, constituent properties of reinforced concrete material is necessary to define the rotation capacities of plastic hinges in compression and tension of beam-column joints. Mode shapes will indicate the participation of higher modes (whether torsional or translational) under the damaged state of the structure. This stage of damage assessment is rather difficult in ocean structures as the *in situ* properties of the materials are difficult to ascertain even with modern NDT methods. Marine growth causes a serious setback during such testing. *Numerical analysis* is an alternate method to the former, which is primarily focused on estimating the stiffness coefficients of structural members. Stiffness properties are derived from nonstationary parameters. Equal levels of difficulty also exist in this method of structural assessment, which can lead to serious errors if not done carefully. *Geometric analyses* include determination of modal flexibility matrix on the basis of damage indices obtained from the influence coefficients, which are determined by experimental investigations. Influence coefficients are displacement responses of the structural members (at nodes) for a predefined (unit) force. Structural joints are simulated under the deteriorated material properties.

Experimental investigations are carried to estimate the influence coefficients under simulated test conditions. This method of condition assessment is rather time consuming and computationally expensive.

It is therefore necessary to integrate a spectrum of experimental investigations and indices that are derived from the methods described earlier to monitor a structural facility over a long period of time (Chandrasekaran and Gupta, 2007a,b; Chandrasekaran and Kumar, 2007; Chandrasekaran and Srivastava, 2007; Chandrasekaran et al., 2007a,d,e, 2011b). In fact, in ocean structures, which are strategically important, it is mandatory in many developed countries such as those of the European Union to deploy a structural health monitoring (SHM) scheme during the construction stage itself. In the absence of a dedicated preinstalled SHM scheme, it is also equally important to visit the constructed facility at some stage of its life cycle to evaluate or to decide the evaluation of vital damage indices, which are nonrealistic. A well-coordinated and well-structured integration of experiments, analysis, and information technologies in the context of structural assessment is critical (Chang, 1997, 1999; Fraldi et al., 2009; Chandrasekaran et al., 2010d; Bhattacharyya et al., 2010).

5.4 NONDESTRUCTIVE TESTING

Noninvasive techniques are used to determine the integrity of materials, components, or structure, or to quantitatively measure some characteristics of objects. NDT is therefore a domain of experimental investigations, which refers to inspect or measure without doing harm to the structure. NDT is used to inspect pipelines to prevent leaks that could damage the environment. Visual inspection, radiography, and electromagnetic testing are some of the NDT methods used. Nondestructive evaluation (NDE) is useful in many ways: (1) flaw detection and evaluation, (2) leak detection, (3) location determination, (4) dimensional measurements, (5) structure and microstructure characterization, (6) estimation of mechanical and physical properties, and (7) stress (or strain) and dynamic response measurements. NDE are used on certain conditions: (1) to assist in product development, (2) to screen or sort incoming materials, (3) to monitor, improve, or control manufacturing processes, (4) to verify proper processing such as heat treating, (5) to verify proper assembly or workmanship, and (6) to inspect for in-service damage. Most common NDE methods are visual, liquid penetration, magnetic, ultrasonic, eddy current, and X-ray diffraction.

5.4.1 Visual Inspection

Visual inspection is carried out using fiberscopes, borescopes, magnifying glasses, and mirrors. Robotic crawlers are also used in cases where the accessibility is limited. In the case of inspection of large tankers storing hazardous chemicals, air ducts, nuclear reactors, and pipelines, they are very useful and safe. Figure 5.1 shows a typical storage tank inspected using a robotic crawler.

5.4.2 Liquid Penetration Test

In a liquid penetration test, liquid with high surface-wetting characteristics is applied to the surface of the defected or damaged member. Liquid is allowed to seep into

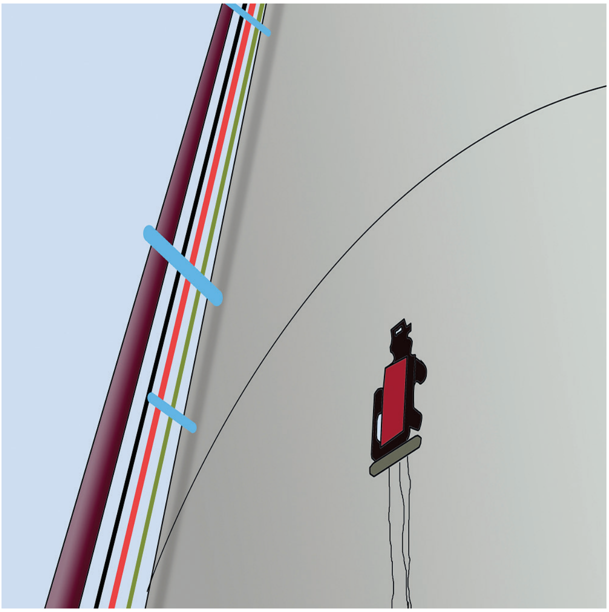


FIGURE 5.1 Inspection using a robotic crawler.

surface breaking defects. Excess liquid is wiped off before applying a developer; this is a powder, which is capable of pulling the penetrated liquid out of the defect and spreading it on the surface where it can be seen. The liquid penetrant, which is used is loaded with a fluorescent dye, and the inspection is done under ultraviolet light to increase test sensitivity. Figure 5.2 shows a typical damaged member, inspected using the liquid penetration test. Cameras fitted on long articulating arms are useful for inspecting underground storage tanks for damage.

FIGURE 5.2 Liquid penetration test.

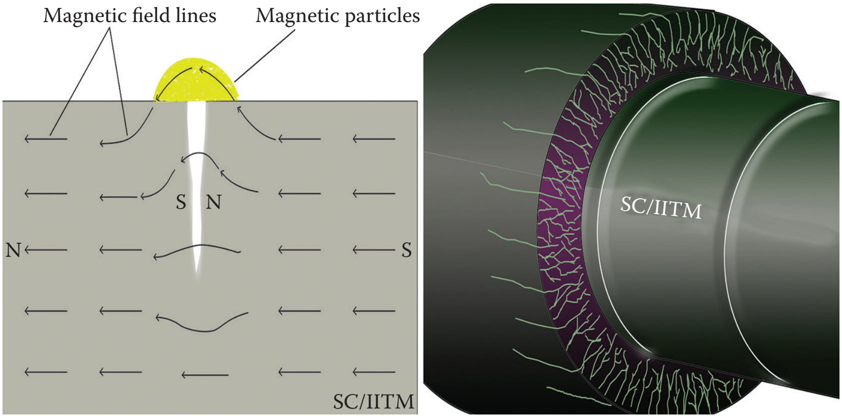


FIGURE 5.3 Magnetic particle test.

5.4.3 Magnetic Particle Inspection

In this method, damaged part of the member is magnetized. Finely milled iron particles, coated with a dye pigment are applied to the specimen. These particles are attracted to magnetic flux leakage fields. They will cluster to form an indication directly over the discontinuity. This indication can be visually detected under proper lighting conditions. Figure 5.3 shows the test method, as applied on a drilling stack union.

5.4.4 Radiography

This method uses radiation energy, which has a shorter wavelength in comparison to the electromagnetic waves that emit light. The source for the radiation should be either an X-ray generator or any radioactive source. Alternatively, film radiography is commonly used. In this technique, the damaged part of the member or specimen is placed in between the radiation source and a piece of film. Because of the physical intervention caused by the specimen, some of the radiation will be blocked; it is obvious to understand that the thicker and denser area will block more of the radiation. The film darkness will vary with the amount of radiation reaching the film through the specimen. Based on the traces of darker areas, the extent of damage can be estimated. Figure 5.4 shows a typical test procedure. Failure of pressure vessels can result in rapid release of a large amount of energy. To protect against this dangerous event, tanks are inspected using radiography and ultrasonic testing.

5.4.5 Eddy Current Testing

Eddy current testing is also another important method by which NDE is carried out. Figure 5.5 shows a schematic view of the eddy current testing procedure. The coil generates a magnetic field when the flux is created, and the conductive material is placed in that part. Eddy currents are developed, as shown in green.

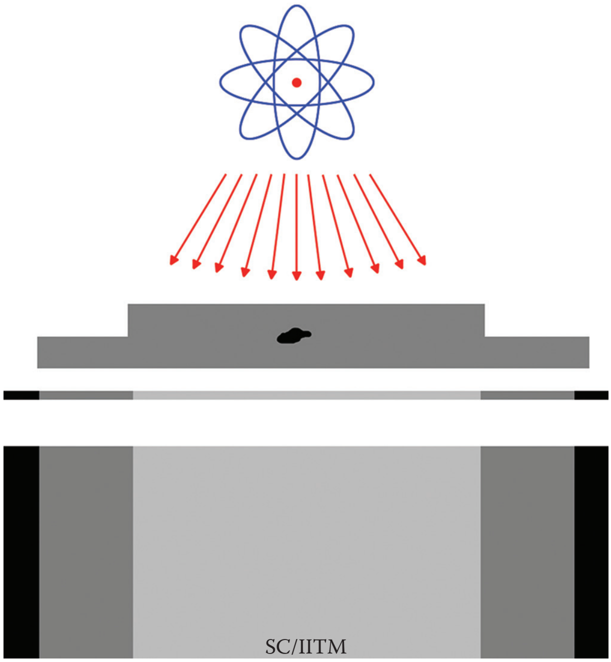


FIGURE 5.4 Film radiography.

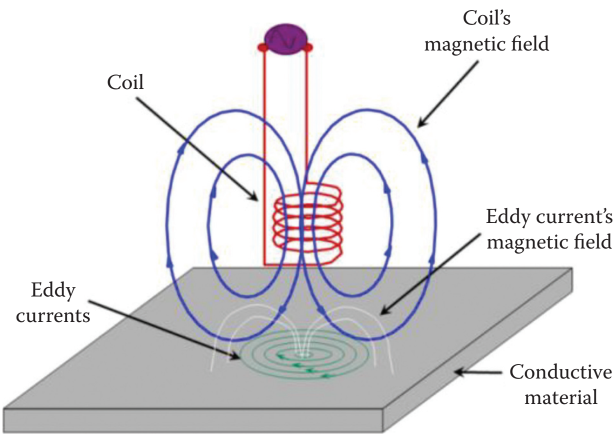


FIGURE 5.5 Eddy current testing.

Based on the intensity of the eddy current, a magnetic field is generated, which is essentially used for NDE. This method is useful in crack detection, in measuring material thickness and coatings, and so on. They are very useful to detect surface defects, whose inspection gives results instantaneously. As only conductive materials can be inspected, this is one of the serious limitations. This is useful to examine

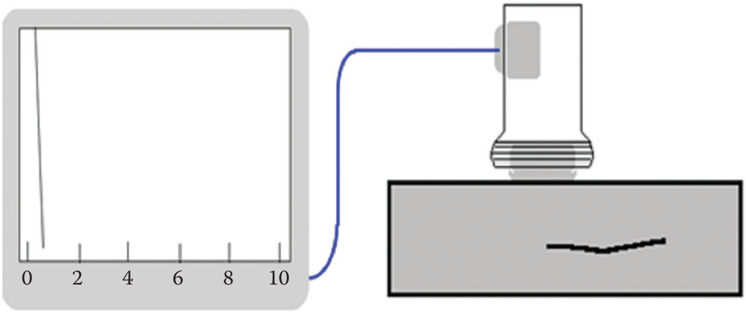


FIGURE 5.6 Ultrasonic inspection.

the corrosion coating thickness and the thickness of sacrificial anode in corrosion protection. Laminar layers, painted surfaces, and members with other surface irregularities cannot be evaluated using this method.

5.4.6 Ultrasonic Inspection (Pulse-Echo)

One of the common methods of NDE is ultrasonic inspection, which is also termed as pulse-echo method. Figure 5.6 shows a schematic view of the ultrasonic inspection method. High-frequency sound waves are introduced into the defected material. They reflect back from the surface or the flaws. The time it takes for the path of the light or the ultrasound waves to hit the material and reflect back accounts for the degree of defect present in the material. Reflected waves can be traced to determine the crack-echo location. Once located, one can also find the depth of the crack, based on the energy level, shown in the plot. As offshore structures are mostly fabricated with steel, inspection follows secondary processing to assess machining, welding, grinding of the welded connections, heat treating, electroplating thickness and so on. Inspection is also carried out to assess in-service damage caused to the structural members: cracking, corrosion, heat damage, and so on.

5.5 NDT FOR UNDERWATER INSPECTION

Inspection of offshore structures needs NDT to be carried out underwater. Materials mostly include steel, concrete, and wood. Problems that are to be identified include crack and other growing defects, which are formed on the material. Wall thinning, which results from corrosion and biological and chemical changes, needs also to be diagnosed. Damage caused by collision of ships and tugboats needs also to be identified. Most of the severe damage to offshore structures happen underwater and therefore becomes difficult to trace by conventional NDT methods. The primary objective of underwater inspection is to ascertain crack prorogation and localized corrosion. Cracks mostly occur at welded zones or other zones that have a high stress concentration. Tubular joints near the splash zone are candidates of such problems.

The main objectives are (1) detection of surface-opening cracks in welded tubular joints and (2) detection of wall thickness in tubular members. As thickness can always get compensated because of corrosive nature of sea water, latter is more important. Checking corrosion systems that have been deployed for corrosion protection measures, can also to be examined using NDT. Mapping of marine growth, scour depth, and debris is an additional advantage of deploying NDE for offshore structures underwater.

5.6 OBJECTIVES OF UNDERWATER INSPECTION

Concrete structures are to be examined for deterioration with age in marine environment. Early detection of surface-opening cracks in the zones of high-bending moment plays a very important role as this initiates corrosion in reinforcing bars. Corrosion of reinforcement anchors in prestressing tendons and other members of steel structures should also be necessarily identified as this can result in loss of mechanical strength with an increase in age. Checking the foundation on the seabed is necessary to ascertain the stability and position restraint of offshore platforms and large floating and production vessels. Main objectives of underwater inspection are (1) detection of surface-opening cracks in welded tubular joints; (2) detection of decrease in wall thickness in tubular members due to corrosion; (3) checking corrosion protection systems; (4) mapping of marine growth, scour, and debris; (5) detection of surface-opening cracks in areas of high bending moment; (6) detection of erosion of concrete in splash zone; (7) corrosion of reinforcement and anchors in prestressing tendons; (8) checking for foundation on seabed for its integrity; and (9) detection of cracks in structural steel members under impact loads and increased stress concentration. Underwater inspection is very useful in cases of pipeline risers. If pipeline risers are damaged underwater, they become unnoticeable for long term. It is also an established fact that pipelines underwater are highly susceptible for damage due to many reasons: (1) excessive deflection caused by seabed scouring, (2) high internal pressure, (3) impact loads caused by vessels and seabed movements, and (4) joint failure. Other reasons could arise due to the following factors: (1) thermal loads; (2) unforeseen environmental loads; (3) relative motion between the platform and the pipeline, in cases of structures with fixed bottom; (4) corrosion growth in fixtures; and (5) marine growth and debris. Several methods, as discussed earlier are useful in underwater inspection. For steel structures, inspection tests based on magnetic particles and eddy currents are useful. For concrete and wooden structures, besides other tests such as rebound hammer, and penetration techniques, it is common to remove a core for detailed examination. Vibration analysis is also used, which has a main advantage: defects anywhere in the structure can be identified. Once identified with location-specific details, detailed examination is carried out subsequently by other NDT methods. Underwater NDT has a serious limitation. As the material surface is obscured by marine organisms and corrosion, it becomes mandatory to clean the surface thoroughly before NDT is performed. This may not be possible for members underwater, without which NDT methods cannot be deployed.

5.6.1 Inspection Methods and Limitations

Visual inspection underwater is carried out by deploying experienced divers. Limitations exist on the reliability of the data reported and their experience to quantify the seriousness of damage caused to the members. Alternatively, remote operated underwater camera and videography are also used, but they are prohibitively expensive. Poor visibility, heavy biofouling, and presence of strong currents make inspection difficult for the divers.

5.6.2 Magnetic Particle Inspection

The primary objective of the method is to detect the fine surface cracks in ferromagnetic materials. Invisible cracks cause major structural integrity problems. The area under inspection should be magnetized by applying liquid suspension of ferromagnetic particles. In the presence of fine surface cracks, these particles will deposit along the crack due to the leakage of magnetic flux at the discontinuity of the material. They are then detected by passing ultraviolet rays. Ultraviolet rays will produce a good contrast between the particle gathering along the crack and the dark surroundings. Sometimes these particles are mixed with the fluorescent agents, making the inference doubtful. Instruments are available for conducting underwater inspection using magnetic particle inspection (MPI) up to a depth of about 120 m (see, for example, the instrument developed by Det Norske Veritas [DNV]). Figure 5.7

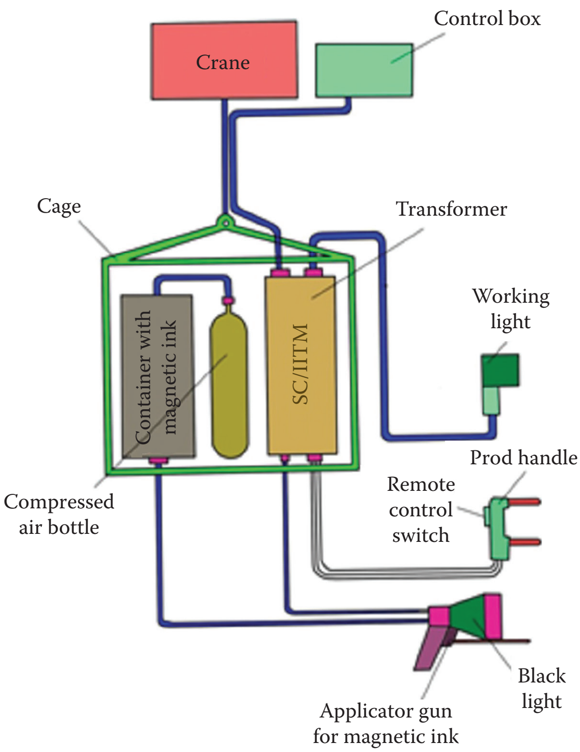


FIGURE 5.7 Instrument used for magnetic particle inspection underwater.

shows a schematic view of the instrument to conduct MPI underwater. By applying a high-amperage alternating current between the two electrodes on the damaged surface, the necessary magnetic flux is obtained. The current is drawn from a transformer, which is located in a waterproof cage that is lowered into the water during the test; the cage also contains the tank for magnetic particle suspension. The tank is fitted with baffles driven by an electric motor to prevent settlement of ferromagnetic particles during operation.

The device shown in Figure 5.7 has different components. Although the crane, which is located above water, is useful to immerse the device during the test, the control box is useful to regulate the measurements taken during conduct of the experiment. A minitransformer is used to supply power for conducting the test. A container supplies the magnetic ink during the test, which is sprayed using the compressed air stored in the air bottle. A prod handle is actually used for detection, which is integrated with the applicator gun for supplying magnetic ink. A remote switch controls the entire operation of the device, and black light is used to improve visibility during the test. The whole device is housed in a watertight, steel cage for protection. Once the gun sprays the magnetic ink, on which the fluorescent material is sprayed, defects can be easily located on the surface of the material (or member).

5.6.3 Ultrasonic Testing for Underwater Inspection

This method of inspection employs high-frequency mechanical stress waves whose operational frequencies are greater than sound. This can be carried out by two methods. The first method is a transducer-based method where the sender and receiver of pulse signals of ultrasonic waves are interpreted to assess the defect. This is commonly used to assess defects on metal surfaces. The second method deploys two transducers by placing them side-by-side and enabling transmission measurements through them. This method is common for concrete and wooden members. The latter method is commonly used to measure thickness of ship hulls using pulse-echo recording but is not effective to measure defects that arise from corroded plates. This is because the pitted surface due to corrosion shows multiple reflections, which may lead to wrong interpretations.

5.7 STRUCTURAL HEALTH MONITORING

Sensing and assessing in marine environment are done for several reasons: (1) to assess the safety of oil exploration and related applications, (2) for global weather predictions, (3) to monitor water quality resulting from oil pollution, (4) to measure parameters detrimental to the health of offshore structures in sea, and (5) for military operations. Structural health monitoring (SHM) is connected with the reduction of ownership costs to increase operational lifetime and to improve safety and operability of the platforms. Aging of ocean structures poses a significant hazard and demands early damage detection systems in place to avoid any catastrophic failure. SHM of ocean structures is more challenging and relatively a new attempt in the domain of ocean engineering.

5.7.1 Specific Objectives

Structural health of ocean structures needs to be monitored through the development of an integrated network of sensors for monitoring and assessing them under the prevailing environmental conditions. The main focus is to enhance their safety and integrity, whereas the other aspect is to develop a knowledge-based and decision-making system for the chosen parameters that are related to their safety and integrity. Analytical studies should be conducted to estimate the vital parameters that can challenge safe operability and even survivability of ocean structures. One of the most attractive and important factors is that they operate in a hostile environment; either no preventive and periodic maintenance is planned ahead. Experimental studies should be conducted on various scaled models to validate the analytical results. Such applications are scarce in the literature. The development of a knowledge-based and decision-making system will suffice as quick remedial solutions in cases of any unforeseen emergency. Raising alert messages, highlighting areas that need immediate technical intervention, and identification of critical zones of damaged members are effective outcome of an automated artificial intelligence system for monitoring and assessment of structural health on offshore structures and underwater marine vehicles.

The following steps are followed in SHM of ocean structures:

- Develop a sensor network to carry out experimental measurements on scaled models of marine structures using the developed network.
- Develop an appropriate knowledge-based and decision-making system for monitoring and assessment to provide remedial solutions with minimum human intervention.
- Develop a sensor network system for real-time data logging response of the structure to ambient loading.
- Arrive at values of damage parameters from dynamic response analysis obtained by analytical methods and validated with experiments on scaled models, so that they can be used to predict the state of health of the structure.
- Develop a central control console with appropriate data analysis programs and graphical user interfaces to monitor and record the dynamic response parameters of the offshore structure continuously.
- Develop damage prediction algorithms that can initiate maintenance and rescue operations based on alarms generated against predecided set point levels of damage parameters.
- Integrate wireless sensor networking (WSN), a damage prediction algorithm, and an appropriate graphic user interface (GUI) to develop a supervisory control and data acquisition (SCADA) type remote real-time monitoring and control station.

The field of monitoring and assessment (M&A), for example, structural health monitoring is concerned with accurately and reliably assessing the safety and integrity of a given structure. It is very important and beneficial to ocean structures as they are expected to withstand cyclic wave loads, severe storms, sea quakes, and corrosive effects of sea water. Furthermore, the process of visually inspecting marine structures has a limited scope technically and economically due to poor visibility, concealment of damage by marine growth, prohibitive cost, nonavailability of properly trained divers, and dependence on weather conditions.

Structural health monitoring is defined as the process of assessing the health state of the structure and determines the need for remedial action as a part of preventive maintenance. It is aided by statistical analysis of variables that are damage-sensitive. Hardware components of the SHM system include sensors and the associated instruments, whereas software includes damage detection algorithm. The NDT techniques lack the ability to address all the needs for health monitoring and performance evaluation of structural systems. It is therefore imperative to use the benefit of advances in sensing technology and data processing algorithms for providing an intelligent solution, which is SHM. The main concept in SHM is to estimate the state of a structural system under the encountered loads through response measurements.

5.8 MEMS DEVICES

Existing monitoring systems use traditional wired sensors technologies and several other devices that are time consuming to install and relatively expensive. Typically, a large number of sensors are integrated through long cables, which pose serious limitations in terms of data loss during transmission, white noise acquisition, poor ordeal, and groping capabilities. However, a wireless monitoring system with micro electro mechanical system (MEMS) sensors can reduce such problems, apart from reducing the cost of networking, if deployed on a large scale. MEMS are small, integrated devices or systems combining electrical and mechanical components. MEMS accelerometers and pressure sensors are quite popular in measuring vibration and loads on structures. MEMS technologies are well suited to improve the performance, size, and cost of sensing systems. MEMS devices may be attached to ships, floating platforms, and fixed-type ocean structures. In addition, MEMS sensors also have successful usage in the field of oil exploration and to detect oil leakage from pipelines. In cases of oil spills, MEMS sensors can help to sense information about ocean currents. It is also possible to predict the oil slick transportation, which can aid the cleaning operation.

MEMS sensors are used in the process of exploring potential oil and gas reserves. MEMS geophones and accelerometers can sense the vibrations sent up from earth's belly. An array of MEMS geophones are installed over a wide area on the seabed. Vibrations are intentionally produced on the ground surface using some techniques. MEMS devices measure the reflection of these waves from different layers in the earth's belly. These readings are then used to create a geological map. This indicates the size and location of the oil or gas reservoir. An array of MEMS sensors, spread on the sea floor, could detect the submarines. MEMS sensors are also used to locate antitorpedo weapons on ships and submarines. MEMS sensors in torpedoes are responsible for detonating the torpedo at night, hitting the target in a crowded environment. Using MEMS sensors poses a few challenges that arise mainly from the complex nature of marine environment. In addition, fouling of sensor surfaces, selection of an appropriate device for a wide range of application, inability to detect extremely low level of chemical concentrations, and inability to resist drifting along with the current will add more limitations to the MEMS sensors in ocean environment.

MEMS sensors proved to offer good advantages due to their simple yet efficient design. They need a very small current to operate, as they function in an open-loop configuration. MEMS sensors are ideal for wireless long-term monitoring, as they offer portability, reliability, and durability. They have a higher bandwidth and have an excellent response at higher frequencies. Miniaturization extends applications to novel devices, which reduces cost by decreasing material consumption.

5.8.1 Challenges in Using MEMS Sensors

Use of MEMS sensors for health monitoring is put to face certain serious challenges:

- *Reliability*: Micron-scale structural thin films are susceptible to premature failure at stress amplitudes as low as half the fracture strength.
- *Residual stresses* are dependent on thin film material properties and fabrication techniques. This affects the fatigue life of the sensor.

MEMS sensors are not limited to recording high noise at ground levels, but in the lower end of ground noise (most accelerometers have 200 μg at their lower end). Limited to 140 dB as in most of the standard cases, the full range of ground motion is from 0 to 200 dB. Magnitude of noise power spectral density (PSD) increases as frequency increases, and the measurement seems to be more accurate in 0 g than in ±1 g.

5.9 SHM System Architecture

A typical SHM architecture, proposed for health monitoring of offshore platform is discussed. Figure 5.8 shows the architecture using radio frequency (RF) communication. The main component of the system architecture is the plant unit, which is composed of sensor network topology, a data acquisition system, and RF communication devices. The RF communication channel is effective only up to a

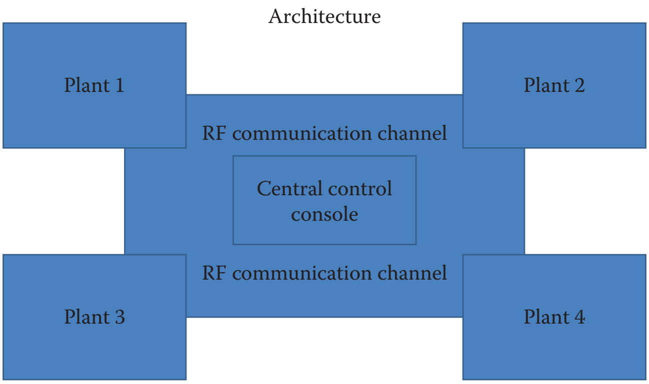


FIGURE 5.8 Block diagram for SHM architecture using an RF channel.

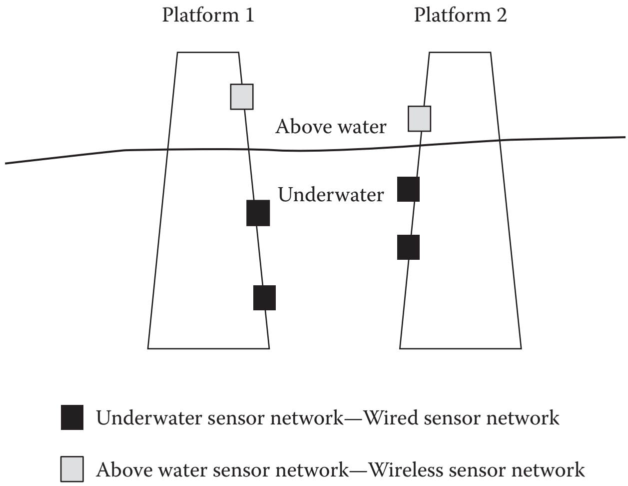


FIGURE 5.9 Schematic diagram of a sensor arrangement.

limited distance, while the satellite transmission through the Internet and intranet services can be adopted for larger distances of communication. The described architecture demands data encryption for secured transmission. The other component in the architecture is the central control console. It is capable of analyzing the data set, which is then tuned to report and alarm generation. It also acts as the data storage for the entire system. Although the sensor network below water is wired, a wireless system is shown for above water. Figure 5.9 shows the sensor arrangement, whereas Figure 5.10 shows the graphical representation of sensor network architecture.

5.10 DEVELOPMENT OF WIRELESS SENSOR NETWORKING

Various stages involved in the development of WSN are discussed as follows:

Stage 1: The basic topology of the WSN is to be established with sensors linked to a sink and a base station. An interface program has to be developed for data recording and display of the data. The SHM systems should be tested by deploying the scaled down experimental model in the test setup.

Stage 2: For experimental investigations, scaled models of offshore platforms are to be fabricated. Sufficient units of sensor network are to be installed at appropriate measurement stations on the deck. Sensors should be connected in the mesh topology with each sensor node having 8–16 sensors. The mesh consists of interconnected sensors, interconnected sinks (routers), and a single gateway or base station (with or without a connected computer). The base station or computer is connected to the intranet, which acts as the central control console. Data backup after screening unwanted data should be done at the control console. Report generation is also a part of this base

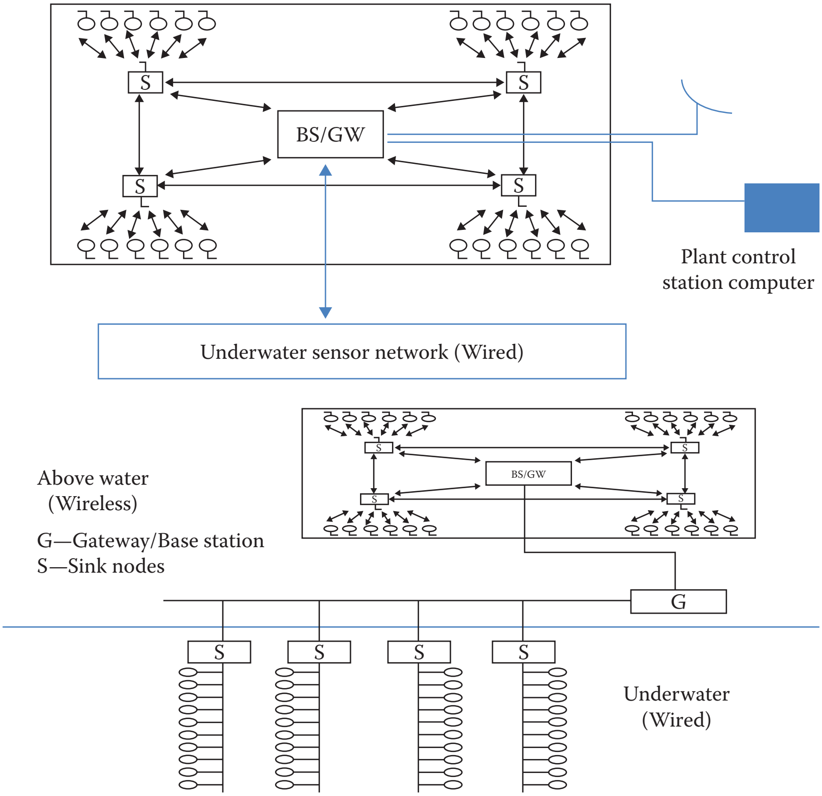


FIGURE 5.10 Sensor network architecture.

station. Based on the comparison of damaged and undamaged parameters of the platform, the system should be able to generate alarm and alert messages that invoke attention for immediate repair or evacuation.

5.10.1 Setting Up a Sensor Network

A hierarchy divides the sensor network into the following three levels: Figure 5.11 shows a schematic view of the sensor network.

- *Sensor interface nodes*—These are the first level devices, which collect the response from the offshore platform. These nodes are generally integrated using RF communications, and the unit is augmented with the necessary data acquisition system. Green dots in Figure 5.11 show the sensor interface nodes.
- *Data sinks*—Collect data from various sensor nodes and send them to the gateway. Blue dots indicate the data sinks.

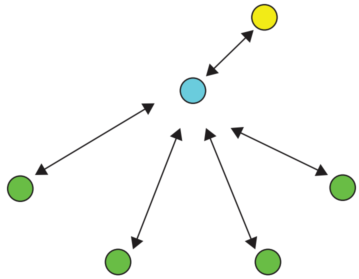


FIGURE 5.11 Sensor network.

- *Data aggregator gateways*—Collect data from sinks and transmits to the central control station by a WAP-based Internet connection, or it is plugged into a plant computer, and then transmitted to central control over the Internet. The yellow dot indicates gateways.

The complete sensor network of wireless and wired networks includes both mesh and star topology in a hybrid hierarchical model.

5.11 WIRELESS SENSOR NETWORKS WITH WASPMOTE AND MESHLIUM

Waspmote Development Kits (from Libelium) can be used as wireless sensor nodes. Libelium also supports the development of mote from a component level using Waspmote and Squidbee devices. This will aid in making custom-made mote applications for specific requirements. Waspmote may be arranged in a star topology with one mote acting as a base station. The sensor interface to the mote may need an additional signal processing requirement of a half bridge, and the full bridge may prompt additional interface circuitry. Similarly, accelerometers may also require additional interface circuitry. The feasibility of resistance to digital converters instead of conventional analog digital converters (ADCs) can be used in mote development. Figure 5.12 shows the development kit along with the components.

5.11.1 Fabrication Materials

The choice of materials for fabrication is not based on electrical aspects, but on the mechanical aspects such as internal stress, processing temperature, and compatibility with other material. Silicon and its substrates (SiN_3 and SiO_2) are primary choices, because of their excellent mechanical strength, stress control, and larger elasticity limit. Various other materials such as polymers, glass, and quartz crystals

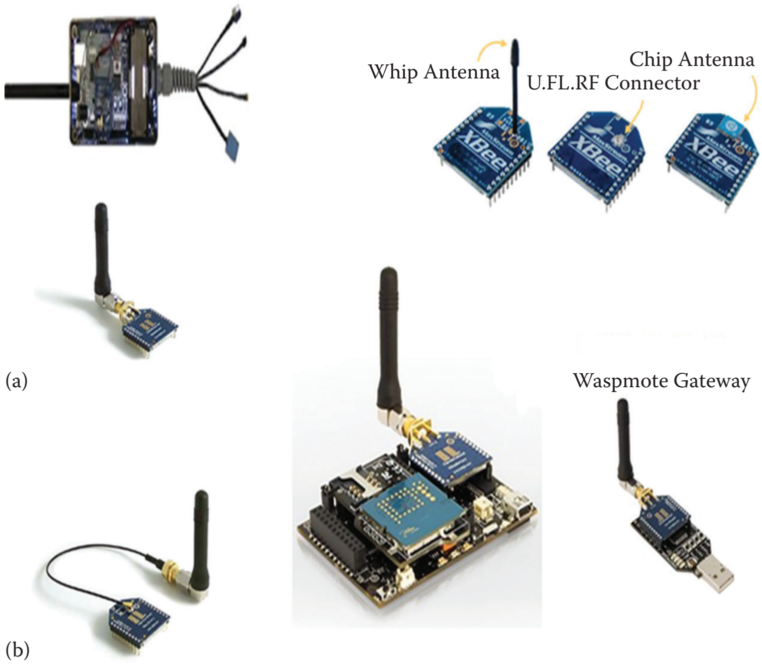


FIGURE 5.12 Waspmote Development Kits: (a) SMA antenna connector; (b) UFL antenna connector.

are also used as alternatives. Common fabrication techniques are bulk micromachining, surface micromachining, and deep reactive ion etching (DRIE).

5.12 WASPMOTE MESHLIUM ARRANGEMENTS

Figure 5.13 shows one of the proposed Waspmote Meshlium arrangements for WSN of offshore structures.

As illustrated in the figure, Waspmotes are connected to the central console. The transmission is through 802.15.4 ZigBee protocol. Two different storage operations are performed with the captured frames. First, acquired data are stored in the local storage system in the central console; second, it is transported through Ethernet, Wi-Fi, or 3G/GPRS interfaces, and stored in the cloud. The data are stored in the external database.

5.12.1 Waspmote Configurations

Waspmote is based on a modular architecture. It is possible to integrate only the modules needed in each device, although these modules can even be expanded, if required. Eight digital pins present in the node can be configured as input or output pins. Each module has an in-built accelerometer, with a 2 or 6 g range of 12-bit resolution; it also includes a temperature sensor. The transmitting unit is the 802.15.4

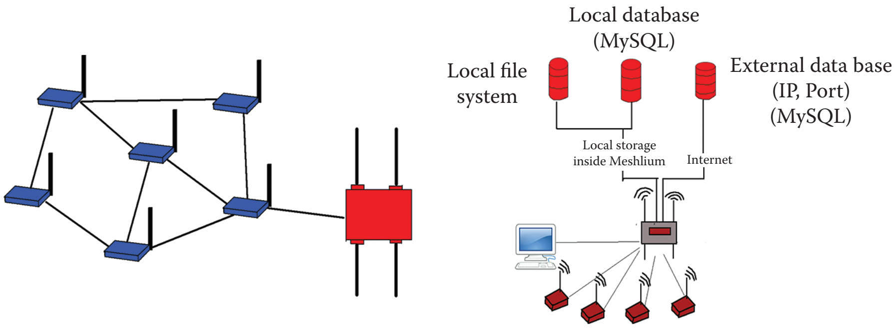


FIGURE 5.13 Waspmote Meshlium arrangement.

ZigBee protocol, which has an application programming interface (API) that enables the user to configure the device. Modules available for integration in Waspmote are categorized as follows:

- ZigBee/802.15.4 modules (2.4 GHz, 868 MHz, 900 MHz) (low and high power)
- GSM/GPRS module (Quadband: 850 MHz/900 MHz/1800 MHz/1900 MHz)
- GPS module
- Sensor modules (sensor boards)
- Storage module: SD memory card

A typical Waspmote configuration is given in the following table, whereas block diagram is shown in Figure 5.14. Table 5.1 shows the Meshlium configurations.

5.12.2 Waspmote Specifications

Microcontroller: ATmega1281
Frequency: 8 MHz
SRAM: 8 KB
EEPROM: 4 KB
FLASH: 128 KB
SD card: 2 GB
Weight: 20 g
Dimensions: 73.5 × 51 × 13 mm
Temperature range: −20°C, +65°

Meshlium includes management software to configure all required parameters through a graphic user interface (GUI). All communication interfaces, such as Wi-Fi, ZigBee, GPRS, and Bluetooth, should be used to manage storage of data, once received. The operating system is Linux, and the configuration software is an open source. Depending on the kind of XBee model, parameters to be configured may vary. The network ID, node ID, power level, encryption key, and MAC has to be configured based on the choice of interfaces. Once they receive the data from the

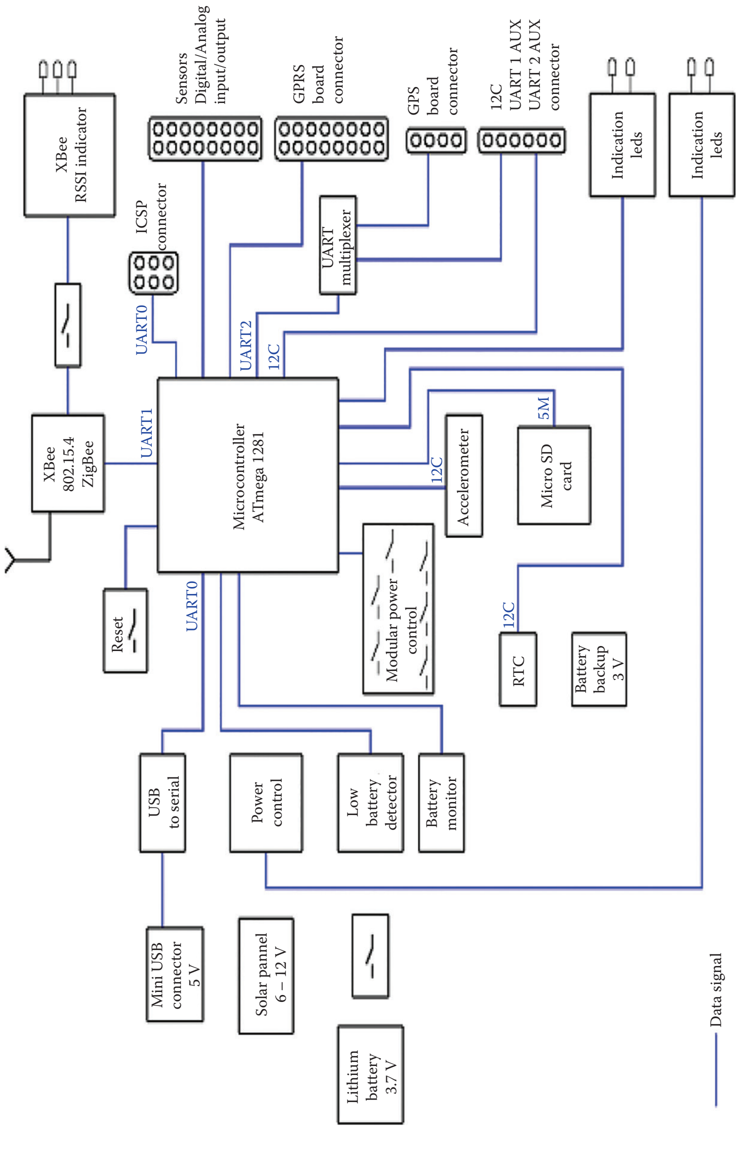


FIGURE 5.14 Block diagram for Waspmote.

TABLE 5.1
Meshlium Specifications

Description	Specification	Function	Range
Processor	500 MHz (*89)	Power	5W (18 V)-POE
RAM	256 MB (DDR)	Storage	8 GB/16 GB/32 GB
Power consumption	270–450 mA	Maximum power current	1.5 A
Temperature range	−20°C, +50°C	Enclosure	IP67
Time to respond/ping over Ethernet	60 s	Service start time	90 s
Weight	1.2 Kg	Dimensions	210*175*50 mm
Wi-Fi AP	802.11b/g, 20 dBm, 2.4 GHz, 500 m	Wi-Fi mesh	802.11a/b/g, 20 dBm, 2.4 GHz/5 GHz, 2 Km
Bluetooth	17 dBM, 100 m	Gsm/GPRS	Quad band: 850 MHz/900 MHz/1800 Mhz/1900 MHz (worldwide usage)
802.15.4/ZigBee/Rf	1–100 mW (2.4 GHz, 868 MHz, 900 MHz)	GPS	−159 dBm sensibility 1s hot start

network, data analysis is initiated at the user interface. The modern class of computers have computing units that have the capability of recording data 24/7 and integrating them with Waspmote configurations with Meshlium. As of today, WSNs remain an underutilized, nonstandardized developing technology. The current market penetration is less than 1%. The nature of WSN makes the incremental deployment as a very attractive possibility. More importantly, WSN promises to change the scope of IT, by enabling even passive infrastructure to intercommunicate.

5.13 CROSSBOW WSN

Crossbow technology is also an alternative for WSN. The hardware platform is the wireless mote, whereas the software platform includes mote network tier, server tier, and the client tier. The server tier is generally a gateway server and the client tier refers to the monitoring and management tools. WSN design generally depends on the types of sensors and nature of data to be acquired. Figure 5.15 shows a typical sensor networking using Crossbow. Figure 5.16 shows a schematic view of the hardware platform used in the network.

The wireless sensor network architecture has individual sinks and sensors connected. Sensors are arranged in a cluster and connected to various sinks and sources of different points. Subsequently, all the sensor nodes are connected through the gateway, which is further connected to the computer for data processing.

5.13.1 Mote

A sensor node, also known as a mote, is a node in a sensor network that is capable of gathering, processing, and communicating sensory information. It also has capability

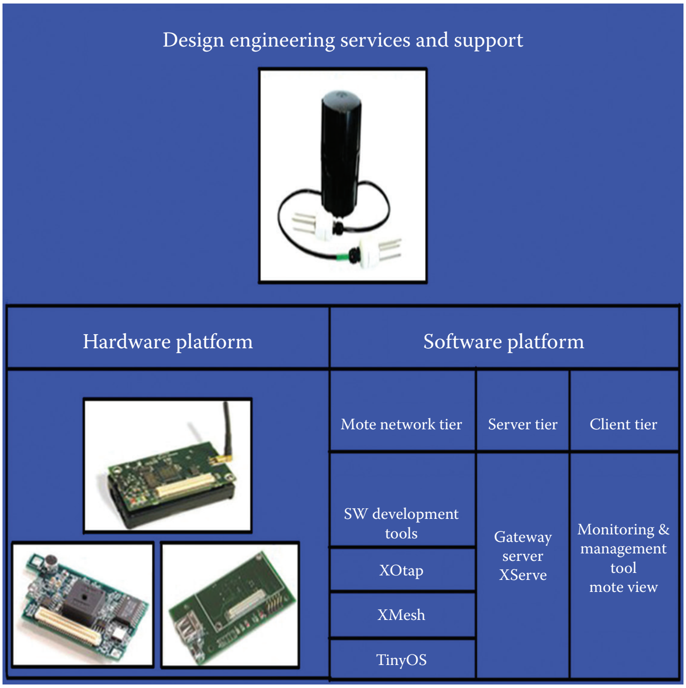


FIGURE 5.15 Wireless sensor networking: Crossbow.

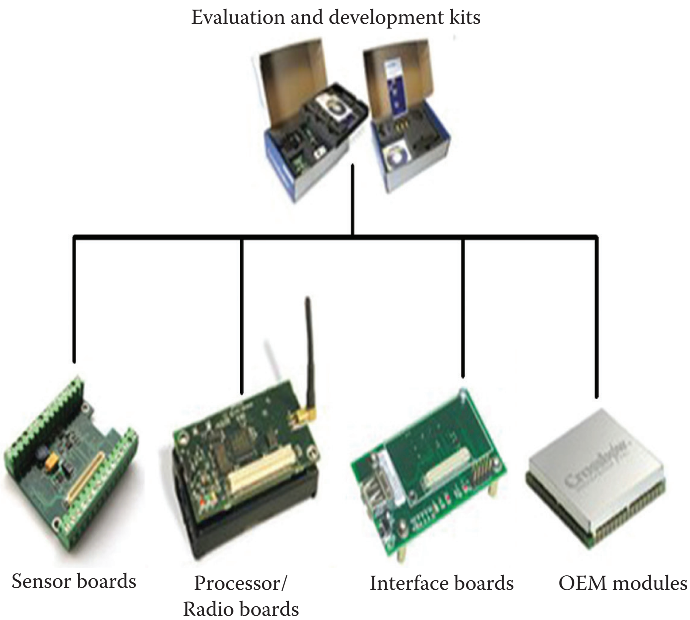


FIGURE 5.16 Hardware platform for WSN.

of communicating with other nodes connected in the network. A mote is a node, but a node is not always a mote. The typical configuration of a mote is as follows:

- Processor/radio board
- Frequency: 2.4 GHz or 916 MHz
- 2.4 GHz motes
- MICAz—MPR2400
- IRIS—XM2110CA
- 916 MHz motes
- MICA2—MPR400CB

Figure 5.17 shows the configuration layout of a mote, whereas a typical hardware component is shown in Figure 5.18. Table 5.2 shows the details of mote configuration, whereas Table 5.3 shows the summary of resource information for the wireless sensor configuration.

The MICAz is a 2.4 GHz mote module used for enabling low-power, wireless sensor networks. It enables the development of custom sensor applications and is specifically optimized for low-power, battery-operated networks. MICAz is based on the open-source TinyOS operating system and provides reliable, ad hoc mesh networking, over-the-air programming capabilities, crossdevelopment tools, server middleware for enterprise network integration, and client user interface for analysis and configuration.

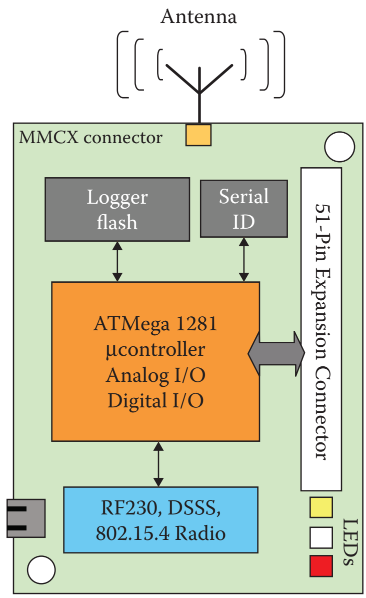


FIGURE 5.17 Configuration layout of an IRIS/MICAz mote.

FIGURE 5.18 Mote core hardware components.

TABLE 5.2
Mote Core Hardware Components

Resource	Value	Information
Program Memory (flash memory)	128K bytes	Stores the application code Programmed through an MIB base station or using OTAP
SRAM	4K bytes	Used to store user application parameters, XMesh variables and TinyOS variables. Also contains the stack
EEPROM	4K bytes	Used to store persistent values such as mote ID, radio frequency, and so on
Timer	4 Timers, two 8-bit, two 16-bit	The two 8-bit timers are used by TinyOS and the two 16-bit timers are available to the users
SPI Bus	1	Reserved exclusively for the radio interface and not available for user applications Used during reprogramming by the MIB units
I2C Bus	1	Standard serial interface to several sensors
UART	2	Can be run in either in asynch. or synch. mode UART0—used for base station communication UART1—available to users, control pins shared with serial flash
ADC	8 channels	10-bit ADC available for users
External clock (High Speed)	7.3228 MHz	Only needed for base station motes that communicate over UART or for communication to external serial devices
External clock (Low Speed)	32 kHz	Used for TinyOS timing (TIMER0) Always running even when mote is sleeping as it is used to wake up the mote after the required sleep interval

TABLE 5.3
Resource Information

Resource	Information
Microprocessor	A low-power microcontroller, which runs TinyOS/MoteWorks from its internal flash memory. A single processor board can be configured to run sensor application or processing and network or radio communications stack simultaneously.
Radio	2.4 GHz IEEE 802.15.4 compliant RF transceiver designed for low-power and low-voltage wireless applications.
	Includes a digital DSSS baseband modem with 250 kbps data rate.
External serial flash (4-Mbit)	For storing data, measurements, and other user-defined info.
	Supports over 100,000 measurement readings.
	Also used for over-the-air reprogramming.
Unique ID Chip	Contains a unique 64-bit identifier.
51-Pin Expansion Connector	Provides a user interface for sensor boards and base stations.
	Includes interfaces for power and ground, ADC inputs for reading sensor outputs, UART interfaces, and I2C interface.
	General purpose digital I/O, and so on.

5.14 TELOSB WIRELESS PLATFORMS

TelosB is an open-source platform for wireless sensor networking. It is capable of data collection and programming through USB. It comprises IEEE 802.15.4 radio with integrated antenna with 250 kbps data rating. It has a low power microcontroller units (MCU) with extended memory with an optional sensor suite to expand the needs of various other sensors to the network. Figure 5.19 shows a layout of TelosB wireless

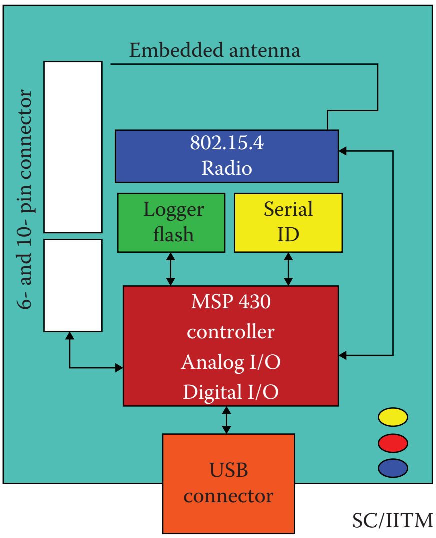


FIGURE 5.19 TelosB wireless platform.

TABLE 5.4
Power Requirements

Operating Current (mA)	IRIS	MICAz	MICA2
Processor, full operation	8.000	12.000	12.000
Processor, sleep	0.008	0.010	0.010
Radio, receive	16.000	19.700	7.000
Radio, transmit (1 mW power)	17.000	17.000	10.000
Radio sleep	0.001	0.001	0.001

TABLE 5.5
Battery Life

	Computed mA-hr Used Each Hour
Processor	0.0879
Radio	0.0920
Logger memory	0.0020
Sensor board	0.0550
Total current (mA-hr) used	0.2369

Computed Battery Life versus Battery Capacity

Battery Capacity (mA-hr)	Battery Life (months)
1000	5.78
2000	11.56
3000	17.35

platform. In most mote applications, the processor and radio are run for a brief period, followed by a sleep cycle. During sleep, current consumption is in microamps as opposed to milliamps. Hence, a very low current is drawn during most of the time, and it draws short-duration spikes while processing, receiving, and transmitting data. This ensures an enhanced battery life of these sensors. Tables 5.4 and 5.5 show the power requirements and battery life of various components in the network platform.

5.15 SENSOR NETWORKING APPLICATIONS

There are many types of equipment introduced by manufacturers throughout the world for various investigations in nature. Often, time-specific new experiments are conducted for assessing *in situ* strength of structural members in ocean environments. Such requirements cannot be done with any general types, as they involve development of suitable sensors with location-specific features. The associated operational features, matching to the nature of the investigations, also make their selection unique. There are a few cases where successful implementation of sensor networking with indigenous technology and efforts are carried out in India. The specialty of these networking of sensors and data acquisition is that they cannot be obtained by any available conventional methods as they belong to most of the applied sciences, where investigations are required to be conducted in an open environment.

- *Underwater ocean monitoring system for Oil and Natural Gas Commission (ONGC)*: Designed and implemented in the Bay of Bengal for evaluating the water mobility and dynamics occurring around the floating platform SEDCO 445 and is operated by the Oil and Natural Commission, engaged in oil prospecting activities.
- *Tide–salinity–temperature recorders for coastal waters for the National Institute of Oceanography (NIO)*: This equipment was designed and installed at 10 locations in the Vemnad backwaters between Alappuzha and Munambam. The work was carried out for NIO-Kochi RC in order to estimate the influence of tides into the estuaries and backwaters. The data acquired were found to be quite different from the general knowledge and was subjected to detailed verification.
- *The temperature chain system for estimation of the thermal wave in the Bay of Bengal*: This system was designed and implemented in the Bay of Bengal off Vaizag for correlating the satellite imagery of ocean thermal imaging with that of the thermal dynamics of ocean depths, as part of the studies conducted by NIO. The equipment consisted of 5 temperature sensors arranged as a chain and installed vertically down to 100 m depth. The temperature data were continuously recorded by computer for several days at different times. The data acquired for the first time in the country showed the complex nature of the ocean thermal dynamics, including thermal waves prominently at around 50 m depth.
- *CODAS (Coastal Oceanographic Data Acquisition System) for coastal engineering*: Developed and implemented in the Alappuzha Coastal area, to monitor 16 coastal engineering parameters in an integrated manner so as to make a systematic analysis of the relevant data and make realistic management of coastal protection and its management. The data acquired included tides, waves, current and direction, water salinity, and water temperature at different water depths and other marine meteorological data: air, temperature, relative humidity (RH), solar radiation, wind, wind direction, and atmospheric pressure.
- *Tide and Wave Telemetering System*: Designed and implemented at different locations along the Kerala Coast, Trivandrum, Alappuzha, Calicut, and Tellichery, continuously for several years during the 1980s, and acquired valuable information of the characteristics of waves, and tides. The equipment designed for installation on the existing piers of the ports consisted of sensors of water level (tide + waves), compatible to continuous installation and operation, with a facility to acquire the data at the laboratories located nearby on the shore. The data were analyzed and became useful for establishing the first-wave energy generator at Vizhinjam. Similar systems were installed in the Gujarat Coast and operated by the Gujarat Maritime Board.
- *Sea HTD (heading–tilt–depth) recorder for the Naval Physical and Oceanographic Lab (NPOL)*: This equipment was developed for testing and recording the multifaced performance of underwater towed platforms, down to 100 m depth, including its heading, tilt, and operational depth.

The data were recorded in underwater pressure capsules, which contained the required sensors, electronics, and battery power.

- *Online HTD (heading–tilt–depth) recorder for NPOL*: This equipment was developed for testing and evaluating the dynamic performance of underwater towed bodies down to 100 m depth. The three dynamic operational parameters, heading, tilt, and operational depth, were displayed online, onboard the ship, and recorded continuously by computer for facilitating an *in situ* analysis of the performance.
- *Wave impact recorder with 17 sensors for IIT Madras*: This equipment was developed for evaluating the impact forces acting on ocean structures. The equipment developed for the Ocean Engineering Centre of IIT Madras consisted of 17 wave impact sensors, electronics, and a computer interface facility. The sensors were installed on the pillars of the ocean structure, operated in the Bay of Bengal, and data were continuously fed to a computer and analyzed.
- *10-channel coastal monitoring system for the Department of Science and Technology (DST) research scheme*: This equipment was designed for making a detailed study on the geophysical aspects of the famous Chakara phenomenon, occurring every year during monsoon season in Kerala Coast. This is the first attempt to record the complex data continuously from 10 sensors mounted on a 10 m tall post in the coastal area. Special sensors were developed for sensing the oscillatory water current caused by the waves. The 10-channel data were fed to a computer for detailed analysis.
- *Underwater environmental meter*: This portable and compact meter was designed to monitor the underwater parameters along the coastal waters including port and harbor areas. The uniqueness of the equipment is the compact sensors that can be inserted into the water depths easily from a small boat or dinky and thus make survey of the water fast without collecting water samples. The data acquired from different depths include water salinity, water conductivity, water temperature, water density, water current, current direction, and underwater radiation. The design features of the sensors and the equipment were established to be suitable for this purpose, since for the first time the stratification characteristics of the Cochin backwaters were recorded with all its dynamic nature. The data monitored during the monsoon showed heavy stratification of water current, current direction, water salinity, and water temperature over short-depth variations of a few meters.
- *Hydrometeorological data acquisition system with 16 sensors for the Orissa Remote Sensing Application Centre (ORSAC)*: The equipment with its 16 hydro-meteorological sensors were designed for the Orissa Remote Sensing Application Centre, Bhubaneswar, for acquiring ground truth data of the coastal area in order to make effective validation of the remote sensing data obtained through satellites. The data acquired included hydrological and meteorological parameters: water level or tide, water current, direction, water temperature, salinity, turbidity, air temperature, RH, wind velocity, wind direction, soil temperature-2 depths, soil moisture-2 depths, solar radiation, and reflected radiation.

- *Online SST and SSS recording system for Sagar Sampada of the Department of Ocean Development*: The FORV Sagar Sampada of the Department of Ocean Development, which is meant for oceanic research activities, required continuous monitoring of SST and SSS to be recorded as the basic data during its voyage, along with others. The equipment designed and installed in the vessel consists of one set of sensors, installed inside the running waterline, and an electronic meter with an optical isolator and computer interface.
- *Coastal wave impact analyzer for coastal wave analysis*: This system is designed for the analysis of the fast-changing characteristics of waves, as they approach the coast. It consists of two sets of sensors mounted 100 m apart along the direction of the approach of the waves to the coast. The sensors include oscillatory water currents and waves. The data from two locations are simultaneously recorded for a detailed analysis of the data. Similar studies have been found to be significant in the recent years, connected with tsunami waves, and their impact on the coasts. Many programs are underway in several coastal and ocean research institutes in India, using model studies in wave flumes.
- *Ship-borne data logger*: This equipment was developed for integrated monitoring of important parameters to be considered during oceanographic investigations, particularly in coastal areas. The 10-channel data pertaining to the water mobility, water quality, and performance of the ship are monitored.
- *Gimbal-stabilized ship-borne data logger for NPOL*: This equipment was designed for precise measurements of 9 of both underwater and meterological data, with meterological sensors mounted on a Gimbal-stabilized device for eliminating errors caused by a ship's motion.
- *Architectural evaluation system*: A series of new microsize sensors that along with their electronics have been developed and implemented for conducting the investigations connected with the performance and behavior of different structures, buildings, and so on, including the comfort of buildings and the comparison among different types of traditional and new buildings and the performance of building materials. A large system with sensors and parameters up to 32 has been implemented in the relevant research laboratories in India.

Ocean structures are assigned with special operations that encompass all tasks associated with the acts of coastal protection. Safe upkeep of these structures ensures stability of the structure throughout their service life. Preventive maintenance of such structures is vital as they are of strategic importance, which precludes a thorough condition assessment of the structure through periodic inspection. Repair and rehabilitation of marine structures require specialized equipment, construction chemicals, state-of-the-art electronic systems to map the existing underwater conditions, electronic surveillance including hydrographic survey equipment, side-scan, sonar imaging, underwater videography or photography, marine borer assessment, and so on. The process of repair and rehabilitation of marine structures is therefore a multidisciplinary task, which needs to be carried out with plenty of research and construction expertise. A few cases of successful implementation of sensor networking with indigenous technology and efforts motivate the engineering community to take up structural assessment as a serious interdisciplinary research. The specialty of these networking of sensors and data acquisition is that they cannot be obtained by any available conventional methods as they belong to most of the applied sciences where investigations are required to be conducted in an open environment. A good spectrum of integration of analytical, experimental, and numerical investigations involved in structural assessment and failure analysis of ocean structures shows that this is not a conventional method of repair; it is an engineering practice that is dependent on detailed analytical and experimental methods.

5.16 NEW GENERATION OFFSHORE STRUCTURES

5.16.1 Offshore Triceratops

Form-based design leads to effective geometric configuration of offshore structures. Innovative geometry enables one to alleviate the encountered environmental loads effectively. Tension leg platforms and spars are the most common dry tree-based configurations successfully attempted for deepwaters. However, a few factors, such as (1) large hulls, (2) complex joints, and (3) complicated station-keeping systems deployed on such platforms, make them expensive (Adrezin et al., 1996). Offshore triceratops is one of the recently proposed structural forms and has been verified to be viable for water depths ranging from 1500 to 3000 m (Chandrasekaran and Madhuri, 2015; Chandrasekaran et al., 2010a,b,c). Buoyant leg structures (BLSs) are positively buoyant structures that are inherently stable in free-floating and tethered mode (Capanoglu et al., 2002). They consist of a circular water-piercing column or hull that supports the deck structure, payloads, structure weight of the BLS, and a restraining system that tethers the column or hull to the seafloor. The BLS resembles a spar due to its deep draft but behaves more like a TLP, as both are heave restrained. While TLP is completely pitch and roll restrained, BLS is only partially restrained if the restraining system is centralized. If the restraining system is circumferential with a group of tethers, the roll and pitch motions of the BLS can be limited. Analytical investigations carried out on offshore compliant structures show less structural response in comparison to other conventional offshore platforms (Chandrasekaran and Nannaware, 2013). The degree of compliancy introduced in the structural system showed encouraging results as in the case of tethered spar. Natural frequencies of such compliant structures are well below the lower bound frequency of encountering sea waves which is an added advantage achieved by the virtue of the compliancy. Earlier studies carried out on the coupled response of BLSs show a good comparison between the experimental and analytical results and confirmed the applicability of BLS as a workable platform with large deck loads (Capanoglu et al., 2002). The concept of the adaptability of BLS for larger deck loads and economic viability of triangular TLPs initiated first hand research on offshore triceratops, which is relatively an innovative concept for deepwater oil exploration. Analytical investigations carried out on triceratops strengthened their suitability for deepwaters; results show that the deck exhibits less roll response under the chosen sea states, highlighting the advantage of ball joint between the deck and BLS (Chandrasekaran et al., 2013). Ensuring the effective control of roll or pitch motion between the deck and BLS units with the presence of ball joints, offshore triceratops show advantageous features to upkeep more facilities on the deck system and comfortable operability during moderate sea states. A critical review of the literature shows that successive attempts are made by researchers to arrive at the innovative platform geometry to suit deepwater oil exploration.

Triceratops consist of a deck structure floated on three BLS units. Deck and BLS units are connected through the large ball joints, making it as a heave-restrained system with the tethers. The platform has nine degrees of freedom: (1) six for the BLS units and (2) three for the deck. Figure 5.20 shows the proposed triceratops, and Figure 5.21 shows the details of a single BLS unit. BLS units consist of a central cylindrical shell of 4.5 m diameter, which is circumscribed by three buoyant tanks of 8.4 m diameter and 130.0 m long. The distance from the outer surface of the central cylindrical shell to the outer surfaces of the buoyant tanks vary from 1 m at the free

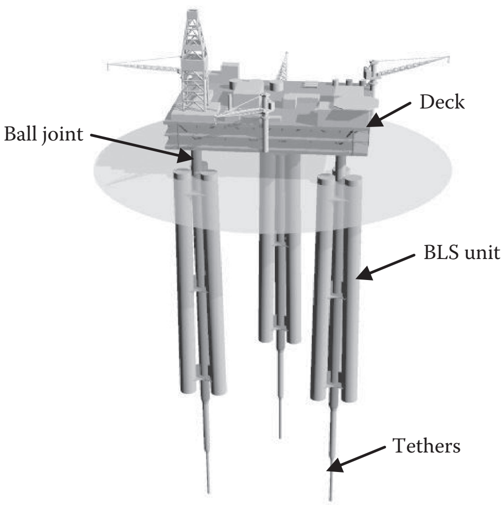


FIGURE 5.20 View of the offshore triceratops.

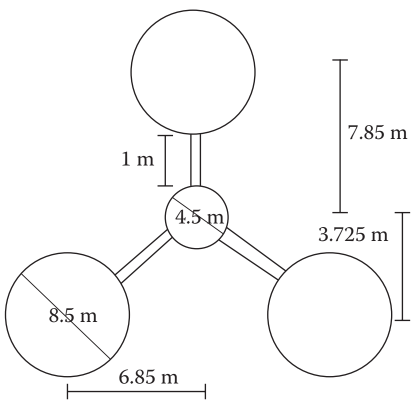


FIGURE 5.21 Details of a single BLS unit.

water surface to 6 m at the keel, which is 121.6 m below the water surface. Each BLS unit is secured to the seafloor with a vertical tethering system.

Ball joints provide rotational compliancy to the hull as a result of which deck always remains horizontal. As BLS is a positively buoyant floating system, buoyancy is in excess of weight; this is subsequently transferred to tethers to ensure the required stability. Though the buoyancy of the triceratops is more than the total mass of the structure, additional ballast is required to achieve the required buoyancy during installation. After the structure is ballasted with additional ballast, it will float freely. The free-floating heave and pitch periods are studied to avoid resonance during installation. Since the displacement of single BLS is less than the triceratops, installation and lifting equipment of larger capacity are not required; this results in significant saving of the installation cost. The natural periods of the free-floating triceratops in the respective degrees of freedom are given in Table 5.6.

Preliminary studies conducted by the authors illustrate the salient advantages of the chosen structural configuration; such new form-based design of offshore platforms will become an effective alternative for ultra-deepwater oil exploration (Chandrasekaran et al., 2010a,b,c, 2015a,b; Chandrasekaran and Madhuri, 2015; Chandrasekaran and Nassery, 2015). Reduced pitch response of the deck in comparison to the BLS ensures a comfortable working environment for personnel onboard for the lower wave periods effectively; this is achieved due to the compliancy offered by the ball joints. The pitch response of the deck is observed due to the transfer of differential heave response from the BLS to the deck. A significant reduction in heave response in comparison to surge response makes triceratops a heave-restrained structure; this makes it suitable for ultra-deepwaters. Although the obtained results are well quantified for the short-period waves, a detailed insight of response behavior of offshore triceratops in long-period waves is necessary. Limitations on the conducted experimental investigations necessitate more detailed investigations on the proposed platform to ensure its suitability for ultra-deepwater oil exploration. Nevertheless, structural advantages derived from the chosen geometric form are highlighted, necessitating a scope for future research on triceratops.

TABLE 5.6
Natural Periods of Offshore Triceratops

Degree-of-Freedom	Free-Floating Triceratops (Model)	Free-Floating Triceratops (Prototype)	Free-Floating BLS (Model)	Free-Floating BLS (Prototype)	Tethered Triceratops (Model)	Tethered Triceratops (Prototype)
Surge	–		–		11.92	145.98
Heave	1.66	20.33	1.60	19.59	0.48	5.88
Pitch	8.04	98.47	1.59	19.47	–	
			% Damping			
Surge	–		–		8.94	
Heave	0.648		1.286		2.71	
Pitch	6.091		1.029		–	

5.16.2 Buoyant Leg Storage and Regasification Platforms

The buoyant leg storage and regasification platform (BLSRP) is the recent innovation in offshore structural engineering, which is essentially designed as a storage and processing platform. The platform rests on six buoyant legs, which support the deck through hinged joints. This is a new hybrid conceptual design that restrains the transfer of both rotational and translational responses from the BLS to the deck and vice versa. BLSs are connected to the seabed through a taut mooring system. Floating, storage, and regasification units of a larger size have found increased applications in the offshore oil and gas fields in the recent past. One of the advantages is that their cost is less than half of that of the onshore facility. Further, they can be fabricated and commissioned within a span of 2–3 years, while it takes about 5–7 years for an onshore plant import terminal. In addition, building of loading and receiving terminals to handle liquid natural gas (LNG) carriers requires huge investments; hence, launching of floating, storage, and regasification unit (FSRU) is preferred, as this is more economical. An attempt is made to arrive at the desired geometric form based on the functional requirements of a typical storage and regasification platform (Chandrasekaran et al., 2015b). A deck with the utilities comprise regasification unit, gas turbine with generator, air compressors, fuel pumps, fire water and foam system, fresh water system, cranes, lubrication oil system, lifeboats, helipad, and LNG Tank. The novelty of the design lies in the development of buoyant leg structure, which is connected to the large deck with the hinged joints. Advantages of the presence of hinged joints on compliant offshore structures are well demonstrated by the researchers in the recent past. Figure 5.22 shows a schematic view of the BLSRP.

Preliminary studies are carried out on the BLSRP at 600 m water depth that highlight its dynamic response behavior under regular waves (Chandrasekaran et al., 2015b).

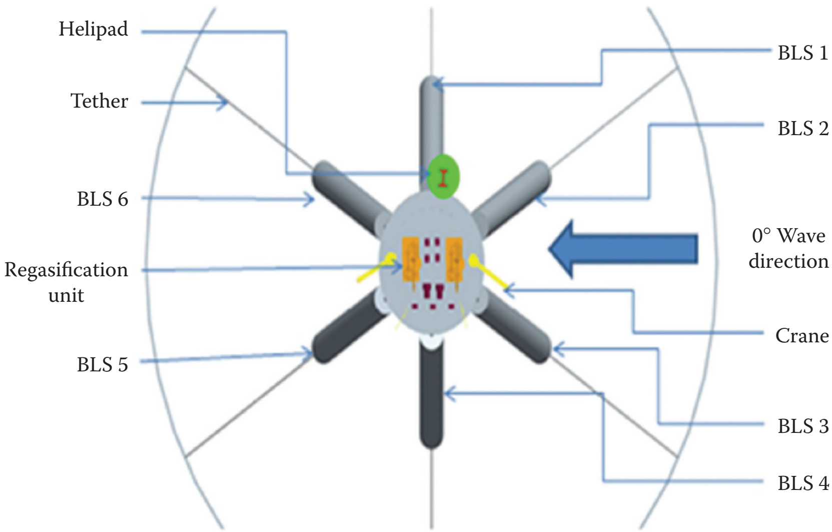


FIGURE 5.22 Geometric model of a BLSRP.

Based on the studies conducted, it is seen that the response of the deck is not influenced by the wave action due to the presence of hinged joints. The proposed structural form proves to be effective in controlling the deck response in all active degrees of freedom. This improves the operational safety and shows high recentering capabilities even under large surge displacements.

Model Paper 1

Time: 3 hrs **Max marks: 50**

READ THIS BEFORE YOU START ANSWERING

Give your answers briefly and to the point. Emphasis should be given to key words of the questions. Include free-hand sketches, wherever necessary.

Use of design codes is not permitted. You shall answer at least one question from each part to qualify for correcting the answer script

Syllabus focus:

Part A (20 marks)

Answer all questions. All questions carry equal marks

1. Discuss the differences between the dry tree and the wet tree.
2. Sketch a semisubmersible and discuss the procedure for wet and dry transportation.
3. Explain the differences between drill ship and jack-up platforms.
4. Discuss the design principles of tension leg platforms. Explain offset and setdown effects.
5. Explain with a neat sketch the functioning of FPSO with subsea systems.

Part B (20 marks)

(a) Objective questions (Answer any five questions. All questions carry equal marks) **(5)**

6. Underwater excavation carried out in shallow sea or freshwater area is called _____________.
7. ____________ is an index to study the behavior of metals under loads.
8. The primary zones of corrosion in any type of offshore structure are ___________________.
9. Name a test that is employed to study the temperature-dependent brittle–ductile transition.
10. Deterioration of materials by chemical interaction with environment is called _____________.
11. "Inspect or measure without doing harm." Associate this caption with structural health monitoring of offshore structures.
12. Area under the stress–strain curve is an index of __________.

(b) Subjective questions (Answer any three full questions)

13a. Discuss the merits possessed by composites when used in marine environment. **(2)**
13b. Describe briefly any one method of nondestructive testing employed to detect corrosion. **(3)**
14a. Write a brief note on hydrogen embrittlement. **(2)**
14b. List any four types of dredging and briefly explain any one of them. **(3)**
15a. Steel used for marine applications is classified in different ways. State how are they classified: (1) based on composition, (2) based on heat treatment, and (3) based on manufacturing methods. **(3)**
15b. State two desirable characteristics of buoyancy materials. **(2)**
16a. List a few serious consequences of corrosion with respect to offshore applications. **(3)**
16b. List the advantages of titanium as a material for the marine environment. **(2)**

Part C (10 marks)

(a) Objective questions (Answer all questions) **(4)**

17. The ratio of the holding power of an anchor to its weight in air is called __________.
18. Statistical analysis of offshore structures is based on _____________.
19. State the condition at which an object can be wind sensitive.
20. Name the two types of anchors.

(b) Subjective questions (Answer any two questions)

21. Draw a neat sketch and explain catenary mooring and taut leg mooring. **(3)**
22. State the Morison equation to compute wave forces on slender bodies and explain the terms. **(3)**
23. What do you understand by marine growth? How is it accounted for in the design? **(3)**
24. Sketch a regular wave and mark the symbols used. What are random waves? **(3)**
25. What are the purposes of anchors? **(3)**
26. List the different types of environmental forces encountered by an offshore structure. **(3)**

------END------

Model Paper 2

Time: 1 hour **Max marks: 25**

Give your answers briefly and to the point. Emphasis should be given to key words of the questions. Include free-hand sketches, wherever necessary. Use of design codes is not permitted.

(a) Objective questions (Answer any 10 questions. All questions carry equal marks)

Rank the following metals/alloys in higher values of their properties given in parentheses.
(1) Tin, cobalt, copper, and iron (toughness); (2) iron, gold, tungsten, and aluminum (ductility); (3) nickel, lead, magnesium, and aluminum (corrosion resistance); (4) bronze, Monel, copper, and bismuth (brittleness).

1. ________ materials have nearly equal rupture strength and ultimate strength as well.
2. Steel alloys suitable for the marine environment should preferably possess two basic properties of greater importance. Name them.
3. What does the area under the stress–strain curve on the tensile test of a steel specimen refer to?
4. Composites are materials consisting of two or more constituents. How are they combined?
5. Concrete has low tensile strength. Do you think that this can be improved to suit marine applications, mention the process briefly.
6. Concrete suffers deterioration during ____________ and ____________.
7. ____________ serve as physical and chemical barriers against attacks from the sea.
8. Charpy V-notch test is the index of ______________.
9. The percentage of elongation, percentage of reduction in area, and bend radius of steel tensile test are the indices of which specific mechanical properties of steel?
10. ____________ occurs in materials when protective films or coatings break down.
11. Hydrogen embrittlement increases with ____________ and ____________.

(b) Subjective questions (Answer any three full questions)

1a. What are syntactic foams? State one of the recent applications of syntactic foams. **(2)**
1b. Discuss few (exclusive) advantages of composites in the offshore environment. **(3)**
2a. List the advantages possessed by aluminum alloys in comparison with steel. **(2)**
2b. List the factors that compel you to study the behavior of materials in the marine environment. **(3)**
3a. List the three steps involved in the corrosion process. **(2)**
3b. Write a brief note on biofouling. **(3)**
4a. Write a brief note on hydrogen embrittlement. **(2)**
4b. List few advantageous properties that make titanium suitable for offshore applications. You should briefly explain them to highlight those properties as necessary for offshore structures, more credit will be given. **(3)**
5a. List few demerits that composites possess when used in the marine environment. **(3)**
5b. List the factors that influence the deterioration mechanism of concrete in the marine environment. **(2)**
6a. Define the yield ratio. Do you feel that it is different from the ductility ratio (DR), then state the explicit difference. **(3)**
6b. How are steel structures protected against fire to ensure offshore maintenance safety. **(2)**

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