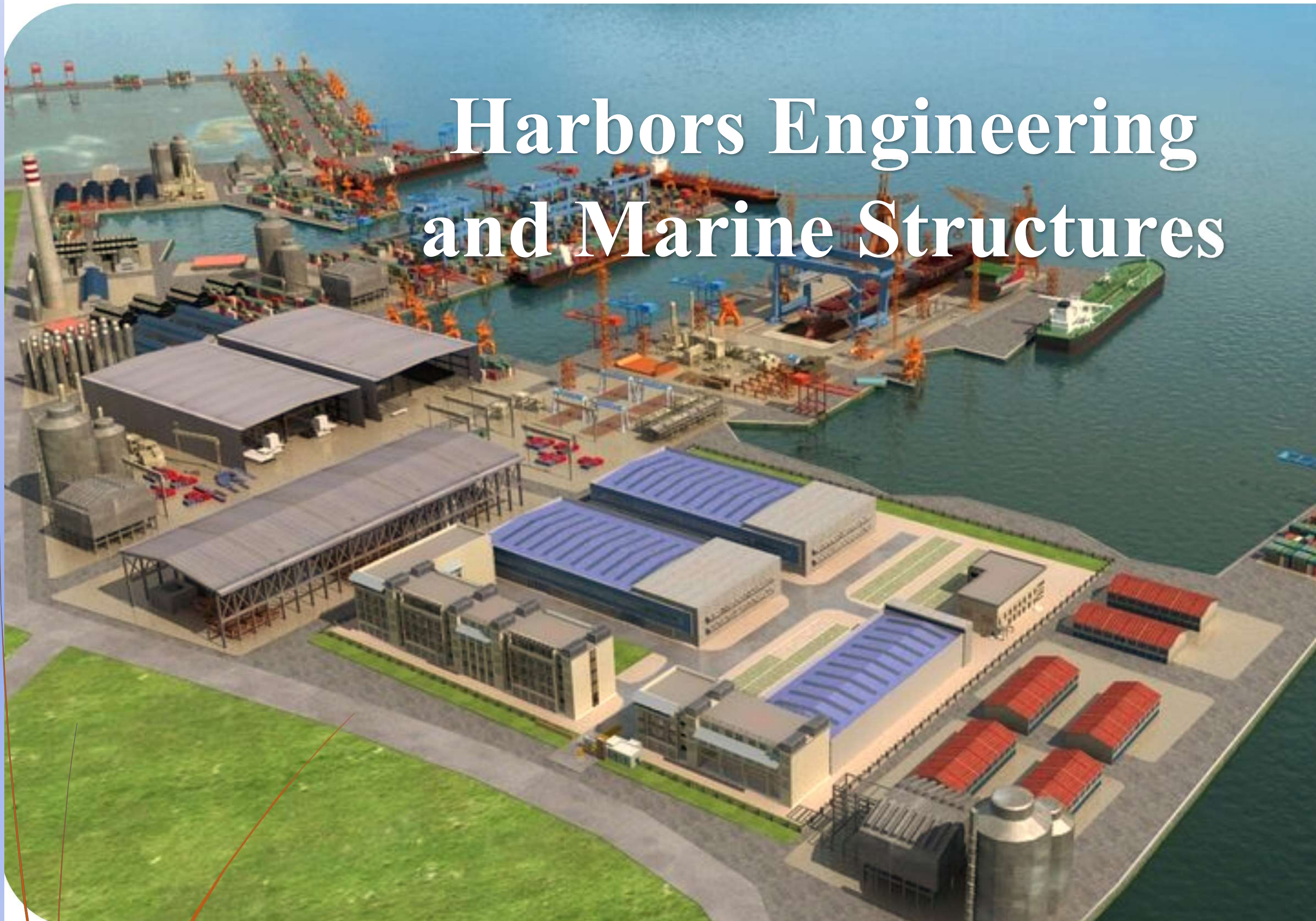
Harbors Engineering and Marine Structures



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Content:

Chapter One

Harbor Engineering and Its Terminology

1.1 Introduction

Harbors -Their Past. Present. and Future

Maritime transportation has generally been the most convenient and least expensive means of transporting goods, and this is why mankind, since ancient times, has been steadily extending its activities into this area. The history of maritime transportation and port development dates back to the year 3500 B.C. and beyond. Over centuries, transport of goods by means of water transportation has been evolved in steps with the needs of world trade and technical capabilities to build larger ships and ship I cargo handling facilities. Initially, waterborne traffic has existed on a local basis where small ships sailed out of river ports for other nearby river ports located in the same river system as showing in **Pic. 1** . With advancing navigational skills the merchants ventured greater and greater distances. Thus, larger ships transporting larger quantities of goods have emerged. As ship . traffic increased, the existing river ports became overcrowded, and in order to permit more ships to berth and at the same time to keep the river usable for more ships, piers had to be constructed along river banks. This stage may be seen as the beginning of the development of modern ports. The ever-increasing demand for shipping and port facilities resulted in construction of the first open-sea ports as showing in **Pic. 2** . Four to five thousand years ago the Phoenicians established open sea ports along the Mediterranean coastline, and the Romans built the famous naval port near Rome on the Tiber River at Ostia. By the end of the first century A.D. a number of large ports had been constructed in the Mediterranean, the Red Sea, and the Persian Gulf.

Pic. 1

Pic. 2

1.2 Terms and Definitions

1.2.1 Harbor is a body of water sheltered by natural or artificial barriers. Harbors can provide safe anchorage and permit the transfer of cargo and passengers between ships and the shore. A harbor is deep enough to keep ships from touching bottom and should give ships and boats enough room to turn and pass each other.

And there are many elements in the port, such as ;

1.2.2 Entrance of Harbor

The port entrance is the most exposed part to the waves, so the depth of the water and the width of the entrance must be greater than in the shipping lane leading to the port. The width of the entrance depends on the traffic density, the number of other auxiliary entrances, the traffic needs and the degree of protection provided to the navigation traffic. As for the width of the entrance, it must be wide enough for the purposes of movement and to avoid dangerous tidal currents, but this width should not be so as to prevent the waves from rising and crashing into the port.

1.2.3 Approach Channel

In principle, the depth of water throughout the port should be sufficient for the purposes of vessel movement throughout the port. The shipping lane leading to the port must be of sufficient width and depth in order to provide a safe passage for ships between the entrance and the berths within the port.

1.2.4 Turning Basin

It is the space that the ship needs for the maneuvering process, when it enters or leaves the berth, and the size of the turning dock depends on the size of the ships visiting the port, and it is preferable that the turning dock be designed in such a way that it allows the ship to rotate continuously without the interference of the auxiliary vessels (Tugs), i.e. The dock should be wide enough to allow for a free rotation of ships, as it is known that ships, like tugs, cannot drive backwards.

1.2.5 Sheltered Basin

It is the water yard protected by the breakwater and the coast. Other port elements such as the anchorage area for ships and berths are present in this basin.

1.2.6 Wharves and Quays

They are built parallel to the beach or breakwater inside the port, and they allow ships to be moored along the quay for the purpose of handling cargo. They are created by filling with soil or other materials and have a wide berth on the surface.

1.2.7 Jetties and Piers

They are either open or closed facilities, with a wide berth above to allow ships to dock along their length. They are constructed offshore or perpendicular to it to reduce alluvial deposits and drilling operations and allow the free flow of tidal currents.

1.2.8 Dry dock and spillways

Its main purpose is to maintain, construct and repair ships. A shipbuilding yard is called a building yard, and it remains dry for ease of work. The dry dock is provided with a gate at the entrance that closes when the vessel enters the dock, and then water is pumped out to keep it dry.

1.2.9 Breakwater

The main objective of the breakwater or breakwater systems is to protect the closed water yard from waves and storms, as they help bring calm inside the port and thus achieve safety for ships inside it and ease of operation. The continuous construction at the top of the breakwater is called the Pier Head.

1.2.10 Ancillaries

They include anchors, hooks, buoys, lights, stores, fire towers and any other services that may be needed.

1.3 Requirements of A Good Harbor: -

The depth of a harbor should be sufficient for every type of visiting ships. The bottom of harbor should provide secured anchorage to hold the ships against high winds. To prevent destructive wave action, break water is provided.

Also Know, how is a Harbor built, Harbors may be natural or artificial. An artificial harbor can have deliberately constructed breakwaters, sea walls, or jetty's or they can be constructed by dredging, which requires maintenance by further periodic dredging. In contrast, a natural harbor is surrounded on several sides by prominences of land.

1.4 Harbor classification:

Harbors are broadly classified as:

➢ Natural harbors.

➢ Semi-natural harbors.

➢Artificial harbors.

Natural harbors: ❖ Natural formations affording safe discharge facilities for ships on sea coasts, in the form of creeks and basins, are called natural harbors.

In below **Pictures 3, 4 & 5** showing live photographically pictures of harbors classifications.

Pic. 3 **Pic. 4** **Pic. 5**

Chapter Two

Natural Phenomena and Technical Studies

Introduction:

Harbors are the cornerstones of international trade. These facilities are so vital in fact that, for many regions, a reduction in capacity in the harbor system carries with it grave economic consequences. Limited redundancy of harbor systems makes them critical nodes in the transportation networks that connect the coast to points inland.

One of the main reasons to success the harbor works, is the natural phenomena around it, so study of natural phenomena is very important to design a success hydraulic structures for harbor.

2.1: Tides.

2.2: Winds.

2.3: Marine Currents.

2.4: Marine Area.

2.5: The Materials Used in Marine Structures and Effect of Seawater on it.

2.6: Waves, Theory & Analysis.

2.7: Soil Properties Studies.

2.8: Test of Hydraulic Models.

Fig (2-1) Harbor

2.1: Tides

Tides are the rise and fall of seawater levels caused by the combined effects of the gravitational forces exerted by Moon, Sun, and the rotation of the Earth.

Tides phenomena is very important in harbor engineering for following reasons:

1- The heights of piers and breakwaters are determined in terms of the change in sea level from the lowest level to the highest level.

2 - The changing in sea level is taken into consideration as a factor that determines the length and height of the side walls of the dry basins.

3- These phenomena give rise to what is known as tidal currents, and it is an important factor that must be studied when determining port entrances.

4- The type of port may depend entirely on the amount of change that occurs in the sea level in the region. If this change exceeds a certain limit, the port may be established in the form of closed basins.

The biggest height tide in the world can be found in Canada's Bay of Fundy (11.7m) as shown in Fig (2-2).

Fig (2-2) Bay of Fundy in Canada

2.1.1 Tides Theory:

Gravity is one major force that creates tides. In 1687, Sir Isaac Newton explained that ocean tides result from the gravitational attraction of the sun and moon on the oceans of the earth.

Newton's law of universal gravitation states that the gravitational attraction between two bodies is directly proportional to their masses, and inversely proportional to the square of the distance between them. Therefore, the greater the mass of the objects and the closer they are to each other, the greater the gravitational attraction between them (with assume: All Earth covered by seawater, Depth of seawater is constant, seawater is non-viscosity and movement of water is slowly so the seawater will consider as statically equilibrium).

Tidal forces are based on the gravitational attractive force. With regard to tidal forces on the Earth, the distance between two objects usually is more critical than their masses. Tidal generating forces vary inversely as the cube of the distance from the tide generating object. Gravitational attractive forces only vary inversely to the square of the distance between the objects. The effect of distance on tidal forces is seen in the relationship between the sun, the moon, and the Earth's waters.

Our sun is 27 million times larger than our moon. Based on its mass, the sun's gravitational attraction to the Earth is more than 177 times greater than that of the moon to the Earth. If tidal forces were based solely on comparative masses, the sun should have a tide-generating force that is 27 million times greater than that of the moon. However, the sun is 390 times further from the Earth than is the moon. Thus, its tide-generating force is reduced by 390^3, or about 59 million times less than the moon. Because of these conditions, the sun's tide-generating force is about half that of the moon, as shown in Fig (2-3).

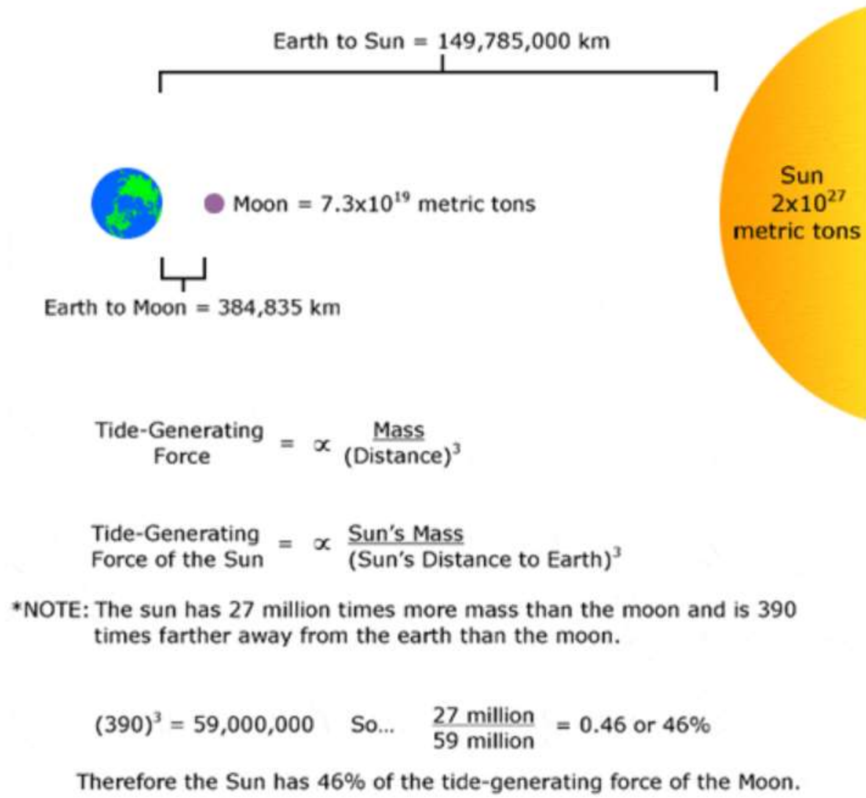


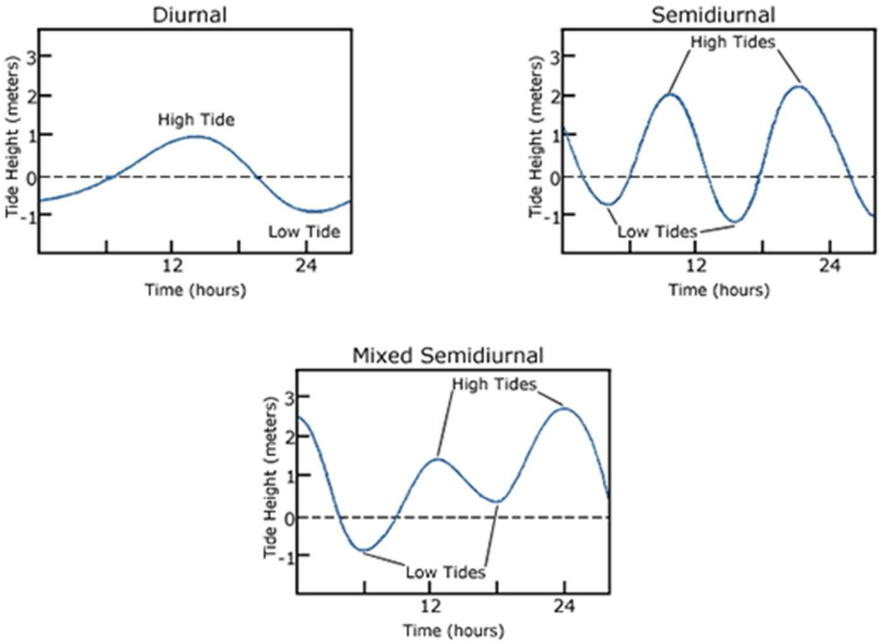
Fig (2-3) Difference between Moon and Sun tides

2.1.2 Types of Tidal Cycles:

Diurnal, Semidiurnal, Mixed Semidiurnal; Continental Interference

Three basic tidal patterns occur along the Earth's major shorelines. In general, most areas have two high tides and two low tides each day. When the two highs and the two lows are about the same height, the pattern is called a **semi-daily** or semidiurnal tide. If the high and low tides differ in height, the pattern is called a **mixed semidiurnal tide**.

Some areas, such as the Gulf of Mexico, have only one high and one low tide each day. This is called a **diurnal tide**, as shown in Fig (2-4) and Fig (2-5).



` Fig (2-4) Types of Tidal Cycles:

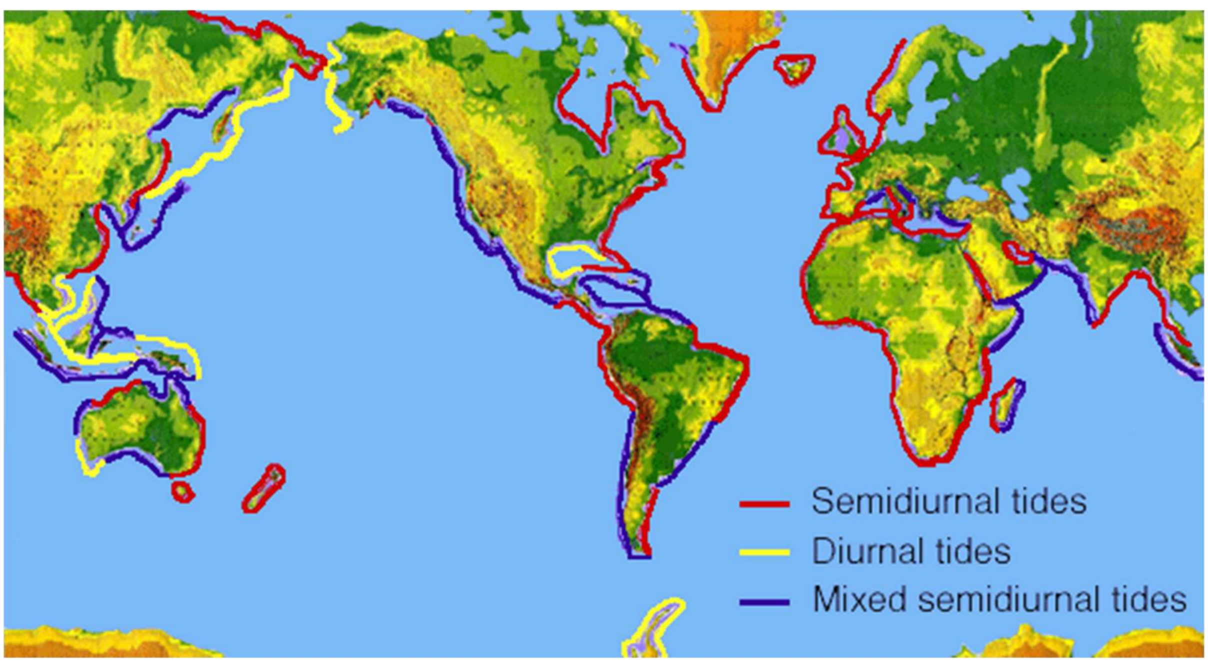


Fig (2-5) Tides types on the world map

2.1.3 Tidal Variations - The Influence of Position and Distance:

The moon is a major influence on the Earth's tides, but the sun also generates considerable tidal forces. Solar tides are about half as large as lunar tides and are expressed as a variation of lunar tidal patterns, not as a separate set of tides. When the sun, moon, and Earth are in alignment (at the time of the new or full moon), the solar tide has an additive effect on the lunar tide, creating extra-high tides, and very low tides—both commonly called **Spring Tides**. One week later, when the sun and moon are at right angles to each other, the solar tide partially cancels out the lunar tide and produces moderate tides known as **Neap Tides**. During each lunar month, two sets of spring tides and two sets of neap tides occur, as shown in Fig (2-6).

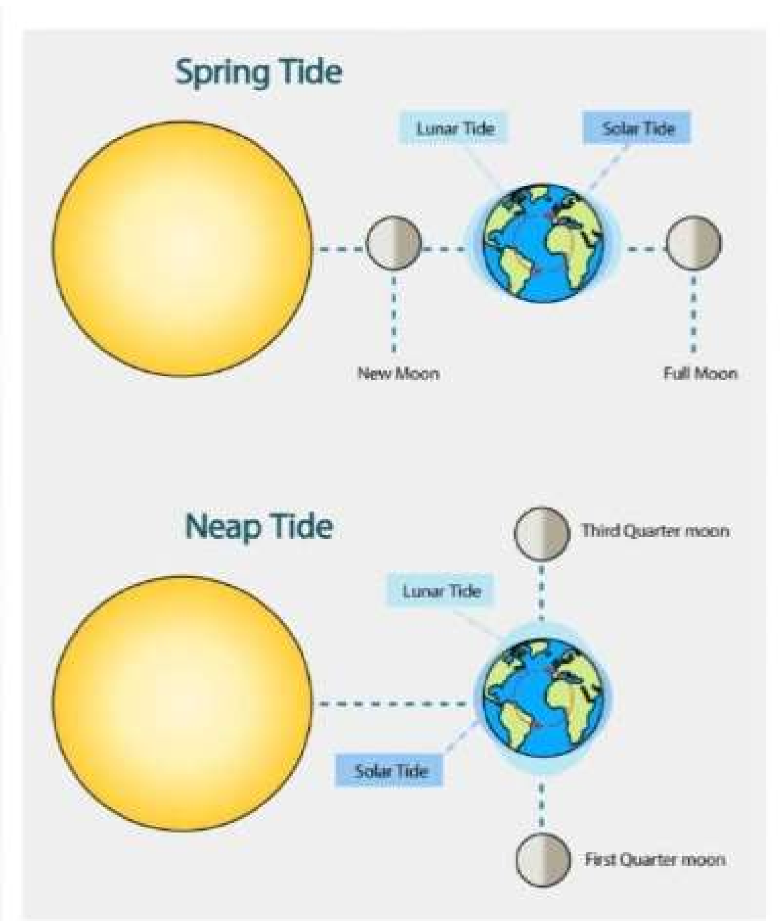


Fig (2-6) Shows Spring tide & Neap tide position

2.1.4 Equinoctial Tide:

It is a special case of spring tides, occurs on 21 March and 23 December every year, when sun and moon in a same line over the equator, which means shortest distance which causes higher tide.

2.1.5 Tidal Range:

Tidal range is the vertical distance through which the tide rises and falls, the difference in water height between low tide and high tide, or, quite simply, the “size” of tide.

2.1.6 Tide Tables:

Tide tables can be used for any given locale to find the predicted times and amplitude (or "tidal range"). The predictions are influenced by many factors including the alignment of the Sun and Moon, the phase and amplitude of the tide (pattern of tides in the deep ocean), the amphidromic systems of the oceans, and the shape of the coastline and near-shore bathymetry. They are however only predictions; the actual time and height of the tide is affected by wind and atmospheric pressure. Many shorelines experience semi-diurnal tides—two nearly equal high and low tides each day. Other locations have a diurnal tide—one high and low tide each day. A "mixed tide"—two uneven magnitude tides a day—is a third regular category.

Fig. (2-7) Low Tide and High Tide

2.2 Winds:

Wind is the movement of air caused by the uneven heating of the Earth by the sun. It does not have much substance—you cannot see it or hold it—but you can feel its force. It is the great equalizer of the atmosphere, transporting heat, moisture, pollutants, and dust great distances around the globe.

Fig (2-8) Wind Direction

Differences in atmospheric pressure generate winds. At the Equator, the sun warms the water and land more than it does the rest of the globe. Warm equatorial air rises higher into the atmosphere and migrates toward the poles. This is a low-pressure system. At the same time, cooler, denser air moves over Earth's surface toward the Equator to replace the heated air. This is a high-pressure system. Winds generally blow from high-pressure areas to low-pressure areas.

The purpose of study this phenomenon to know the directions of wind with time and speed, for following reasons:

1- To specify the speed, direction and time in the region to consider that during design the harbors as well as the directions of marine paths.
2- To calculate the pressure which caused by winds during design cranes and
3- The wind is important reason to establish the waves (later)

2.2.1 Wind Speed Classification:

Table (2-1) Wind Speed Classification

Beaufort Scale	10-minute sustained winds		General terms
	Knots	Km/h	
0	<1	<2	Calm
1	1–3	2–6	Light air
2	4–6	7–11	Light breeze
3	7–10	13–19	Gentle breeze
5	17–21	31–39	Fresh breeze
6	22–27	41–50	Strong breeze
7	28–33	52–61	Moderate gale
8	34–40	63–74	Fresh gale
9	41–47	76–87	Strong gale
10	48–55	89–102	Whole gale
11	56–63	104–117	Storm
12	64–72	119–133	Hurricane

2.2.2 Record Wind information:

The wind speed is measured by an anemometer, which consists of 3 rotating cups, and its unit of measurement (knot, m/s, km/hr), the wind direction is by means of vane wind, which consists of a blade or arrow whose front points to the direction from which the wind blows. The two devices are placed at a height of 10 meters from the ground level, shown in Fig (2-9).

Fig (2-9) Anemometer

There are a lot of ways to record the wind information, the famous one is called rose diagram, shown in Fig (2-10):

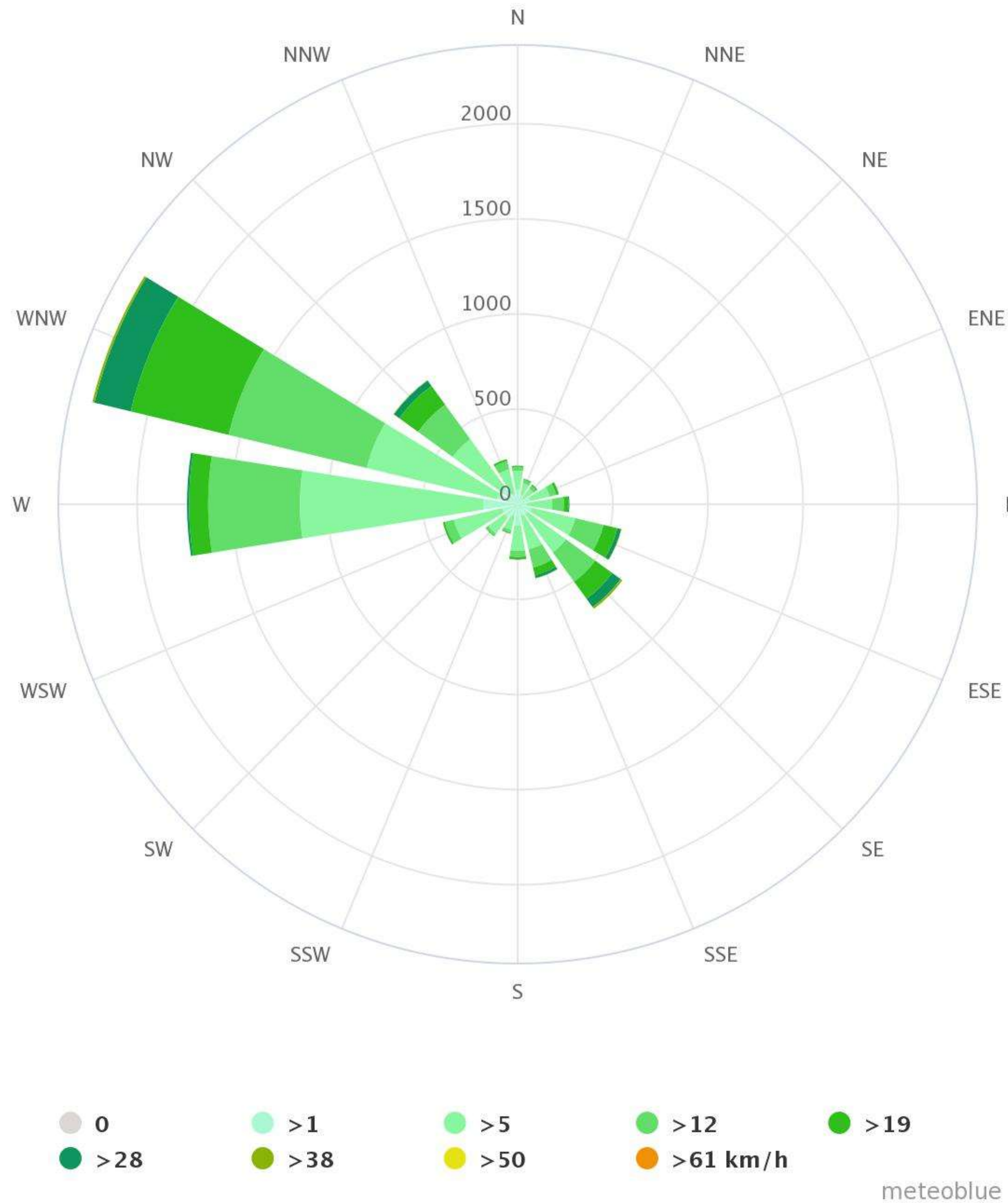


Fig (2-10) Annual Rose Diagram for Basra (shows how many hours per year the wind blows from the indicated direction) (2)

2.2.3 Wind Pressure:

When moving air wind is stopped by a surface - the dynamic energy in the wind is transformed to pressure, as shown in Fig (2-11). The pressure acting the surface transforms to a force

$F_w = p_d\,A$

$= 1/2\,\rho\,v^2\,A \qquad\qquad (1)$

Where F_w = *wind force (N)*
A = *surface area* (m^2)
p_d = *dynamic pressure (Pa)*
ρ = *density of air* (kg/m^3)
v = *wind speed (m/s)*

Note - in practice wind force acting on an object creates more complex forces due to drag and other effects.

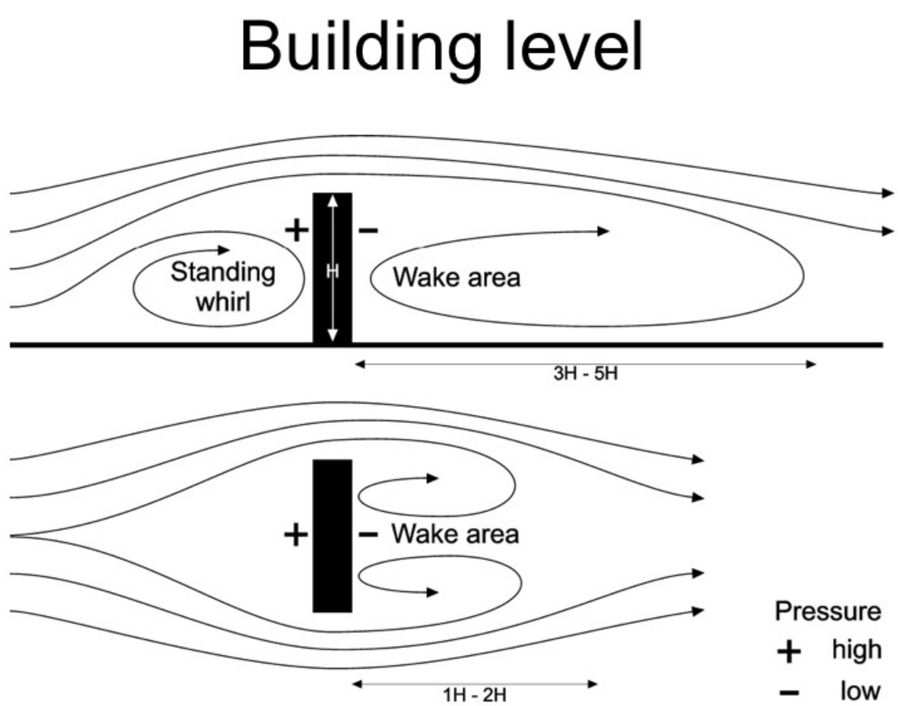


Fig (2-11) Wind Pressure applied on a building

2-3 Marine Current:

Marine currents are driven by tides, wind and water density differences.

Marine currents describe the movement of water from one location to another with sediment or not, as shown in Fig (2-12). Currents are generally measured in meters per second or in knots (1 knot = 1.85 kilometers per hour or 1.15 miles per hour).

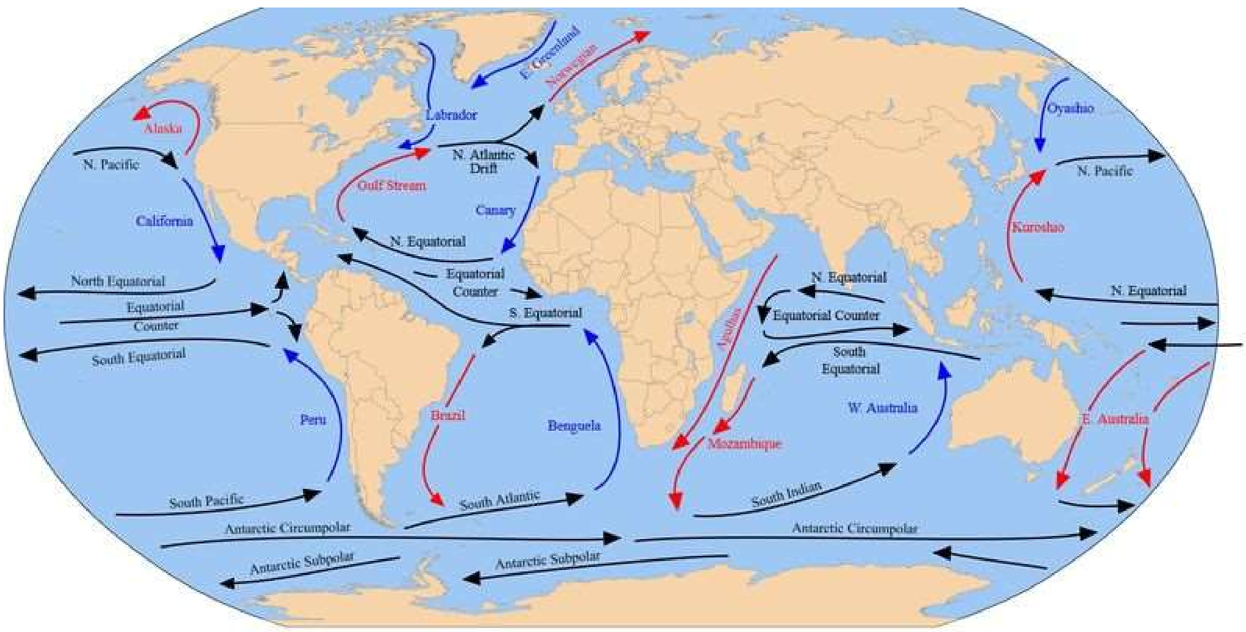


Fig (2-12) Marine Current in Earth

Marine currents are driven by three main factors:

1. **Tides Current:**

   Tides create a current in the oceans, which are strongest near the shore, and in bays and estuaries along the coast. These are called "tidal currents". Tidal currents change in a very regular pattern and can be predicted for future dates. In some locations, strong tidal currents can travel at speeds of eight knots or more.

2. **Surface Current:**

   Winds drive currents that are at or near the ocean's surface. Near coastal areas winds tend to drive currents on a localized scale and can result in phenomena like coastal upwelling. On a more global scale, in the open ocean, winds drive currents that circulate water for thousands of miles throughout the ocean basins.

3. **Natural Current:**

   This is a process driven by density differences in water due to temperature and salinity variations in different parts of the ocean. Currents driven by thermohaline circulation occur at both deep and shallow ocean levels and move much slower than tidal or surface currents. Currents affect the Earth's climate by driving warm water from the Equator and cold water from the poles around the Earth.

2.4 Marine Area:

The marine area combines the requirements of the marine engineer at sea and the requirements of the structural engineer in the areas close to the shore, so, we will limit the following to the marine works engineer’s needs for marine surveying works, as shown in Fig (2-13). In this case, the engineer’s needs can be summarized as follows:

1- Surveying works on the land to define the shoreline.

2- Surveying works on the sea to determine the depths of piers, breakwaters and other marine installations, as well as to determine the locations of islands and submerged rocks and the levels of the sea floor in the area of the shipping lanes.

Fig (2-13) Marine area in front of a harbor

2.5 The Materials used in Marine structures and the effect of sea water on it:

Many of materials used in construction of hydraulic structures in the harbors, but related to un-normal conditions, the used materials must be in high quality and able to withstand what they are exposed to in terms of forces and external factors for the longest possible period.

Maritime facilities are exposed to dynamic forces resulting from wave shocks and shocks resulting from ship movement or shocks that may arise accidentally during leave ships in addition to the friction forces that They are present during traction, whether between the ship and the pier front, which must be protected, or if these forces result from the movement of the mooring ropes.

Marine installations are also subjected to recurring wetness and drought that results from changing water levels in the tides or as a result of storms and their winds from spraying, and this phenomenon has a harmful effect on some of the materials used in construction.

As well as, the change in temperature, the chemical effect of sea water, and the effect of insects and marine worms on some of the materials used.

Special specifications must be provided in the materials used to be able to resist all the aforementioned factors:

1- The material used in construction must be able to withstand the forces of pressure and friction when the structure is exposed to those forces.

2 - To carry the harmful effects resulting from frequent wetness and drought, the constant change in temperature and the chemical effect of sea water, as well as insects and marine worms.

2.5.1 Materials used in marine installations:

1- Natural Stones.
2- Iron and Steel.
3- The Woods.
4- Ordinary Concrete.
5- Reinforced Concrete.

2.5.2 Seawater Composition:

Seawater Composition Seawater is a complex solution of inorganic, organic, and biological components. These can interact with materials to cause corrosion and to degrade their properties.

Table (2-2) CHEMICAL COMPOSITION OF SEAWATER, 19ppt Chlorinity

CHEMICAL COMPOSITION OF SEAWATER, 19ppt Chlorinity
Salinity (ppt) = 0.03 + 1.805 Cl^-

Anions	g/kg of water	Cation	g/kg of water
Chloride	19.35	Sodium	10.76
Sulfate	2.70	Magnesium	1.29
Bicarbonate	0.14	Calcium	0.41
Bromide	0.067	Potassium	0.39
Borate	0.0044	Strontium	0.0079
Fluoride	0.0014		

- Average weight of chemical composition of seawater is (38 g/kg) different from region to other.
- Specific weight for seawater is 1.025 ton/m3

Table (2-3) Chemical Composition Seawater for Arabian Gulf

Parameter	Range (ppm)		
Sodium	11536.00	to	12433.00
Magnesium	1490.00	to	1543.00
Potassium	469.00	to	459.00
Calcium	378.00	to	404.00
Copper	<0.05	to	0.05
Zinc	<0.05	to	0.05
Iron	<0.05	to	0.5
Manganese	<0.05	to	0.1
Chloride	21933.0	to	22014.00
Sulfate	3200.0	to	3273.00
Bicarbonate	156.0	to	161.00
Ammonia	0.02	to	1.13
Nitrate	0.002	to	0.025
Sulfide	0.005	to	0.2
Free chlorine	0.02	to	0.2
Total organic-carbon	8.0	to	8.0
pH	7.84	to	8.24

2.6 Waves:

Waves are the result of multiple factors, which may be r: seismic, such as the movement of ships or explosions near the surface of the water, or natural such as earthquakes, tides, and waves formed by the action of winds are the most important types in marine works engineering, and their study becomes necessary for the full reasons:

1- Huge forced generated from waves and effect on the marine structures such as breakwaters, so must consider that forces during the design phase.

2- If the waves formed within the marine area, will cause many troubles in loading and unloading, as well as, make the ships touch the piers roughly, so accordingly by studying the waves and their refraction and spread is very important.

3. The studying of waves is very important when planning and design the breakwater, the entrance and shipping lanes.

4. The waves contain an energy working continuously to change the coast parameters, so, must consider the wave study with other natural phenomena.

Fig (2-14) Waves

2.6.1 Waves Theory:

Ocean waves are mainly generated by the action of wind on water. The waves are formed initially by a complex process of resonance and shearing action, in which waves of differing wave height, length, period is produced and travel in various directions. Once formed, ocean waves can travel for vast distances, spreading in area and reducing in height, but maintaining wavelength and period as shown in Figure (2-15):

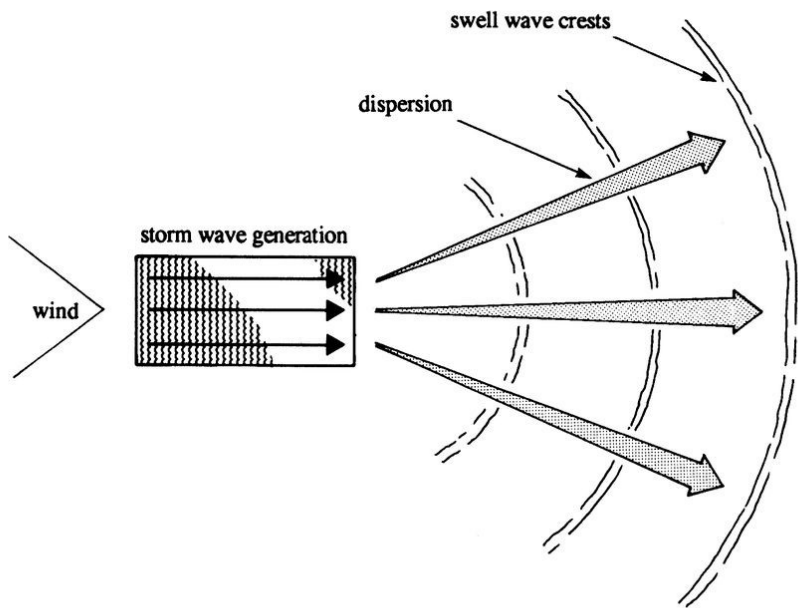


Fig (2-15) Wind Waves

In the storm zone generation area high frequency wave energy (e.g. waves with small period) is both dissipated and transferred to lower frequencies. Waves of differing frequencies travel at different speeds and therefore outside the storm generation area the sea state is modified as the various frequency components separate. The low frequency waves travel more quickly than the high frequency waves resulting in a swell sea condition as opposed to a storm sea condition. This process is known as dispersion. Thus, wind waves may be characterized as irregular, short crested and steep containing a large range of frequencies and directions. On the other hand, swell waves may be characterized as fairly regular, long crested and not very steep containing a small range of low frequencies and directions, as shown in Fig (2-16).

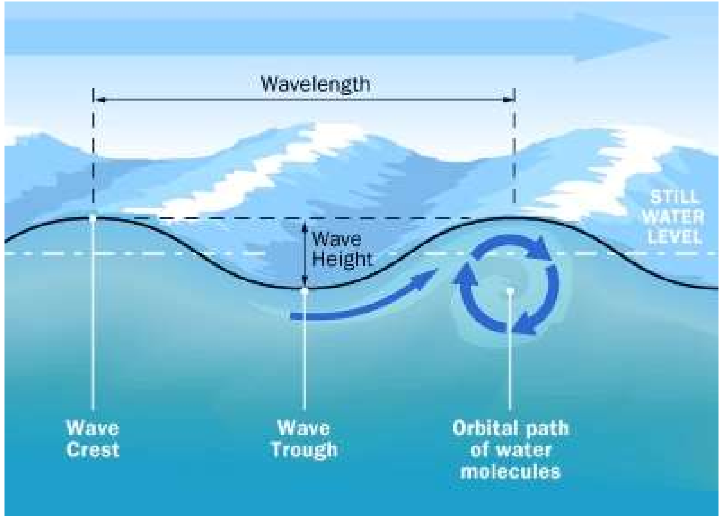


Fig (2-16) Wave Details

2.6.2 Wave Types:

All waves are primarily classified according to the generation and restoring mechanisms, which induce oscillations within a wide band of periods and associated wavelengths. The resulting waves and induced motion through the water column affect a large number of marine processes and engineering activities that take place on the surface and in the most superficial sublayers of ocean. These include the exchange of heat and gases, ocean mixing, transport of sediment, coastal morphology, seakeeping, offshore engineering, and renewable energy among many others. A description of the different types of ocean surface waves and their concurrent classification is presented.

Table (2-4) Ocean Wave Classification

Classification	**Period band**	**Generating forces**	**Restoring forces**
Capillary waves	<0.1 s	Wind	Surface tension

Classification	Period band	Generating forces	Restoring forces
Ultra-gravity waves	0.1–1 s	Wind	Surface tension and gravity
Gravity waves	1–20 s	Wind	Gravity
Infra-gravity waves	20 s to 5 min	Wind and atmospheric pressure gradients	Gravity
Long-period waves	5 min to 12 h	Atmospheric pressure gradients and earthquake	Gravity
Ordinary tidal waves	12–24 h	Gravitational attraction	Gravity and Coriolis force
Transtidal waves	>24 h	Storms and gravitational attraction	Gravity and Coriolis force

2.6.3 Capillary Waves:

The shortest-period waves, and the first to be noticed on the ocean surface when wind starts blowing, are the capillary waves, which resemble cat's paws ripping the otherwise smooth surface. This peculiar wavy structure is generally forced by a light breeze of speeds of about 3 m/s (taken at a reference height of 10 m from the water level) and assumes a fine structure of small ripples with a wavelength of less than 1.5 cm and period less than 0.1 s (an example of capillary waves is shown in Fig (2-17)).

Fig (2-17) Capillary Waves

2.6.4 Gravity Waves: Wind Sea

A consistent blowing of wind over a substantial fetch (i.e., the distance over which the wind blows) forces waves to become much longer than the threshold wavelength of 1.7 cm. As the wavelength grows longer than 1.5 m (i.e., wave period becomes larger than 1 s), surface tension becomes negligible and gravity remains the sole restoring mechanism. Under these circumstances, waves are classified as gravity waves. It is worth mentioning, in this regard, that gravity acts on wave dispersion by inducing wave phases to propagate faster than wave groups and thus reversing the effect of surface tension. Generally speaking, gravity waves assume periods ranging from a minimum of about 1 s up to maximum of approximately 25 s (i.e., wavelength varies roughly between 1.5 and 900 m), as shown in Fig (2-18).

Fig (2-18) Gravity Wave

2.6.5 Infra-gravity Waves:

Nonlinear interactions between wave components convert part of the energy associated to wind-generated gravity waves into subharmonics with periods ranging from about 20 to 30 s up to a maximum of approximately 5 min. These long oscillations, which are driven primarily by swell, are bound to the generating wave trains and are normally known as infra-gravity waves by filtering period lower than 30 s, as shown in Fig (2-19).

Fig (2-19) Infra gravity Waves

2.6.6 Long-Period Waves (Tsunamis, Seiches, and Storm Surges):

Well-defined waves with periods longer than 5 min are routinely recorded in the ocean. Although different originating mechanisms can be responsible for such waves, meteorological conditions and earthquakes remain the primary cause. Normally, long oscillations generated by atmospheric conditions are known as seiches and storm surges, while tsunamis identify waves originated from earthquakes. Despite the long wavelength, the restoring mechanism is still dominated by gravity, as shown in Fig (2-20).

Fig (2-20) Long period waves

Tsunamis are long waves with period varying between 1 and 20 min (wavelength from a few kilometers up to a few hundreds of kilometers) that are generated by sudden tectonic changes to the sea bed or landslides that are usually attributed to earthquakes and submarine volcanic activity. In the open ocean, tsunamis have very small amplitude (only rarely wave height exceeds 1 m) and generally pass completely unnoticed. Propagation into shallower waters, however, makes wave shoal, compressing the shape of the oscillation. As a result, its speed diminishes of about one order of magnitude (from about 800 to <80 km/h), while its wavelength reduces to less than 20 km with a consequent substantial growth of wave height.

2.7 Soil Properties Studies:

It is required achieved sufficient technical studies on soils to determine its natural and mechanical properties, which required for following:

1- Estimating the total cost of a new harbor where the type of facilities that are suitable for the quality of the soil, as well as determining the type of soil in which the drilling will pass, and usually the study, in this case it is not intense.

2- Related to the results of soil properties, two choices, first; the cost of the new harbor will very expensive according to the type of soil, so required cancel the location and search for other, or second, the location will be accepted.

2.8 Test on the Hydraulic Models:

In many cases it is necessary to do hydraulic researches in lab before start the design or construction for large projects, which that require a special study according to the recognized engineering theories. Hydraulic researches applied on models of required structures, surrounding area and the forces affecting them, these models must be completely corresponding for each structure.

The purpose of doing the testing to determining the direction and location of breakwaters, protective heads on the beaches, study the sediment or erosion expected after the establishment of an insulting facility after coasts formation with protect it and test the balance of breakwaters.

In general, Tests are required when the designing department cannot be satisfied with the engineering theories or from his previous experiences which will cause any mistake to loss of money.

For example, generation of waves, generation of tides and marine currents, as shown in Fig (2-21) and Fig (2-22).

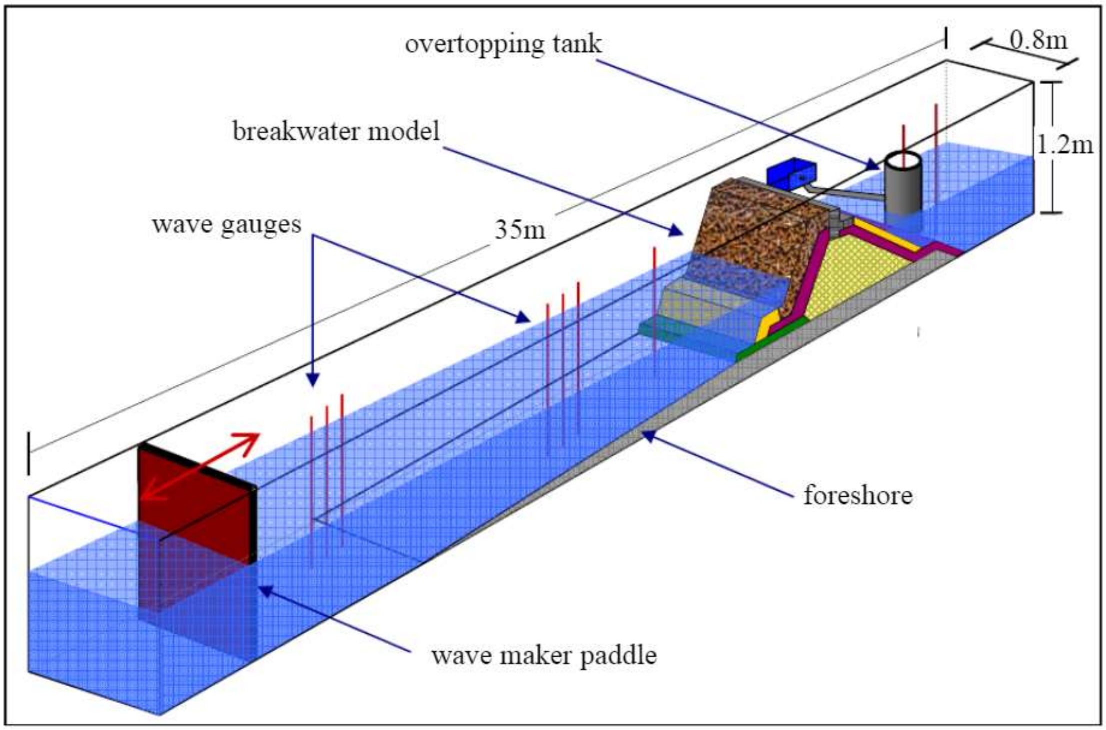


Fig (2-21) Waves simulator with water breaker

Fig (2-22) Water breaker model

Chapter Three

Planning of Harbors

Introduction

One of the most important reasons to make any harbor is successful and effective, the planning of harbors which give the ships comfortable lanes, loads and unloads and waiting period, so, in below will discuss and show the planning of harbors rules, as follows:

3.1: Factors of Harbors Design.

- 3.1.1: Properties of Ships.
- 3.1.2: Nature of Location.
- 3.1.3: Purpose of Construction a New Harbor.

3.2: Planning of harbors:

- 3.2.1: Navigation Lanes Rules.
- 3.2.2: Entrance of Harbors.
- 3.2.3: Water Area in front of Harbor.
- 3.2.4: Planning of Breakwater.

Fig (3-1) Planning of Baltimore Harbor, MD, United States

3.1: Factors of Harbors Design:

It is depending on many factors can be summarized as follows:

1- Properties of ships that are expected to come to the harbor, length, width, draft and maximum cargo capacity.
2- The nature of the proposed construction site and the possibilities of natural protection.
3- The purpose for which the harbor is established.
4- Various natural phenomena in the construction area.
5- Water depths in the construction area and the shape of the shoreline.
6- The type of soil forming the bottom.

It is very important study and plan each of the following:

1- Navigational lanes leading to the harbor entrance in terms of its layout, depth and breadth.
2- The harbor entrance (may have more than one entrance) in terms of its location, depth, breadth and sides if necessary.
3- The water area that ensures ease of movement inside the harbor (or other purposes) without increasing its area and exposure to waves.
4- Dividing the harbor into special areas according to needs.
5- Planning the sidewalks and determining their dimensions and the depths of the water in front of them.
6- Planning roads and railways inside the harbor.
7- Determining the dimensions and heights of the necessary warehouses and storage yards, as well as determining the tasks required for loading and unloading.
8- Determine the locations of maintenance basins, such as dry basins.

3.1.1: Properties of Ships:

The shipbuilding industry is developing very quickly while the proportions between the length and width of the ship are constantly changing, and it is not necessary when studying the harbor planning project that the engineer know the characteristics of the different ships that roam the seas around the world, but it is sufficient to make a thorough study of the sizes and tonnages of the ships that are expected to deal with the harbor. For example, if a harbor is to be established on any location, then it is necessary to study foreign trade, as well as, which harbors expected to deal with during next Ten years. If those harbors are identified, it is necessary to know sufficient data about the ships that frequented them. A comprehensive study is made on the ships that have been frequenting them in the past ten or twenty years, and from which it is possible to know the rate of growth in the volumes and tonnage of ships and thus what is expected to be in the following years, figure (3-2) shows the ship dimensions names.

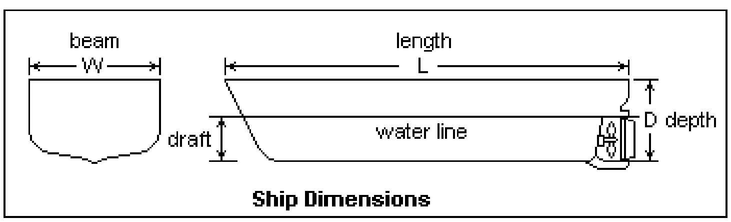


Fig (3-2) Ship dimensions

These are the same ships that are expected to transport trade to and from the proposed harbor, that is, they are the ships that are expected to fly to this harbor, and the information that must be specific for ships that frequent the harbor are:

1- Length, width and draft of the ship when it is loaded with maximum load.
2- Area floating above water exposed to wind pressure when the ship is empty.
3- Empty weight of the ship and its maximum load up to the loading mark.
4- Number of ships expected to be inside the harbor at any time.
5- Type of trade to and from the harbor, its quantities, and the seasons for its loading or unloading.

3.1.2: Nature of Location:

The harbors are classified to Three sections according to locations as follows:

1- **Natural Harbors:** It protected fully by nature protection without need for artificial works, such as the harbors on rivers, figure (3-3) shows Al Maaqal Harbor on Shatt Al Arab river.

Fig (3-3) Al Maaqal harbor (Natural harbor)

2- **Semi-Natural Harbors:** It is protected partially by natural protection, but need some artificial and manmade construction.

3- **Artifical Harbors:** It have no natural protection but artificial arrangement are made to protect the harbor from storm and wind, the construction costs are expensive.

3.1.3: Purpose of Construction a New Harbor:

Each harbor has requirements according to the purpose for which it is established.

These requirements are divided into two parts, requirements specific to the uses of the harbor, and these are determined by the specialists, as is the case in military harbors in terms of defending the harbor and so on, and engineering requirements that depend on the uses of the harbor, and these are determined by the designer.

Harbors are classified according to their purpose as follows:

1- **Commercial Harbors:** Ships are almost visited it regularly for commercial purposes, including loading, unloading goods and passengers. It must be located on public navigation routes and be easily connected to the interior roads of the country, its industrial and agricultural, and other production centers. And this connection is either by railways, roadways, navigation channels, airlines or all of them, and the following must be available in such harbors:
   - a- Full protected from waves and storms so that ships can loading and unloading.
   - b- Protected from tides or at min. range, by even construct internal basins if required.
   - c- Depth of berths are sufficient for all ships, and construct sheds to store goods temporarily or for a long time.
   - d- Provision of fixed and mobile cranes on berths to load & unload the goods in shortest time.
   - e- Equipping with maintenance docks such as dry docks, floating docks, as well as its workshops for repairing ships.
   - f- Lighting for harbor and supply the ships with fuel, food and fresh water.

Fig (3-3) Umm Qasser Harbor (Commercial Harbor)

2- Military Harbors: It constructed to serve warships and to be a base for it, figure (3-4) shows naval Norfolk harbor as military harbor, the following must be available:

a- It connected with the country by railways - roads - navigational channels and airline lines for the ease of transporting the insured, soldiers and ammunition from the interior of the country.

b- Ease of defending it by making the areas around it suitable for the necessary defense means.

c- The entrance must be narrow so that it can be closed with a window to protect the harbor from acts of sabotage, with considering the distance must be enough for allows the exit of ships quickly.

d- The harbor must have more than one entrance, as the enemy may sink one of the ships at the entrance to prevent the movement of navigation in/out the harbor.

e- The harbor must have a large water area to accommodate the largest number of naval vessels and allow them to carry out rapid movements.

f- There must be enough land around the harbor to establish the necessary land installations.

g- It is not required to have a large number of berths in such harbors, as it is rare for these ships to be left on the docks except at the time of supply, but it must be provided with maintenance docks, both dry and floating, repair workshops and stores of spare parts.

Fig (3-4) Naval Norfolk harbor, Virginia, USA (Military harbor)

3- **Fishing Harbors:** Its type of commercial harbor with usually small size, but the following conditions must be met in them:

a- It located in places where a lot of fishing on the coasts.

b- The water area in the harbor must be enough to accommodate all the expected ships at one time.

c- Width of entrance must be enough for Four fishing ships at once, and must be not less that (50m).

d- It is be connected to interior of country by a network of roads and railways so that the fishes can be transported immediately to the distribution centers before they are damaged.

e- The harbor must have long berths enough for ships to unload their fishes at once as soon as possible.

f- This type of harbor requires sufficient land areas to set up repair workshops, places to spread and repair fishing nets, as well as areas for fish processing and all additional industries that are necessary.

Fig (3-5) Half Moon Bay California, USA (Fishing harbor)

4- **Free-zone Harbors:** It is type of commercial harbor in which goods can be loaded and unloaded without paying any customs duties. Ships can enter and exit without inspections, and without paying customs duties except for goods that are transported outside the customs area into the country, - as for the goods that remain inside the free-zone harbor for re-processing and then transferring them to another harbor, no fees are paid.

The establishment of these harbors attracts commercial ships that pay entry fees, berth returns, fuel prices, supplies, rents for cranes, maintenance docks, and workers' wages, and these harbors are prepared with warehouses for storing transit goods. Factories for re-manufacturing and packing of good, figure (3-6) shows the biggest free zone in the World.

Fig (3-6) Free zone in Dubai Harbor (Free-zone harbor)

5- **Safety Harbors:** Ships used it when faced strong storms, or when supplies or fuel were needed. This type of harbors was almost non-existent due to the great progress that had occurred in shipbuilding, so that most of them were able to withstand storms at sea, in addition to Harbors are expanding, and accordingly, there is no longer a need to establish harbors for this purpose.

This is in addition to the harbors that were established for use as a helipad, and this type also ceased to exist.

In all types of harbors, regardless of the purpose of their establishment, harbor planning includes both of the following elements:

3.2: Planning of Harbors:

3.2.1: Navigation Lanes Rules:

Sometimes the depth of water is changed related to some submerged islands, it becomes necessary to determine a navigational road leading to the entrance to the harbor, called the navigational lane, the depths of the water at a sufficient length for the safety of navigation, and the necessary works in the design of navigational lanes are:

1- **Navigational Lanes Planning:**

In planning navigational lanes, it is considered that the movement of ships along the lane is easy and does not require many maneuvers, and that strong currents do not intersect with it. Also, it is considered in planning that the passage is not subject to sedimentation, which reduces the depths of water in it.

If a change in the direction of the path is required in its length, it is preferable to do so by means of a group of short tangents that connect with short curves as well. If the angle is 30 degrees, then the length of the tangent should not be less than 300 meters, and the radius of the curve should not be less than 900m, but in most cases the distance of vision is not less than About 800 meters.

The width of the navigation channels (or corridors) should increase at the bends to facilitate navigation, several methods are followed in this case, one of that is increase the navigation channel width at the intersection point of the two inner tangents by 3.3 meters for every one degree of the angle of change in direction.

The sides of the navigational lanes are identified at their beginning and along their extension by signs consisting of floating bodies that are fixed almost in their places by attaching them to hooks or concrete blocks resting on the bottom, as shown in figure (3-7). These bodies are called buoys, it must be distinguished day and night by providing them with illuminated lights.

Fig (3-7) Floating Signs (Buoys)

2- **Width of the Navigational Lane:**

The width of the navigation lane is determined on the basis of the distance between the two lower feet of the slopes of the sides, or it is the width in which the bottom is at the design depth, and the width of the navigation corridor depends on the following factors:

a- Width and speed of ship.

b- If it allows passing one ship in a row or two ways for two ships.

c- The depth of the bottom in the navigation lane.

d- Layout of lane, whether it is bordered by sides with a full depth or if it is in a wide sea.

e- Balance the sides of the depth in the lane.

f- Wind, waves and currents.

Some navigation authorities recommend that the width of the lane be equal to Four times the width of the ship if the lane is used to cross ships in a row, while the width is equal to six times the width of the ship if any ship is allowed to cross another ship in the same lane. Based on laboratory and field studies, another method has been found to calculate the width of the navigational lane, where the total width is divided into:

- The Capacity that ship occupies while at sea:

It depends on the amount of control over the ship as explained in table (3-1), which is determined as follows:

1- Very good control of the attacking military ships.

2- Good control for military and new commercial ships.

3- Poor control for old ships.

Table (3-1) Width of lane according to type of ships

Control Level	Very Good	Good	Poor
Width of lane vs ship width %	160%	180%	220%

- The clearance between the two ships:

A minimum clearance is (30) meters.

- Clearance between ship and sides of channel:

This clearance ranges between 60% and 150% of the ship's width, depending on several factors, namely:

1- Degree of control over the ship.
2- Speed of ship, where the clearance increases as the speed is permitted.
3- Wind and currents.
4- The ability of the sides of the drilling to be eroded by the movement of ships.
5- Width increases if the sides of the shipping lane are submerged with water.

3- Depth of Navigational lanes:

It depends on several factors:

1- The maximum draft of the largest vessel crossing the runway.
2- Tides.
3- Change in water density.
4- The amount that the ship sinks due to the decrease in the water surface around it during its movement.
5- The inclination of the ship in the longitudinal direction and the transverse direction when it is intercepted by the waves.
6- The ship's full draft is not fixed, which may increase at the front or at the back according to the speed at which the ship is moving.
7- Situational factors related to the ease of movement of the ship and the efficiency of operating its engines in addition to the safety factor, and it is usually added to the depth to achieve these factors, half to one meter.

3.2.2: Harbor Entrances:

The harbor entrance is open in the breakwater to allow ships to enter or exit the water area, while the harbor may have more than one entrance, which is a necessary condition in military harbors, and the entrance design includes the following elements:

1- The entrance is located along the breakwater.

2- Determining the width that guarantees the safety of the movement of ships through it, and at the same time, it must not be more big to prevent waves into the water area of the harbor.

3- Determine the depths of the bottom at the entrance.

4- The necessary protection works for the entrance.

The main factors to choose the location of entrance are depends on winds in the area, waves and water currents and their ability to erode and settle, usually the entrance is located at the edge of the breakwater.

To determine the entrance depth, the same conditions are applied to it that were mentioned in determining the depth of the navigational corridor, considering that the waves are at greater heights at the entrance, and that the sides are defined by breakwaters.

Width of Entrance must be meets the requirements of navigation through the entrance, by ensuring the safety of the crossing ships, and that it achieves as much as possible not to enter the largest possible amount of wave energy, the number of entrances in a single harbor, the depth of the bottom and the characteristics of the winds, waves and currents, the entrance width ranges between 125 meters and 300 meters depending on local factors, to determine the width of the entrance according to the size of the harbor, as explained in table (3-2).

Table (3-2) Width of Entrance

	Small Harbor	Medium Harbor	Big Harbor
Width of Entrance	90m	125m – 150m	150m-250m

3.2.2: Water Area of the Harbor:

The water area of the harbor is determined according to the number and sizes of the ships, as well as according on depth of water in front of the harbor, In general, the required space is sufficient to establish the berths necessary for the berths of used ships for the harbor, in addition to a space sufficient for the rotation of ships so that they can navigate out of the harbor with their engines, and the water area of the harbor may increase to the extent that is sufficient for ships to dock on their anchors when they come to the harbor pending procedures Required or waiting to prepare the berth to which the ship will land, or for loading and unloading some goods to and from terminals.

Since the area required to establish the berths is limited to the size of trade and the number of ship sizes, the effective factor in reducing the water area is due to what is allocated to the circulation of ships, and ships can change their directions in one of the methods. The choice of one of the methods used for the circulation of ships inside the harbor depends on the aforementioned factors as well as on the amount of movement within the harbor, figure (3-8) shows the area of water in front of harbor.

Fig (3-8) Water area in front of a harbor

4. Planning of Breakwater:

Breakwaters are created to protect the water area of artificial harbors and semi-natural harbors from waves, the position of the breakwater is determined according to the following factors:

1- Direction of waves in the region, especially high height waves.
2- Shape of the shoreline in the region and the expected expansion of the harbor.
3- The water area that should be available in the harbor.
4- Topography of the seabed in the region.

The water area should not be less than the needs of the harbor, and it should not be large to the extent that it creates internal waves, and the water area may be greater than the required amount if there are shallow areas over which the breakwater can be established, so that its sector can be minimized and thus the costs are reduced. Its construction, however, the waves are an important factor in planning the barrier in terms of shape and in terms of the location of the entrance, and the direction of the waves is seldom fixed in the area, and therefore it is preferable that the harbor be two entrances so that ships use the least turbulent entries, and the figure shows different models for planning breakwaters and locations the entrance.

In general, the planning of breakwaters is what expands the distance between the two barriers after the entrance quickly and then continues to widen to the base, just as one of the barriers protrudes from the other to protect the entrance, and that sharp angles are not allowed.

Fig (3-9) Water breaker in AL Fao Harbor

Chapter four

(Protection of harbor against wave)

Introduction:

The waves cause a great force that affects the marine structure, as they cause erosion of the facilities and the change of the shore semblance and cause many troubles in loading and unloading, as well as, cause the body of the ship to hit the wall of the piers to a degree that represents a danger to all of them .

Therefore, protecting the harbor from the impact of the waves has become necessary. Below the Hydraulic Structures Used to Protect Harbor**:**

4.1 Breakwater:

They are facilities constructed inside the sea, either parallel to the shore line or connected to the shore, in order to reduce wave energy Arriving to the shore, then, to protect the shore from erosion as a result of wave attacks or for the purpose of determining the water area of the port and protecting it from waves and currents, and thus the ships are safely moored to complete the shipping and transfer of goods, show figure (1) , Structurally, breakwaters can be divided into two main types, namely:

1.Fixed Breakwaters.

2.Movable Breakwaters.

Fig(4.1) Breakwater

4.1.1 Fixed Breakwaters:

This type of barrier is constructed to protect large ports that are exposed to high waves, as it is divided into:

4.1.1.1 Vertical Breakwaters(Up Right Breakwaters):

This type of barrier is based on the reflection of waves without dispersing their energy and thus the wave reaches the barrier at its full height and collides with the barrier, so the water rises near the barrier by the height of the wave surface, the wave bounces back and meets the wave falling on the barrier forming a standing wave that is almost twice as high as the wave falling on the barrier and thus it is tried to be The level of the barrier surface towards the sea is higher than the level of the highest tide of the height of the incident wave on the barrier added to the height of the wave surface

Crest level>Run up=1.5H+TR   TR= Tidal wave

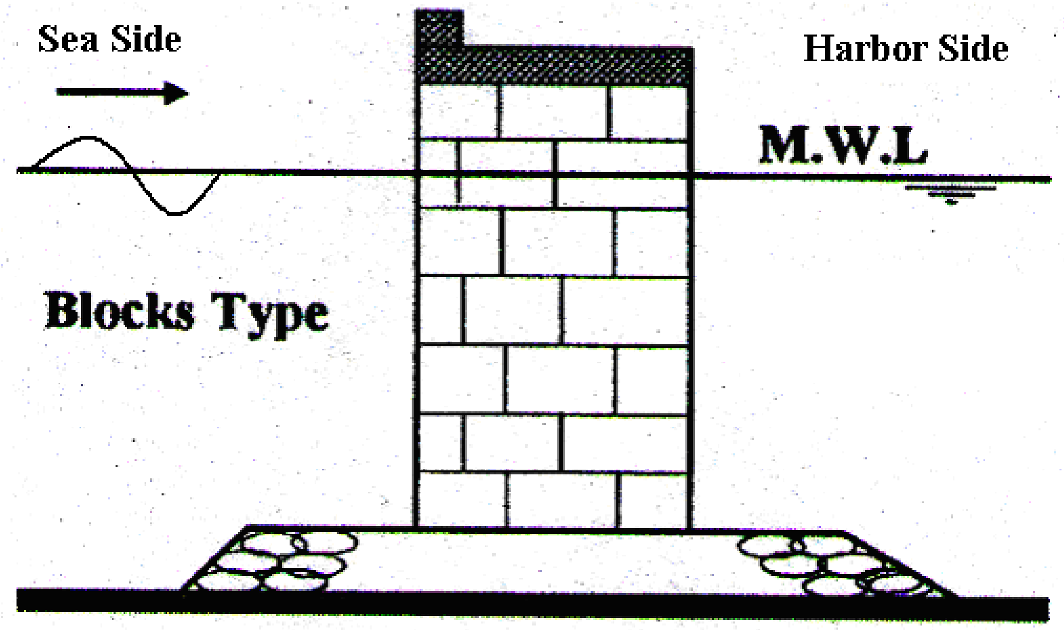
These types of barriers are installations that have vertical edges towards the sea that work to respond to the falling waves They have different types as shown in(Figure 4.2) They are:

A-Block Type Breakwaters

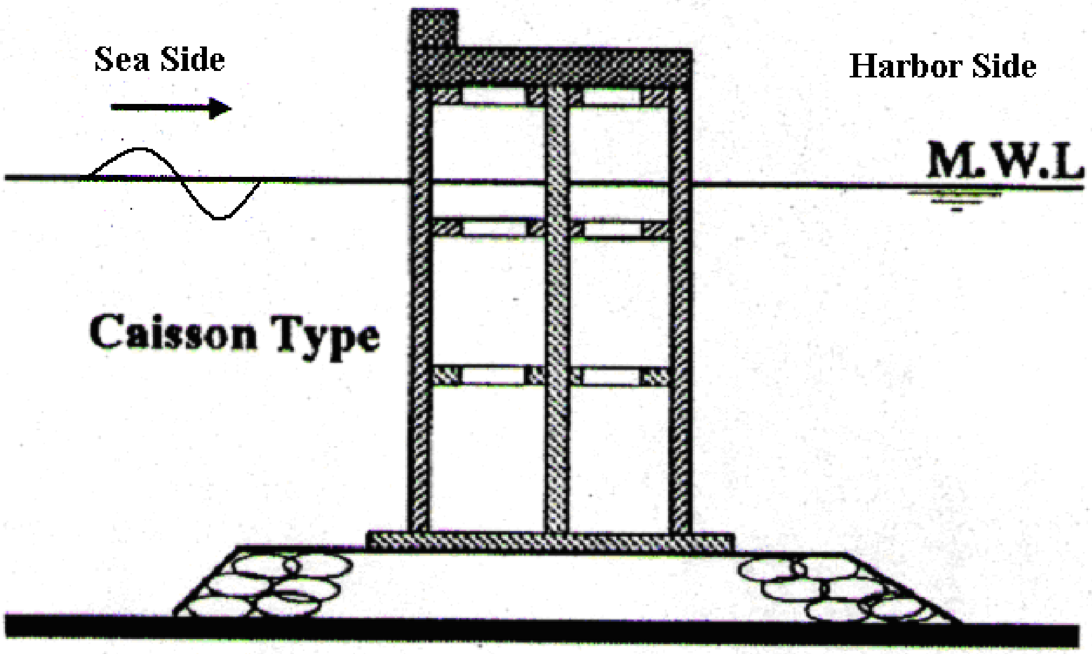
B-Caisson Type Breakwaters

C-Sheet pile Breakwaters

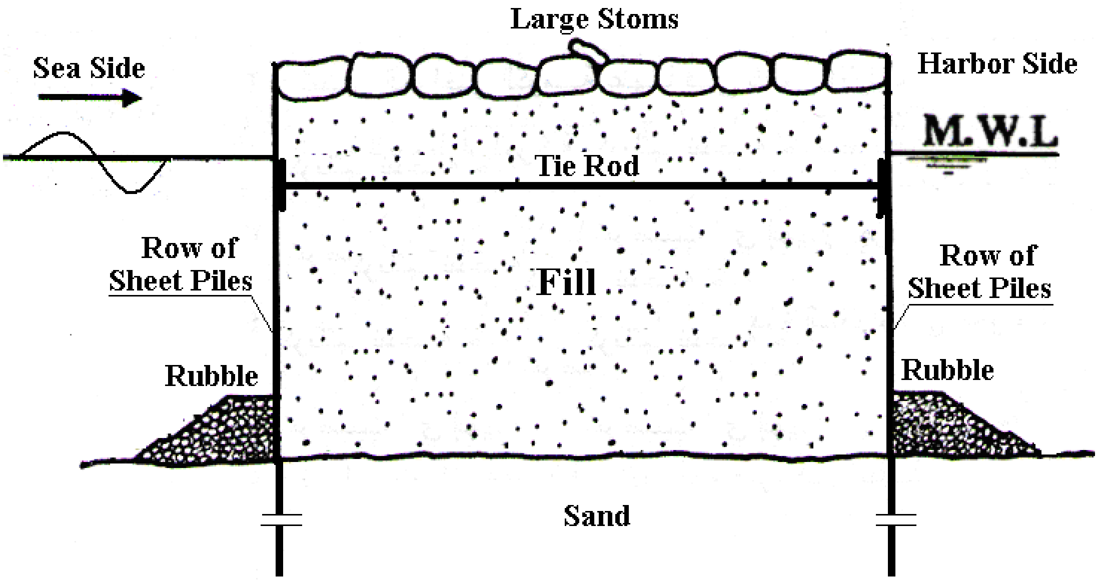
D-Cellular Type Breakwaters



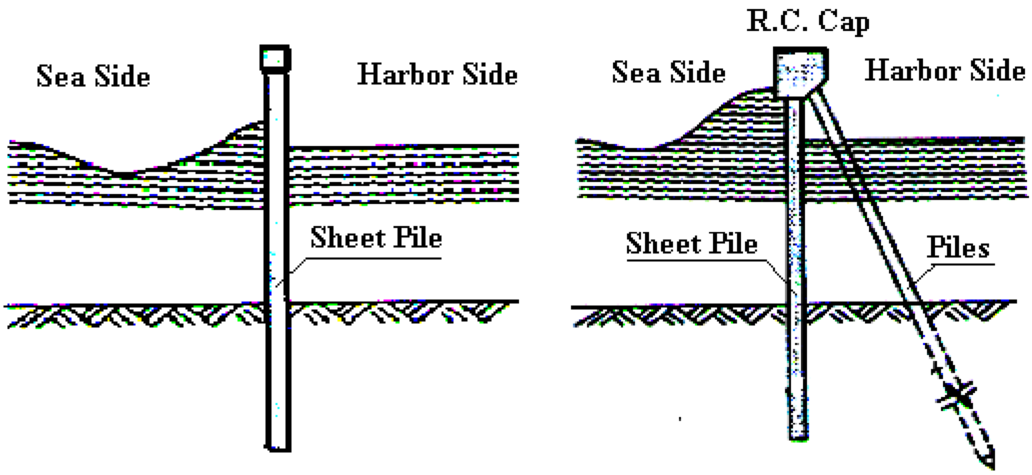
fig(4.2.A) Block Type Breakwaters



Fig(4.2.B) Caisson Type Breakwaters



fig(4.2.C) Cellular Type Breakwaters



fig(4.2.D) Sheet pile Breakwaters

-Advantages of vertical breakers:

1.When prayer stones are not available, this type can be used

2. The construction time is small

3. It gives a large water area to the port

4. It is possible to dock on small ships and service units

5. It is possible to make accurate design for all parts

6. Low maintenance costs

7. Can be constructed in difficult marine conditions

-Disadvantages of vertical breakers:

1. Needs high tolerance soil

2. Needs to level the bottom below the structure before construction

3. The collapse of the origin is due to the subsidence of the bottom below the origin due to the high loads

4. Restoration and maintenance are difficult

5. Needs high building skills

4.1.1.1.2Design of vertical breakwaters:

To design such barriers**, the forces affecting them must be identified, which are:**

1.Wall Weight = Dry Weight + Submerged Weight.

....)2.Live loads on the surface of the barrier:(cranes, wagons, lorries,

3.Horizontal forces resulting from wave shocks

4.Forces resulting from the anchorage of navigational units on the checkpoint if it is used as a dock

-Wave force on vertical walls:

1.Non-breaking Zone:(Sain flou Formula) $\frac{H}{d} < 1.0$

$$\boldsymbol{h_0} = \frac{\boldsymbol{k\,h^2}}{\mathbf{2\,tanh}(\boldsymbol{kd})} \_\_\_\_\_\_\_\_\_\_\_\_\_\_(1)$$

$$\boldsymbol{p_2} = \frac{\boldsymbol{\gamma H}}{\boldsymbol{cosh}(\boldsymbol{hd})} \_\_\_\_\_\_\_\_\_\_\_\_\_\_(\mathbf{2})$$

$$p_1 = (\gamma d + p_2)\left(\frac{H+h_0}{H+d+h_0}\right) \_\_\_\_\_\_\_\_\_\_\_\_\_\_(3)$$

K=$2\pi \backslash L$______________(4)

$\gamma = 1.03t\backslash m^3$

$h_0 = height\ wave\ up\ the\ water\ surface$

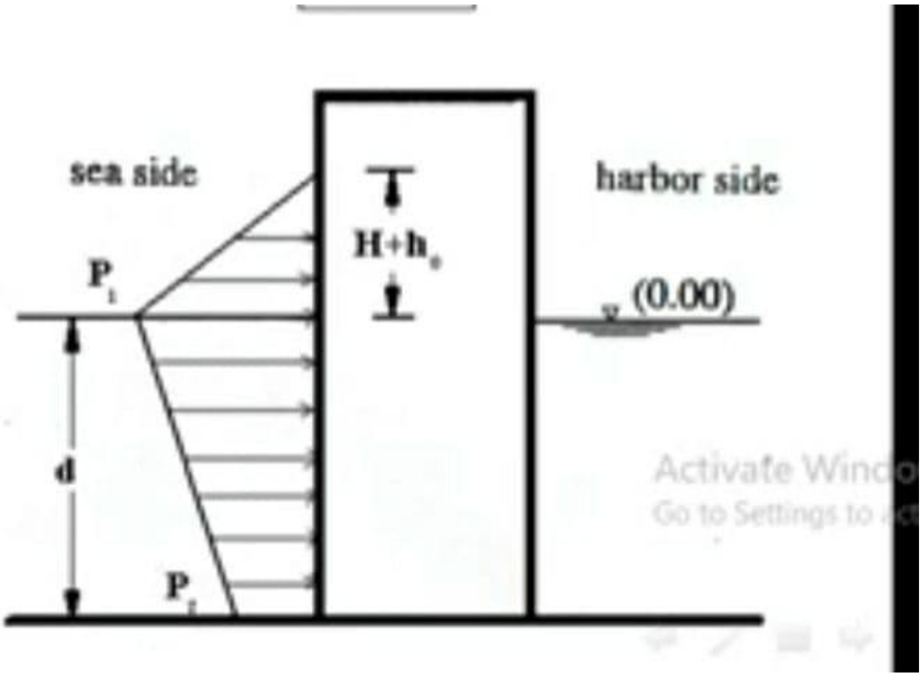
$p_1 = pressure\ up\ water\ surface$

$p_2 = pressure\ under\ water\ surface$

H = Wave Height

L=length of wave

d=depth of water



Fig(4.3) **Non-**breaking **Zone**

2 . breaking Zone:(Miniken formula)) $\frac{H}{d} \geq 1.0$

$P_{st} = \frac{\gamma H}{2}$ ________(5)

$P_{dyn} = \left(K\frac{d_1}{d}\right)(\gamma H)\left(\frac{d+d_1}{2}\right)$____________(6)

K=$2\pi \backslash L$____________(7)

$P_{st} = water\ pressure\ static$

$P_{dyn} = water\ pressureDynamic$

H = Wave Height

d =depth of water

$\gamma = 1.03 t \backslash m^3$

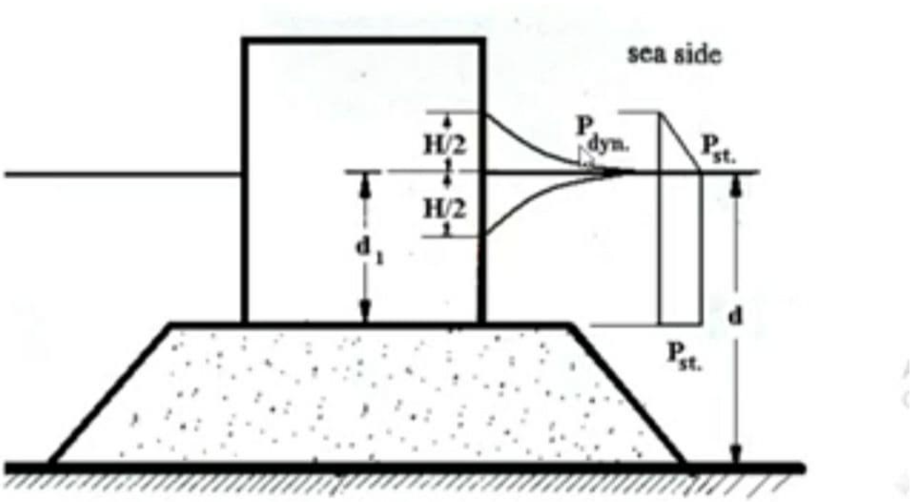


Fig (4.4) breaking Zone

3.Chech of stability of Breakwater:

- Check of sliding:

(F.S)sliding= $\mu \ \frac{N}{F}$ __________(8)

F.S$\geq$ 1.5

F=wave force(horizontal force)

N=total vertical force

μ=the friction coefficient in Table(1)

Condition	μ
Concrete on concrete	0.5
Concrete on rock bed	0.5
Concrete on soil bed	0.5
Concrete on rubble bed	0.6
Rubble mound on ruble	0.8

Table (1) friction coefficient

-Check of overturning:

(F.S)overturning=$\frac{N.x}{F.y}$ ______________(9)

F.S ≥ 2

N= total vertical force

X,y are the arm of normal and lateral force from the turning point

-Check of stresses:

$$f = -\frac{N}{b}\left(1 \pm \frac{6e}{b}\right) \leq fall \qquad (10)$$

b =width of Breakwater base

e=The eccentricity *e=(b\2)-(*$\frac{M_{net}}{N}$)

$M_{net} = M_{st} - M_{ov}$

$M_{st} = N \bullet x$

$M_{ov} = F.y$

F=wave force

N=Total vertical force

$M_{net} = moment\ net$

$M_{st} = moment\ static$

$M_{ov} =$ moment overtrning

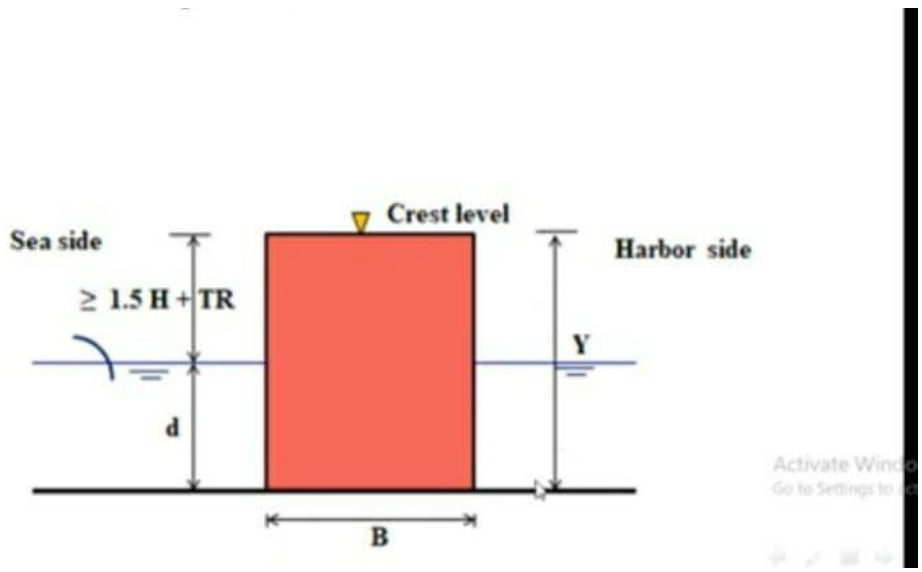
2.Empirical Dimension of Break water:

Crest level ≥ Run up=1.5H+TR…………………(11)

Height of Breakwater =Y=Crest level+high water depth……….(12)

Width of Break water =B=(0.7-1)×Breakwater height………..(13)

ix



4.1.1.2 Rubble Mound Breakwaters(Inclined Breakwaters):

This type of barriers arises from the small stones resulting from the remnants of quarries (the core of the barrier), where they are covered with a layer of natural or heavy concrete stones (shield layer) to resist waves and to ensure that the core stones do not leak from the voids of the shield layer are placed between the two layers natural stones of medium weights called the filter layer . And the barrier has a slight inclination towards the sea so that the waves break at this mile and lose most of their energy, while the slope of the barrier towards the port is a severe one where there are no waves. It consists of three main layers:

A- The outer protective layer (Armor Layer) and the weight of the stones is large, which is either natural or

.Industrial

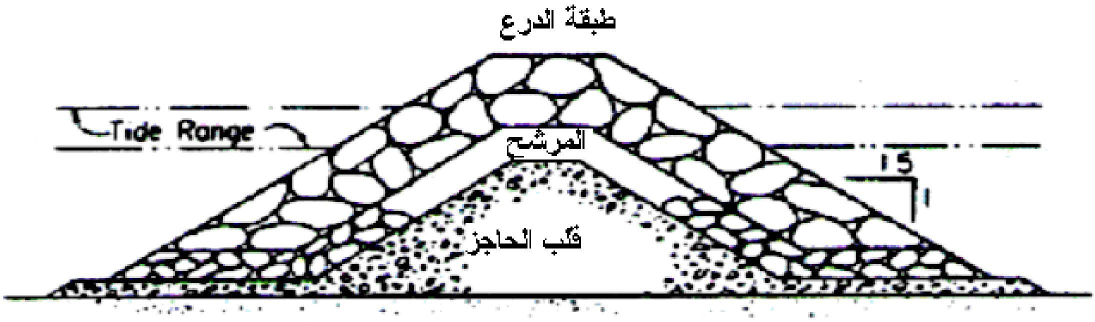
B - the filter layer and the weight of the stones is smaller and arises from .the natural stones

C- The core of the septum and the weight of the stones in it is small and .arises from natural stones

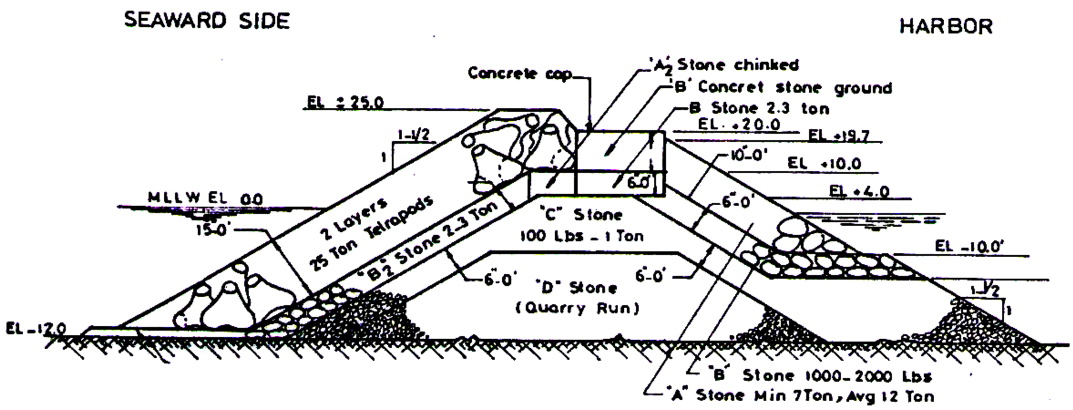
This type of barrier absorbs the wave energy by tilting towards the sea and creates either a whole of stones

Natural stones as in(Figure 4.6A) or the diaphragm core and the filter are formed from natural stones and the protective layer is formed

External of artificial stones made of regular concrete or lightweight concrete, as in(Figure 4.6B)



(Figure 4.6A) the filter are formed from natural stones



(Figure 4.6B) the filter are formed from artificial stones

This type of barrier has advantages and disadvantages, including:

Advantages of Rubble Mound Breakwaters:

1. Easy to constrected

2. It is constructed on soils of medium carrying capacity

3.It is raised on an uneven bottom

4.Maintenance is easy

5.We get a calm water space in and out of the harbor

Disadvantages Rubble Mound Breakwaters:

1.It needs a lot of stones

2.Difficult to create in high waves

3.Reduces the water area of the port

4.Mooring boats is difficult for this type

5.Maintenance is expensive

6.The construction time is large

Design of Rubble Mound Breakwaters :

To design these barriers, it is necessary to know the following:

1.The inclination of the sides of the barrier towards the sea and also towards the water area of the port

.2.Weights, types and number of layers of stones forming each layer

3. thickness of each layer (shield layer and filter layer)

4.The different levels of the barrier (surface level at sea, port, road level, core surface level - filter surface level)

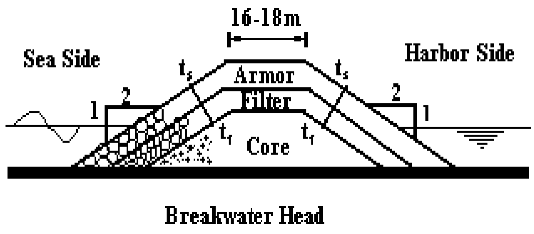
5.The width of the barrier surface and the determination of the road width .on the barrier

.6.Verify the integrity of the stresses on the bottom soil below the septum

7.Verify the stability of the sector against collapse by slipping

Side Slope:

Figure (4.7) shows two different sectors in the breakwater, one at the head of the barrier and the other in the stem of the barrier, where the .figure shows the preferred lateral tendencies of the barrier



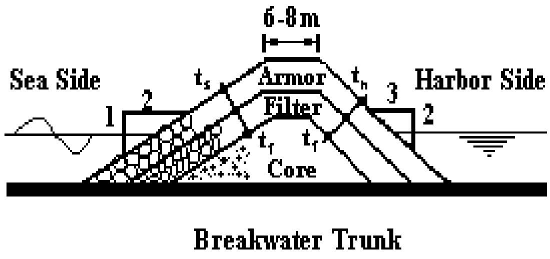


Figure (4.7) shows two different sectors in the breakwater

- Layer weights and thicknesses:

1.Filter Layer:

W_{filter} = W_{armour} ÷ (10)……… $K_D \geq 12$
W_{filter} = W_{armour} ÷ (5) ……..$K_D < 12$ W_{armour} : Natural
$t_{\,armour}$ =$K_{\Delta.NnV^{1/3}}$ ________________ (14)
V = $\frac{W_{filiter}}{\gamma_s}$______________(15)

2.Core Layer

W_C = W_{armour}÷ (200 - 6000) , W_{armour} : Natural______________(16)
Core Surface Level = +H/2 For End Method ______________(17)
= -H/2 For Floating Method

3. Crest Level and Width

•Level of armor layer = M.S.L. + H/2 + t_{filte} + $t_{\,armour}$ _________ (18)
•Maximum Run up = 1.5 H + TR__________(19)

(Over Topping)

•Take the Road Width over the breakwater = 18.0 m at Head

= 8.0 m at Trunk

4. Armor Layer

$$W_{armour} = \frac{\gamma H^3}{K_D (S_r - 1)^3 \cot\alpha} \qquad \text{\_\_\_\_\_\_\_\_\_\_\_\_}(20)$$

$$t_{armour} = K_\Delta \, n \, V^{1/3} \qquad \text{\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_}(21)$$

$$V = \frac{W_{armour}}{\gamma_s} \qquad \text{\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_}(22)$$

γ_S= 2.2t/m3 Natural Blocks

= 2.4 t/m3 Artificial Blocks

Sr =$\frac{\gamma_s}{\gamma_w}$, γ_w = 1.03 t/m3

$\cot\alpha$ =2.0

H = Wave Height

n = 2 Two Layers

Wfilter=weight of filter

$W_{armour} = weight\ of\ armour$

W_c=weight of core

$t_{armour} = thiknes\ of\ armour$

K_D and K_Δ From Tables (2) and (3)

Armor Units	n	Placement	Layer Coeff. (K_Δ)	Porosity (P) %
Quarry Stone (Smooth)	2	Random	1.02	38
Quarry Stone (Rough)	2	Random	1.15	31
Quarry Stone (Rough)	≥ 3	Random	1.10	40
Cube (Modified)	2	Random	1.10	47
Tetrapod	2	Random	1.04	50
Quadripod	2	Random	0.95	49
Hexapod	2	Random	1.15	47
Tribar	2	Random	1.02	54
Dolos	2	Random	1.00	63
Tribar	1	Uniform	1.13	47

Table (2) The Layer Coefficient, K_Δ

Armor Units	n	Placement	Structure Trunk		Structure Head		cot α
			K_D		K_D		
			Breaking Waves	Non-Breaking Waves	Breaking Waves	Non Breaking Waves	
Quarry Stone (Smooth)	2	Random	2.1	2.4	1.7	1.9	1.5 to 3.0
Quarry Stone (Rough)	2	Random	3.5	4.0	2.9	3.2	1.5
					2.5	2.8	2.0
					2.0	2.3	3.0
Quarry Stone (Rough)	≥ 3	Random	3.9	4.5	3.7	4.2	1.5 to 3.0
Cube (Modified)	2	Random	6.8	7.3	-	5.0	1.5 to 3.0
Tetrapod and Quadripod	2	Random	7.2	8.3	5.9	6.5	1.5
					5.5	5.1	2.0
					4.9	4.1	3.0
Hexapod	2	Random	3.2	9.5	5.0	7.0	1.5 to 3.0
Tribar	2	Random	9.0	10.4	8.3	9.0	1.5
					7.8	8.5	2.0
					7.0	7.7	3.0
Tribar	2	Uniform	12.0	15.0	-	9.5	1.5 to 3.0
Dolos	2	Random	22.0	25.0	15.0	16.5	2 to 3
					13.5	16.0	3

Table (3) The Equilibrium Coefficient, K_D

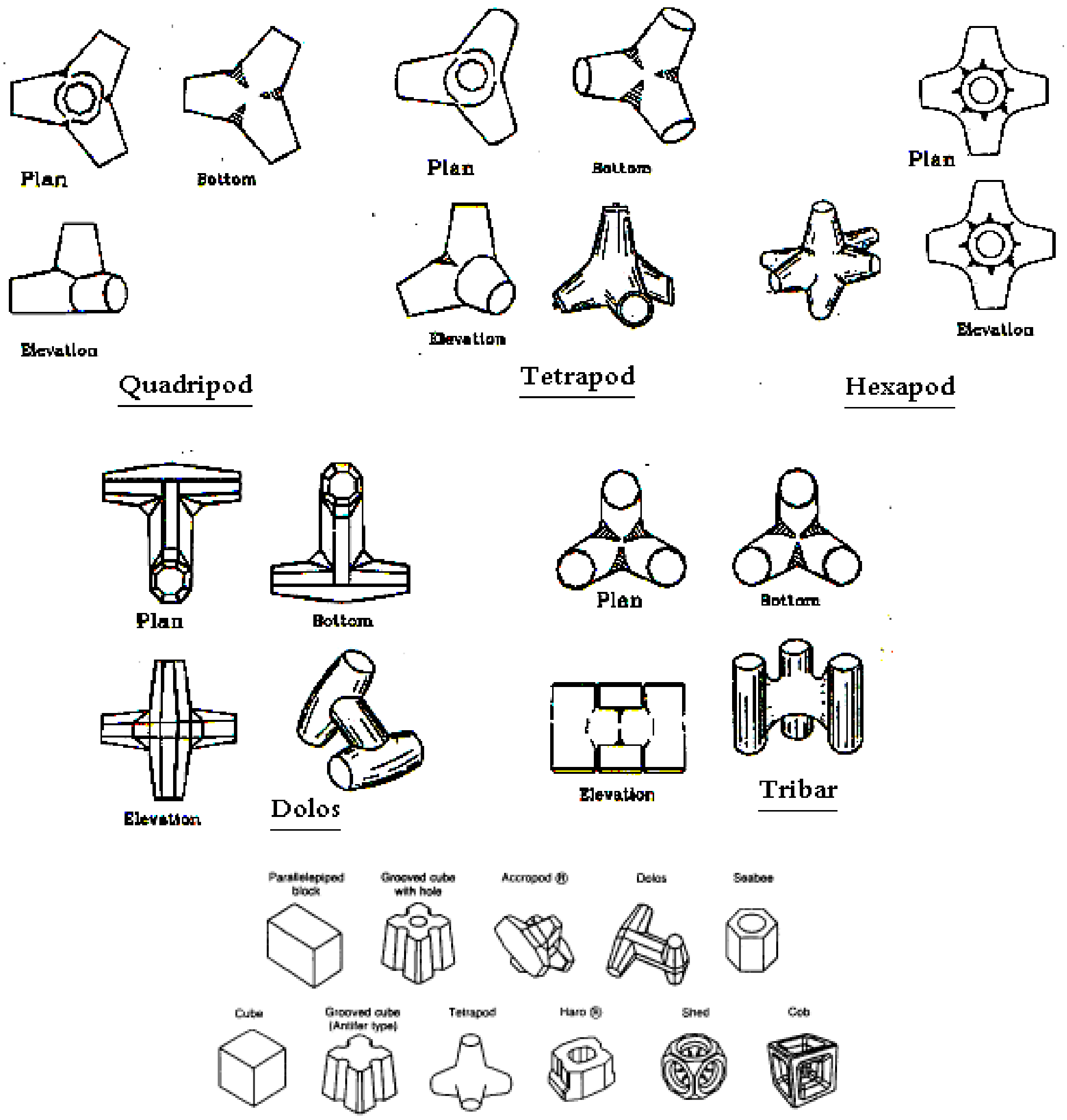
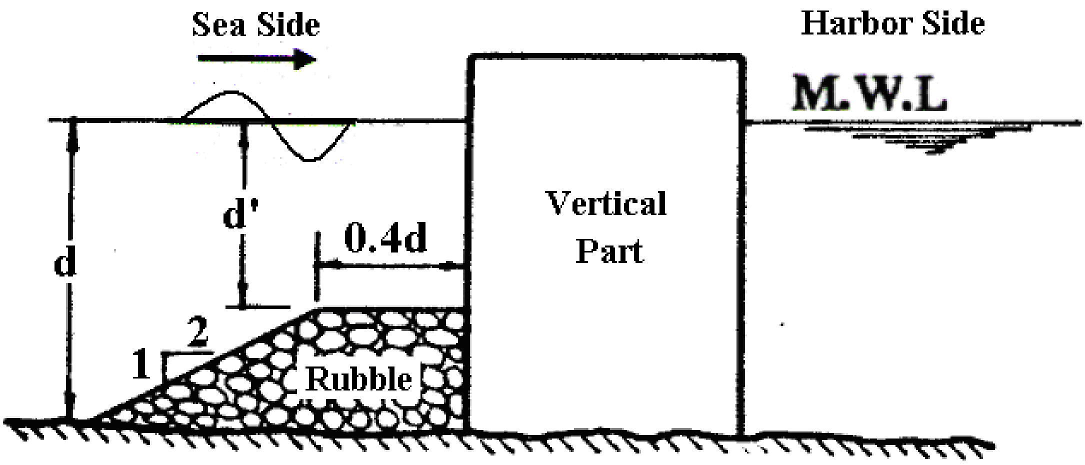


Fig. (4.8) Some common forms of artificial stones

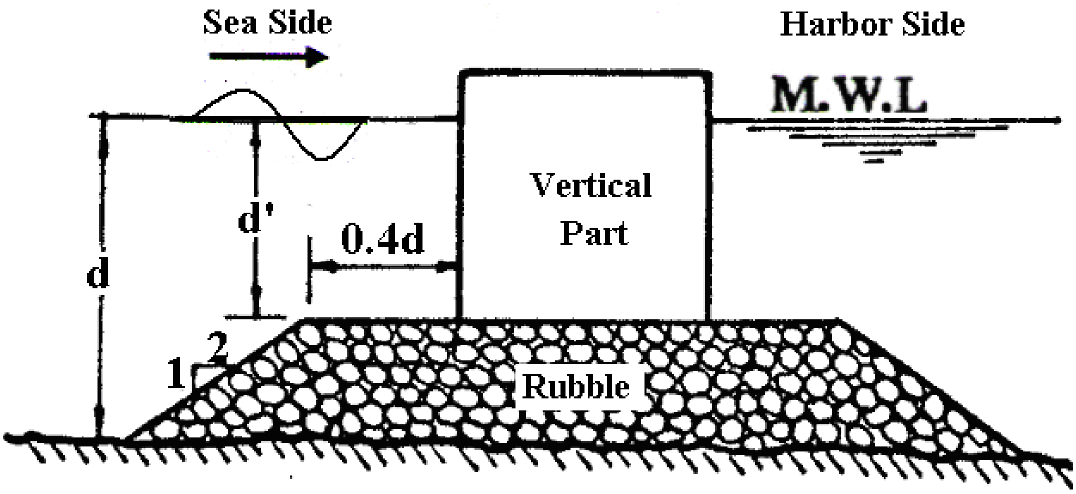
4.1.1.3 Composite Breakwaters:

It is a mixture of vertical barriers and bulk barriers, where it is used when the depths of water are large, and in this type of barriers the comic part is either to protect the head of the vertical part from slashing (Toe Protection as in Figure 4.9) or is used as a basis for the vertical part of the Foundation for the transfer of interpretations

On the soil safely as in Figure 4.10



Fig(4.9) Rubble as Toe Protection



Fig(4.10) Rubble as Foundation

Composite Breakwater Design:

The vertical section designs the same as the vertical breakers while the Rubble section is designed as follows:

$$W_{rubble} = \frac{H^3 \gamma_r}{{N_s}^3 (s_r - 1)^3} \text{\_\_\_\_\_\_\_\_\_\_\_\_}(23)$$

$\gamma_r = 2.2t\backslash m^3$

H =Design wave height

$S_r = \frac{\gamma_r}{\gamma_w}$ ________________ (24)

S_r = specific gravity

$\boldsymbol{\gamma_r}$ = unit weight $natural\ blocks$

$\gamma_w = 1.03t\backslash m^3$

$W_{rubble} = weight\ rubble$

$\gamma_w = unit\ weight\ sea$

$N_s{}^3 = the\ stability\ Number\ from\ chart\ (1)$

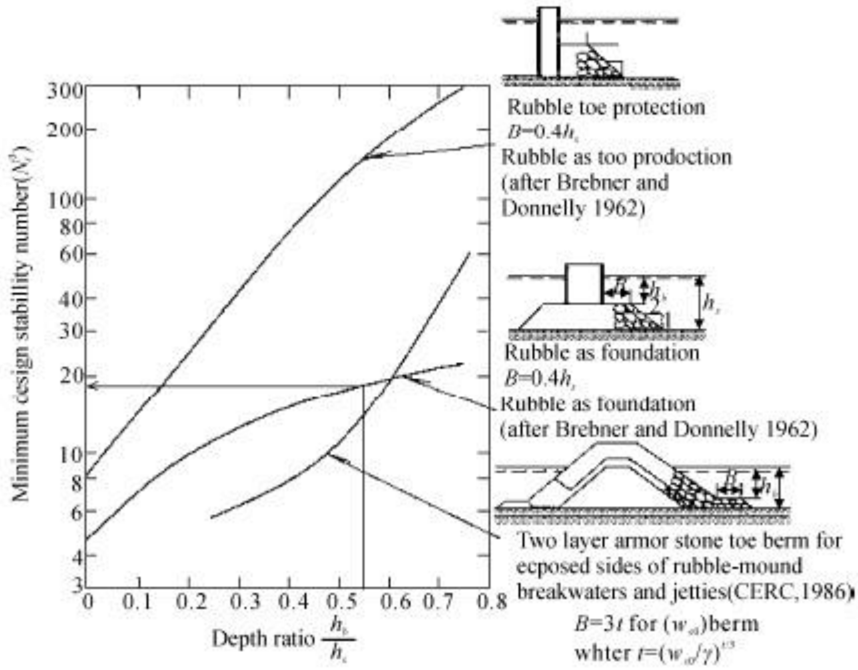


chart (1)the stability Number

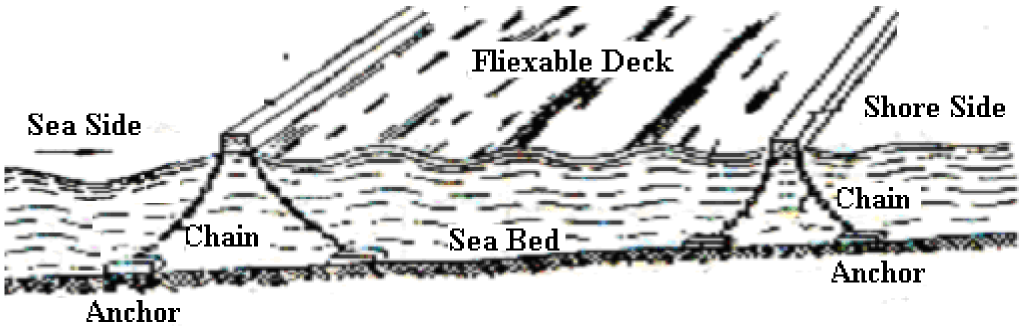
4.1.2 Movable Breakwaters:

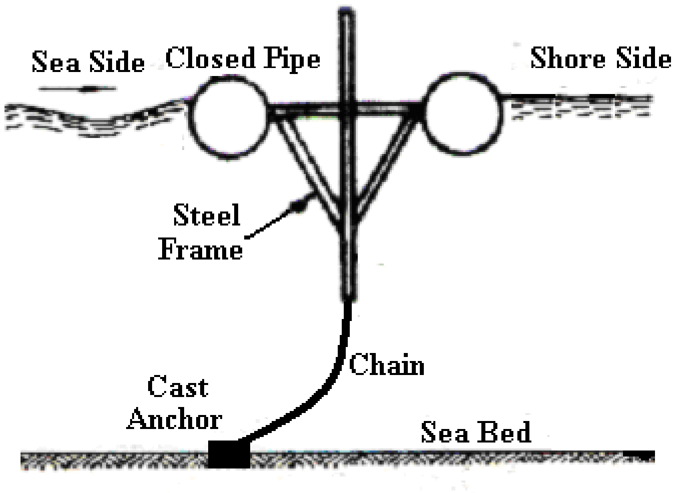
These barriers prevent part of the wave energy from reaching the intended beach or water area

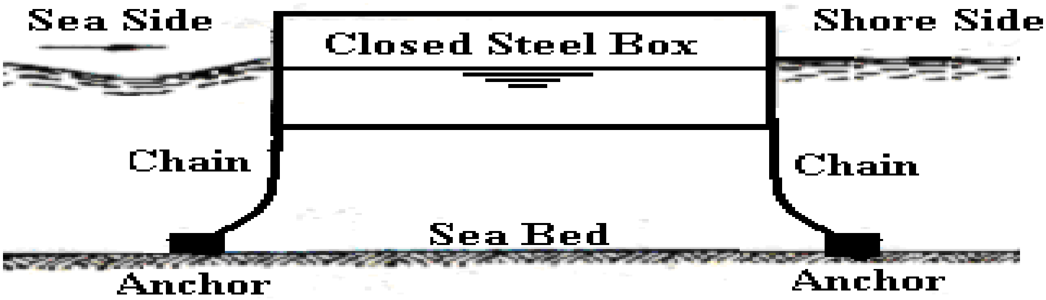
Protect it so the waves calm down. It is also called temporary barriers, as it can be used for a specific purpose, such as protecting a new port site during its construction period to protect from high waves, as it calms the waves towards the construction site, which helps the construction process safely and then moves to protect a new location. Among its types:

4.1.2.1 Floating Breakwaters

These barriers consist of bodies floating on the surface of the water and tied at the bottom with wires or chains, Fig. (4-11) such as barrels partially filled with water or closed boxes of reinforced concrete or steel and partly filled with water, rubber or wood. The draft of these barriers is determined according to the degree of protection require



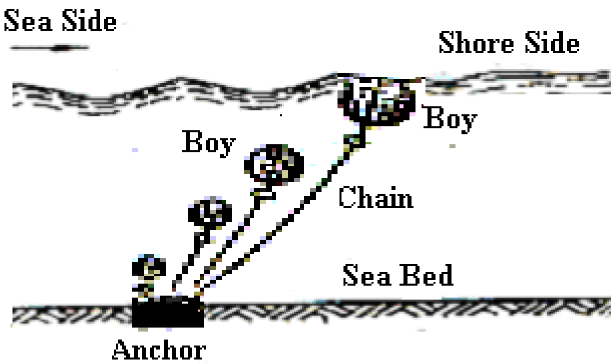


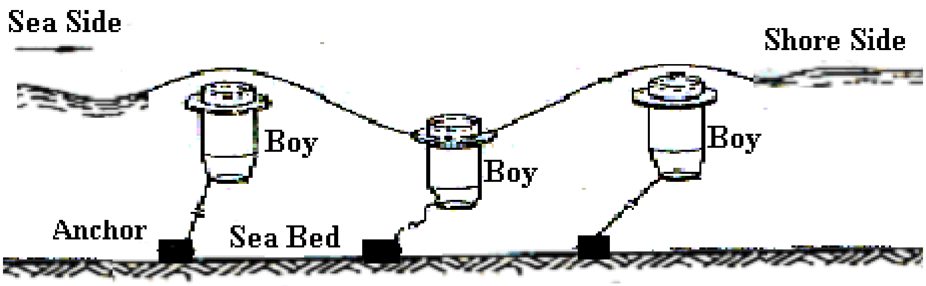


(Figure 4.11) Some types of floating breakwaters)

4.1.2.2 Submerged Breakwaters:

These barriers consist of bodies submerged under the sea surface of various shapes and types, as shown in Figure 4.12 Some of these are forms.





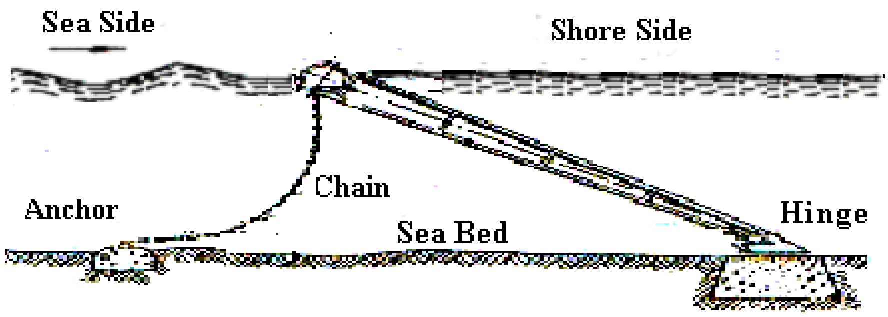
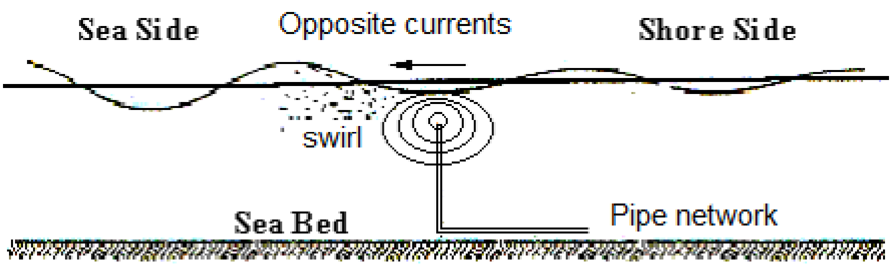


Figure 4.12 Some of Floating Breakwaters

4.1.2.3 Hydraulic breakers :

These barriers consist of a network of perforated pipes that are placed below the sea level and water is pumped into and out of

Holes, which create water currents and swirls opposite the direction of the waves, which helps to disperse part of the energy of the waves, so that only a few of them enter the water area to be protected, as shown in Figure (4.13)



Hydraulic breakers : Figure (4.13)

4.1.2.4 Pneumatic Breakwaters:

Much like hydraulic barriers except that it uses compressed air instead of water where it works

The compressed air emerging from the holes are air bubbles that are directed towards the surface of the sea and create currents

The swirls of water opposite the direction of the waves, which helps to disperse part of the energy of the waves, as shown in(Figure 4.14)

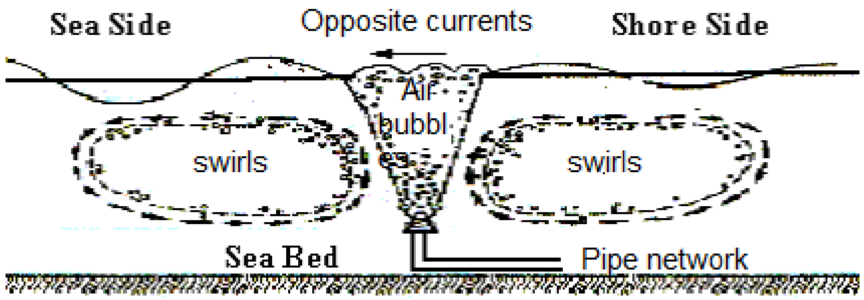


Figure 4.14 Pneumatic Breakwaters

4.2 The type of breakwater proposed for a given area is determined by a specific set of factors:

1.The depth of the sea floor along the breakwater

2.The properties of the bottom soil, its viability, and its behavior under the influence of the different forces created by the barrier

3.The natural phenomena in the region in terms of the properties of waves and the extent of tides and water currents

4.Restrictions imposed on the expansion of water areas and the amount of space the proposed barrier occupies

5.The cost of construction materials that are used in the construction of the breakwater

6.Experiences supervising the implementation, types of available equipment and manpower

Chapter Five

Types of Pier Sea and Its Design

5-1 The Concept of Pier

A pier is a structure extending outward at an angle from the shore into navigable waters and normally permitting the berthing of vessels on both sides along its entire length. A traditional definition of a pier is a raised walkway supported by pilings or pillars. A pier extends from a shore over water and supported by pillars. Fixed piers are attached to the shore and are supported by wooden pilings driven into the shore land. A pier is used to secure, protect, and provide access to ships or boats. A pier is a structure, usually of open construction, extending out into the water from the shore, to serve as a landing place, recreational facility, etc., rather than to afford coastal protection. Concrete piers are hard to beat in terms of strength. However, the cost, complexity, esthetic considerations, and environmental footprint. Piers may be open, closed, partially open or partially closed.

Piers are the wharves built at angle with the shore on both sides of a pier berths are provided see figure (5-1)

Figure 5-1 Pier

The Difference between wharf and pier

Wharf vs Pier - Both wharfs and piers are level areas to which vessels may be moored/docked for loading and unloading. A wharf is typically built "parallel" and adjacent to a shoreline which drops steeply to allow vessels sufficient draft to dock. A pier is a projection out to deeper water, typically "perpendicular" to a shore, to allow vessels sufficient draft to dock as this comparison that show the difference between them.

	BUILT ON PILES	BUILT ON FILL
PARALLEL TO SHORE	WHARF	QUAY
EXTENDING OUT FROM SHORE	PIER	JETTY

5-2 Structural Types of Pier

The three major structural types for piers and wharves are open, solid, and floating.

Open type piers and wharves are pile supported platform structures that allow water to flow underneath. Figure (5-2) illustrates the open type. Another type of an open type pier is a jack-up barge(5-3)

Solid type uses a retaining structure such as anchored sheet pile walls or quay walls, behind which a fill is placed to form the working surface. Solid type will prevent. stream flow underneath. Figure (5-4) illustrates the solid structural type

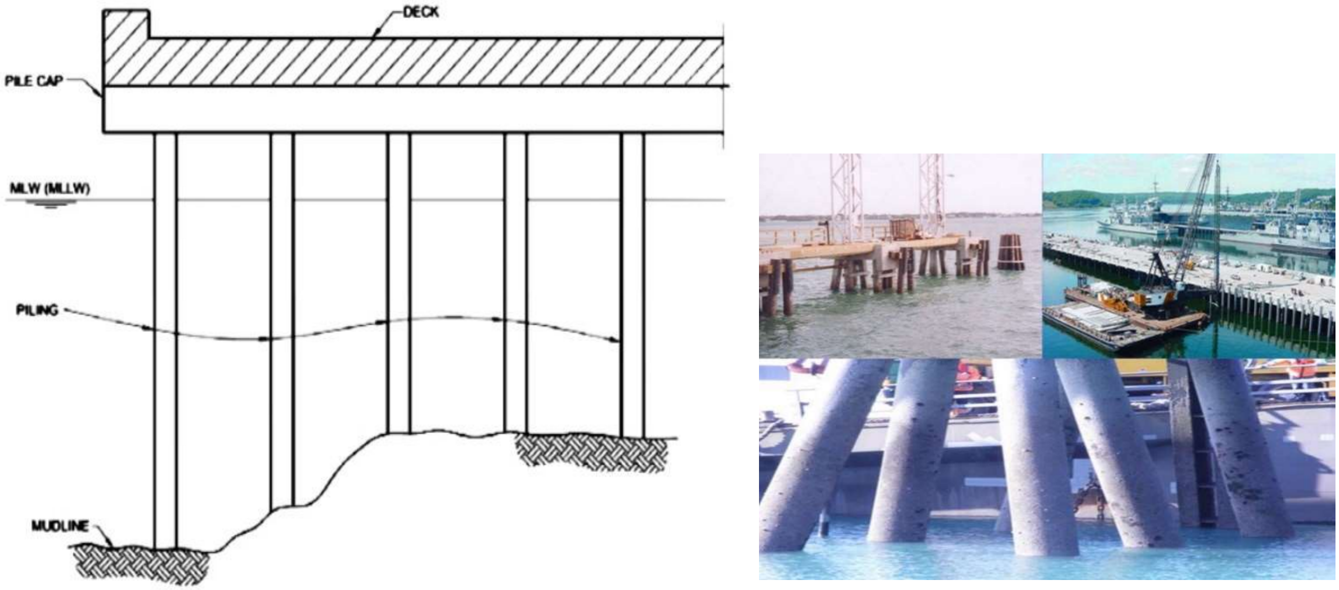


Figure 5-2 Open Type

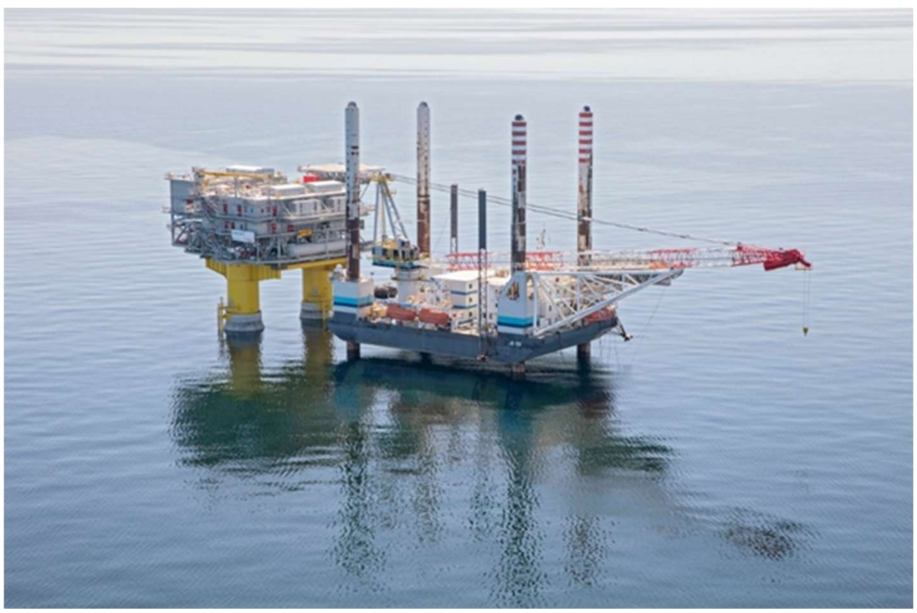


Figure 5-3 jack-up barge

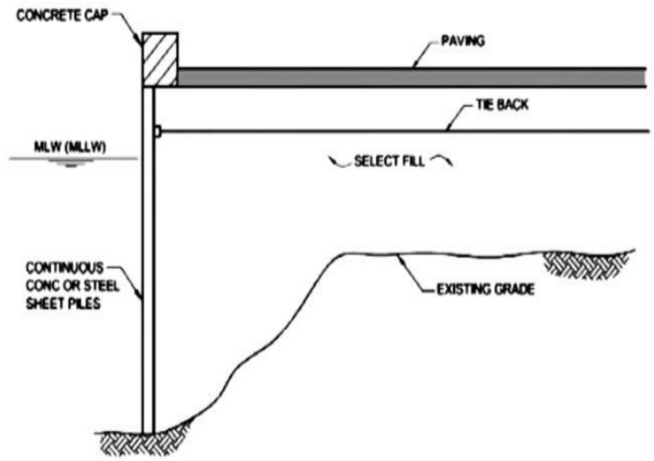


Figure 5-4 Solid Type

Floating type is a pontoon structure that is anchored to the seabed through spud piles or mooring lines and connected to the shore by bridges or ramps. A floating double deck pier is shown in Figure (5-5)

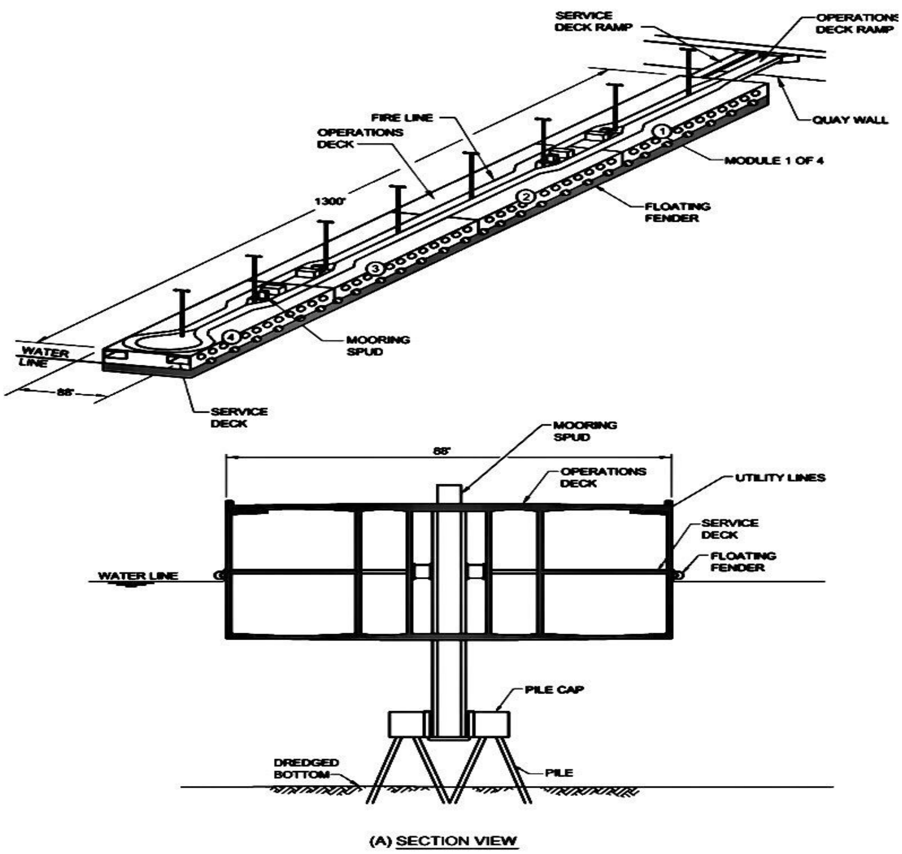


Figure5-5

5-2-1Open Type

***a. Platform on Piles and a Curtain Wall at the Inshore Face**. See Figure (5-6). The underwater slope should be as steep as possible, as limited by both constructional and geotechnical parameters, thus making the pile-supported platform narrow and more economical. In seismically active areas, where hydraulic fill susceptible to liquefaction is used for upland fill, a rock dike may be used instead of the granular fill dike to resist the lateral forces caused by liquefaction of the fill. The use of a filter fabric also should be considered at the hydraulic fill interface.

*b. Platform on Piles and a Sheet Pile Bulk head at the Inshore Face.** See Figure (5-7). The sheet pile bulkhead permits a narrower platform. The cost tradeoff between platform width and bulkhead

height should be investigated as the bulkhead may be found to cost as much or more than the pile supported platform width saved.

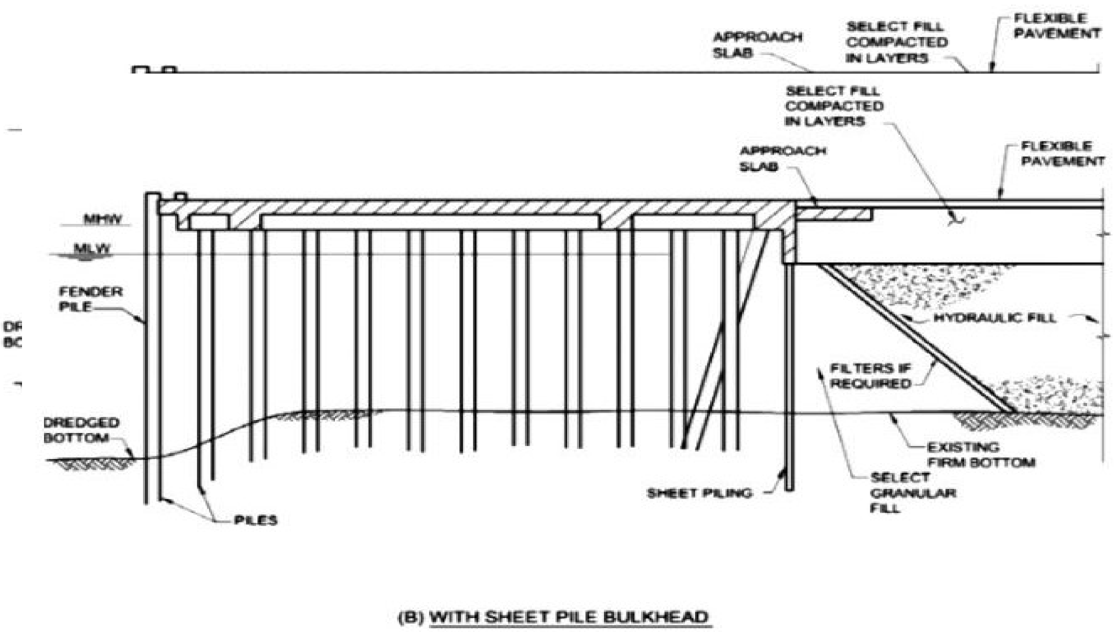


Figure 5-6 pier with curtain

Figure 5-7 pier with sheet pile bulkhead

5-2-2 Solid Type

a- Sheet Pile Bulkhead.

See Figure 5-8. The bulkhead consists of a flexible wall formed of steel or concrete sheet piling with interlocking tongue and groove joints and a cap of steel or concrete construction. The bulkhead is restrained from outward movement by placing an anchorage system above the low water level. Many types of anchorage systems can be used. The most common types in use in the United States consist of anchor rods and deadman anchors. The latter could be made of concrete

blocks, steel sheet piling, or A-frames of steel, concrete, or timber piles. In countries outside the United States, an anchorage system consisting of piles, attached near the top of the sheet pile bulkhead and extending at batters up to 1 on 1 to embedment in firm material, is often used. Rock or earth anchors consisting of high-strength steel rods or steel prestressing cables are sometimes preferred in place of the anchor batter piles. Provide granular free-draining material adjacent to sheet pile bulkheads, extending from dredged bottom to underside of pavement on grade. Grade this material to act somewhat as a filter to limit subsequent loss of fines through the sheet

pile interlocks. Placement of free-draining material should be in stages, commencing at the intersection of sheet piling and dredged bottom and progressing inshore. Eliminate mud and organic silt pockets. In general, do not consider hydraulic fill for backfill unless provision is made for the effects of fill settlement, potential liquefaction of fill in seismic zones, and high pressure exerted on sheet piling. Consider vibro-compaction for consolidation of hydraulic fill. In areas with tidal ranges greater than 4 feet (1.2 m), provide 2 inch (51 mm) diameter weep holes for the sheet piles above the mean low water level. If a waterline is located behind the

bulkhead, provide weep holes. When weep holes are used, provide graded filters to prevent loss of finer backfill material. Provide openings in pavement or deck for replenishment of material in order to compensate for loss and settlement of fill. In general, flexible pavement using asphaltic concrete is preferred over rigid pavement with Portland cement concrete, as it is more economical to maintain and better able to accommodate underlying settlement.

b- Sheet Pile Bulkhead and Relieving Platform.

The relieving platform is used in conjunction with a sheet pile bulkhead to reduce the lateral load on the sheet piling created by heavy surcharges and earth pressures. See Figure (5-9). Lateral restraint is provided by the batter piles supporting the relieving platform. A variation of this type of construction is to use only vertical piles for the relieving platform and to furnish an independent anchorage system consisting of tie rods and deadman, similar to the types specified for sheet pile bulkheads

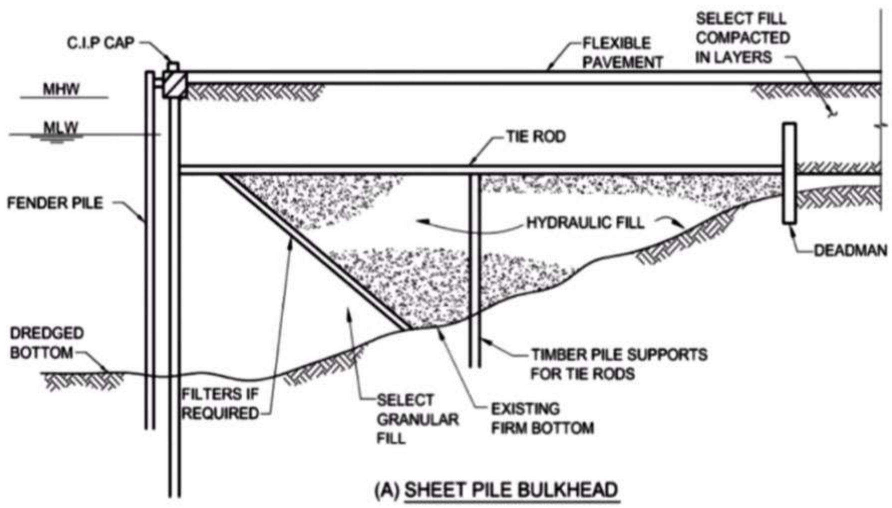


Figure5-8 sheet pile bulkhead

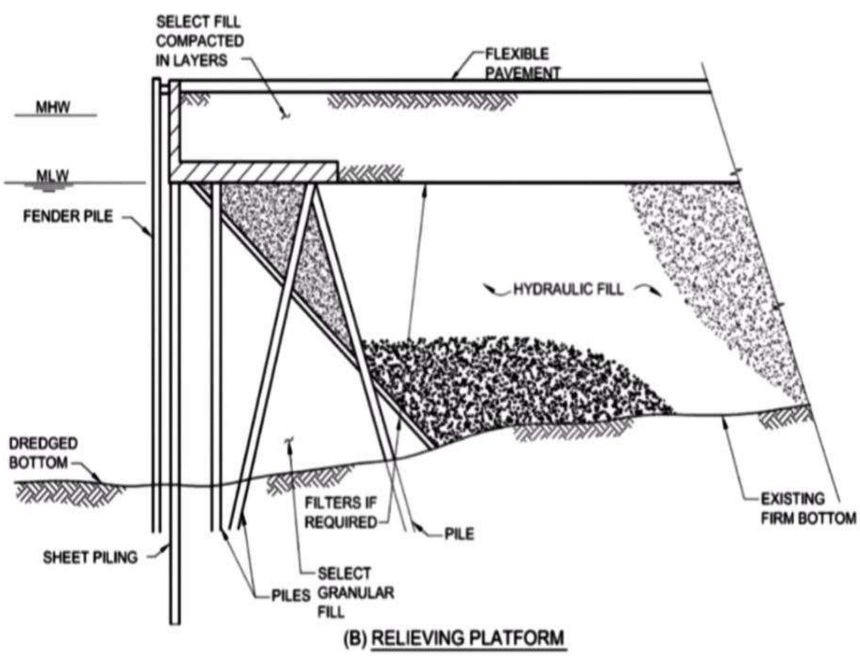


Figure5-9 relieving platform

c- Cellular Construction Consisting of Sheet Pile Cells.

Design of Sheet Pile Cellular Structures, Cofferdams, and Retaining Structures. Cellular structures are gravity retaining structures formed from the interconnection of straight steel sheet piles into cells. Strength of cellular structures derives from resistance to shear caused by friction and tension in the sheet pile interlocks and also from the internal shearing resistance of the fill within the cells. Accordingly, clean granular fill materials such as sand and gravel are usually used to fill the cells. Exercise extreme care in the construction of cellular structures because excessive driving onto boulders or uneven bedrock may cause ruptured interlocks, which can later unzip under hoop tension (from filling) and cause failures of the cell. Similarly, carefully control all aspects of fill placement, as cofferdams can unzip and cause sudden (little to no warning) failures of the cell. Compensate for movement and expansion of cells during construction of the cells and carefully control fill placement to satisfactorily maintain alignment of the face of the wharf. A concrete facing may be employed to protect the steel within the tidal zone. Cellular structures are classified according to the configuration and arrangement of the cells. The basic types are discussed below and are shown on a Figure below 5-10

*it may be

-circular

-Diaphragm

-Cloverleaf

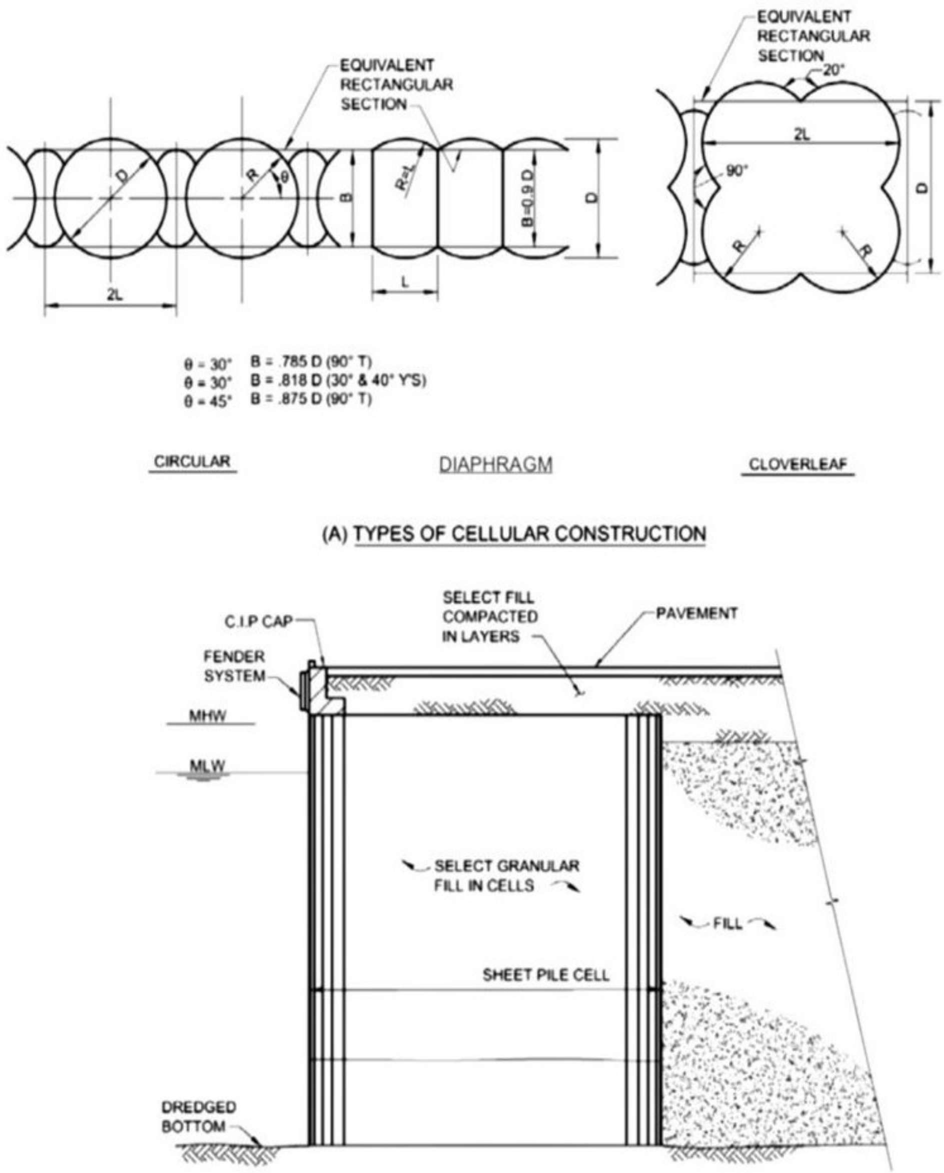


Figure 5-10 types of cellular construction

d- Reinforced Concrete Caisson.

See Figure (5-11). In this type of construction, concrete caissons are cast in the dry, launched, and floated to the construction site where they are sunk on a prepared foundation. The caisson is filled with gravel or rock and a cast-in-place retaining wall is placed from the top of the caisson to the finished grade. This type of construction is prevalent in countries outside the United States. Reinforced concrete caissons come in two variations: closed and open end caissons (caissons without the bottom slab). Closed end caissons are normally floated and then sunk in place. Open bottom caissons are barge delivered, but are easier to install and have better rotational stability due to mobilization of the infill soil shear action (see cellular cofferdams). Closed end caissons require sub base leveling and sub base stabilization for prevention of the caisson roll off, particularly in areas with extreme wave climate. Sometimes, caissons are fabricated as the ring modules, barge delivered and preassembled at the site prior to installation.

e- Precast Concrete Blocks.

See Figure (5-12). This form of solid wharf is a gravity type wall made up of large precast concrete blocks resting on a prepared bed on the harbor bottom. A select fill of granular material is usually placed in the back of the wall to reduce lateral earth pressures. This type of construction is popular outside the United States.

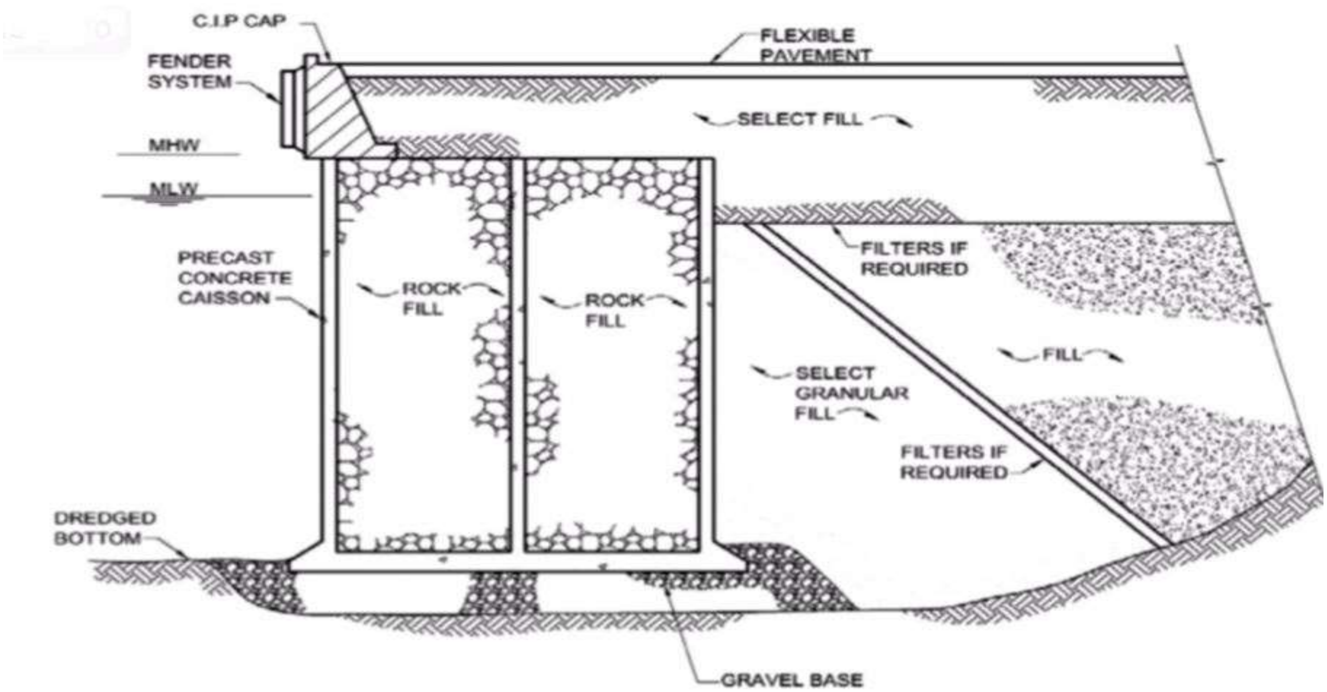


Figure 5-11 precast concrete caisson

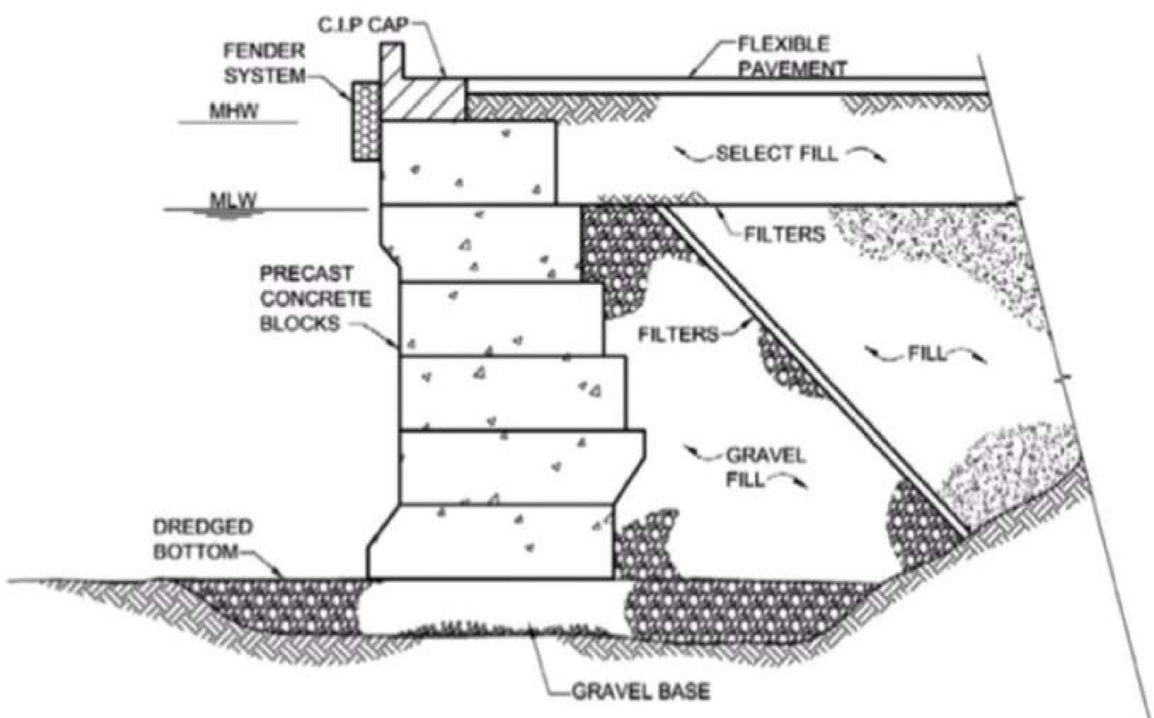


Figure 5-12 precast concrete blocks

5-3 The meaning of HYDRAULIC FILL

The soil drawn up by the suction head of a dredge, pumped with water through a pipe, and deposited in an area being filled or reclaimed is referred to as "hydraulic fill." At port and terminal facilities, where land is not available onshore and where dredging is required to provide adequate water depths for vessels at berths and approach channels, hydraulic fill is commonly used for land reclamation because of its availability and low cost.

5-4 CATEGORIES OF PIERS SEA

Piers are grouped into four (4) primary types as follows:

5-4-1 Type I – Fueling, Ammunition, and Supply

Fueling These are dedicated piers equipped with facilities for off-loading fuel from ship to storage and for fueling ships from storage.

Ammunition These are dedicated piers and wharves used for discharging ammunition for storage and for loading ammunition on outgoing ships.

Supply

Supply piers is used primarily for the transfer of cargo between ships and shore facilities.

5-4-2 Type II – General purpose

Berthing General-purpose piers is used primarily for mooring ships. Furthermore, berthing facilities may be active, as when ships are berthed for relatively short times and are ready to put to sea on short notice, and inactive, as when they are berthed for long periods in a reserve status. Depending upon intended pier usage, i.e. active berthing, maintenance/repair, inactive berthing, consider appropriate mooring service type as it relates to design/capacity of mooring fixtures. Activities that typically take place on berthing piers and wharves are personnel transfer, maintenance, crew training, cargo transfer, maintenance, and waste handling. Under some circumstances, fueling and weapons system testing may also be carried out in these facilities.

5-4-3 Type III – Repair

An important consideration for repair piers is the need to provide heavy weather mooring capability. This includes properly sized and spaced storm bollards and a compliment of heavy weather mooring lines. This consideration is predicated upon the fact that ships under repair may not be able to get underway during a heavy weather event.

Repair piers are constructed and equipped to permit overhaul of ships and portions of a hull above the waterline. These structures are generally equipped with portal cranes or designed to accommodate heavy mobile cranes.

5-4-4 Type IV – Specialized

*Training, Small Craft, and Specialized Vessels. These piers and wharves are typically light structures designed for specific but limited functions.

5-5 Flexibility of Berths Design

Typically, piers and wharves are designed to provide space, utility service, and other supporting facilities for specific incoming or homeported ships. However, berthing plans and classes of ships berthed change with time. While it is not economically feasible to develop a single facility to accommodate and service all known ship classes, design the facility with a certain amount of built-in flexibility to allow for anticipated future changes in the functional requirements. This is especially true for berthing piers and wharves that will be used to accommodate different classes of ships as well as support a variety of new operations.

5-6 Steps of Pier Design

Design Step 1 - Obtain Design Criteria

The first design step is to identify the appropriate design . defining material properties, identifying relevant superstructure information, determining the required pier height, and determining the bottom of footing elevation.

$$H_{super} = H_{par} + \left(\frac{t_o + H_{hnch} + D_o + t_{bf}}{12 \frac{in}{ft}} \right)$$

Where :-

H_{super} = superstructure depth

H_{par} =Parapet Height

t_0 =Deck overhang thickness

H_{hnch} = Haunch thickness

D_0 = Web depth

t_{bf} = Bot. flange thickness

Design Step.2 - Select Optimum Pier Type

Selecting the most optimal pier type depends on site conditions, cost considerations, superstructure geometry, and aesthetics. The most common pier types are single column (i.e., "hammerhead") see figure 5-13, solid wall type, and bent type (multi-column or pile bent)

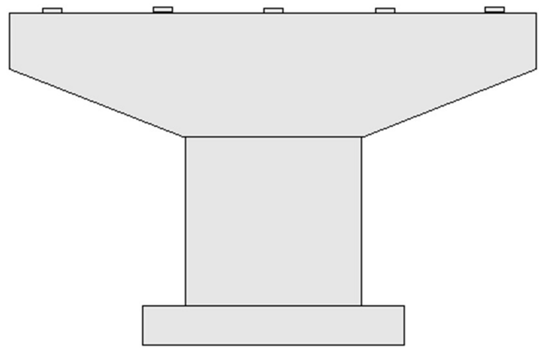


Figure 5-13 Typical Hammerhead Pier

Design Step 3 - Select Preliminary Pier Dimensions

Since the Specifications do not have standards regarding maximum or minimum dimensions for a pier cap, column, or footing, the designer should base the preliminary pier dimensions on state specific standards, previous designs, and past experience. The pier cap, however, must be wide enough to accommodate the bearing

Figure 5-14 show the preliminary dimensions selected for the pier design example

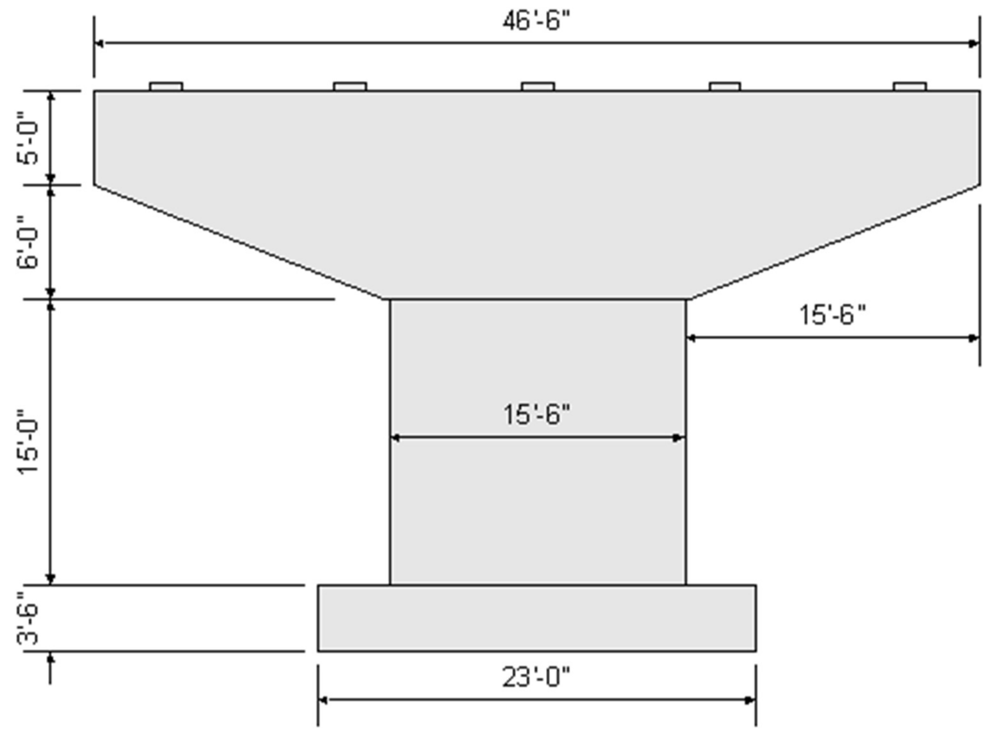


Figure 5-14 Preliminary Pier Dimension

Design Step 4 - Compute Dead Load Effects. Once the preliminary pier dimensions are selected, the corresponding dead loads can be computed. The pier dead loads must then be combined with the superstructure dead loads. In addition to the dead loads, the weight of the soil on top of the footing must be computed

Design Step.5 - Compute Live Load Effects.For the pier design , the maximum live load effects in the pier cap, column and footing are based on either one, two or three lanes loaded (whichever results in the worst force effect) . See figure 5-15

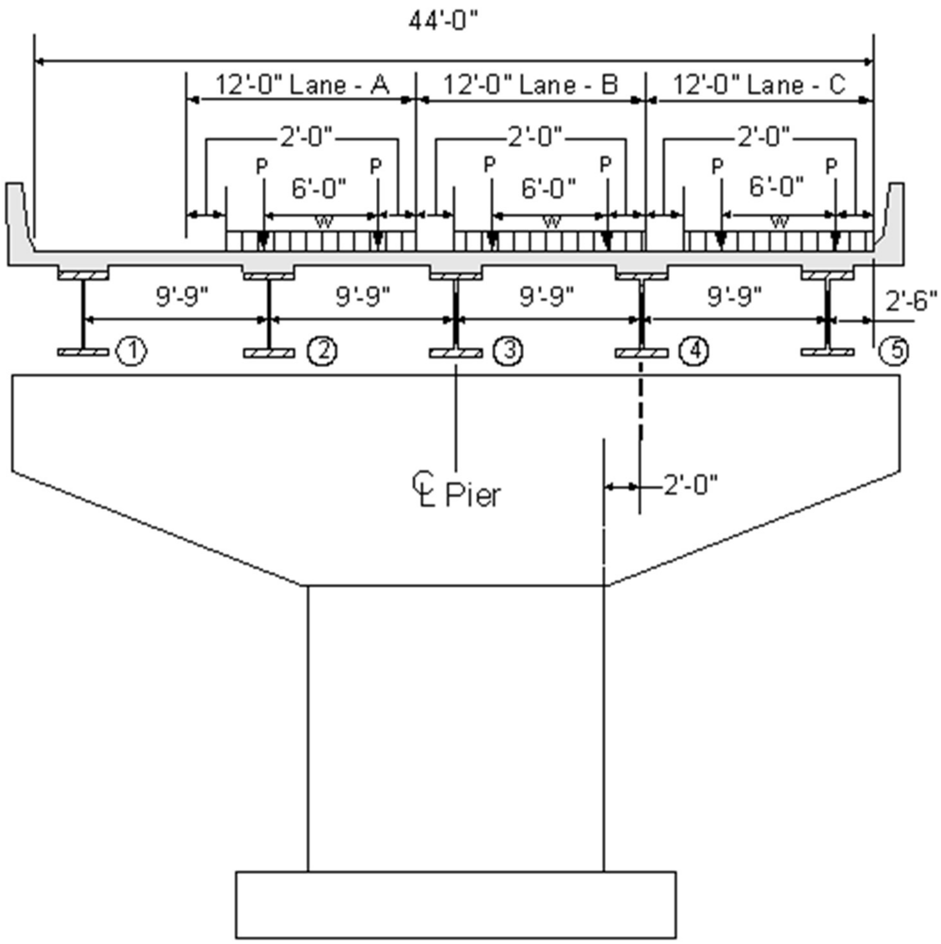


Figure 5-15 Pier Live Loading

$$R_{5\_c} = \frac{P_{wheel} \cdot (4.25ft + 10.25ft) + W_{lane} \cdot 10ft \cdot 7.25ft}{9.75ft}$$

$$P_{wheel} = \frac{R_{truck}}{2} \cdot (1 + IM) \cdot (0.90)$$

$$W_{lane} = \frac{R_{lane}}{10ft} \cdot (0.90)$$

Where :-

R 5-c = Fifth lane reaction

R_{truck} = Truck reaction (weight)

IM = Dynamic Load Allowance

P_{Wheel} = Wheel load

W_{lane} = Distance between wheels

Design Step 6 - Compute Other Load Effects

Other load effects that will be considered for this pier design include braking force, wind loads, temperature loads, and earthquake loads.

The braking force per lane is the greater of:

25 percent of the axle weights of the design truck or tandem

5 percent of the axle weights of the design truck plus lane load

5 percent of the axle weights of the design tandem plus lane load

$$BRK = \max\left(BRK_{trk}, BRK_{tan}, BRK_{trk\_lan}, BRK_{tan\_lan}\right)$$

Where :- BRK = Braking force

BRK_{trk}=Braking force of the design truck

BRK_{tan}=Braking force of the design tandem

$BRK_{trk-lan}$=Braking force of the design truck plus lane load

$BRK_{tan-lan}$=Braking force of the design tandem plus lane load

Design Step 7 - Analyze and Combine Force Effects

The first step within this design step will be to summarize the loads acting on the pier at the bearing locations The loads along with the pier self-weight loads need to be factored and combined to obtain total design forces to be resisted in the pier cap, column and footing.

Design Step 8 - Design Pier Cap

Prior to carrying out the actual design of the pier cap, a brief discussion is in order regarding the design philosophy that will be used for the design of the structural components of this pier.

Design Step 9 - Design Pier Column

the critical section in the pier column is where the column meets the footing, or at the column base.

Design Step 10 - Design Pier Piles

The foundation system for the pier is a reinforced concrete footing on steel H-piles. The force effects in the piles cannot be determined without a pile layout. The pile layout depends upon the pile capacity and affects the footing design.

Design Step 11 - Design Pier Footing

This includes the punching (or two-way) shear check at the column and estimating the applied factored shear and moment per foot width of the footing when adjacent pile loads differ.

Design Step 12 - Obtain Final Pier Schematic see figure 5-16

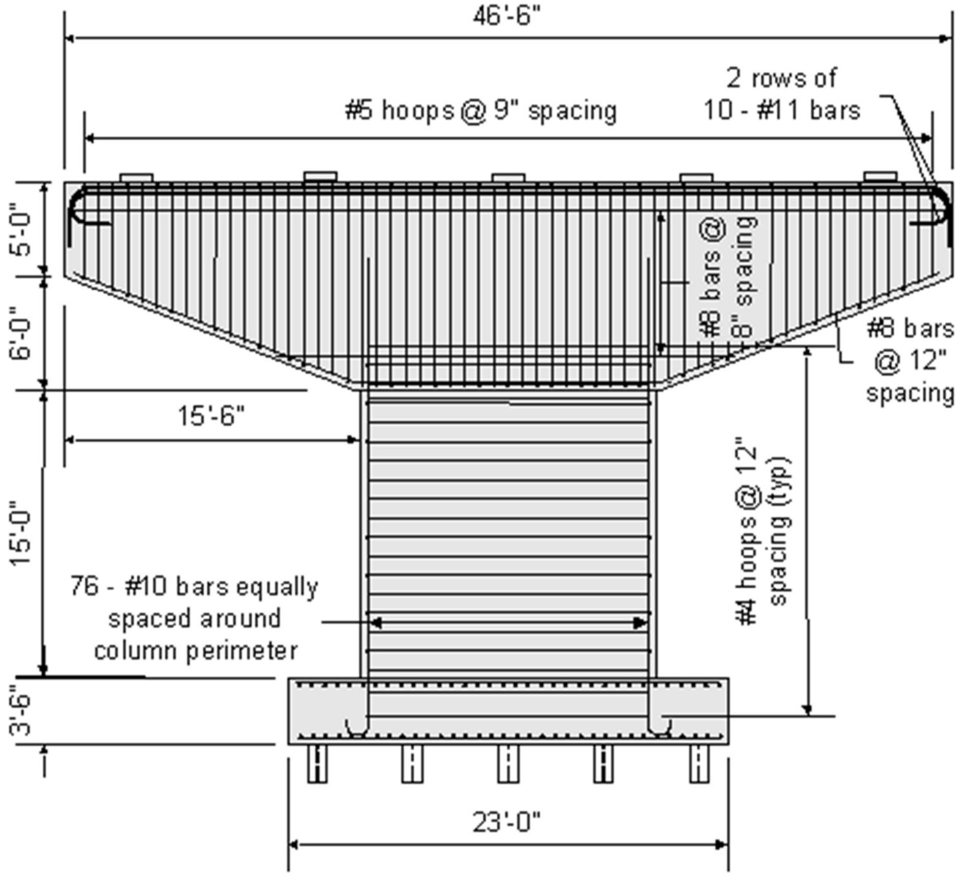


Figure 5-16 Final Pier Schematics

5-7 Dimensions of Pier

5-7-1 Pier Length see figure 5-17

Single Berth

The length of pier or wharf should equal the overall length of the largest ship to be accommodated, plus an allowance of 50 feet (15.2 m) at each end of the ship. For aircraft carriers, increase the allowance at each end of the vessel to 100 feet (30.5 m). Multiple Berths The length of a pier or wharf should equal the total overall length of the largest ships simultaneously accommodated, plus clear distance allowances of 100 feet (30.5 m) between ships and 50 feet (15.2 m) beyond outermost moored ships. Container and RO/RO Berths

The length of berths used for container or RO/RO berths should account for the requirements of the container cranes or special ramps. Where shipboard ramps are used, provide adequate berth length to allow for efficient vehicle maneuvering Submarine Berths.

For most classes of submarines, a 50-foot (15.2 m) end distance to a quaywall or bulkhead is adequate. The nose-to-tail spacing for multiple berthing should also be a minimum of 50 feet (15.2 m). However, large submarines such as the Ohio class (Trident) require 150 feet (45.7 m) or more nose-to-tail spacing and clearance to bulkhead or quay wall. Where explosive safety distance considerations require the use of fragmentation barriers, or specific separation distances, provide spacing adjusted per the requirements of NAVSEA OP-5, Ammunition and Explosives Safety.

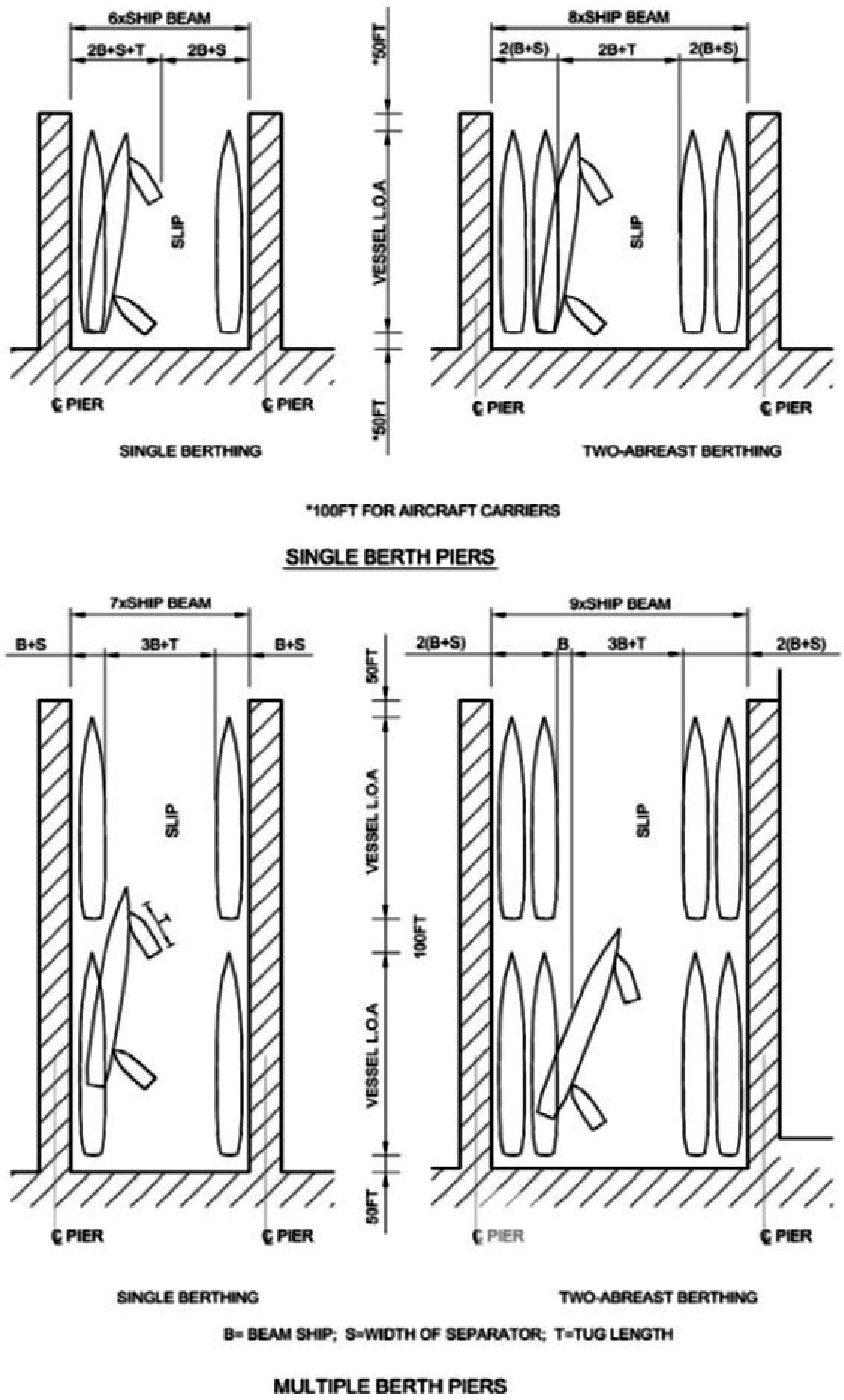


Figure 5-17 show the different types of piers according to the length

5-7-2 Pier Width

Pier width is indicated in Figure 5-18 . This definition also holds for U-, L-, and T-type wharves see figure 5-14. However, with reference to wharves, the width should be the dimension to a building, roadway, or other identifiable obstruction. . Review with specific functional requirements of the individual installation in mind before a final selection on width is made. Functional requirements include space for: cargo loading operations, line handling, ship maintenance, maintenance of utilities and layout of cables and hoses, solid waste collection, brows and platforms, crane operation, and other operations. For crane operation, consider crane outriggers, tail swing of crane counterweights, and overhang of vessels. For CVN's, coordinate the tail swing of gantry cranes with the overhang of the flight deck and elevators considering available camels and potential list of the ship. Also, these dimensions should not be less than the widths determined by geotechnical and structural considerations see table 5-1.

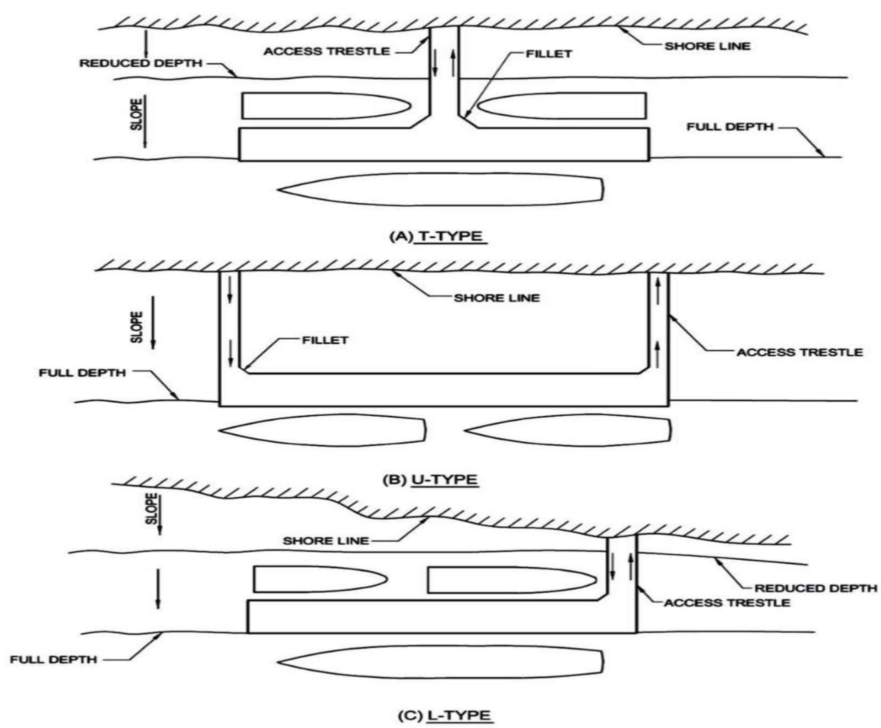


Figure 5-18 piers with U/L/T Type

Table 5-1 Typical Piers Width

Function Classification	Ship Type	Typical Pier Width (feet)		Typical Wharf Apron Width (feet)	Railroad Track (Standard Gage)	Rail Mounted Cranes
		Single Deck Pier	Double Deck Pier			
Ammunition	Ammunition	100	-	100	-	-
General Purpose Berthing	Auxiliary	115	93	65	-	-
	Surface Warfare	115	93	65	-	-
	CVN	150	93	90	-	-
	Submarine[1]	65	-	65	-	-
	Submarine[2]	85	-	65	-	-
Repair	Auxiliary / Cruiser	125	-		4 tracks; 2 each side	(2) 40-ft gage; 1 each side
	CVN	150				
Fueling	Auxiliary	50	40/50 [3]		-	-
Supply (General)	Auxiliary	125		100	2 tracks	-
Supply (Container)	Auxiliary	125		100	Up to 3 tracks	(1) 100-ft gage

[1] Pier width is shown for a pier with only in-shore berths
[2] Pier width is shown for a pier with in-shore and off-shore berths (multiple berths along its length)
[3] Operational deck = 40 feet (12 m); utility deck = 50 feet (15 m)

CHAPTER SIX

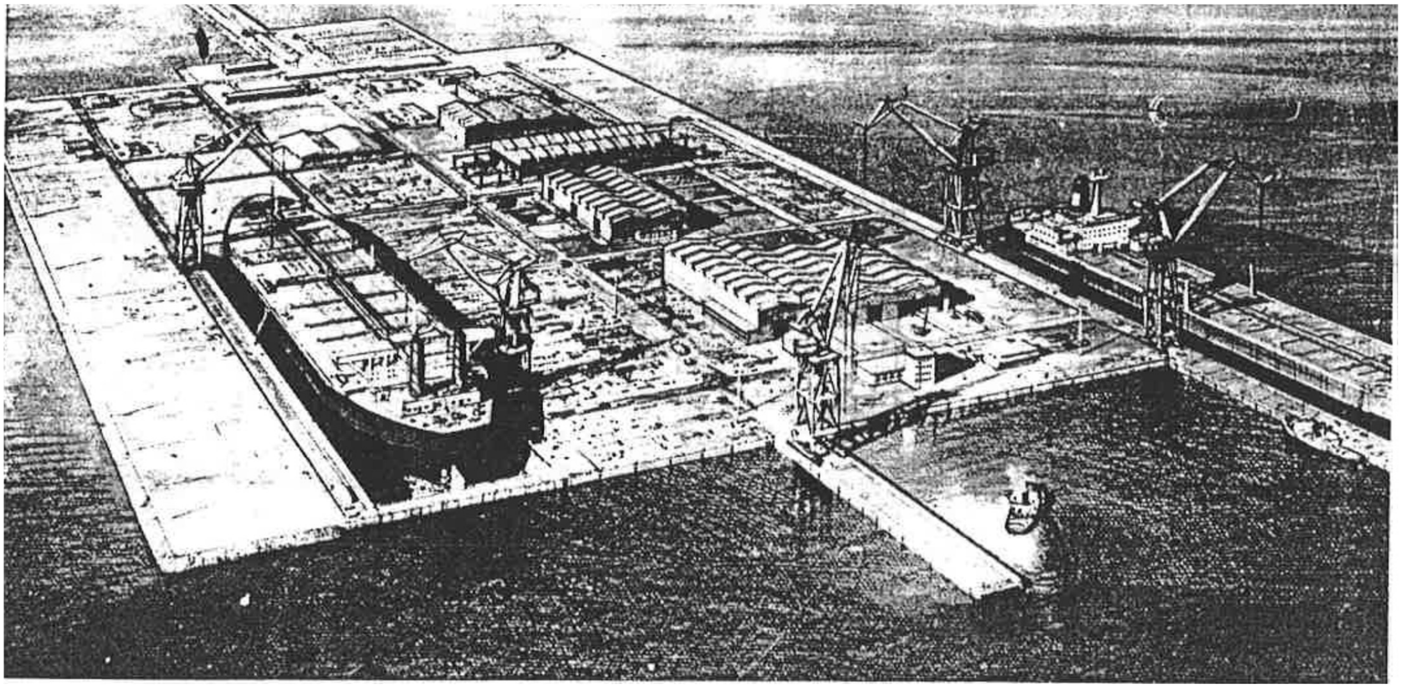
DRY DOCKS

INTRODUCTION

Every machine need repairing, maintenance and constant upkeep for smooth functioning. Dry Docking is the process followed for the periodic repair and maintenance of ships, boats and watercrafts as shown in Fig (6.1).

Throughout history and since the concept of drydocks where invented, techniques differed from one way to another, one of the first pioneers in this field were the Chinese in about 1088 A.C. In 965 A.C., a couple of dragon type vessels with length of 63 m each, where presented to the emperor to show loyalty, after lot of cruises bodies decayed and needed repairs, but repairs where impossible offshore due to their sizes, so in the period of king “Hsi-Ning”, the official of the palace suggested, a dock was dug in a lake that was capable to contain this type of vessels. Construction procedure was with the use of huge crossed beams located on a piled raft foundation. After that a hole was dug to make water flow into the dock, so the ships can move in the basin, after the ship settles in the basin the hole closed and the water was pumped away using wheel pumps, so the ship could rest in the air, after repairs finished, basin was filled again so the ship can sail again, finally the basin was roofed so the ships can be protected during repairs (Levathes, 1994. Europeans, however, also made drydocks in a quite similar way, in England the first and oldest surviving drydock (since 1495), was established at “HMNB Portsmouth” in the era of “Henry VII” of the “Tudor” dynasty, This basin nowadays keeps the oldest battleship, which is “HMS Victory” (Noel, 1988).

Meanwhile, it is possible that the earliest floating drydock was established in Vince in 1590, an anonymous inventor asked for a method of rescuing damaged vessels, and then proceeded and described his approach, the method was flanking the shop by a couple of natant platforms, creating a ceiling above the ship. The ship is located in a proper position by using bonds, fixed to the structure (Sarton, 1946).



FIG(6.1) SHIP REPAIR YARD ON AN ISLAND __BAHRAIN REPAIR YARD (PROF ABRIL ,Lisbon,portugal)

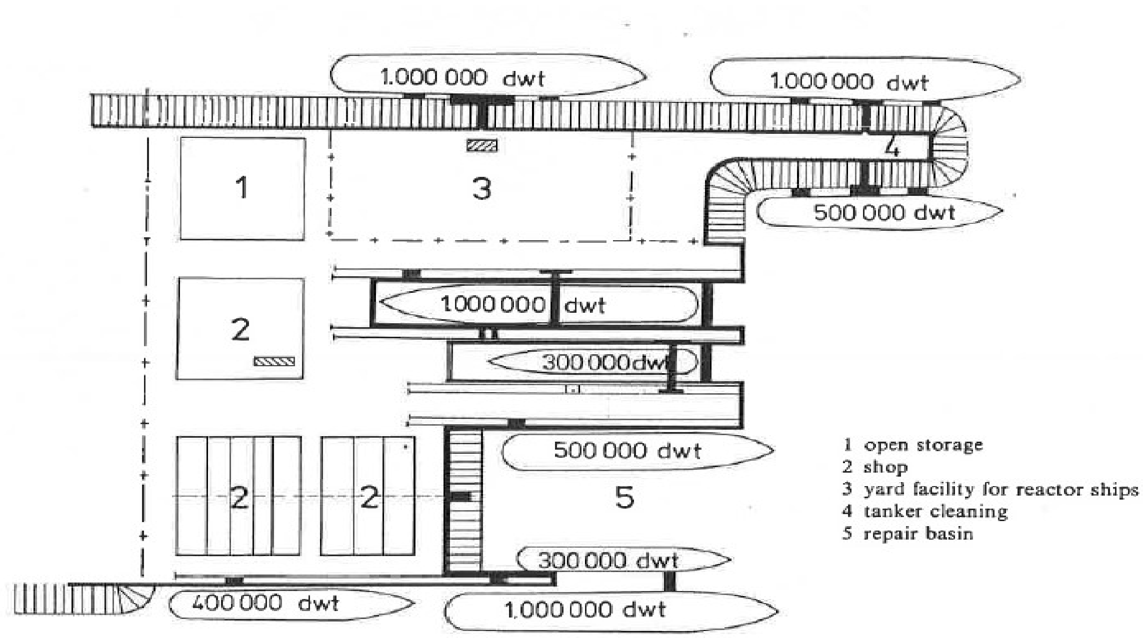
6.1: DRY DOCKS DIFINITION

A dry-dock is a narrow basin or a vessel which can be flooded to make a marine craft resting on the docking blocks on it to float and vice versa, i.e., water can be drained off by some mechanism to make the structure rest on the docking blocks. These can be used for the construction and repair of ships by docking and then by floating. This thesis work involves the structural design and stability of a floating dry-dock. Floating dry-dock as the name suggests, itself is floating. Since it is a floating structure, it works on controlled buoyancy to lift any structure out of water for any kind of repair works or inspections. It needs to be stable floating, carrying the intended load during the docking operations and should also have the strength to withstand the intended loads. Floating dry-docks are generally U-shaped or channel like structures, with less complex geometry. The common structure is pontoon type consisting of water tight chambers, the pontoon and wing walls. The draft of the floating dock is adjusted by controlling the water ballast during the docking operations.

Drydocks are exposed to special conditions such as thee xtreme degree of humidity, marine conditions, special loading phases, and other conditions related to themarine environment. Docks subdivide into two types, wet docks, and dry docks, for wet docks, these structures are built for human services, like trading, ship anchoring, military services, and other services, while dry docks are designed and constructed for vessels services as shown in Fig (2).

Dry Docking is necessary to view and correct mechanical defects in the vessel in its original, dry form for better results and effective functioning. Basically, to clean, scrub and prepare the ship by de-scaling all rusted areas of the hull, and finally paint the ship's hull so that speed and fuel consumption are restored to it's original.

Before a ship is taken for repair, a surveyor inspects the vessel in clear water where the hull is visible on the outer surface. The surveyor analyses and points out defects which are then corrected through Dry Docking.



)Fig (6.2).ship repair yard on apeninsula (after Knegjens ,1968

6.2: Types of Dry Dock

Different types of dry docks are used for repairing and cleaning a ship as shown in Fig (6.3). The main ones are:

1. Graving dock
2. Floating dock
3. Marine Rail Dock
4. ship lifts
5. Marine mobile lifts

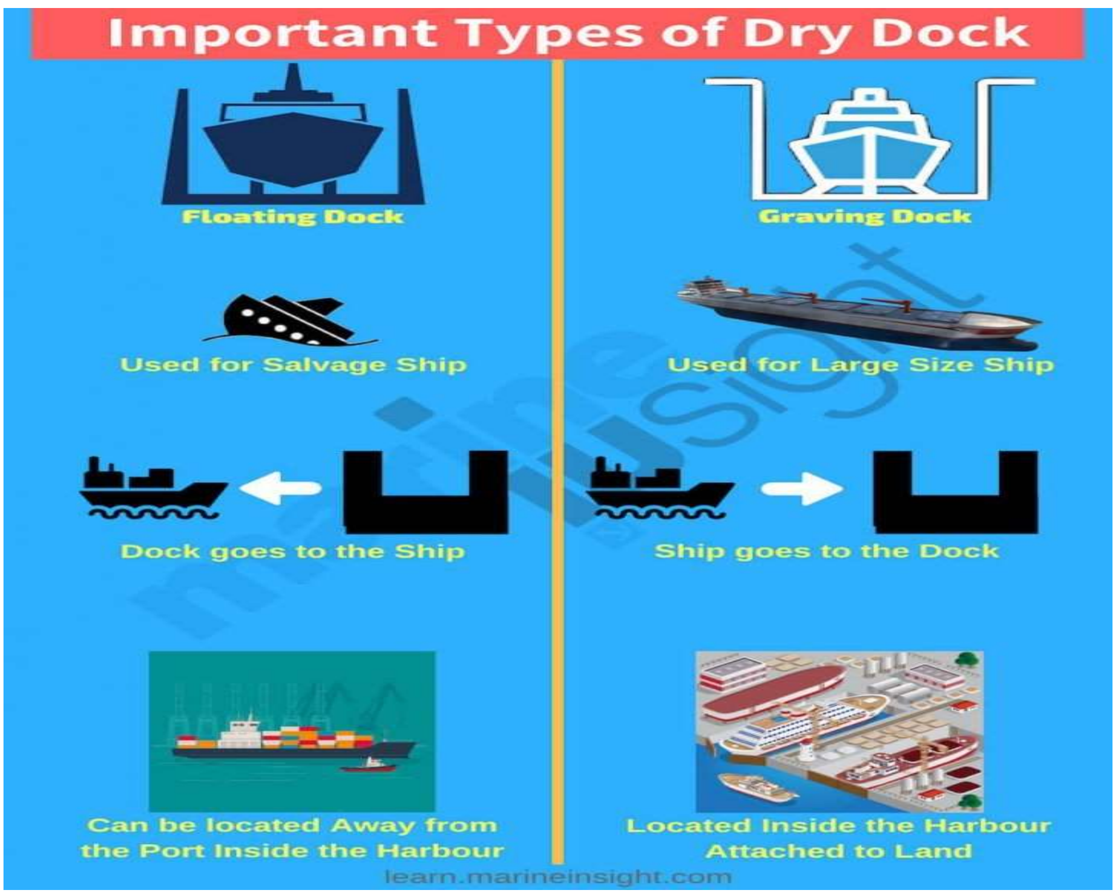


Fig (6.3) Types of Dry Dock

Among these, the marine mobile lifts and ship lifts are mainly used for small vessels such as recreational yachts, tugs pilot boats etc.

Nowadays, there are mainly two types of dry dock procedures from the above list that are used for seagoing vessels:

(6.2.1) Graving Dry Dock:

This type of dry dock is normally constructed on land near the coastal waters with a rectangular solid concrete construction with blocks, walls, and gates as shown in Fig (4) and Fig (5). The vessel is shifted inside the dry dock and rested on the blocks. After the ship is in the required position, the gate is closed and water is removed.

Fig (6.4) gravity dry dock

Fig (6.5) 3D gravity dry dock

In the earlier days, the graving dry docks were constructed using stones and timber. Now, steel and concrete structure is used to make the enclosure and a heavy steel gate is used to seal the dock to stop the ingress of water once the ship is standing on the blocks.

The gates can be in two parts with each hinged to the sides and hydraulically operated or one solid steel structure supported on roller over the track, which can be retracted inside the dry dock walls when opening the gate as shown in Fig (6.6).

Fig (6.6) Graving Dry Dock Gate

(6.2.1.1) Advantages of Graving Dry Dock:

1. It can accommodate bigger size vessels when compared to other dry-docking systems.
2. It is cheaper for dry-docking a similar-sized vessel as compared to other types.
3. The graving dry dock can be used to perform retrofitting, modification etc. which is difficult to achieve in other types.
4. The supply of spares, machinery, services to graving dock is very much accessible due to its location-based near the land.
5. New advanced graving docks have welding, hot-work and other workshop located inside the dock in an elevated surface, (above the water surface when the dock is filled) giving quick access and workflow in the dock.
6. Retractable ramps in new types of graving docks make it easy to supply spare, machinery and saves a lot of time and manpower to transfer them inside the dock.
7. A bigger graving dock can be used to repair more than two ships at a time and some modern graving docks have two gates at both ends, making it easier to repair and re-float the ships independently.

6.2.1.2: Disadvantages of Graving Dry Dock:

1. When re-flooding the dry dock, all the machinery and equipment needs to be taken out from the dock, which takes time.
2. The maintenance cost of the graving dock increases as per the age of the dock and becomes very high.
3. Any problem with the dock gate will make the whole dock non-operational.
4. The docking and undocking process in the graving dock takes time as compared to other types.

5. If the dock holds multiple ships for repair, the complete operation needs to be stopped if anyone of the vessels needs to be taken out of the dry dock as it will require filling of water for refloating.

(6.2.2) Floating Dry Dock:

A floating dock is in the form of "U" structure which is mainly used in salvage, to carry ships that have met with an accident and is damaged to an extent that has made them unable to sail further to a coastal dock as shown in Fig (6.7) and Fig(6.8).

However, now many regular sea-going, small and mid-size vessels are also dry docking in a floating dock. Several "U" type floating docks can be joined to carry a large vessel.

The ship is brought near the channel where the floating dry dock will partly submerge itself and the ship slides inside the dock.

Once the ship is in the position, the floating dock is then de-ballasted to drain the water from its hollow floors and walls to support the vessel on the blocks arranged on the floor of the floating dock.

A valve is provided which can be opened to fill up the chambers with water and which will make the dock immersed in water so that the ship can sail out.

The water is pumped out of the chamber which will allow the dry dock to rise, exposing the underwater area of the ship for maintenance or carrying the ship repairs.

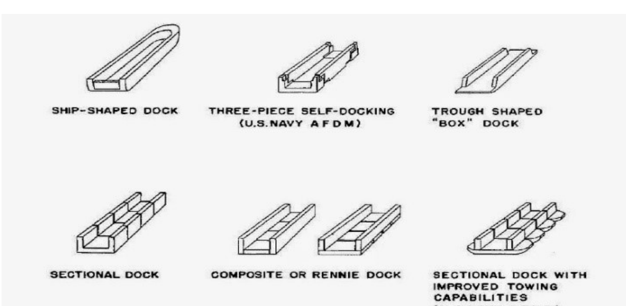


Fig (6.7) SOME FLOATING DRY DOCKS BASED ON SHAPE AND MODULAR STRUCTURE

Fig (6.8) Side Floating Dry Dock

Fig (6.9) 3D Floating Dry Dock

(6.2.2.1) Advantages of a Floating Dry Dock:

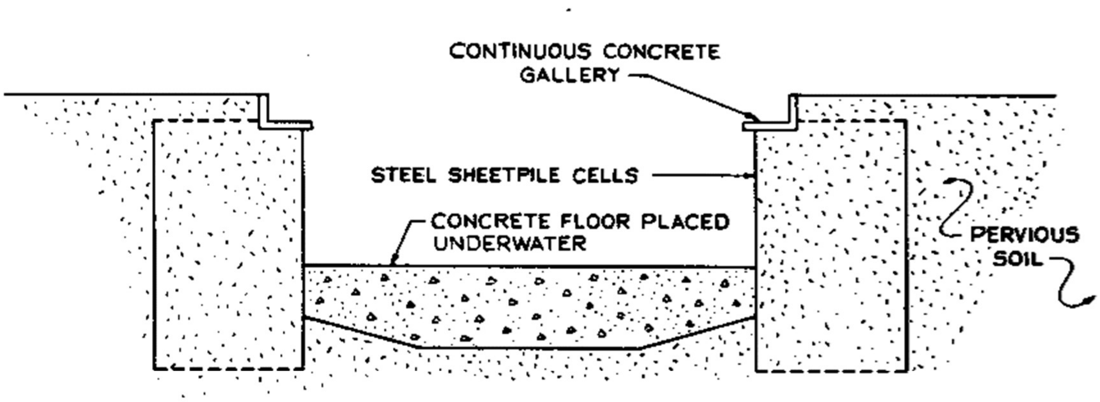
1. They can be propelled to the location of a salvage vessel near the harbor.
2. They are cheaper to maintain as compared to graving docks and can get a higher resalable return.
3. They can be installed near or away from the shore inside the harbor, making them a portable and space-saving structure without taking space of the shore facility.
4. The complete floating dry dock can be aft or forward trim by ballasting the dock, which further assists the ship or the damaged vessel which cannot be given a trim.
5. Additional mooring equipment is needed for the floating dry dock to make it stable.
6. The floating dry dock can be altered and increases in size in all dimensions by extensive retrofitting/ rebuilding.
7. They can also be split into two different floating docks independent of each other.

(6.2.2.2) Disadvantages of Floating Dock:

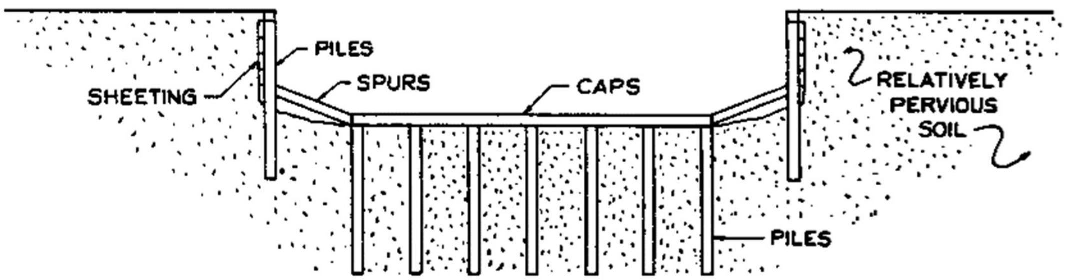
1. The supply of store, equipment, and manpower is usually done from one access point gangway which makes the operation slow.
2. The maintenance cost of the floating dry dock is similar to that of a ship as the hull of the floating dock is submerged in the saltwater.
3. The floating dry dock operation will effect if there are tides or during windy weather.
4. When re-flooding the dock, all the machinery and equipment needs are to be taken out from the dock which takes time.

(6.3) Miscellaneous Types:

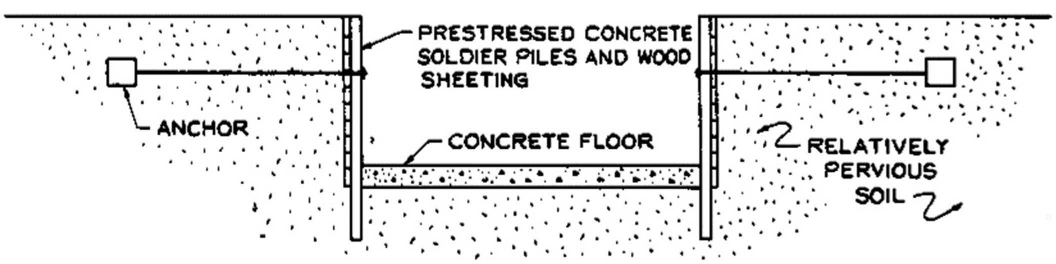
For drydocks of temporary or semi-permanent nature, a great variety of types may be used. These types are so much different in general character from the conventional naval drydock that classification in accordance with the method of solving the water pressure problem is not entirely definitive. These drydocks are generally for shipbuilding or for building other types of floating structures, and take a great variety of shapes and forms see Fig(6.10). For these drydocks, provide the simplest drainage systems. Either the floor or walls, or both, may not be watertight, and the water may seep through them into the dock chamber and run off the floor into trenches or pump sumps for disposal by pumping.



(a) SEMIPERMANENT SHIPBUILDING DRYDOCK



(b) TEMPORARY, ALL TIMBER DRYDOCK, CONSTRUCTED
IN OPEN CUT FOR BUILDING FLOATING STRUCTURES



(c) TEMPORARY DRYDOCK, CONSTRUCTED IN OPEN CUT
FOR BUILDING TUNNEL SECTIONS

Fig (6.10) Miscellaneous Types of Drydocks

6.4: STRUCTURAL DESIGN

6.4.1: SCOPE. This section presents criteria on structural design of drydocks, with particular reference to dead loads, hydrostatic pressure, earth pressure, live loads, special conditions of loading, materials and design stresses, and methods of analysis all these loads see in Fig (6.11) and Fig (6.12).

6.4.2: DEAD LOADS. Dead loads are of special significance because the deadweight of the structure, including all mobilizable earth weight plus friction and hold down piles, must be greater than the maximum buoyancy.

(6.4.2.1) Weight of Concrete Structure. For design purposes, compute the weight of reinforced concrete structures on the basis of 2403 kg/m3 (150 lb/ft3) (weight in air).

(6.4.2.2) Weight of Earth. In computing the total resistance to uplift, include the weight of earth engaged by any extension of the slab beyond the outside of the wall. Earth below the drydock floor, when engaged by hold down piles or other devices, is included in the computation of the total weight.

(6.4.2.3) Weight of Earth on Floor Slab Projections

(6.4.2.4.) Specific Weights. Unless special, lightweight soils are encountered, use 961 kg/m3 (60 lb/ft3) for submerged soils and 1602 kg/m3 (100 lb/ft3) for soils above water levels.

(6.4.2.5) Computation of Volume. To compute volume with a dock empty and with mean high water, use the soil weight above the slab projection between a vertical plane at the outer edge of the projection and the back of the wall. With a dock empty and extreme high water, add the weight of earth wedge between the vertical plane and an intersecting plane sloping 15 degrees outward from the vertical plane.

(6.4.2.6) Weight of Earth Engaged by Floor Slab Holddowns

(6.4.2.6.1) Specific Weights. This earth is always submerged. For ordinary soils, use 961 kg/m3 (60 lb/ft3.) Since this weight is usually very important, determine the correct weight by test if there is any indication that the soil may be of a greater or lesser weight.

(6.4.2.6.2) Computation of Total Weight. The holddown capacity of individual piles may be computed by methods given in DM-7.02 Foundations and Earth Structures.

The total holddown capacity is not necessarily the sum of the individual capacities of a pile group and may never be larger than the weight of the block of soil included in the pile group.

(6.4.2.6.3) Weight. In computing the weight of this block, assume its plan dimensions to extend beyond the outer rows of piles by a distance of one-half the typical pile spacing.

(6.4.2.6.4) Depth. For the depth of block, assume the block bottom is above the pile tips by a distance of one-half the typical pile spacing. Where spacings are different in each direction, use the larger of the spacings.

6.4.3: HYDROSTATIC PRESSURE

(6.4.3.1) Weight of Water. In computing pressures, use 1025 kg/m3 (64 lb/ft3) for seawater and 1002 kg/m3 (62.5 lb/ft3) for fresh water.

(6.4.3.2) Buoyancy Computations. Make all buoyancy computations for three water levels, as follows:

(6.4.3.3) Extreme high water. To check safety against uplift with the maximum (15 degrees) earth wedge mobilized.

(6.4.1.2.4) Mean high water. To check safety against uplift with the minimum earth block and friction mobilized.

(6.4.1.2.5) Extreme low water. With a ship in dock to determine maximum downward load on foundation soil or piles.

6.4.4: EARTH PRESSURE

(6.4.4.1) Variations. Acting against a dock structure, the resultant outside earth pressure will vary considerably according to pressure and weight conditions inside the dock. Resultant earth pressures will be different when a dock is full of water, when a dock is dry but contains a vessel, and when a dock is empty.

(6.4.4.2) Water or Ship in Dock. Active pressure is to be used because the rotation of the wall with respect to the floor is negligible. Do not use surcharge for computing pressure on drydock walls except where railroad rails on ballast are near the wall.

(6.4.4.3) Dock Empty. Assume partial passive pressure to be operative where there is structural continuity at the juncture between sidewalls and floor because, with the dock empty, sidewalls of full hydrostatic docks have a tendency to rotate outward against the backfill. The amount of passive resistance shall be determined by assuming a uniform increase in resistance to occur throughout the sidewall height, starting from zero value at the top to an ascertained maximum bottom value. The rate of increase is based on the condition that the total internal work of the bending stresses throughout the dock cross section has a minimum value.

(6.4.4.4) Inconsistency in Partial Passive Pressure Assumption. The earth pressure at floor level should be no greater than active pressure, because the horizontal displacement of sidewalls is zero at about floor level, which is the approximate center of rotation for the sidewalls. Nevertheless, the method of approximating the total passive resistance for the condition of dock empty, as described previously, has proved satisfactory for existing structures so designed.

(6.4.4.5) Upward Pressure

(6.4.4.5.1) Full Hydrostatic Dock. The distribution of upward pressure beneath a dock designed to resist full hydrostatic pressure is known when the dock is empty, because the dock weight is nearly equal to total buoyancy.

(6.4.4.5.2) Relieved Floors. For relieved floors, earth pressures are not uniform because they are dependent on slab deflections induced by concentrated ship loads and moments at the wall bases. For the solution of elastic foundation problems and associated soil pressures.

(6.4.4.5.3) Friction on Sides. In addition to the dock deadweight, friction piles, and earth weight over projections, the frictional resistance between backfill and sidewalls also is effective in preventing uplift.

(6.4.4.5.4) Mean High Water. To determine the frictional resistance for an empty dock at mean high water, the lateral force acting against the sidewalls (that is, the force corresponding to the active pressure of submerged earth) is multiplied by the coefficient of friction for the earth material on the sidewall material.

(6.4.4.5.5) Extreme High Water. Stability against uplift at extreme high water is computed using a deadweight of the earth wedges as described in section (6.12), instead of frictional resistance.

6.4.5: LIVE LOADS.

For design purposes, conditions comprising live loads are:

(6.4.5.1) Shiploads. Determine shiploads on the floor for the specific class ship. Figures 5-1 and 5-2 indicate shiploads that have been used for four basic types of ships. Note the extra heavy loads at the stern overhangs. Nuclear powered ships have extra heavy loads under the reactor.

(6.4.5.2) Thin Floors. For thin floors, investigate the effect of these extra heavy loads, and reinforce the floor locally as necessary.

(6.4.5.3) Positioning. Base the blocking arrangement for design of the floor on any likely positioning of ships in the dock. Larger ships may be docked only on the centerline of the graving dock. For docks wide enough to permit multiple docking of ships abreast, or long enough to permit various placement fore and aft, apply the load pattern for such smaller ships multiple docked in odd positions to the floor as well as the load pattern of larger ships docked on the centerline.

(6.4.5.4) Wheel Loads. The typical wheel loadings for a 36287 kg (40-ton) locomotive crane, and 22680 kg (25-ton), 31750 kg (35-ton), and 45359 kg (50-ton) portal cranes are given in MIL-HDBK-1025/1 Piers and Wharves.

- **Shiploads** applied to dock floors through blocking.
- **Wheel loads** from crane wheels, railroad tracks, and trucks applied to local beam and slab supports.
- **Local static and moving loads** on roofs and floors or pumpwells.
- **Railroad track loadings**, as a surcharge of earth pressure, from tracks carried on ties and ballast adjacent to sidewalls of docks and walls of pumpwells.
- **An impact allowance** of 15 percent is made for moving loads for structural members forming the primary support for the moving loads.

(6.4.5.5) Full Hydrostatic Graving Docks. Crane wheel loads do not normally influence the design of the main wall of graving docks designed to resist full hydrostatic pressure, because of the extensive longitudinal distribution of the wheel loads by the walls. Wheel loads from cranes operating around full hydrostatic graving docks, therefore, usually are significant only in the design of local beam supports under rails crossing the overhead of service tunnels, pump wells, and other auxiliary structures.

(6.4.5.6) Relieved and Partially Relieved Graving Docks. For relieved and partially relieved graving docks, crane wheel load may influence the design of main dock walls as well as the design of local beam supports.

(6.4.5.7) Mobile Crane Loads. Mobile cranes are to some extent replacing locomotive cranes. Use truck crane wheel loads as given in MIL-HDBK-1025/1 Piers and Wharves or the crane manufacturers wheel load specifications (covering many of the larger ~ 220-Ton mobile cranes used by PWCs/Shipyards/Private Contractors) for beams, slabs, and the overhead structure of the pumpwell, where crane track loading does not govern.

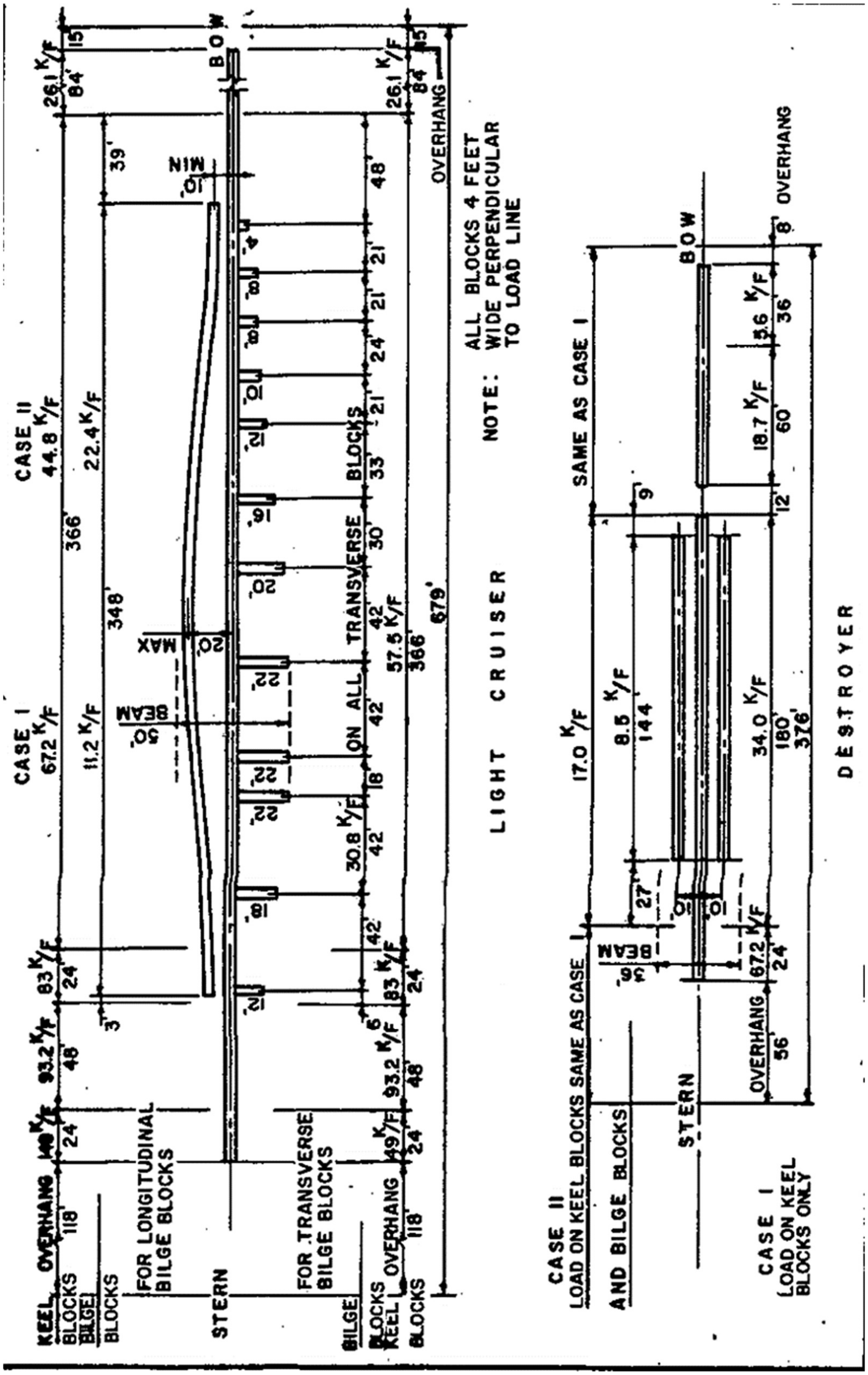


Fig (6.11) Ship Blocking Loads

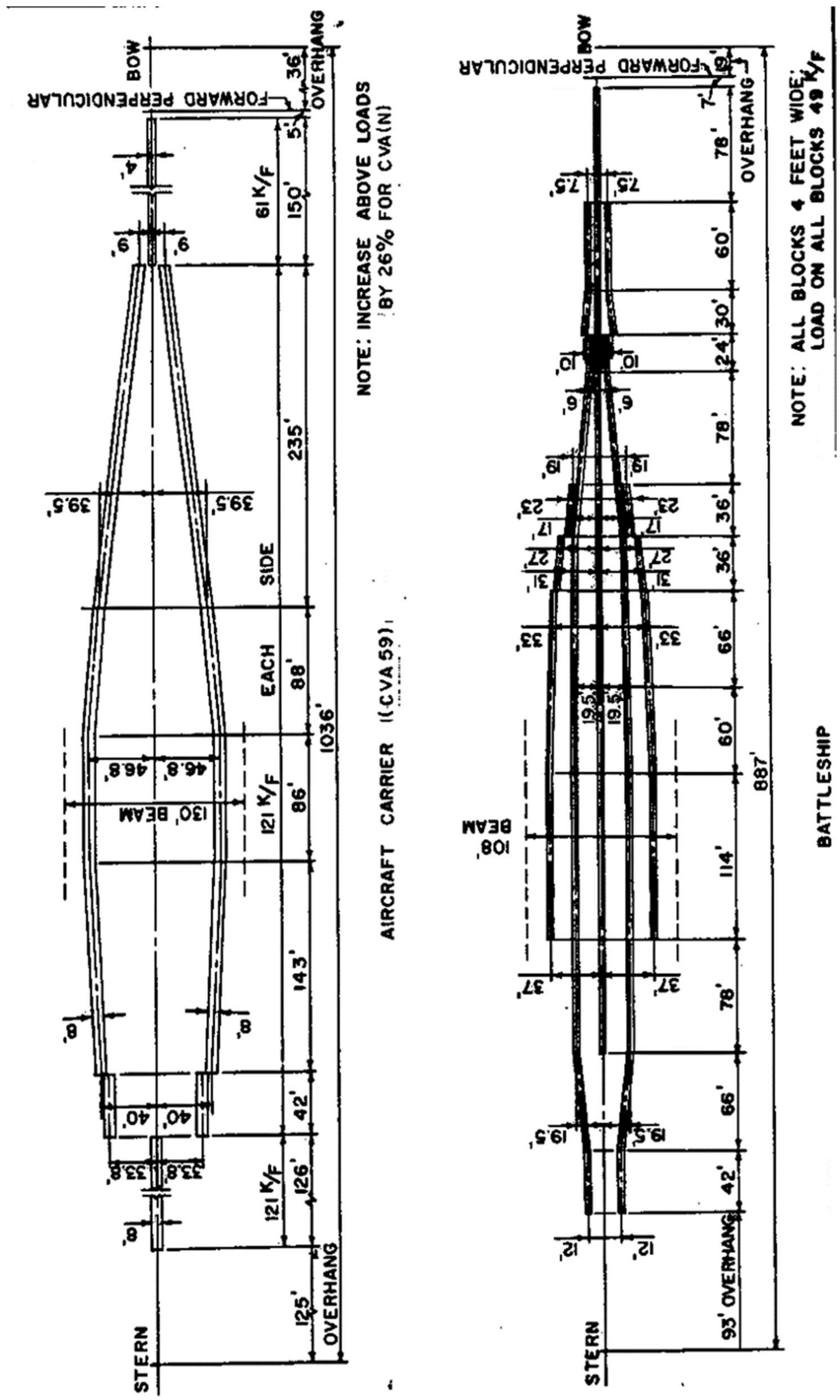


Fig (6.12) Ship Blocking Loads

(6.4.5.8) Loads on Pump well Overhead and Floors. Pumpwell overhead should be designed for a uniform load of 2929 kg/m2 (600 lb/ft2) and for truck crane wheel loading when it is at ground level. The critical load for floors supporting main pumps usually corresponds to the maximum upward pressure. Use a uniform load of 1464 kg/m2 (300 lb/ft2) for floors not subject to upward hydrostatic pressure; also, these floors are to sustain loads from operating machinery placed thereon either in a permanent operating position or in a temporary overhaul position. Include vibrations induced by reciprocating and rotating equipment in the design.

(6.4.5.9) Earthquake Forces. In regions of earthquake probability use design criteria contained in MIL-HDBK 1025/1 Piers and Wharves. A drydocking facility is a major capital investment and, therefore, the structural system will be subject to a rigorous dynamic analysis of seismic probability effects in accordance with well- established principles of mechanics. Design the drydock to resist the forces of two- thirds of the site adjusted Maximum Considered Earthquake (MCE). The MCE is an earthquake with an acceleration equivalent to an earthquake having a 2 percent probability of exceedance in 50 years. Site adjustments are based on the characteristics of the soil underlying the site. Design lateral acceleration should be at least 0.12g.

(6.4.5.10) Bomb and Blast Resistance. Drydocks are not usually designed to resist bombing or blast effects because of the massive size of the structure involved. In some locations, consideration should be given to protective construction for the upper part of the pump well and the service tunnels. Additional guidance for Anti- terrorism/Force Protection is contained in MIL-HDBK 1025/1 Piers and Wharves.

(6.4.6) SPECIAL CONDITIONS OF LOADING

(6.4.6.1) Full Hydrostatic Drydocks. Although there are many special loading conditions to be considered in the design of a graving dock (for example, nonsymmetrical loads, wave action on exposed walls, earthquake, and unusual water differentials), the design of full hydrostatic pressure docks generally is concerned with four especially critical conditions.

- Case I. Dock under construction.
- Case II. Dock empty. Maximum hydrostatic uplift.
- Case III. Maximum ship load. Minimum hydrostatic uplift.
- Case IV. Dock full of water.

For application of these loadings, refer to American Civil Engineering Practice, Volume II.

(6.4.6.2) Partially and Fully Relieved Drydocks. Critical conditions for partially and fully relieved designs are similar to those for full hydrostatic drydocks, except for appropriate allowance for decreased upward and lateral water pressures in accordance with the degree of lowering of hydraulic gradients.

(6.4.6.3) Drydocks Built by Underwater Methods. When completed, these drydocks are always of the full hydrostatic type. In some cases, however, this method of construction involves loadings not encountered with construction in the dry. These cases occur when walls are built entirely in the dry in cofferdams set on slabs previously constructed underwater. Under these conditions, when wall cofferdams are unwatered, the partially completed structure does not have the benefits of wall and finish floor slab weight to overcome the buoyancy of the cofferdams, or the full sidewall thrust to overcome the tension in the slab. For examples of two such conditions, see Fig (6.13).

These drydocks are frequently associated with the use of holddown piles, necessary if the weight of the floor slab is insufficient to overcome the total buoyancy including that of the empty cofferdam or cofferdams. Note in (a) of Figure (6.13) there is a tendency to develop tension in the slab bottom without benefit of axial compression from a sidewall thrust. In (b) of Figure (6.13), there is a tendency to develop tension in the slab top with a side thrust that is much smaller than will be developed against the walls of the completed dock.

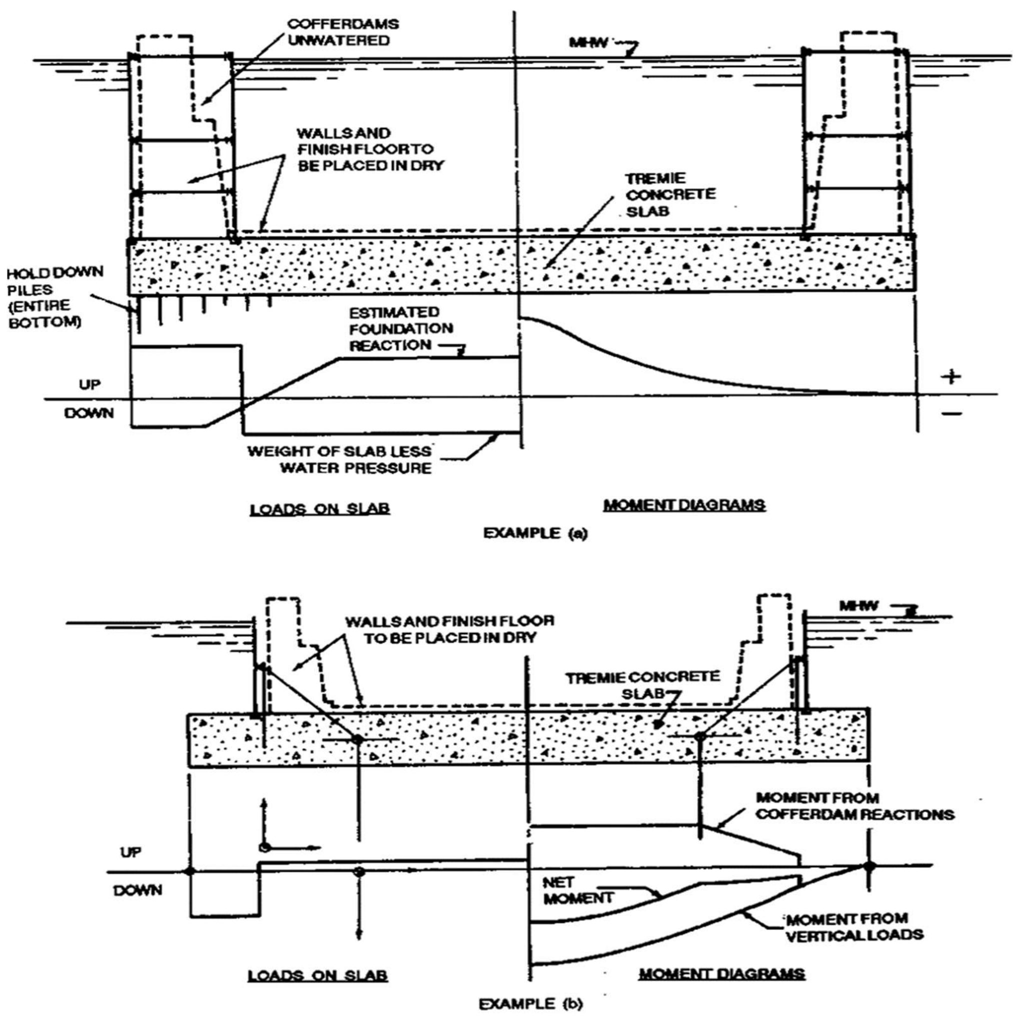


Fig (6.13) Examples of Drydocks with Slabs Constructed Underwater and Walls and Finish Floor Constructed in Dry

(6.4.7) MATERIALS AND DESIGN.

This section contains special provisions applying to concrete for drydock walls, floors, and general cross section.

(6.4.7.1) Concrete

(6.4.7.1.1) Classes of Concrete. Recommend using concrete as specified in UFGS 03311 Marine Concrete. Use minimum 24,132 kPa (3,500 psi) estimated 28 days compressive strength cast in place concrete for the main body of the dock. Classes of greater strength may be used in accordance with structural requirements. Do not specify mortar intrusion concrete (see Glossary) for permanent drydocks.

(6.4.7.1.2) Admixtures. Admixtures may be used to produce air entrainment, higher strength, greater durability and better workability, up to maximum percentages detailed in the project specifications**.**

(6.4.7.2) Reinforcing Steel

(6.4.7.2.1) Cover. Minimum concrete protection for reinforcement is as follows:

- 152.4 mm (6 in) where face of concrete will be in contact with soil.
- 76.2 mm (3 in) for formed or finished surfaces not in contact with soil.
- 50.8 mm (2 in) over bearing pile tops.
- At piles intended as hold downs, and having a considerable length of embedment in the concrete, place the reinforcement as in the first bullet above.

(6.4.7.2.2) Reinforcing Bars. Reinforcement should have a minimum yield strength of 275,790 kPa (40,000 psi), and conform to ASTM Specification A 615, Deformed and Plain Billet-Steel Bars for Concrete Reinforcement, Grade 40. High strength, or special large size reinforcement should conform to Grades 60 and 75. Consider use of epoxy coated, stainless steel, cladded stainless steel, or galvanized reinforcement.

(6.4.7.2.3) Foundations. Evaluate safe soil bearing capacity by methods set forth in DM-7.01 Soil Mechanics and DM 7.02 Foundations and Earth Structures. Where the safe capacity of the soil is exceeded, provide structural support. Types of structural supports applicable to the foundation of drydock proper and to supplemental structures are: timber piles, concrete piles, steel H-piles, pipe piles with open or closed ends, and caissons. For pile capacities, analytical treatment, information on range of capacities for various types of piles, and capacity of caissons, refer to DM-7.02 Foundations and Earth Structures.

(6.4.8.) Design

(6.4.8.1.) Load and Strength Reduction Factors. In design of reinforced concrete structures, proportion members for adequate strength in accordance with provisions of the latest edition of ACI 318, Building Code Requirements for Reinforced Concrete, using load factors and strength reduction factors (phi) specified.

(6.4.8.2) Service Load Stresses. Alternatively, no prestressed reinforced concrete members may be designed using service loads and permissible service load stresses in accordance with provisions of ACI 318, Appendix B, Alternative Design Methods.

(6.4.8.3) Buoyant Condition Increases. For a buoyant condition of a continuous U cross section, which might be produced in a relieved or partially relieved dock resulting from failure of the pressure relief system, or for construction stages, the ordinary working design criteria may be increased 50 percent.

(6.4.9) METHODS OF ANALYSIS

(6.4.9.1) Full Hydrostatic. For analysis of four basic conditions of loading, refer to American Civil Engineering Practice, Volume II

(6.4.9.2) Fully or Partially Relieved. Where these drydocks have relatively thin floors, concentrated ship blocking loads and wall reaction produce deflections resulting in variations in foundation pressures and requiring methods of elastic foundation analysis. The problem is to be treated as two-dimensional. For typical methods of solution, refer to DM-7.02 Foundations and Earth Structures. The elastic foundation method may be used to assist in estimating foundation pressures for the special loading conditions discussed in section (6.4.8).

(6.4.9.3) Computer Analysis. The Naval Facilities Engineering Service Center has a three-dimensional computer program for analyzing or designing drydocks.

6.5: SAFETY CONSIDERATIONS

(6.5.1) Basic Safety Standards. For general safety standards see OSHA Part 1915, Occupational Safety and Health Standards for Shipyard Employment.

(6.5.2) Safety Features Peculiar to Drydocks. Observe the safety features described in sections 5-9.2.1 through 5-9.2.7

(6.5.3) Coping Railing. It is necessary for coping railings to be removable to avoid fouling lines when docking and undocking ships. The removal and replacement must be accomplished with as little hazard as possible, because of the seriousness of the accident should a person fall into an empty dock. Chain rail with removable stanchions is often used, but maintaining adequate chain tension is a common problem. Solid metal pipe or fiberglass railing, provided in 1.8-3.0 m (6-10 ft) sections for ease of removal/reinstallation, is preferred.

(6.5.4) Stairways. Use open mesh treads on all framed stairways. Use non-slip treads for concrete stairways. Provide closing chains at top of steep, infrequently used stairways. Preferably, stairways should have pipe handrails. Chain and stanchion handrails may be used as an alternate.

(6.5.5) Toe Guards. Provide toe guards at all handrails wherever possible. At the coping edge, a curb may serve as a toe guard

(6.5.6) Obstructions to Mooring Lines. Keep the top of coping clear between the edge of the coping and the line of capstans and bollards, except for luffing and fairing line handling fittings.

(6.5.7) Stepdown in Tunnels and Culverts. Avoid unprotected stepdowns in all unlighted tunnels and culverts. Use guardrails, or an arrangement of rails and gratings, to protect personnel while still retaining the water carrying capabilities of the tunnel or culvert

(6.5.8) Spillways. Provide ladder rungs or handrails, depending on steepness, up flooding and discharge spillways, to aid in access for inspection of sluice gates and stoplogs.

(6.5.9) Dock Floor Irregularities. Graving dock floors should present an unbroken surface. Cover drainage conduits with gratings that do not protrude above the floor level. Run service pipes near the sidewalls and bridge them at stairways. Floor finish should be on a true plane and sufficiently rough to prevent slipping, but not so rough as to injure blocks.

(6.5.10) Painting. Paint obstructions with a striped pattern composed on contrasting colors as outlined in UFGS 09900N. Paints and Coatings.

- Service pipes are customarily painted according to established color codes, but those below headroom or otherwise forming obstructions should be painted with contrasting colors in stripes.

- Channel pedestrian traffic through safety zones by marking the borders of pedestrian passageways with traffic zone paint.

6.6: Main dimensions of the dry dock

dry dock generally speaking, a chamber or basin separated from the adjacent harbour waters by dock gate see Fig (6.14)

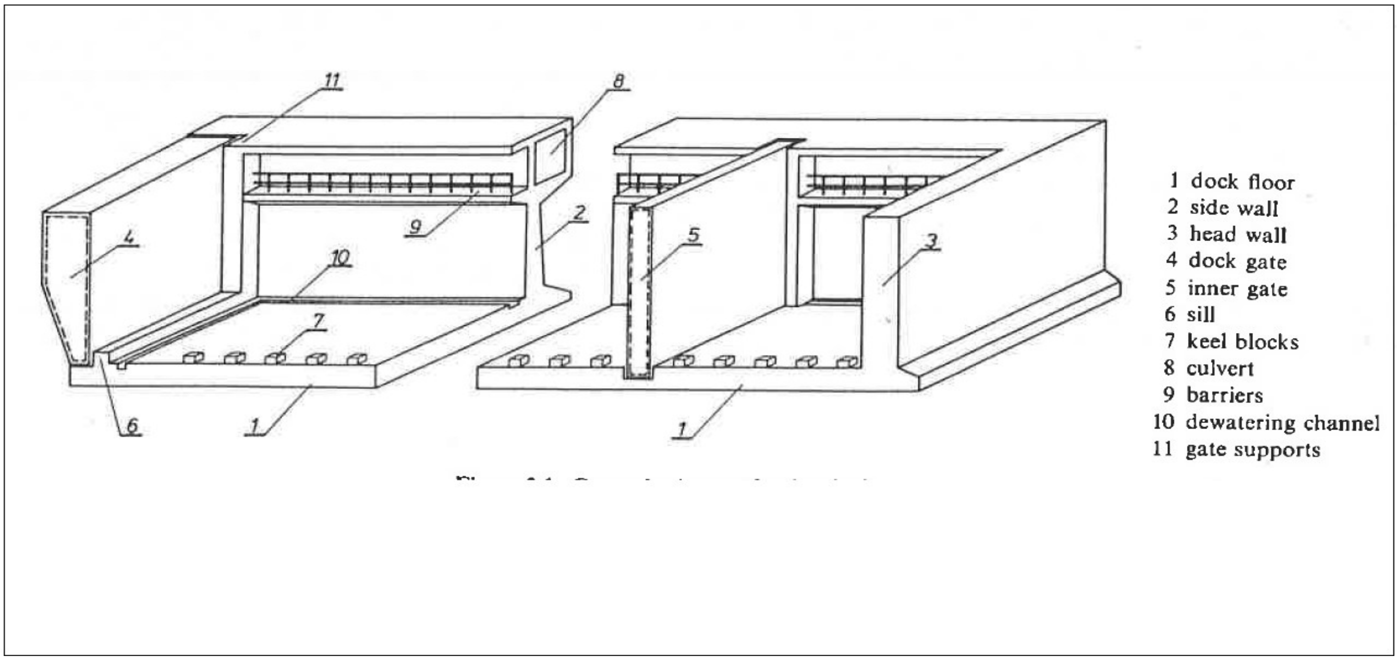


Fig (6.14) General Scheme of a dry dock

Each dry dock is characterized by physical dimensions which define the size of ships that can be built or repaired in it. Those data relate to the following Fig (6.15)

1. Total length of the dock structure**, Ld**.

2. Effective length of the dock, Lu. This length should be up to about 3-4 m greater than the total length of the ship using the dock .If the screw shaft in the hull has to be installed or removed, the free space should be extended by 10 to 30 m. If double seats for the dock gate are provided, then the length L_u should be measured up to the gate face placed on the seat nearer to the head wall.

3. Total width of the dock structure, $\mathbf{S_d}$ **.**

4 . Effective width of the dock , Su .very often this is the distance between the fenders. The effective width is established by taking the greatest width of the ship plus an allowance on each side for working. This allowance ranges from 3 to 6 m on each side depending on the kind of dry dock, i.e., building or repair .In existing docks and docks under construction the ratio of effective length Lu and effective width Su is usually between 5 to7 .Because of the system of ship-building in halves and later welding together in the water, as well as the shifting of sections from one part of the dry dock to another, this ratio for building docks can of course, change rapidly. For the future, however the following relation between the effective width and the effective length of a dry dock can be assumed:

$$\mathbf{S_u = (0.15 \text{ to } 0.18)\ L_u}$$

5. The width of the dry dock at dock coping level, S_t.

6. The width of the dry dock at dock floor, S_b.

7. The width of the dry dock, S_k, i.e., the average distance between the side walls equal to

$$\mathbf{S_h = 0.5(S_t + Sb)}$$

8. The width at entrance,$\mathbf{S_w}$. It should be 1.5 to 2.0 m wider than the beam of the biggest ship to be docked.

9. The depth of entrance**,** $\mathbf{h_w}$, i.e., the depth of the sill at the design water level. For design purposes in seas without tide, or with small tides in relation to the total water level changes. the depth corresponding to the average level of low water should be taken. In cases when no waiting of a ship can be allowed, the depth of entrance is determined with respect to absolute lowest sea level. Extending the depth of a dock in this manner, however considerably increases construction costs, so the correct level can only be decided when all available water level data and the possible production procedure in the dry dock have been analyzed. The comparison of the estimated costs of the ship's idle time and the increased costs of dock construction will determine the proper design level. In docks where warships are to be serviced, docking must be ensured even at the lowest water levels.In tidal harbors, the largest ships are usually docked during high tide. The level at high tideis taken and the choice of the design level is made according to this level. Because of the regularity of tides, the waiting period for the correct water level is relatively short. Thus the lowest predicted High Water Neap Tides are used for design.The depth of entrance is the basic feature distinguishing the types of dry docks. Thus, the necessary depth entrance is defined on the basis of the ship's draught with cargo, without cargo as of the ship's draught with cargo, without cargo as well as the critical draught ofa damaged ship. Thus, the depth of entrance, h_w, to accommodate a ship of maximum draught, $\mathbf{T_c}$, is

$$\mathbf{h_w = T_c + r}$$

where **r** is the required free space between the keel and the sill, usually 0.5 m.

10. The depth of dry dock at the entrance, $\mathbf{h_{kw}}$

11.The depth of dry dock at the head wall, $\mathbf{h_{kt}}$

Depths $\mathbf{h_{kw}}$ and $\mathbf{h_{kt}}$, may be identical or different. They are different when the floor has a longitudinal inclination towards the sill. The depth at the entrance $\mathbf{h_{kw}}$ is greater than that of the head wall $\mathbf{h_{kt}}$, according to

$$\mathbf{h_{kw} = h_{kt} + L_u s}$$

where $\mathbf{s}$ = the longitudinal inclination of the dock floor. It is assumed that the depth of the dry dock at the entrance h_{kw} should not be less than the sum of the depth of entrance hw and the height of the keel blocks h_s, so that

$$\mathbf{h_{kw} \geq h_w + h_s}$$

Hence the depth at the head wall should be at leas

$$\mathbf{H_{kt} \geq h_w - L_u s}$$

In docks serving warships it is assumed that the sill is at the dock floor level, i.e , $\mathbf{h_{kw} = h_w}$

12. The depth of the dry dock, $\mathbf{h_k}$, equal to

$$\mathbf{H_k = 0.5(h_{kw} + h_{kt})}$$

13. The effective dry dock depth, $\mathbf{h_u}$, i.e., the depth of the upper level of the keel blocks measured from the design water level.

14. The height of the coping edge, $\mathbf{h_p}$. i.e., the distance between dock wall coping level and water level. This height should be at least 0.5 m above the highest sea level possible.

15. The height of the dock structure, $\mathbf{H_k}$.

16. The height of the dock walls at the entrance, $\mathbf{H_{cw}}$.

17. The height of the dock walls at the head wall, $\mathbf{H_{ct}}$.

18. The height of the dock walls at the sill, $\mathbf{H_p}$.

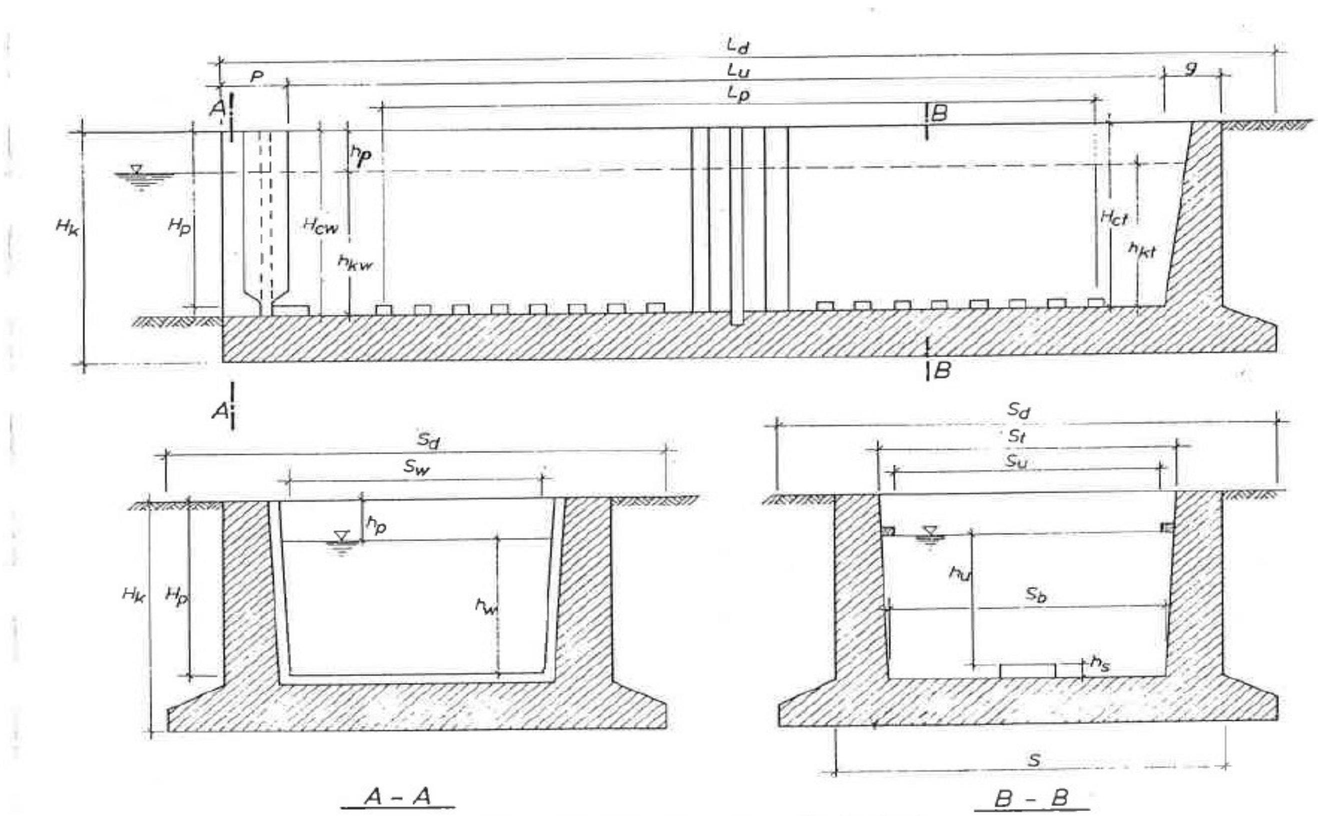
19. Dock volume, **V**, i.e., the volume of water which at the design water level, must be pumped out of the dry dock for complete dewatering. For a rectangular cross-section of a dry dock and rectangular longitudinal section, this volume can be calculated by

$$\mathbf{V = L_U h_K S_K}$$

20. The factor of dock capacity, w_p, i.e., the ratio of the displacement of the greatest shi9p possible for docking **W** and the dock volume V

$$\mathbf{W_p = W/V}$$

It ranges usually from 0.4 to 0.6 but it may be much lower in docks of great width.



Main dimensions of dry dock Fig (6.15)

EX: A shipyard dry-dock, initially emptied from water is used to construct and build a new ship of a mass of 100 metric tonnes. See *Fig* **(6.16)** below. The ship has a width of 8 m, height of 5 m and length of 20 m. The ship rests on 10 support blocks each of dimensions of width 1 m, height 1 m and length 9 m. The dock will be refilled with seawater to match the sea level outside the dock gate, while the ship floats and rises. The dock gate has a width of 10 m and a height of 5 m, while the length of the dock is 40 m. Assume the level of the seawater outside the dock is inline with the top of the gate.

Find:

(i) the resultant force *and* the location of the centre of pressure on the dock gate when the dock is empty.

(ii) the volume of seawater pumped into the dock at the moment when the ship becomes buoyant.

(iii) the time taken to fill the dock until the water level inside the dock equals the sea level outside, if 10 identical pumps are used each having a volumetric flow rate of $0.1\ m^3/s$.

(iv) the metacentric height of the ship if a point-load of 30 metric tonnes is placed at the centre of the top deck. Take the centre of gravity of the ship at the centre of volume of the ship.

Take density of seawater to be 1.025 x density of fresh water.

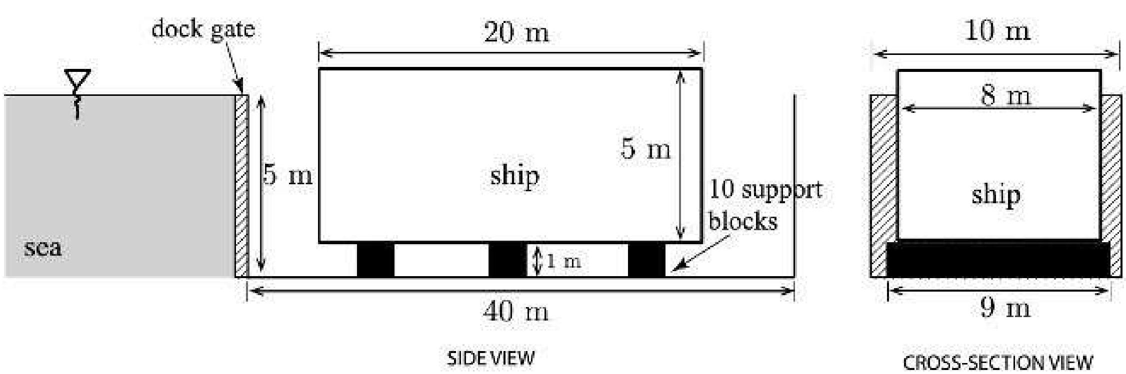
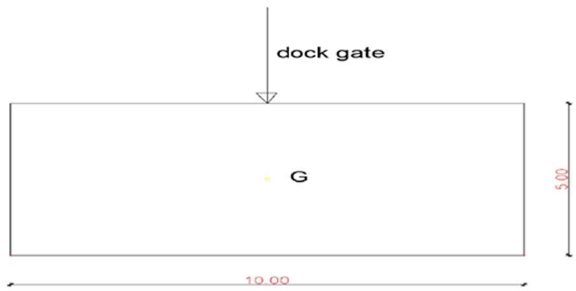


Fig **(6.16)** - Dry-dock setup

SOLUTION



$I_G = bd^3/12$

$\rho = 1025\ Kg/m^3$

$F = \rho\ g\ A$ x (distance of CG from the surface = ꞇ)

$= 1025 \times 9.81 \times 10 \times 5 \times 5/2$

$= 1256906.25$

Center of pressure

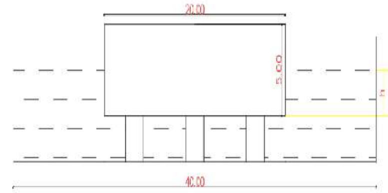
$= I_G/Ah + h'$

$= (10 \times 5^3 \times 2)/(12 \times 10 \times 5 \times 5) + 5/2$

$= 3.3$ m from top

(ii) The ship becomes by just

when



$Mg = \rho\ g\ h \times A$

$100 \times 103 \times 9.81 = 1025 \times 9.81 \times h \times 8 \times 20$

$h = 0.60975$ m

h' = high of the ship immersed in water

A′ = Area of the base

Volume of the sea water pumped = (h + 1) 40 – volume of immersed part of ship -volume of blocks

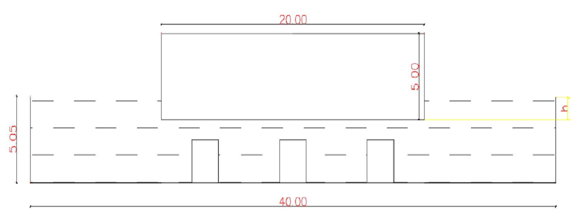
= (h + 1) 40 x 10 – h x 20 x8 – 9 x 1x1 x 10

456.34 = m

(iii)

Volume of sew water = 5 x40 x 10 – 20 x 8 x h – 9 x 1 x1 x10

=1812.44 m^3



Time taken = 1812.44/10x0.1 =1812.44 sec

(iv)

130 x 10^3 x g = ρ x 20 x 8 x g x h

h = 0.792682 m

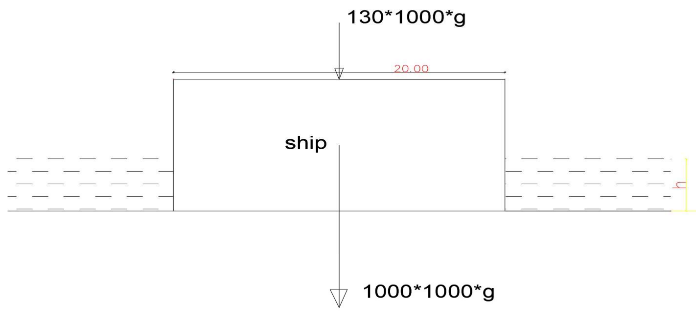
AB = h/2 = 0.396341 m

AG = 5/2 = 2.5 m

GM = 2.103659 m

GM = I/v -BG

$= 8 \times 20^3 / 12 \times 8 \times 20 \times (0.792682) - 2.10369 = 39.947$ m



Chapter Seven

Berthing Facilities

7. Introduction

7.1 Water transportation

is concerned with conveyance of people in what is concerned with conveyance of people and goods in vehicles that float upon water these vehicles may be hollow vessels of wood or metal or a combination of both. which are made to pass over water by natural or mechanical power .The small vessels are often called boats and large , these vessels ships are generally classified according to size construction power and purpose the waterway for water transportation is any navigable water which is free of obstructions and of sufficient depth of carry vessels, these navigable Waters exist on inland Lakes streams and Rivers estuaries bays seas and oceans most of these waterways are natural but some are manmade like canals .[1] fig(6-1)

Fig (7-1) Water transportation

7.2 Waterways:

can be classified broadly as oceanic waterways and inland waterways oceanic. waterways are concerned with the conveyance of people and goods primarily across the ocean between continents or island, but inland and Rivers waterways consist of water transportation on rivers lakes and canals within the main land. (1)

7.3 harbor:

the protected area within a break water area within breakwater is the end of maritime transport ,it consists of wave barriers that surround the water area, the entrance to the port, the navigation channel that connects the ship to the port and the rotational water area in which the ship rotates, which is a circular shape designed with a specific rotational radius(2) .

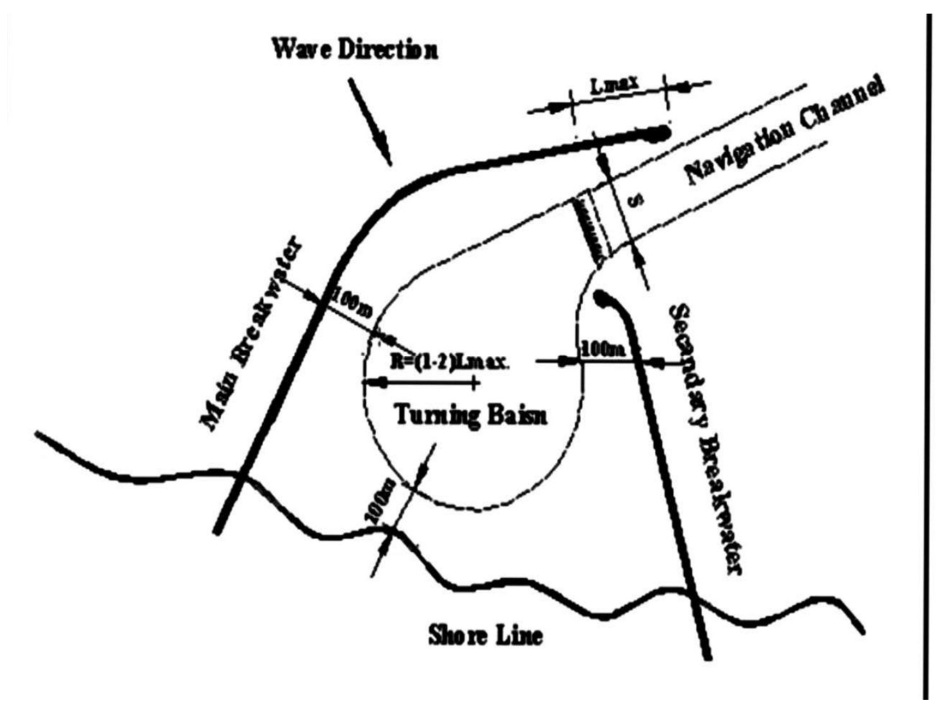


Fig (7-2) harbor

7.4 Port

The water area in addition to the land area, which includes breakwaters, docks, storage, areas and all service facilities located on that land area, and it is also includes maintenance facilities for ships [2].

• **Function:**

- Transfer cargo between various transport modes.
- The other major function is storage.

• **Services:**

- Sea-related services (buoys and moorings, berths, tugs
- Land-related services (cargo handling by port
- Labor, cranes, equipment, storage ... etc), and
- Delivery services (handling rate, warehousing, port's own transport system .... etc.

Fig (7-3) Port – harbor

7.5 Port structure [3]

• berth

Any place where a ship can safely lie alongside a quay, pier or dock, at anchor or a buoy, and where she can carry out loading/discharge operations or embark and disembark passengers

• wharf:

It denotes any structure of timber, masonry, cement, or other material built along or at an angle to the navigable waterway, with sufficient depth of water to accommodate vessels and receive and discharge cargo or passengers. Fig (6-4)

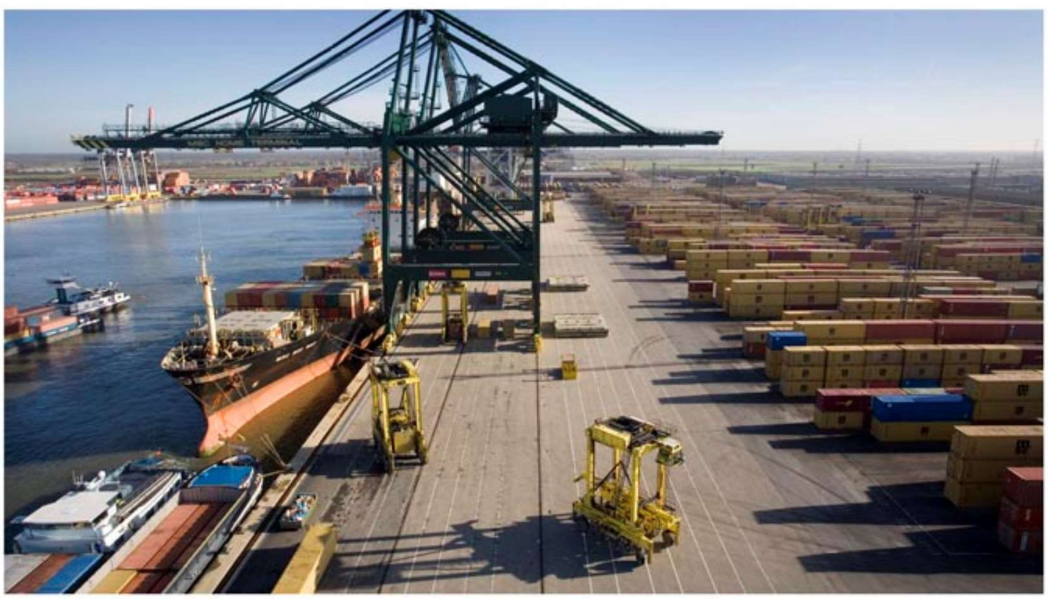


Fig (7-4) wharf

• pier

A ***pier*** is a construction work extending into the harbor with sufficient depth of water alongside to accommodate vessels, also used as a promenade or landing place for passengers**.**

Fig (7-5) pier

• jetty

A ***jetty*** is a small pier, usually made of timbers for boats, yachts or fishing boats *(fisherman jetty),* but it also refers to large ships.

• dock:

has a number of meanings it is an artificially enclosed basin into which vessels are brought for inspection and repair.

Fig (7-6) : Jetty

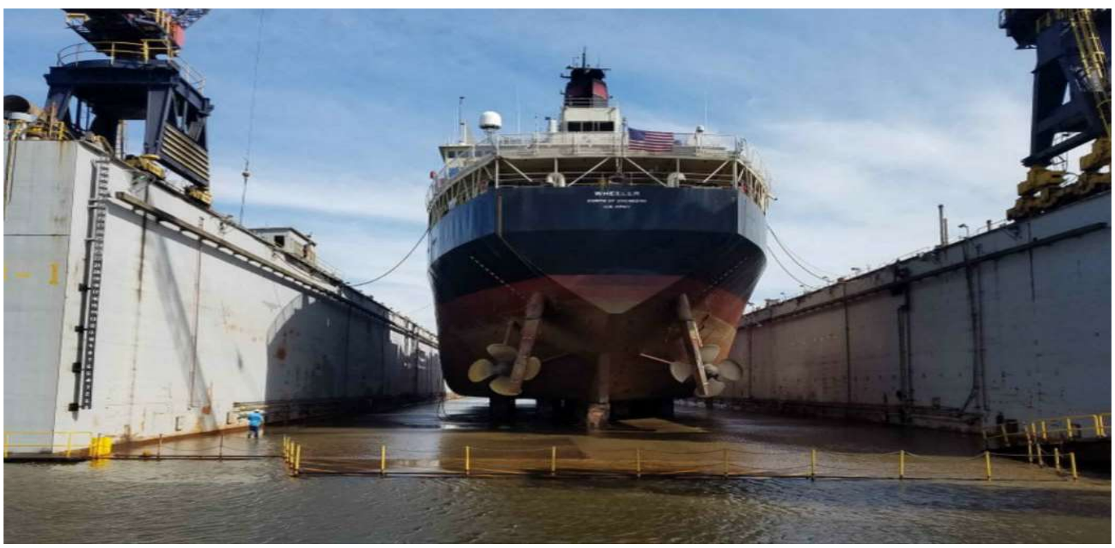


Fig (7-7) : dock

7.6 Berthing Facilities [4]

Nautical structures built in ports when docks are installed. The chief purpose of berthing facilities is to facilitate a ship's approach and mooring process.

Such structures set along the shore are called quays, while those that protrude into the p ort waters at an oblique or right angle to the shore are called piers.

6.6.1 Classification berthing according to shape [2]

Open-type:

The section of the berth allows wave motion under or through the berth. They are usually constructed on piles or floating deck types. fig (6-8)

Closed-type (Solid-type)

The section of the berth is almost impermeable and does not allow the wave motion through the berth. Fig (7-9)

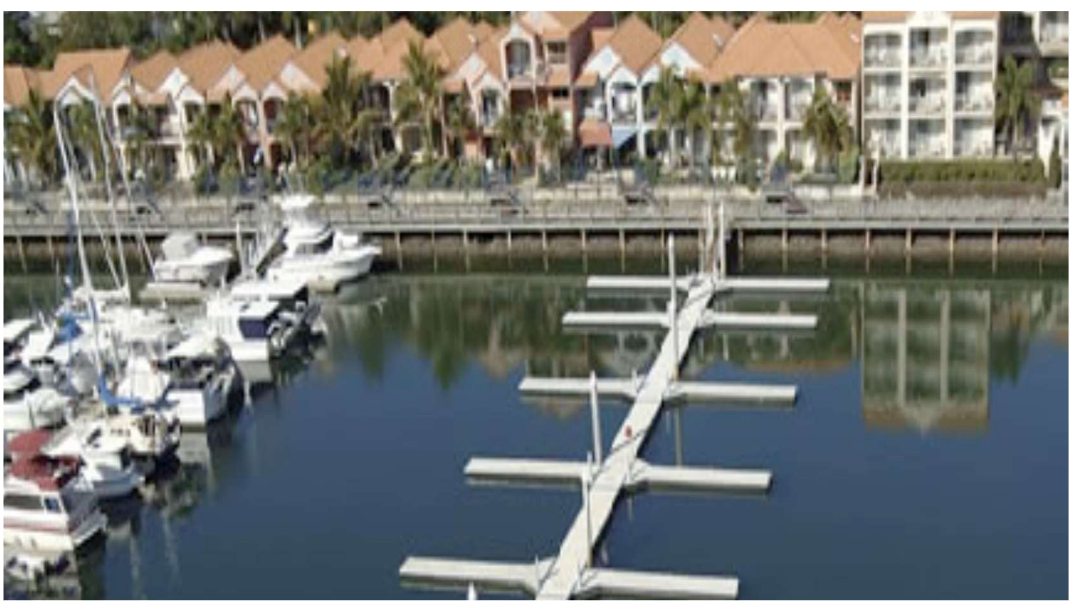


Fig (7-8) open type berthing

Fig(7-9) closed type berthing

6.6.2 Type of berthing facilities according to design

1. **cantilever type - wall (pile):**

   t is a row of blinds running along the front of the marina structure to depths below sea level in front of the structure, so that these depths are sufficient to stabilize the walls.

2. **Anchored –bulk heads (pile):**

   It is a row of curtains like the previous type, but these curtains can be linked to back clamps that help achieve wall balance, in addition to what the soil does from the support process along the curtains below the bottom line. Fig (7-10)

3. **Cellular type –quay wall:**

   They are formed by the method of driving the curtains in different arrangements to have cells between them and fill with sand or an appropriate type of fill. Fig (7-11)

4. Relative plate forms :

The structure in this case consists of a platform working of reinforced concrete concentrated over a group of vertical and inclined piles and piling the pillars parallel to the longitudinal direction of the structure at the front or rear edge of the platform and these pillars support part of the backfill that is located behind the structure. Fig (7-12)

5. Walls consisting of precast blocks made of regular concrete:

These blocks are poured into the pouring yard close to the beach in the form of blocks one next to the other to form a wall fig (7-13)

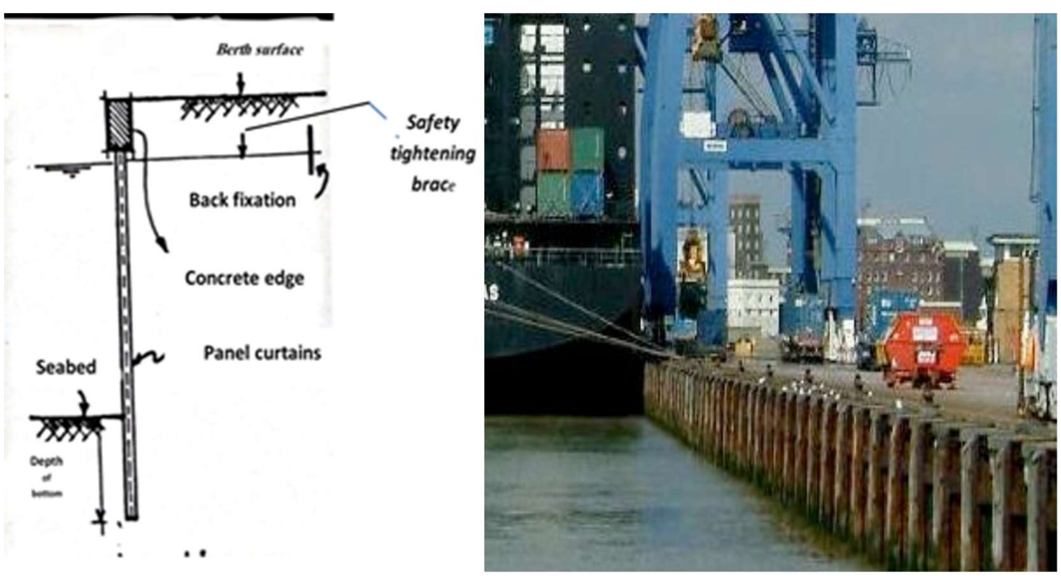


fig (7-10) Anchored –bulk heads

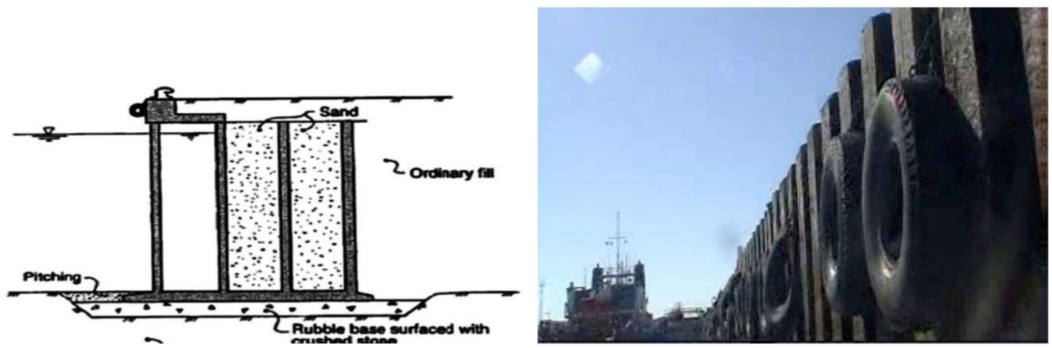


Fig (7-11) cellular type –quay wall

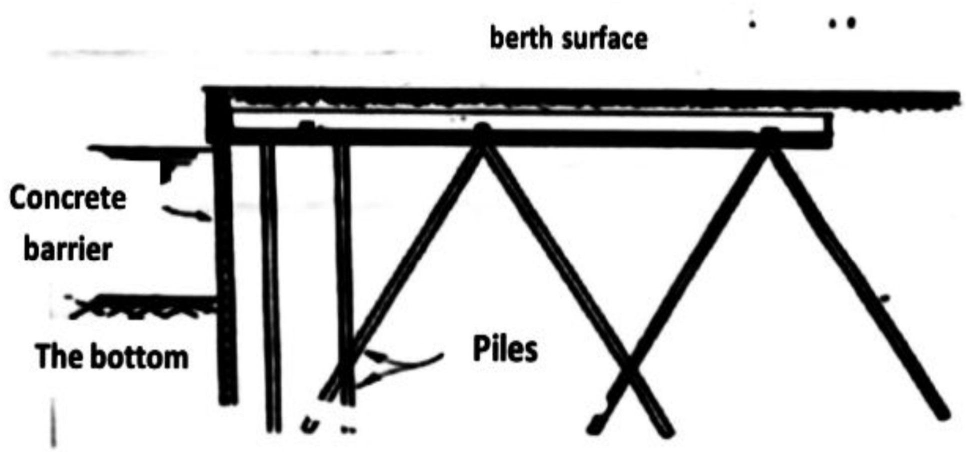


Fig (7-12) relative plate forms

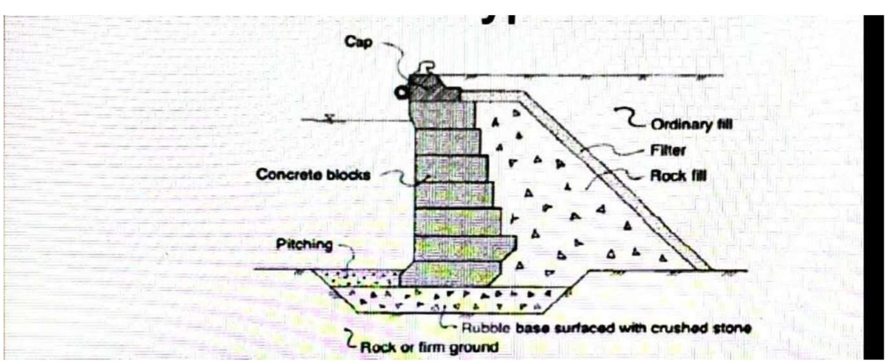


Fig (7-13) Walls consisting of precast blocks made of regula

7.7 The design of the marina facilities depends on the following factors:

1- Determining the pavement surface level, depending on its uses and the extent of the tide.
2- The width of the slope of the sidewalk façade.
3- The length of the structure and the depth of the bottom in front of it, and this depends on the load of the ship and the type of vessel g.
4- Loads above the sidewalk surface.
5- Forces that act to displace the ship, such as forces resulting from wind pressure on the ship's deck, the force of clouds of water currents, and forces resulting from wave shocks on the ship's deck that floats above the water surface.
6- The nature of the bottom: the availability of a contour map of the seabed and knowledge of the physical and mechanical properties of the bottom soil.

7.8 Location and area of turning basin [2]

Area of basin for ship maneuvering shall exceed an area of a circle with a radius of 1.50 times the overall length of the ship. fig (8-1)

$$R_{b} = L_{s} \times 1.50$$

R_b= radius of area circle basin

A basin used for anchorage may have a water area exceeding the area of a circle with a radius given by:

a) single buoy mooring R = L + 25 m,

b) double buoy, rectangular (L + 50 m) (L/2)

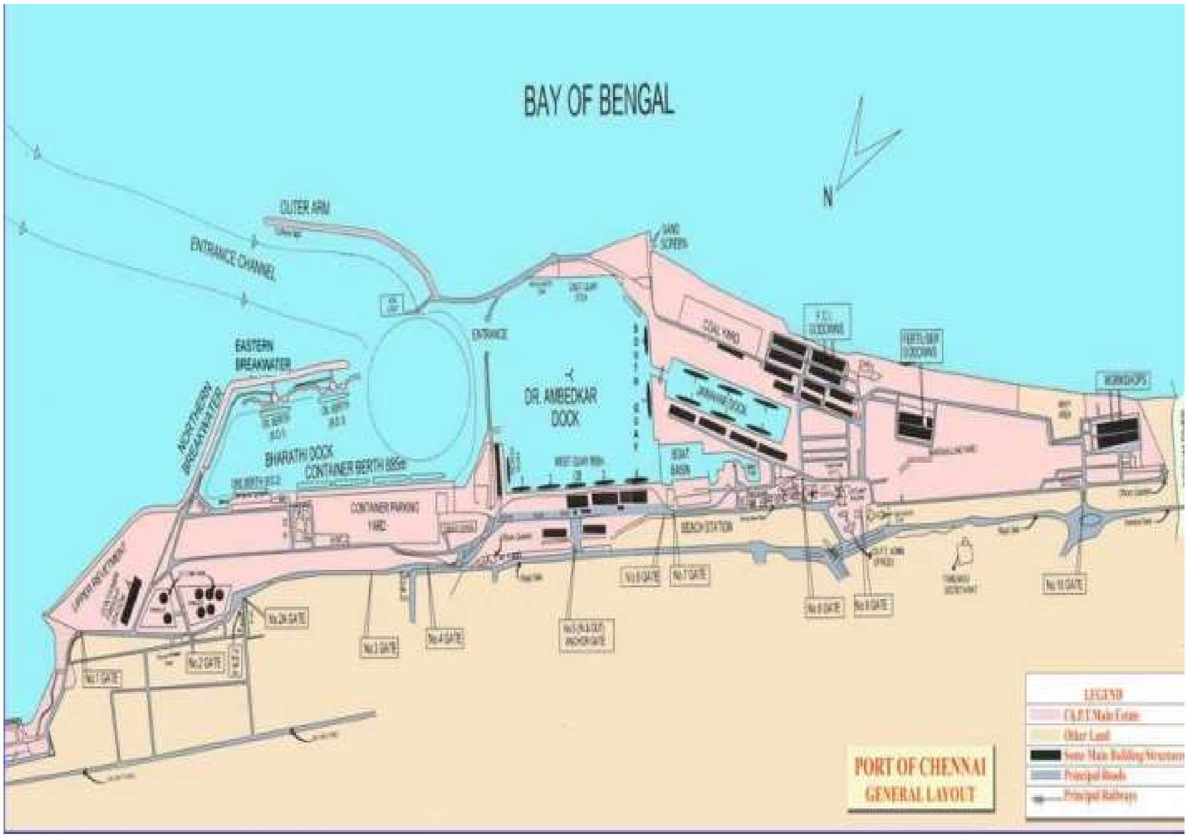


Fig (7-14) basin for turning ship area and anchor

7. 9 Berth Planning

For straight line berths:

Length of berth = Ls + 2 (10-15) meters

Ls: is the overall length of ship= 1.1 ship wetted length

For a basin berth:

Width of basin = 4 - 5 B for 2-ship berth.

B is the ship beam width

Width of basin = 7-8 B for more than 2-ship berth

7.10 Harbor entrance channel:

The entrance must be located to minimize the

effects of cross currents.

The layout should be as straight as possible.

The minimum net under keel clearance is:

0.50 m for sandy bottom

1.0 m for a rocky bottom.

The width >1.8 B of largest vessel (one way).

>4.6 B of largest vessel (two way).

Under keel clearance should be determined by three types of[5] factors:

- Water level factors in the Sea. These include the reference of water level, mainly due to meteorological effects, which can have a positive or negative value.
- Ship related factors. These include ship's maneuvering characteristics, as given in the IMO resolution A. 601(15) on Provision and Display of Manoeuvring Information on Board Ships. The effect of water density should further be considered.
- Sea bottom related factors. These include allowance for sea bed level uncertainties and allowance for bottom changes etc. Experiences have shown that these effects could have a value of approximately up to two meters in some area fig (8-1)

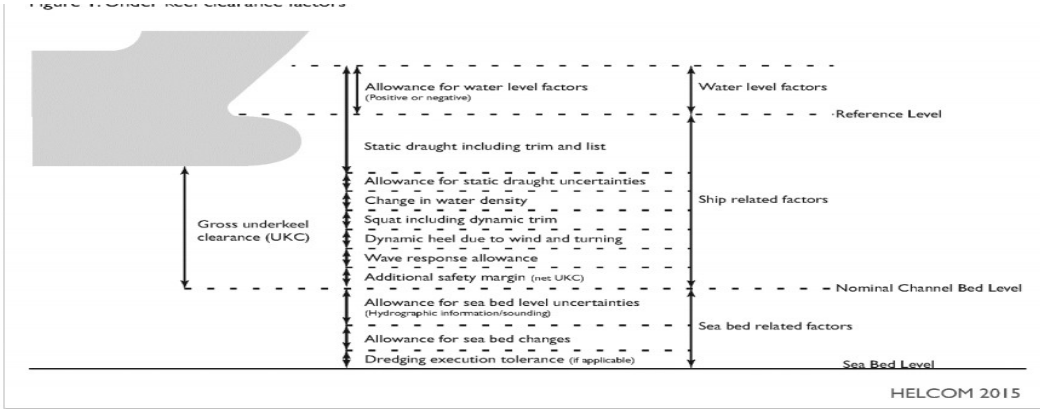


Fig (7-10.1) Under keel clearance factors

Depth of Navigation Channel:

h =Dr + Zmax + G + I + A + C

where

h = Total depth of channel

Dr = Draft of vessel (loaded)

Zmax = Max. squat of the vessel at the speed allowed

G = Deviation of the water level

A = Allowance for the bed fluctuations

C = Under-keel clearance

I = Allowance for vertical motions due to waves

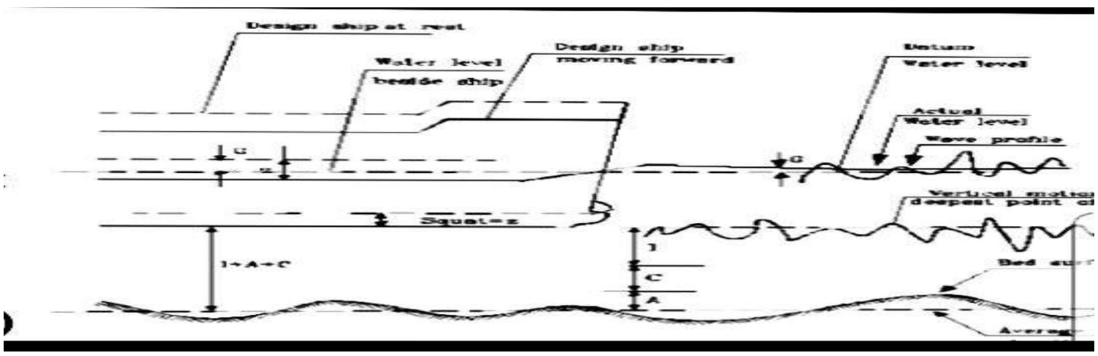


Fig (7-10-2) depth of navigating channel

Chapter Eight

Navigation Locks and Fish Ladders

8-1 Navigation locks

8-1-1 General

Locks are device have used to provide safe passage for navigation of boats, ships and other watercraft between stretches of water of different levels on river and canal waterways. The distinguishing feature of a lock is a fixed chamber in which the water level can be varied whereas in a caisson lock for example, a boat lift because the chamber itself (usually then called a caisson) can rises and falls. Locks used also to allow a canal to cross-land that is not leveled. Later canals used more and larger locks to allow a more direct route to be taken. The figures (8-1 & 8-2) below show specific navigation lock in Panama Canal.

Fig (8-1) Panama Canal lock

Fig (8-2) Panama Canal navigation

8-1-2 Pound locks

A pound lock is a type of lock that is used almost exclusively nowadays on large waterways. A pound lock has a chamber with gates at both ends that control the level of water in the pound. In contrast, an earlier design with a single gate was known as a flash lock. The water level could differ by 4 feet (1.2 m) or 5 feet (1.5 m) at each lock and in the Grand Canal and the level was raised in this way by 138 feet (42 m). This type is illustrated in fig (8-3) below.

Fig (8-3) pound lock

8-1-2-1 Primary Components of pound locks

1. Upper approach

The canal immediately upstream from the lock is referred to as the upper approach. The guide wall serves to align and to guide a down bound tow into the lock chamber and is usually a prolongation of one wall of the chamber. The guard wall provides a barrier that prevents the tow from entering an area having hazardous currents or potentially damageable or damaging structures. See fig (8-4)

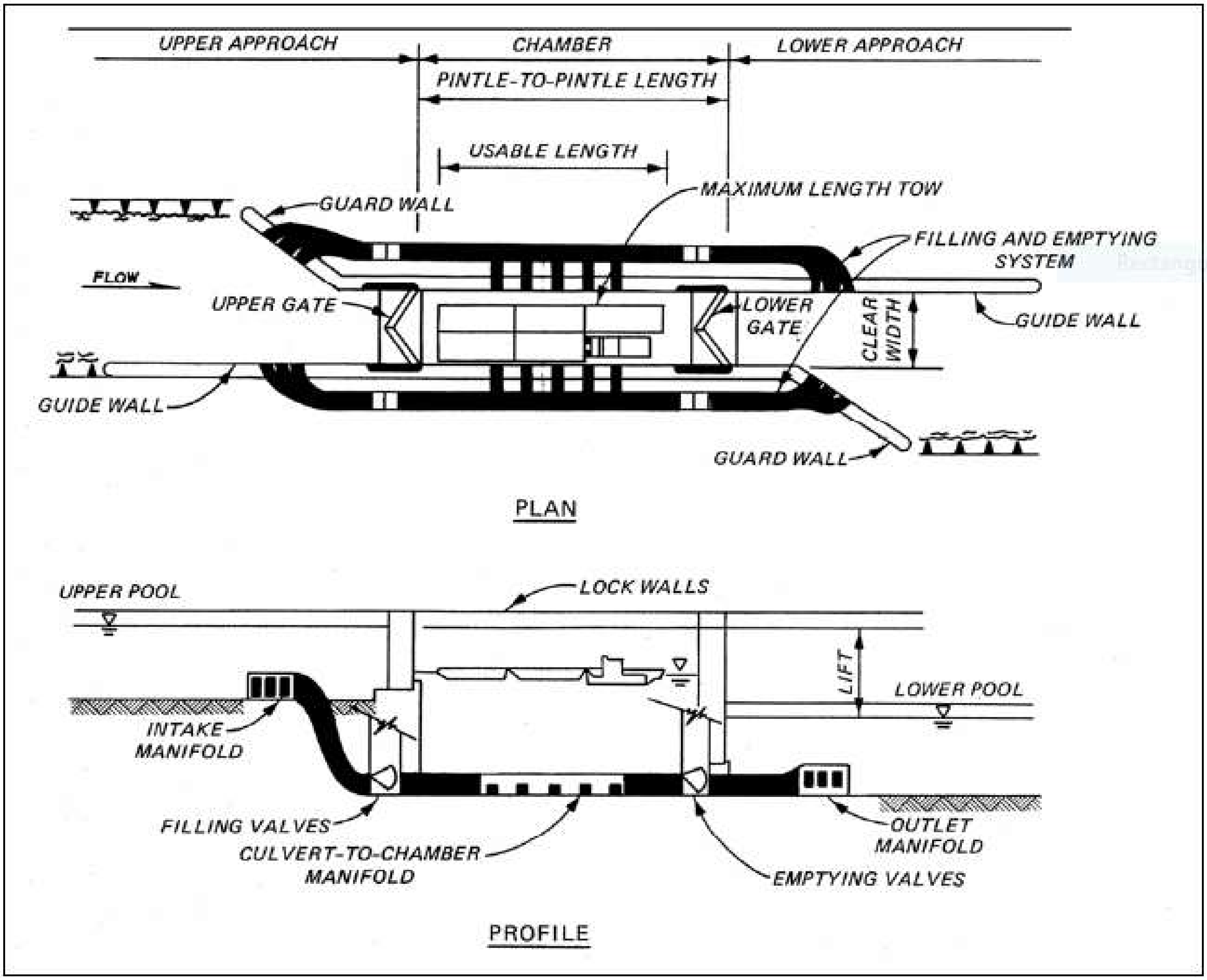
2. Lock chamber and gates

The down bound traffic is lowered to lower pool and the up bound traffic is raised to upper pool within the lock chamber. The upper and lower gates are movable barriers that can be opened to permit a vessel to enter or exit the chamber. The gates used, as mitre gate, rolling gate and caisson gate will discussed later separately. See fig (8-4)

3. Filling and emptying system

For a lock filling operation, the emptying valves are closed. The filling valves are opened. Flow enters the intake manifolds and exits by means of the culvert-to-chamber manifolds into the lock chamber. For emptying, the filling valves are closed and the emptying valves are opened. Flow enters the culvert-to-chamber manifolds and exits by means of the outlet manifolds. See fig (8-4)

4. Lower approach. The canal immediately downstream from the chamber is referred to as the lower approach. Guide, guard walls are used and defined similarly as upper approach. See fig (8-4).



Common pound lock features and component with culverts in the sidewall Fig (8-4)

8-1-2-2 Classification Systems of pound locks

1. Locks classification duo to lifting as follow in the table (8-1)

Table (8-1) classification duo to design lift

Range of Maximum Design Lift (ft to ft)	Project Classification
0∨ to 10	Very low lift
10 to 30/40	Low lift
30/40 to 100	High lift
100 to ___ (Undefined)	Very high lift

2. Locks classification duo to filling/emptying system as shown in the Fig (8-6).

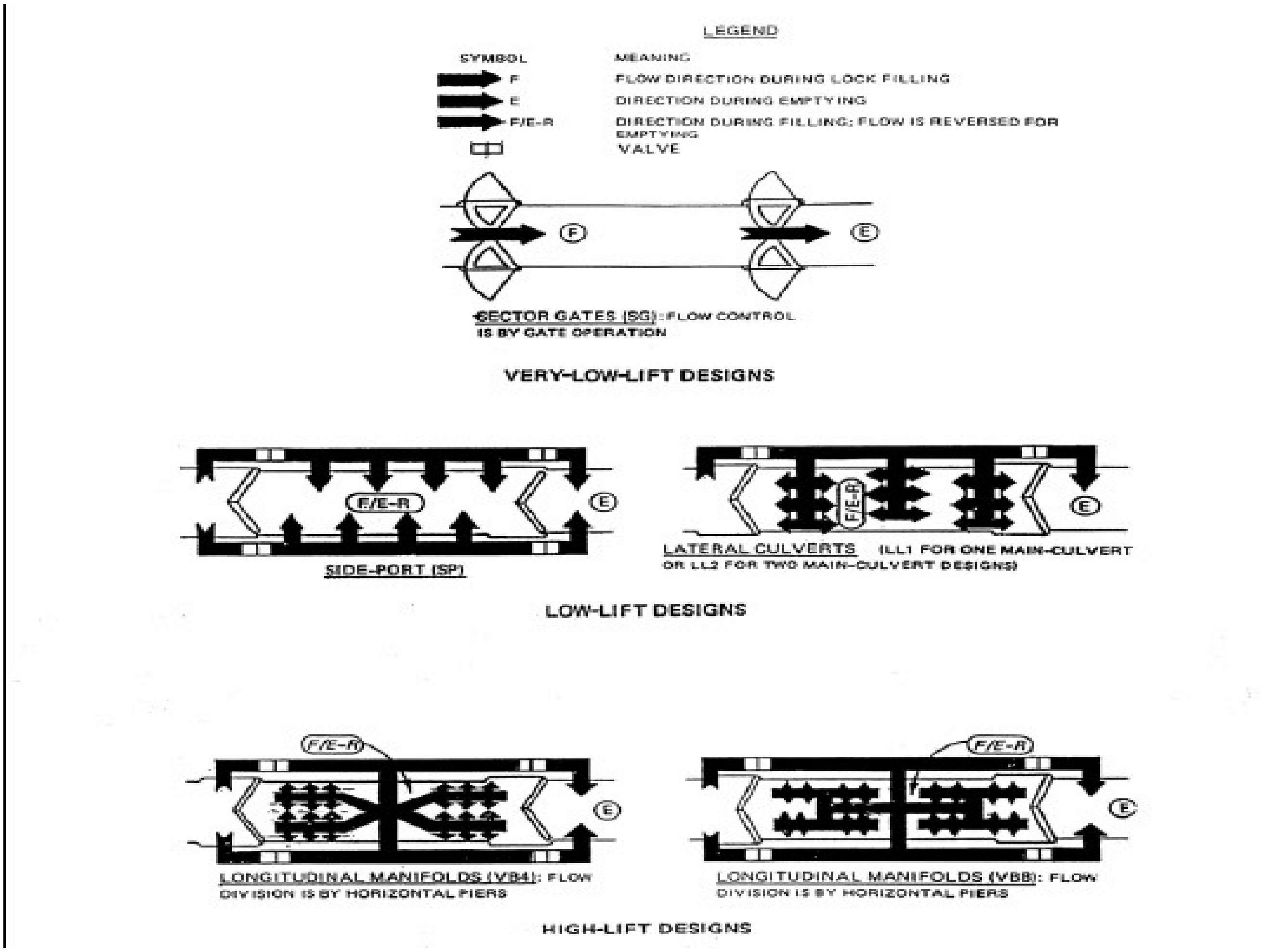
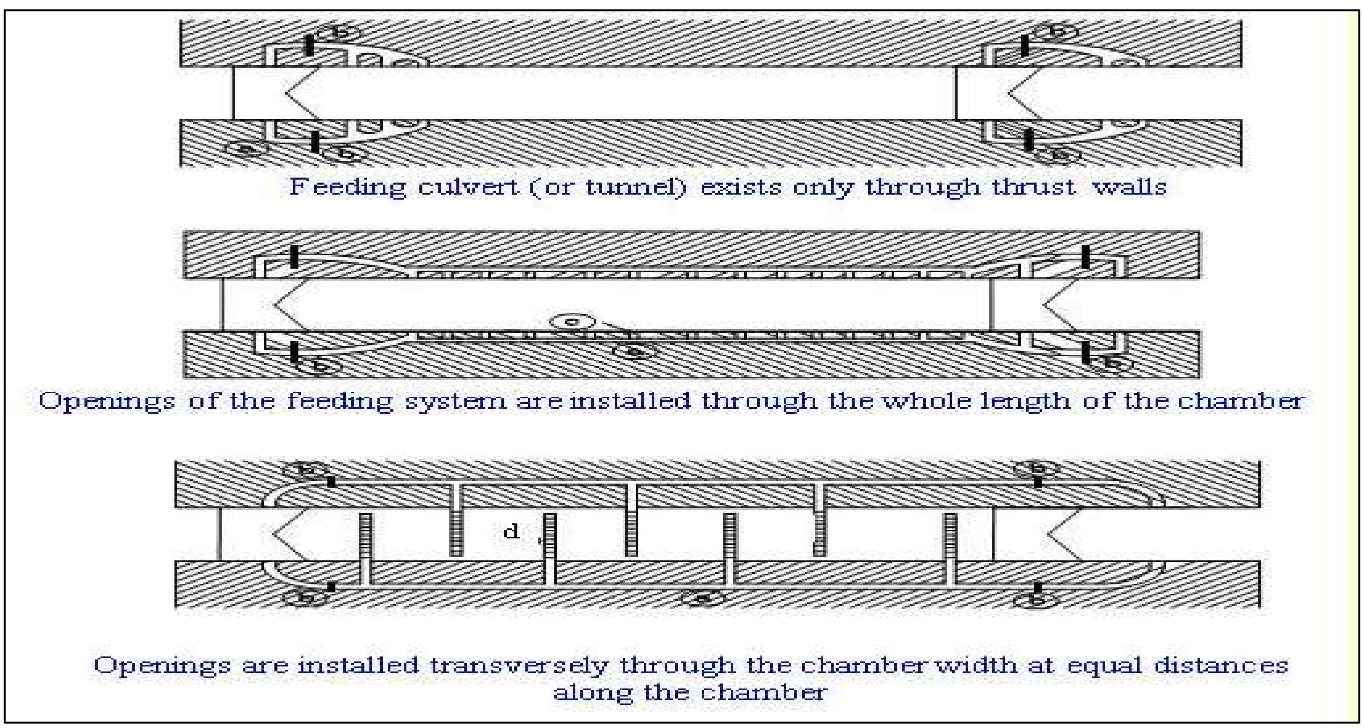


Fig (8-6) Flow distribution (filling and emptying) systems of recommended design

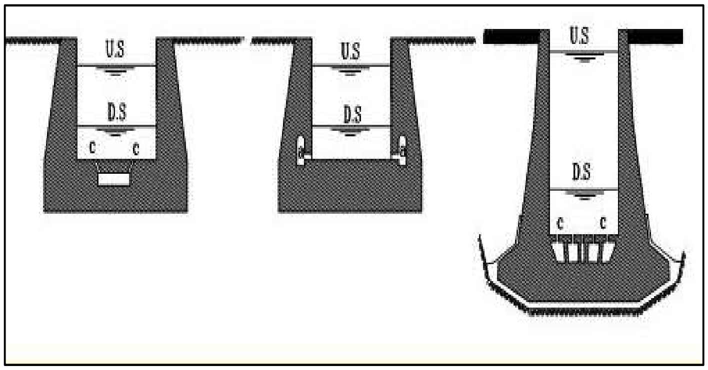
8-1-2-3 Different types of feeding systems for pound locks

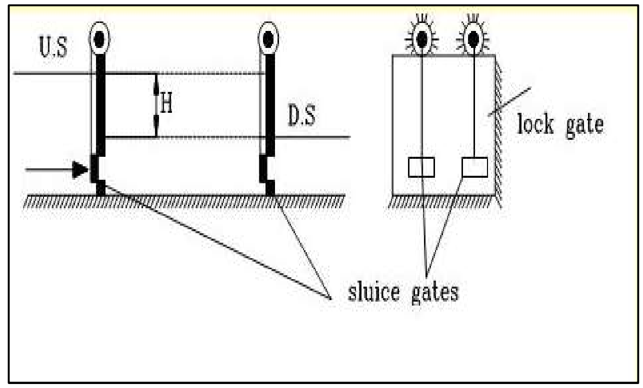
1. culvert (or tunnel) at floor
2. sluice gate at thrust wall
3. longitudinal culverts along landing wall
4. Cross pipes (or openings) above the floor level.

A-side port system



B- End culvert, lateral side ports and openings of pipes along the lock length





C- Side section of lock with culvert system D- using sluice gate in feeding system

Figs (8-7) A,B,C,D are different types of feeding systems

8-1-3 Classification of vessels

8-1-3-1 PIANC (International Navigation Association) defined a number of classes for standard navigation vessels that could be used for designing of locks as shown in table (8-2).

Table (8-2) Recommended large vessel dimensions

R/S klasse	maximum permitted vessel dimensions (m)			minimum bridge passage height (m)
(River/Sea)	length	width	draught	
1	90	13.0	3.5 of 4.5	7.0 of 9.1
2	135	16.0	3.5 of 4.5	≥ 9.1
3	135	22.8	4.0 of 4.5	≥ 9.1

8-1-3-2 ECMT (European Conference of Ministers of Transport) specify their own classification for standard inland vessels as illustrated in table (8-3)

Table (8-3) ECMT classes of vessels

CEMT class	type vessel	length L (m)	width B (m)	draught T (m) empty	loaded	clearance H (m)
I	Spits - Peniche	39	5.1	1.2	2.2	5.0
II	Kempenaar- Campinois	55	6.6	1.4	2.5	6.0
(IIa) **)	Hagenaar	56 of 67	7.2	1.4	2.5	6.3
III ***)	Dortmunder	67 of 80	8.2	1.5	2.5	6.3
IV	Rijn-Hernekanaalschip	85	9.5	1.6	2.8	6.7
Va	Big Rhine barge Push barge	110	11.4	1.8	3.5	6.7/8.8 *)
Vb	Pushed convoy	186,5	11.4	1.8	4.0	8.8
VIa	Side-by-side formation	110	22.8	1.8	4.0	8.8
VIb	Pushed barge train	186,5	22.8	1.8	4.0	8.8

8-1-3-3 In Netherlands, the CVB (Commissie Vaarweg Beheerders) formulated the Guidelines on Waterways for Recreational navigation vessels as illustrated in table (8-4)

Table (8-4) Recreational navigation classes

Category	class	height (m)	draught (m)	width (m)	length (m)
Sailing boats	1	8.50	1.25	3.00	9.00
	2	12.00	1.50	3.50	10.00
	3	12.00	1.75	3.75	11.00
	4	>>12.00	1.90	4.00	12.00
Motorboats	1	--	0.90	3.50	10.00
	2	2.75	1.10	3.75	12.00
	3	2.75	1.40	4.00	14.00
	4	3.40	1.50	4.25	15.00
Traditional boats	bv1	>>12.00	1.20	5.50	25.00
	bv2	>>12.00	1.40	6.50	30.00

Some Common Dimensions of boats, as illustrated in table (8-5) and fig (8-8)

Table (8-5) Common dimensions of vessels

Type of Boat	Length (m)	Width (m)	Draft (m)
Ships	80-85	10-12	3-3.5
Steam boats	40-60	7-10	2-2.75
Barges	20-40	6-8	2
Tugs	20-25	4-5	1.75
Sail boats	15-25	6	2

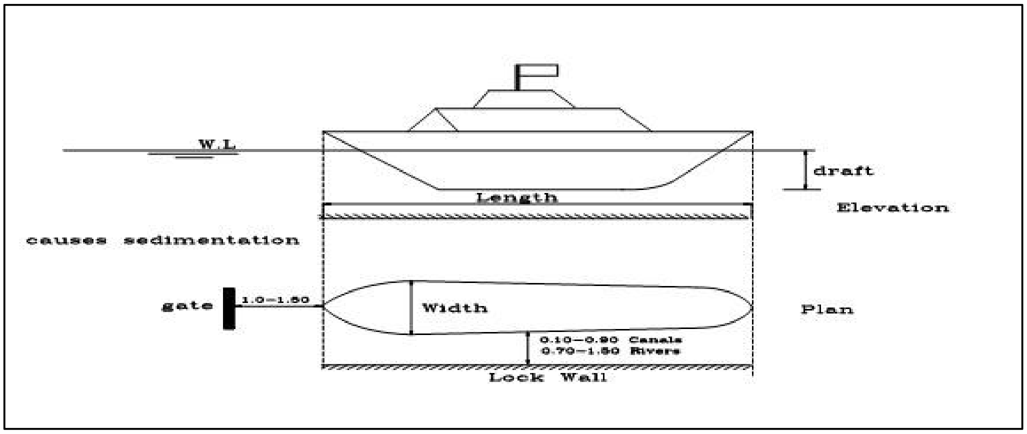


Fig (8-8) Basic dimensions of vessel

8-1-4 Guidelines for effective horizontal chamber dimensions

8-1-4-1 Marine navigation locks

There is no literature available for chamber dimensions for marine locks. However, the wet cross-section of the chamber (width x depth) at minimum locking levels should be at least 25 to 30% larger than the cross section of the normative vessel and the length between the gate recesses = 1.15 x length of normative vessel.

8-1-4-2 Inland navigation locks

The effective chamber dimensions of a minimum lock (for little navigation volume) for inland navigation are illustrated in the table (8-6)

Table (8-6) Dimensions of inland locks

waterway class	dimensions normative vessel (m)	effective chamber length 1) (m)	effective chamber width 2) (m)	sill depth 3) (m)
I	39 x 5.1 x 2.2	43	6.0	2.8
II	55 x 6.6 x 2.5	62	7.5	3.1
(IIa)	67 x 7.2 x 2.5	75	8.0	3.1
III	67 x 8.2 x 2.5	75	9.0	3.1
(IIIa)	80 x 8.2 x 2.5	90	9.0	3.1
IV	85 x 9.5 x 2.5	95	10.5	3.5
Va	110 x 11.4 x 3.5	125	12.5	4.2
Vb	186.5 x 11.4 x 4.0	210	12.5	4.7

The effective chamber width is about 1.10 width of the normative vessel (for class I and II it is about 1.15). The effective length of the chamber is about 1.12 length of normative vessel.

8-1-4-3 Recreational navigation locks: the dimensions are illustrated in table (8-7)

Table (8-7) Dimensions of recreational locks

Navigation volume (yachts/annum)	Determining chamber dimensions
about 10.000	• normative vessel length and width: exceeded by 30% of the vessels; • chamber width: double the width 30% -yacht + 1 m, but no more than 8 m; • chamber length: double the length 30%-yacht;
10.000 - 25.000	• chamber width 8 m and chamber length between double the length 30%-yacht and 60 m; • if more length is required, width also increases: 10 to 12 m
> 25.000	• simulations will yield a width of 10 -12 m at a length of 80 - 120 m

8-1-4-4 Chamber Dimensions of pound locks due to specific classification

Table (7-3) illustrate some lock chamber dimensions

Table (8-8) lock chamber dimensions

Type of Lock	Length (m)	Width (m)	Depth (m)
First class lock	125	16	3-4
Locks on River Nile	80	16	4.25
Locks on Nile branches (2nd class)	80-65	12	3
Locks on Main Canals (3rd class)	65-55	8-10	1.75-2.0
Small water ways	40-25	6-8	1.5-1.75

In addition, due to some reference, most locks that have built in United States since 1950 for commercial traffic have usable horizontal dimensions as listed below

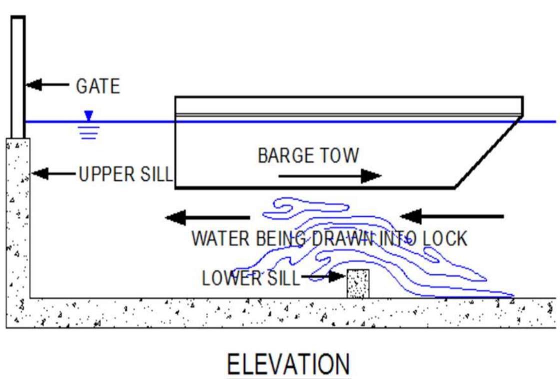
Table (8-9) Common locks horizontal dimension in USA since 1950

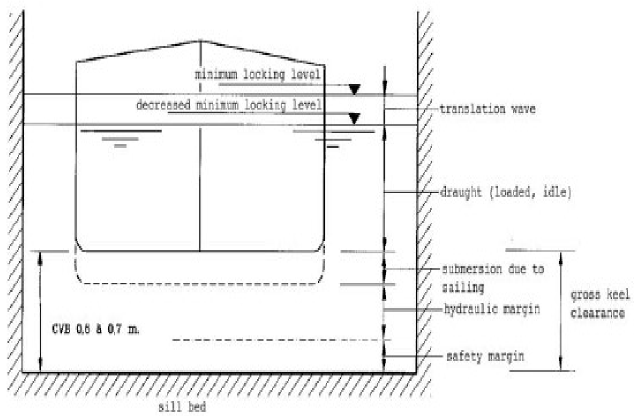
Lock Width ft	Usable Lock Length ft
84	400
84	600
84	720
84	1,200
110	600
110	800
110	1,200

8-1-5 Depth on Lock Sills

The minimum lock sill depth for shallow-draft large locks should be approximately two times the design draft of the tows that use the waterway, at least the depth of sill reaches to 1.7 times of tows draft. For some lock projects, this criterion may also govern the floor elevations, as operations and maintenance needs usually that the lock floor be at least 2 feet below the sills.

For deep-draft locks, vessels having drafts between (16-25) feet would only be used in locks with 25 to 30 feet of depth on the sills.





See fig (8-9)

Fig (8-9) sill of navigation lock

8-1-6 Depth in Lock Chamber

Depth in a lock chamber is governed by the depth on the sills and by requirements for the filling system cushion depth. For operation and maintenance purposes, the lock floor should be at least 2 feet lower than the lock sills. For instance, considering a design draft of 9 feet, the required sill depth would be 18 ft with the required 2 ft clearance (sill to lock floor) the lock chamber depth would be more than 20 feet.

Vessels having drafts between 16 and 25 feet would only be used in locks with 25 to 30 feet of depth on the sills. For vessels with deep drafts (25 feet and greater), lock sills should be placed low enough to provide ample allowance for the vessels to be permitted to enter or leave the lock under their own power. If vessels are to be moved into the lock by towing engines or winches, it may be possible to reduce the sill depth slightly.

For large vessels, over 100,000 dwt, a minimum clearance between the sills and the hull of the vessels of about 5 feet should be provided. Vessels of this size would not be permitted to enter or leave a lock under their own power.

Capstone is the word traditionally used to indicate the top of the lock coping. In common usage, this is also synonymous for height of the chamber wall.

The height of the chamber wall above the maximum locking level is mainly determined by the requirement that it is supposed to supply visual as well as physical guidance. For visual guidance, it is favorable if the chamber wall always is higher than the bow of the vessel so that the vessels should be able to sail by without risk.

8-1-6-1 For inland navigation locks

The height of capstone above maximum locking level is illustrated in the table (8-10).

Table (8-10) Height of lock coping

CEMT class	Height of lock coping above maximum locking level	
	chamber wall	lock head
I - IV	1.5 m	same as chamber
V	2.5 m	4.5 m

8-1-6-2 For recreational navigation locks

Capstone height is given with respect to the normative high summer level. Recreational navigation locks consists of both very small vessels and larger vessels of 10 to 15 meters or even more. For the smallest vessels, a height of 0.75 to 1.0 m is acceptable. Even for larger yachts, 1 m is suitable if the height is not less than 0.5 m above the maximum locking level.

8-1-7 Clearance at the navigation lock

In general, the clearance at locks is related to fixed bridges across the lock or to lift gates. The level of the underside of the fixed bridge or lifted gate is determined by the clearance. The preceded table (8-3) has shown recommended values of clearance due to ECMT, also it can compute from the following formula.

Clearance = mast height from normative water level + 0.3 m (Safety margin)

8-1-8 largest lock in the world

world's largest lock was until 2016, the Berendrecht Lock, giving access to the Port of Antwerp in Belgium, see fig (8-10). In 2016 the Kieldrecht Lock in the same port became the largest, see fig (8-11). The lock is 500 m (1,600 ft) long, and 68 m (223 ft) wide and drops 17.8 m (58 ft), and has four rolling lock gates.

Fig (8-10) the Berendrecht Lock

Figs (8-11) the Kieldrecht Lock

8-1-9 Type of gates used for pound locks

1. Mitre gate

Fig (8-12) Mitre gate

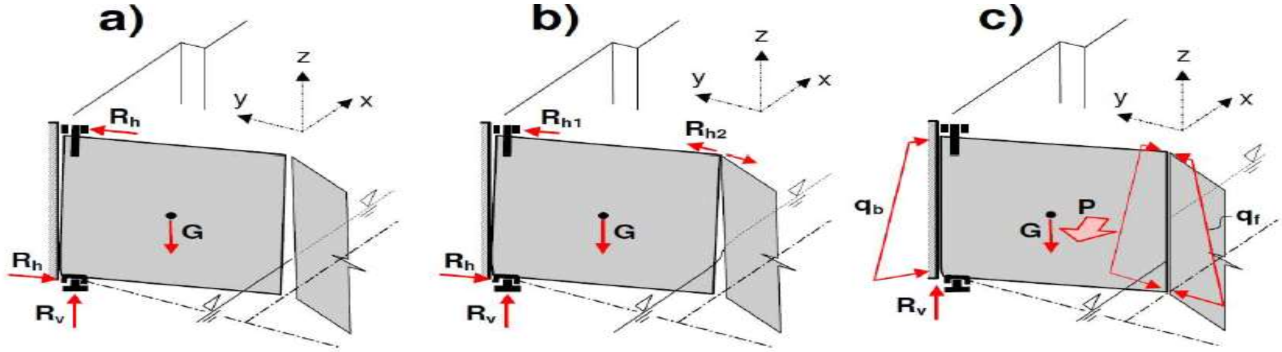


Fig (8-13) Mitre gate changing support condition during operation

2. Sector gate


Fig (8-14) Sector gate

3. Submersible tainter gate.

Fig (8-15) Tainter gate

4. Sliding caisson gate

Figs (8-16) Caisson gate

5. Rising sector gate

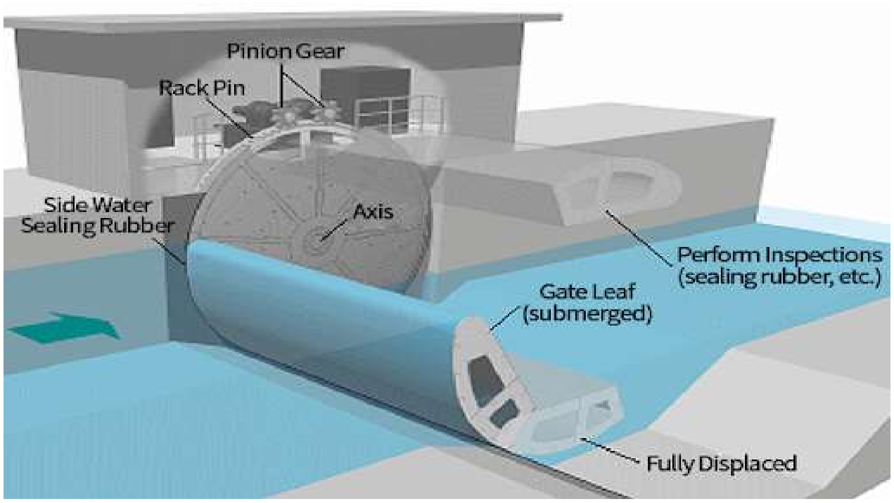


Fig (8-17) rising sector gate

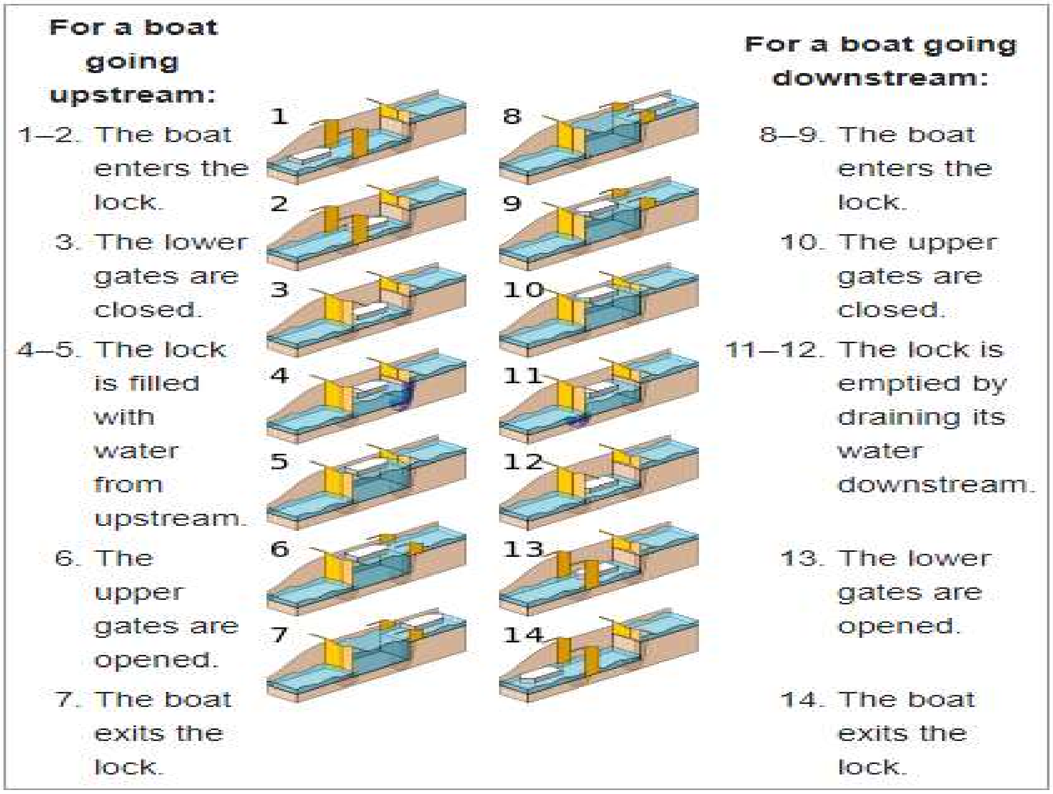
6. Rolling gates for large locks as which provided for Berendrecht Lock

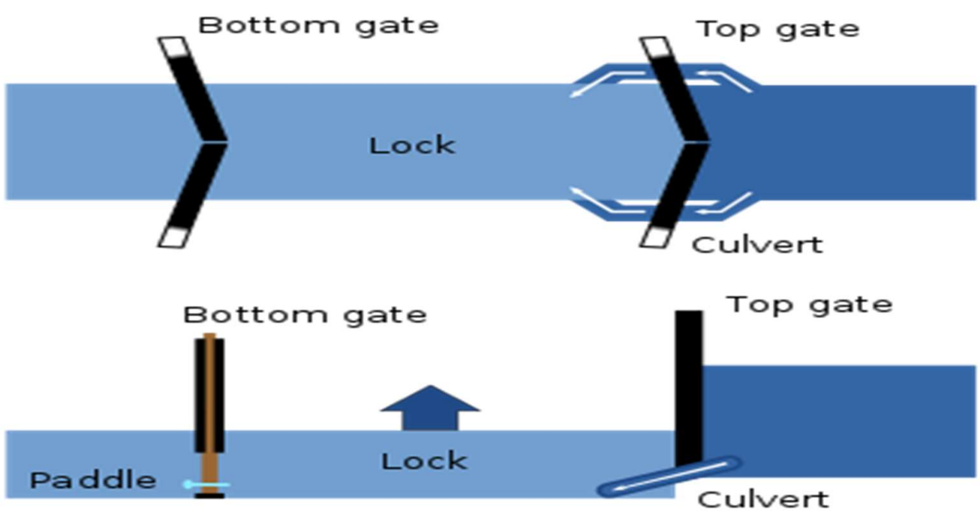
8-1-10 Basic operation to up-stream /down-stream for pound locks

The principle of operating in lock is simple. For instance, if a boat travelling downstream finds the lock already full of water:

1. The entrance gates are opened and the boat moves in.
2. The entrance gates are closed.
3. A valve is opened, this lowers the boat by draining water from the chamber.
4. The exit gates are opened and the boat moves out.

   If the lock were empty, the boat would have had to wait 5 to 10 minutes while the lock was filled. For a boat travelling upstream, the process is reversed; the boat enters the empty lock, and then the chamber is filled by opening a valve that allows water to enter the chamber from the upper level. The whole operation will usually take between 30-40 minutes, depending on the size of the lock and whether the water in the lock was originally set at the boat's level, See figs (8-18)





Figs (8-18) principles of lock operations

8-1-11 Time taken by vessel sailing through the Lock as shown in table (8-11)

Table (8-11) Different operation times in locks

filling (or emptying) Lock Chamber	5 -10 min
Opening of US gates to Enter the Lock	1 min
Boats to enter into chamber	8-10 min
Closing US gates	1 min
Emptying (or filling) Lock chamber	5-10 min
Opening DS gates	1 min
Boats to leave Lock chamber	6-8 min
Close DS gates	1 min
Average time taken for sailing a boat from the US to the DS through a lock = 30-4 min Time taken depends on: dimensions of lock chamber, head between US & DS wat levels, efficiency of opening & closing system of gates; efficiency of the mechanic system of controlling the valves of side culverts	

8-1-12 Time derivation for emptying system and culvert area calculation

Q dt = A dh **(1)**

cd*a*[(2gh)^0.5]* dt = A* dh **(2)**

(L.H.S) = volume of water through culverts

(R.H.S) = volume of water in chamber of lock

By integration, we obtain the following relation**:**

$$\boldsymbol{T = \frac{2As(\sqrt{H1} - \sqrt{H2})}{A * cd * \sqrt{2g}}} \qquad \textbf{(3)}$$

Where:

T: time for emptying (sec)

As: Area of water surface in lock chamber (m^2)

A: area of openings of culvert (m^2)

cd: coefficient of discharge

H1: Initial head (m)

H2: final head (m)

The equation (3) above cannot used directly to estimate the time required for filling/emptying the chamber.

Modifications to the equation above have been developed that provide a reasonably satisfactory means of determining operation time for locks with end filling systems and locks with wall culvert systems.

For locks with end systems.

$$T = \frac{2A_s}{C_L A\sqrt{2g}} \sqrt{H} + Ut_v \tag{4}$$

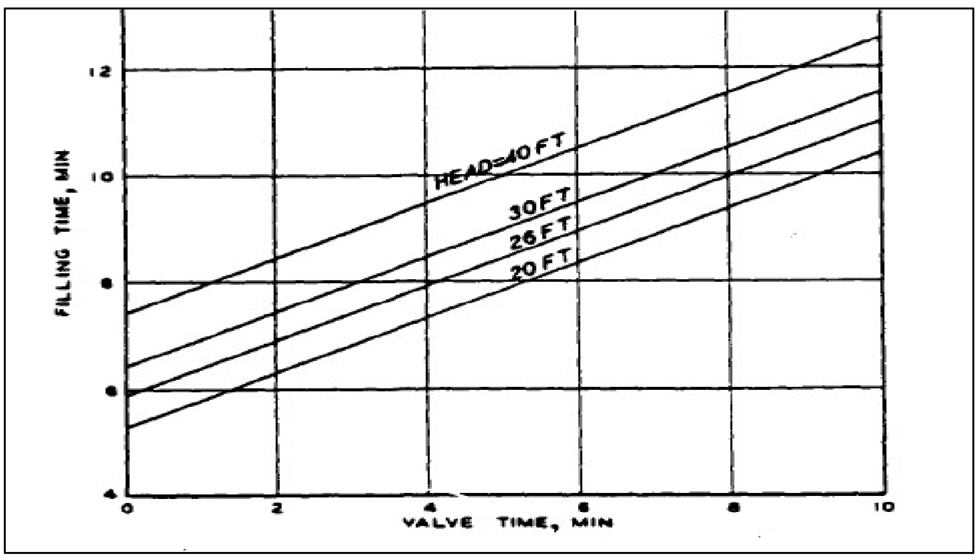
T: Time required for filling/emptying system.

As & A: Area of water surface in lock chamber & area of opening respectively.

H: Is represent H1 while H2 is equal to zero.

CL: Overall lock coefficient.

U: valve time coefficient provides an approximation of the effect on operation time by



adding a portion of the valve opening time t_v. This parameter is shown in fig (8-19).

Fig (8-19) valve time coefficient

For locks with wall culvert systems.

Pillsbury's equation is:

$$t = \frac{2A_s}{C2A_c\sqrt{2g}} \left(\sqrt{H_1 + d} - \sqrt{H_2 + d}\right) \tag{5}$$

t: time required for the lock to fill or empty from H1 to H2 after the valves are fully open.

2A_c: It is the area of the culverts at the valves sections as we considered 2 culverts.

As, H1, H2: Are define above

C: Discharge coefficient

d: Lock overfill or over empty, can be computed from fig(2-20) below

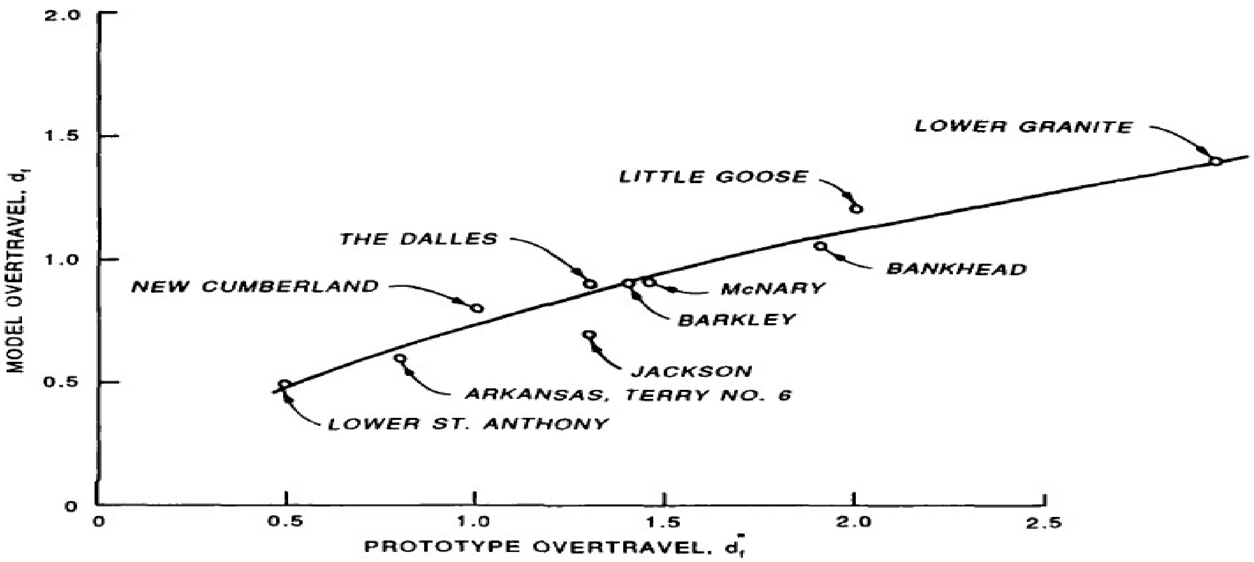


Fig (8-20) lock overfill depth versus "j"

By applying the effect of valve time to the preceding equation we get:

$$T = \frac{2A_s}{C_L 2A_c \sqrt{2g}}\left(\sqrt{H + d} - \sqrt{d}\right) + Ut_v \quad \textbf{(6)}$$

The equation **(6)** above is by taken that (H2=0), so it is completely filling or emptying of system. I.e. that the net required time for filling/emptying whole system is:

$$T - Ut_v = \frac{2A_s}{C_L 2A_c \sqrt{2g}}\left(\sqrt{H + d} - \sqrt{d}\right)$$

(7)

T: Operational time required to fill the lock

tv: Valve opening time

As: Surface area of lock chamber

2Ac: Area of culverts at valves (Assume 2 culverts with 2 valves)

CL: Overall coefficient, which include all head losses from intake until inside of the chamber.

H: Required head or lift

d: Overfill or over empty depth

U: Valve time coefficient

By rewriting the preceding equation **(7)** of net time, we could get the required area of culverts system.

$$2A_c = \frac{2A_s(\sqrt{H+d} - \sqrt{d})}{C_L\sqrt{2g}\ (T - Ut_v)} \quad \textbf{(8)}$$

Where "f" in the equation denote to the value related to the actual model

$$2A_c = \frac{2A_s(\sqrt{H+d_f} - \sqrt{d_f})}{C_L\sqrt{2g}\left(T_f - Ut_{v_f}\right)} \quad \textbf{(9)}$$

Equation above is not a very exact statement of the relationship between the parameters that are involved. The value of d_f, the overfill depth is utilized to allow for the effects of inertia head, but it does not provide an accurate measure of the inertia effects on head during the valve opening period when the mass of water in a culvert is being accelerated.

Locks designed in the United States in the last 40 years have had model valve time coefficients, U, ranging from about (0.45 to 0.70). And normally fall between 0.50 and 0.60. As illustrated in table (8-12) below

Table (8-12) Range of overall coefficient value

Type of System	C_{L_f}
Wall culvert side port	0.68 to 0.74
four-manifold bottom longitudinal	0.60 to 0.68
eight-manifold bottom longitudinal	0.52 to 0.65

As illustrate in the fig (8-20) above, where we applied value of "j" to find value of d_f.
The value of "j" can be computed from equation **(10):**

$$j = \frac{2A_c L_c}{A_s} \qquad \textbf{(10)}$$

Where **(L_c)** is the length of the culvert.

Now, Value of A_c in equation **(10)** can be calculated as trial from equation **(11)** below;

$$2A_c^* = \frac{2A_s\ H}{\sqrt{2g}\left(T - Ut_v\right)} \qquad \textbf{(11)}$$

At the start of a lock design study H and A_s are known. The operations time T_f, the valve time coefficient U, and the valve time t_v can be estimated. With a value for $2A_c$ from equation- **(11)** and the length of the culverts from the preliminary layout, a first trial value of d_f can be obtained from Figure (8-20) after calculating "j" from equation **(10).** By using the values of U, T_f, d_f and C_L, a tentative size for the main culverts can be calculated with Equation **(9)**. At this point, the value of $2A_c$ can be used to recalculate a new value of j, and a new value of d_f can be read from Figure (8-20). The new d_f value can then be used to go back and check the first calculated value of A_c obtained from equation **(9).** In all probability, the difference in A_c that results from use of the new d_f value will be insignificant.

Culvert shapes are usually square or rectangular and the rectangular shapes have height to width ratios varying from about 1.10 to 1.15.

8-1-13 Type of feeding system by using culverts

- Wall culvert- port systems
- Bottom lateral culvert system
- Bottom longitudinal culvert system

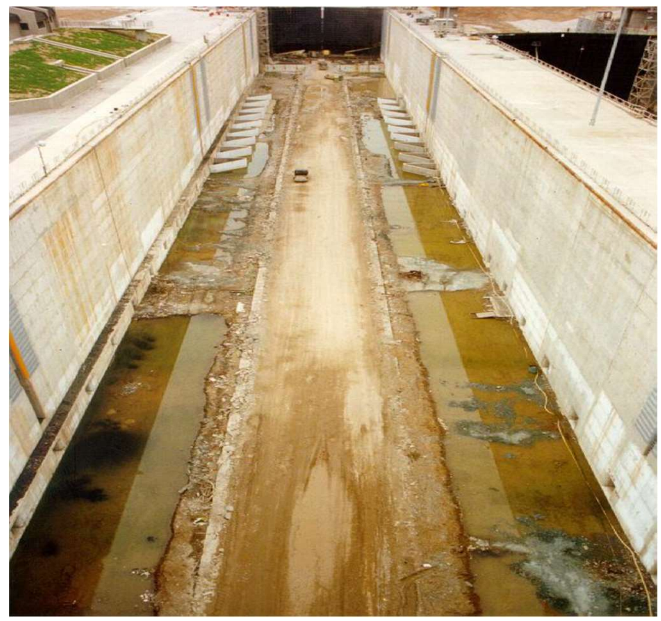


Fig (8-21) Wall culvert- port systems

Fig (8-22) longitudinal culvert system

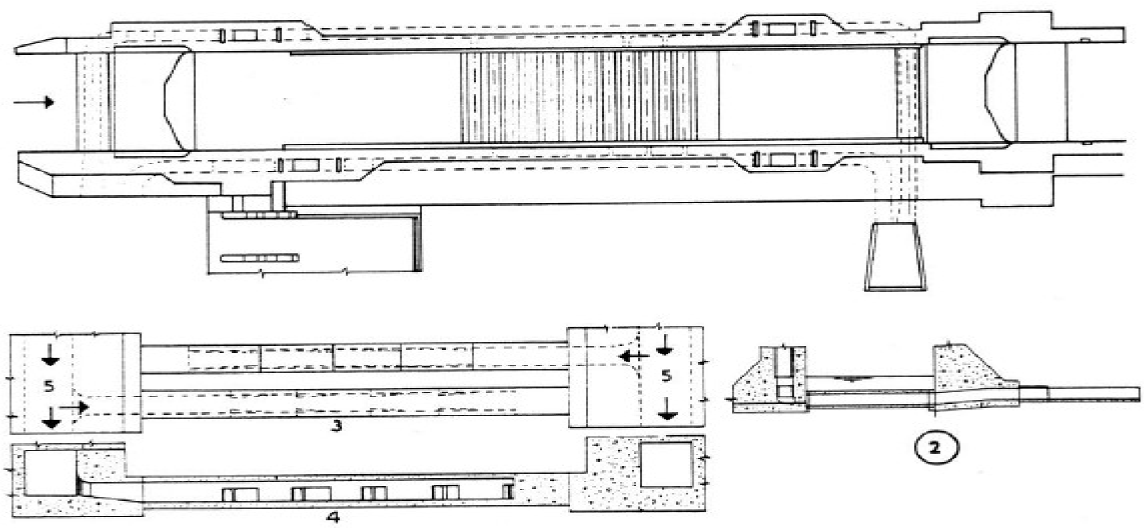


Fig (8-23) lateral culvert system

8-1-14 Design of feeding system by side ports

In a wall culvert port system, water is discharged into the lock chamber through short rectangular passages between the culvert and the lock chamber. The portion of wall culvert in which the water passages or ports are located is known as the manifold section of the culvert. The number, size, location, spacing configuration, and elevation of the port with respect to the lower pool level are all critical factors in design of a wall port system.

....
....

Instruction for design wall culver-port system

- The ports should occupy about (50% to 60%) of the length of the lock chamber and should be centered around the midpoint of the chamber.

- The total throat area of the ports in one culvert manifold should be from (90% to 98%) of the culvert area, preferably about 95%.

- Making this ratio $\frac{\sum Ap}{Ac}$ smaller than 0.95.

- Port sizes are influenced largely by lock size and culvert size. For instance, 1270 ft *110 ft lock, port sizes of 10.0 to 11.0 square feet give satisfactory results. 670 ft * 110 ft lock, a port size of 9.0 to 10.0 square feet will give good results. In 655 ft * 84 ft locks, ports of 6 to 7 square feet in cross section are satisfactory.

- Port spacing is influenced more by lock width than by any other feature. By staggering the port in one wall with respect to the ports in the opposite wall and providing proper port spacing, an arrangement can be developed that permits the expanding jet issuing from a port to cross to the opposite lock wall without directly colliding with an opposing jet. The objective is to dissipate most of the energy through boundary friction in the areas of contact between opposing jets. If the jets are permitted to collide, head-on, there will be upwelling, severe turbulence, and unstable conditions that will cause surging in the chamber.

The following port spacing, which illustrated in table (8-13) below for the widths of locks that are most widely used in the United States are:

Table (8-13) specific port spacing

Lock Chamber width	port spacing
120ft	32ft
110 ft	28 ft
84 ft	20 ft
75 ft	18 ft

These data apply only to locks designed specifically for **shallow-draft traffic**.

Since the port spacing, port size, number of ports, manifold length, and culvert size are all interrelated, a tentative procedure for establishing the number of ports would be as follows:

By consider 655 ft * 84 ft lock

Manifold length = 55% of 655 = 360.25 ft

Port spacing = 20 ft from the table above

Number of ports = 360.25/20 =18

Port size = 6 ft^2

Port to culvert ratio = assumed to be 0.95

Total port area = 6 *18 = 108 ft^2

Culvert area = 108/0.95 = 113.76 ft^2

Assume height to width ratios varying from about 1.10 to 1.15 and take it equal to 1.15

Use culvert 10 feet wide by 11.5 feet high = 115 ft^2

$\frac{\sum Ap}{Ac}$ = 108/115 = 0.94 which is accepted.

- Baffles for wall port systems consist of placing low concrete walls on the lock floor around the exit of each port or placing the culvert floor low enough so that the port can discharge into a shallow recess in the lock floor at the port exit, as shown in fig(8-24)

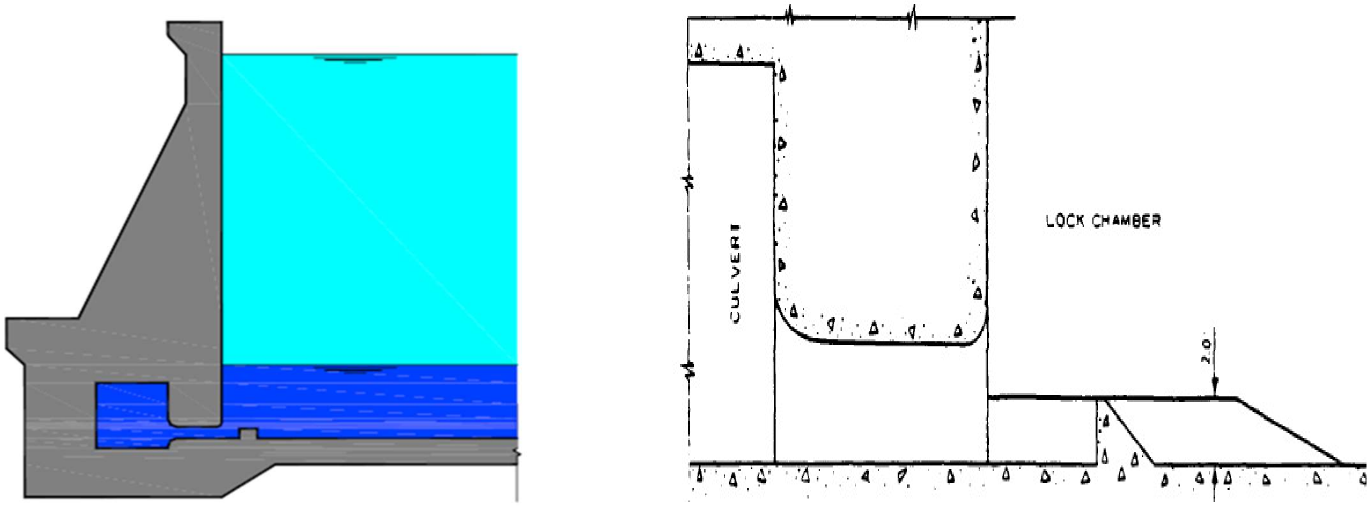
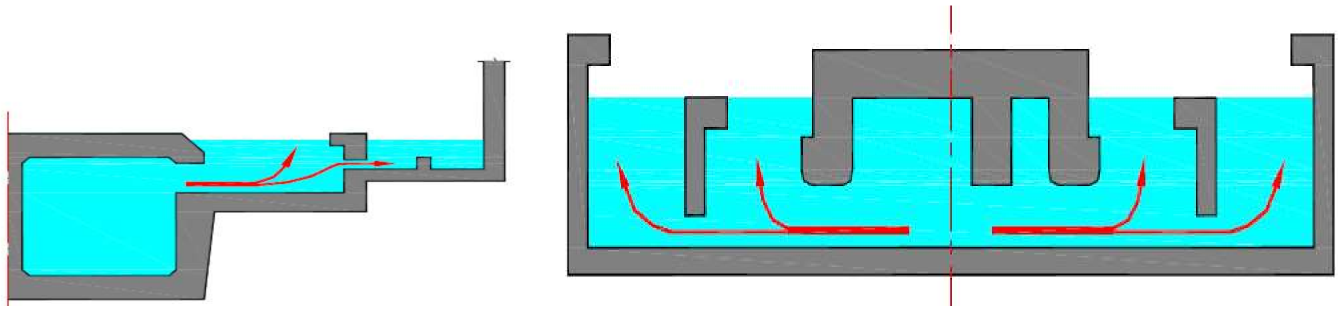


Fig (8-24) Using baffles for wall port system to dissipate the energy

8-1-15 Energy Dissipation System

To avoid dangerous currents at the outlets of filling and emptying system, measures should be taken for energy dissipation. Double ditches are effective system for energy dissipation. The baffles used for energy dissipation are illustrated in the figures below.



(a) (b)

Fig (8-25) Double ditches used for energy dissipation; (a) bottom lateral system (b) in ILCS

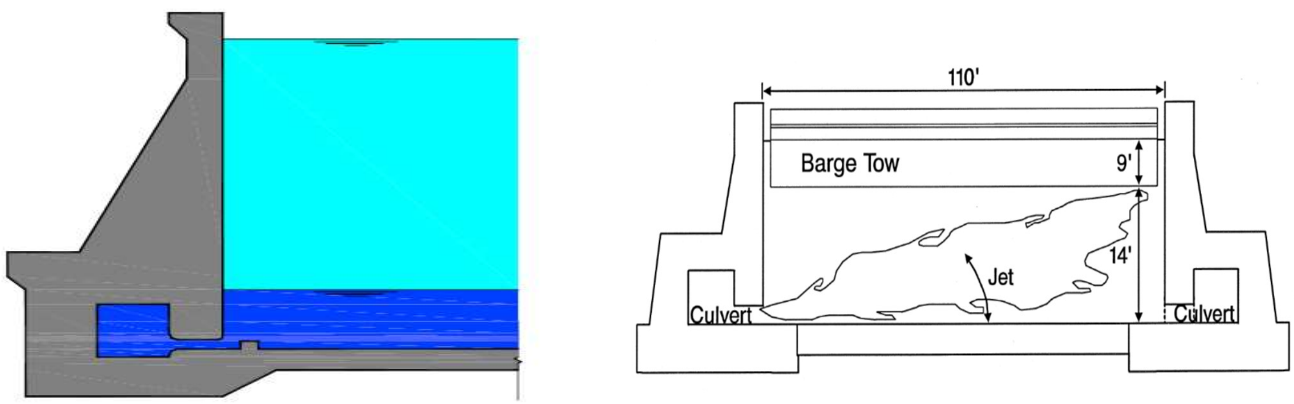


Figure (8-26) Side Port System with Port Deflectors

8-1-16 Head Losses

Head losses in lock filling systems are the hydraulic losses that occur because of water flowing through gated openings, into and through conduits, through valves, through conduit transition sections and ports, and into or out of a lock chamber.

For a steady flow condition, such losses can be represented as a discharge coefficient for the entire system by Equation $Q = c\sqrt{2gh}$. This discharge coefficient can be converted to actual head loss coefficient by Equation $k = \frac{1}{c^2}$

k_i: Is the loss coefficient for the culvert intake.

k_r: is the loss coefficient for resistance to flow in the portion of conduit being considered.

k_b: is the coefficient for any bends in the culvert.

k_s: is the coefficient for bulkhead slots.

k_{vw} is the coefficient for the valve well (valve fully open).

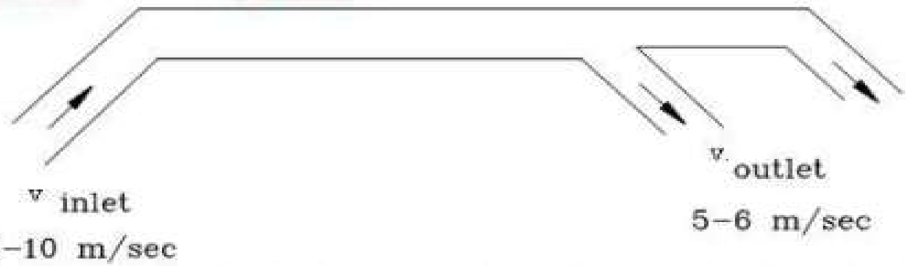
$$k_i + k_r + k_b + k_s + k_{vw} = k_{uv} \quad \text{... (12)} \qquad H_{L_{uv}} = k_{uv} \frac{V_c^2}{2g} \quad \text{... (13)}$$

Where $H_{L_{uv}}$ is the total head loss from the upper pool Z_u to the valve skin plate and V_c is the mean velocity in the culvert at the section where the valve is located.

8-1-17 Relation between Time of filling, Area of tunnel system and Velocities

Area of tunnel outlet > area of tunnel inlet

v at outlet < v at inlet



Sketch showing inflow and outflow through side culvert

$$\frac{area\_outlet}{area\_inlet} \approx 1.3 - 1.5$$

v_{inlet} (just after opening the culvert sluice gate)

$$= C_d \sqrt{2gH} \quad \text{(flow through orifice)}$$

Cd = 0.65 for short culverts

= 0.60 for long culverts

V inlet = 0 at Head = 0

At Head = H, velocity = V_{max}

V max x Area of chamber = v max x a culverts

V min = 0

$$V_{mean} = \frac{V_{max} + V_{min}}{2} = \frac{V_{max}}{2} \leq 4\,cm/\sec$$

where : V = velocity of vertical movement inside lock chamber

v = velocity of water flow through side culverts

N.B.

► Assuming the velocity of vertical upward or downward movement of water inside lock chamber during filling or emptying of chamber to be V

► This value is the same of the velocity of movement of boats inside the lock chamber

Fig (8-27) relation between Time of filling, Areas and Velocities

8-1-18 Different Structural Types of locks as shown in fig (8-28)

1. Concrete Lock
2. Sheet-Pile Cellular Locks
3. Sheet-Pile Lock with Tieback Anchorage

4. Earth Wall Locks with Concrete Gate Bays

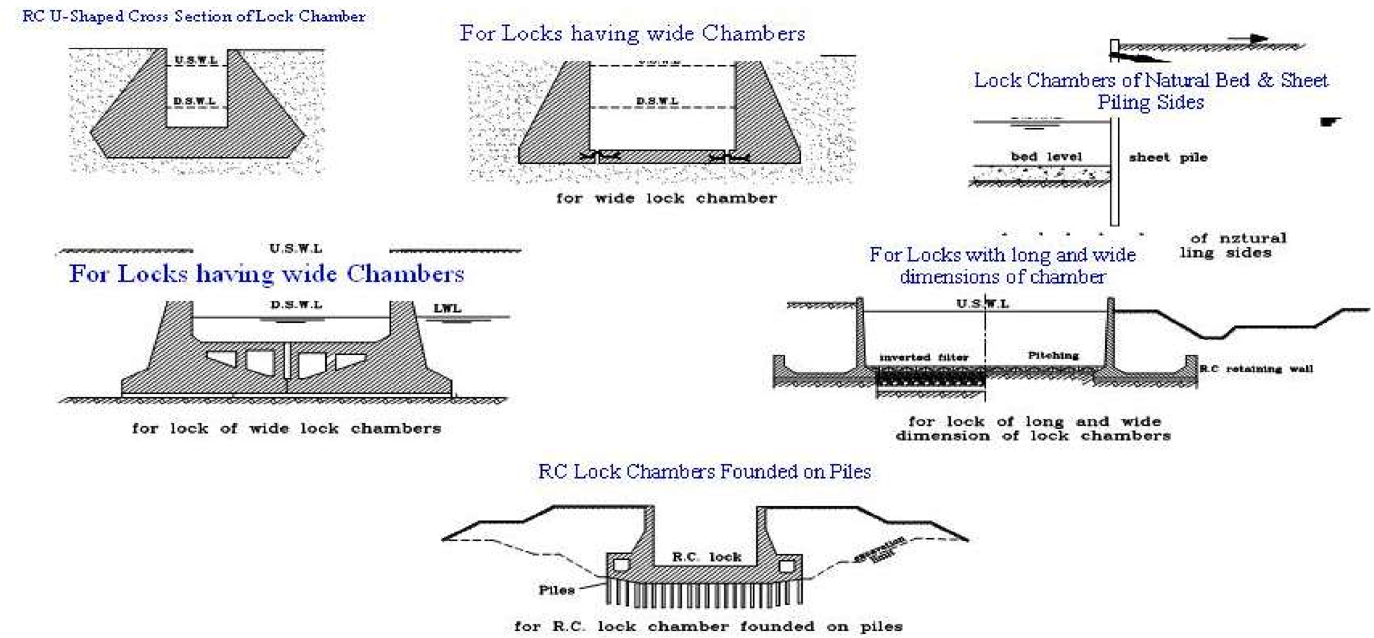


Fig (8-28) different structural types of locks

8-1-19 Loads and Pressure Distribution:

- **8-1-19-1 Symmetrical lock Chamber as illustrated in fig (8-29) below**

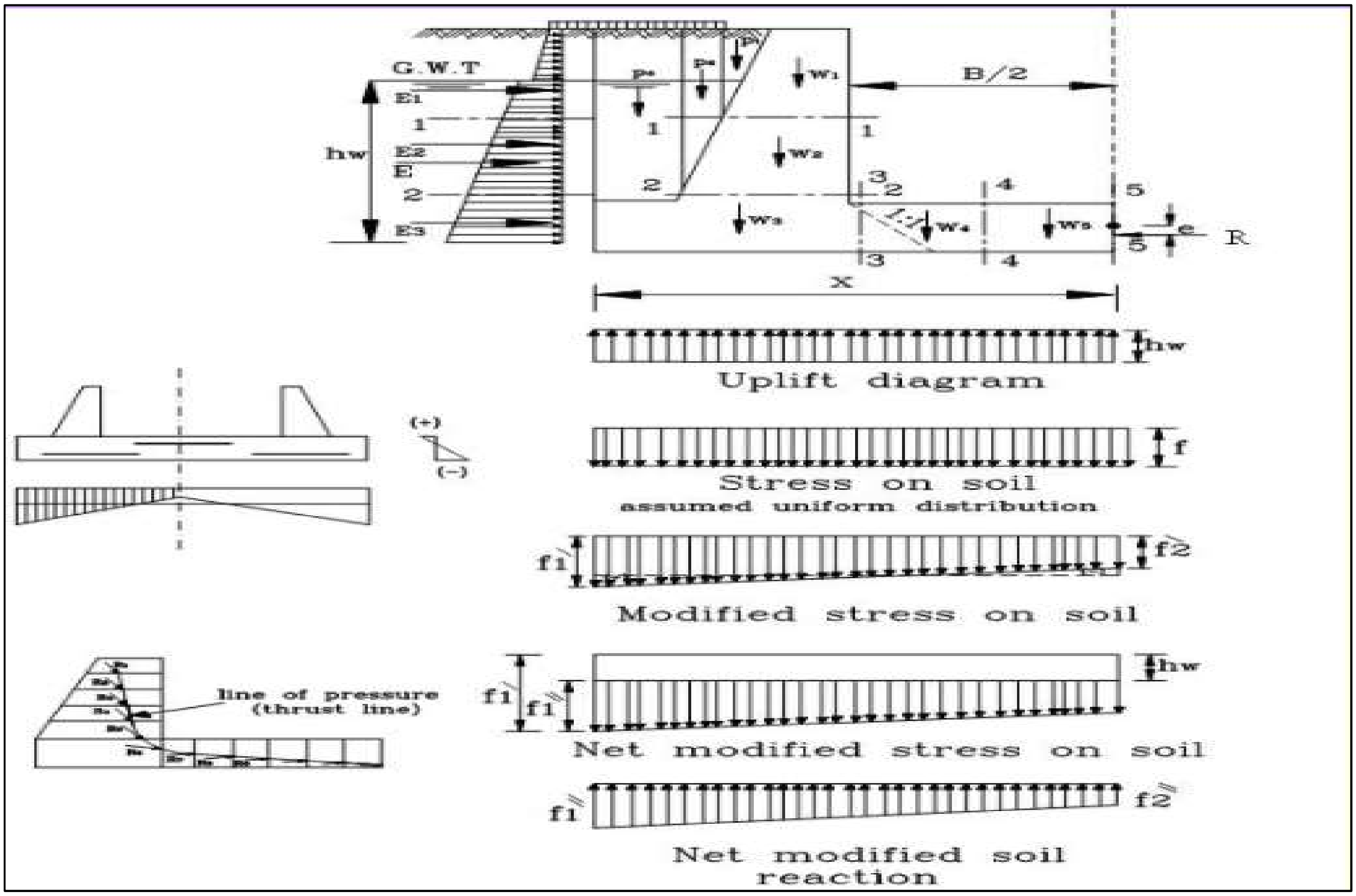


Fig (8-29) load and pressure distribution for symmetric locks

- **8-1-19-2 Unsymmetrical lock Chamber as illustrated in fig (8-30) below**

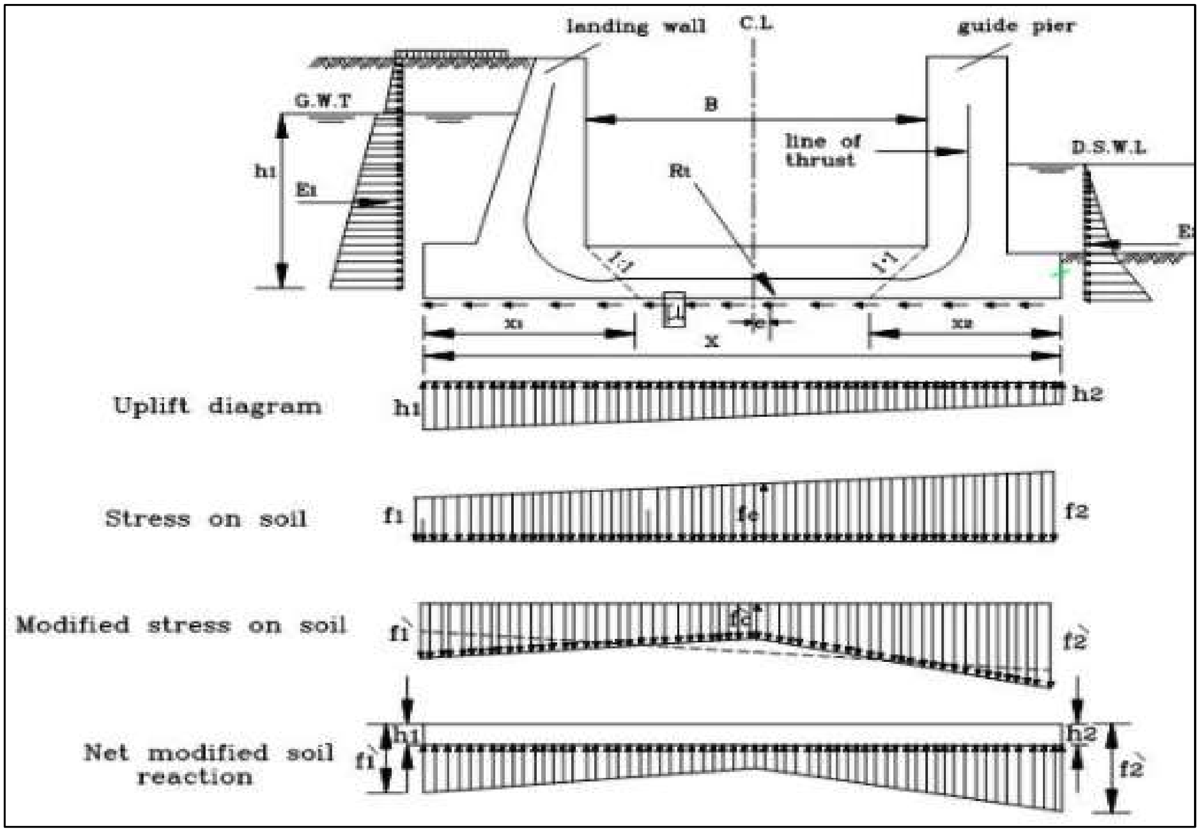


Fig (8-30) load and pressure distribution for symmetric locks

- **8-1-19-3 Stability of retaining walls**

Design loads

The retaining wall has been designed for the following loads:

a) Dead load (self-weight of structure).

b) Static earth pressure.

c) Dynamic increment in earth pressure due to earthquake.

Stability check

Stability of the retaining wall is checked for the following condition

• Lock is empty

• Maximum water level, upstream at H.F.L.

• Backfill soil is saturated

a) Overturning:

Safety against overturning is checked about the point of rotation at the bottom end of toe of retaining wall in the horizontal direction.

$$\text{Factor of Safety }(FOS) = \frac{\text{Restoring Moments}}{\text{Overturning Moments}}$$

... (14)

b) Sliding

$$\text{Factor of Safety (FOS)} = \frac{\text{Resisting forces}}{\text{Sliding forces}}$$ **... (15)**

c) Foundation base pressure

$$Base\,Pressure = \frac{W}{b.L}[1 \pm \frac{6.e}{L}]$$ **... (16)**

b & L: Foundation base width and length (m)

e: Eccentricity of load

W: Algebraic sum of the vertical forces

The factor of safety must be more than those given in table (8-14) below.

Table (8-14) Requirement of factor of safety

Minimum factor of safety	Normal	Seismic
Sliding	1.5	1.2
Overturning	2	1.5
Base pressure	<500 KN/m²	<750 KN/m²

- **8-1-19-4 Stability of base slab by using piles**

The base slab is destabilized due to uplift pressure. Provision of sufficient floor thickness and tension piles prevents the failure against uplift pressure; the table (8-15) below illustrate the (F.O.S) for piles system

Table (8-15) Safety factors geotechnical working load capacities of the piles.

Load	FOS
Skin friction on tension piles	(SF)=3.0
Lateral load	(SF)=2.0

Fish ladder

8-2 Fish ladder

A fish ladder, also known as a fish-way, fish pass or fish steps, is a structure on or around artificial and natural barriers (such as dams, locks and waterfalls) to facilitate fishes (lampreys, trout and salmon) natural migration go upstream to spawn. Most fish-ways enable fish to pass around the barriers by swimming and leaping up a series of relatively low steps into the waters on the other side. The velocity of water falling over the steps has to be great enough to attract the fish to the ladder, but it cannot be so great that it washes fish back downstream or exhausts them to the point of inability to continue their journey upriver, See fig (8-31) below.

Fig (8-31) Fish Ladder on the Elbe River, Germany

8-2-1 Type of fish way

Different types of Fish ladders are as follows:

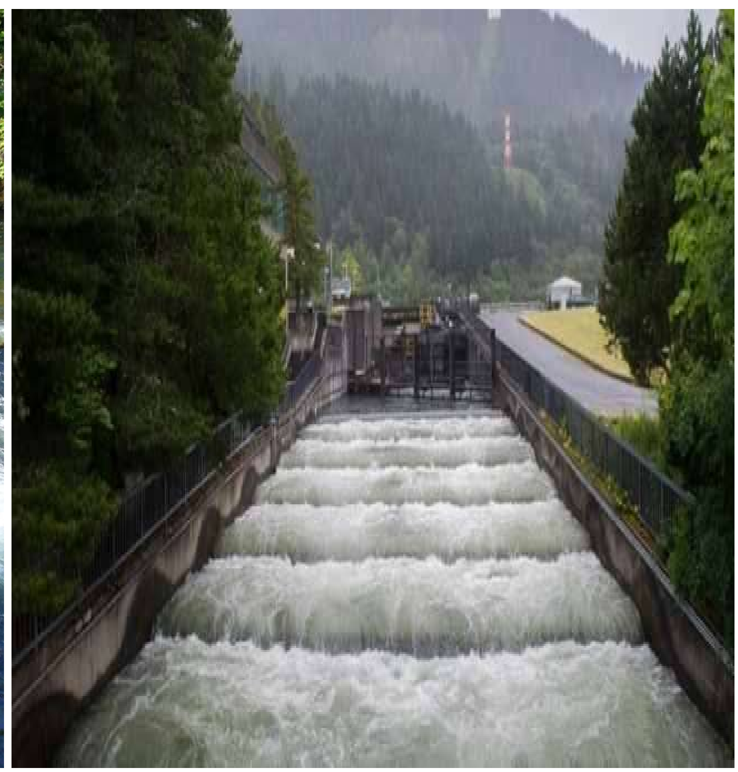
1. Pool and Weir fish ladder
2. Pool and Orifice fish ladder

3. Vertical slot fish ladder
4. Baffle fish-way
5. Rock ramp
6. Fish elevator
7. Siphon fish-way
8. By-pass fish-way

....

1. pool-weir fish-way

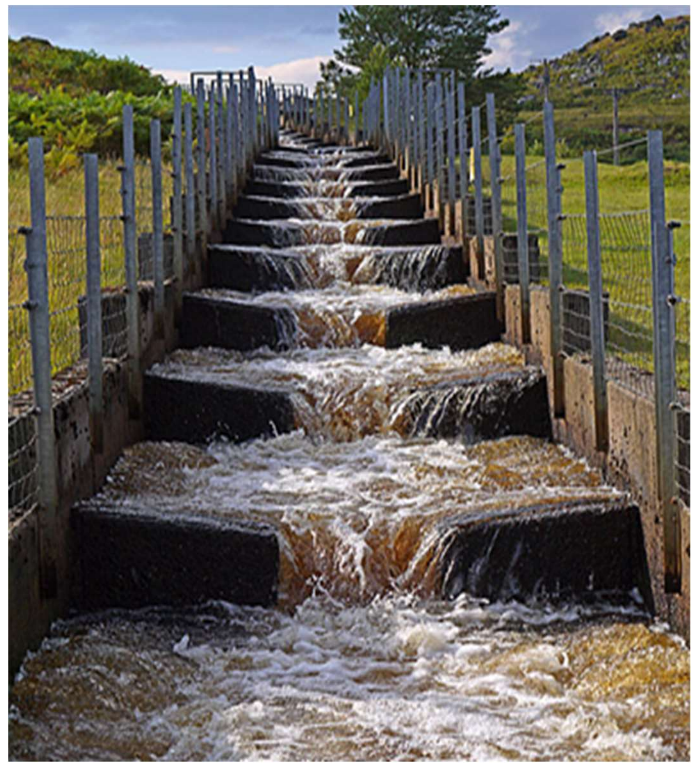
One of the oldest styles of fish ladders. It uses a series of small dams and pools of regular length to create a long, sloping channel for fish to travel around the obstruction. The channel acts as a fixed lock to gradually step down the water level, to head upstream. Fish need to jump from one pool to another to migrate to the upstream. Pool and weir fish ladders are suitable for all types of structures either it is small scale or large scale, as illustrated in fig (8-32)

Figs (8-32) Pool and Weir type Fish Ladder

2. Pool and Orifice Fish Ladder

Pool and orifice fish ladder is almost similar to pool and weir fish ladder and the only difference, in this case, is the overflowing weir is provided with a submerged orifice within its body. Hence, in this case, the fish can travel to upstream by just passing through each



orifice rather than jumping over the weir crest as illustrated in fig (8-33) below.

Figs (8-33) Pool and Orifice Fish Ladder, Tilpa Weir, Australia

3. Vertical slot fish-way

Vertical slot fish-way is another variation of pool and weir fish-way. In this case, the weirs are replaced by walls with vertical slots so that the fish can pass through these slots from pool to pool and to the upstream easily. Vertical slot fish ladders allow the fish to swim at their preferred depth. This type of fish ladder is recommended where there is a huge amount of fish migration in the river. See fig (8-34) below.

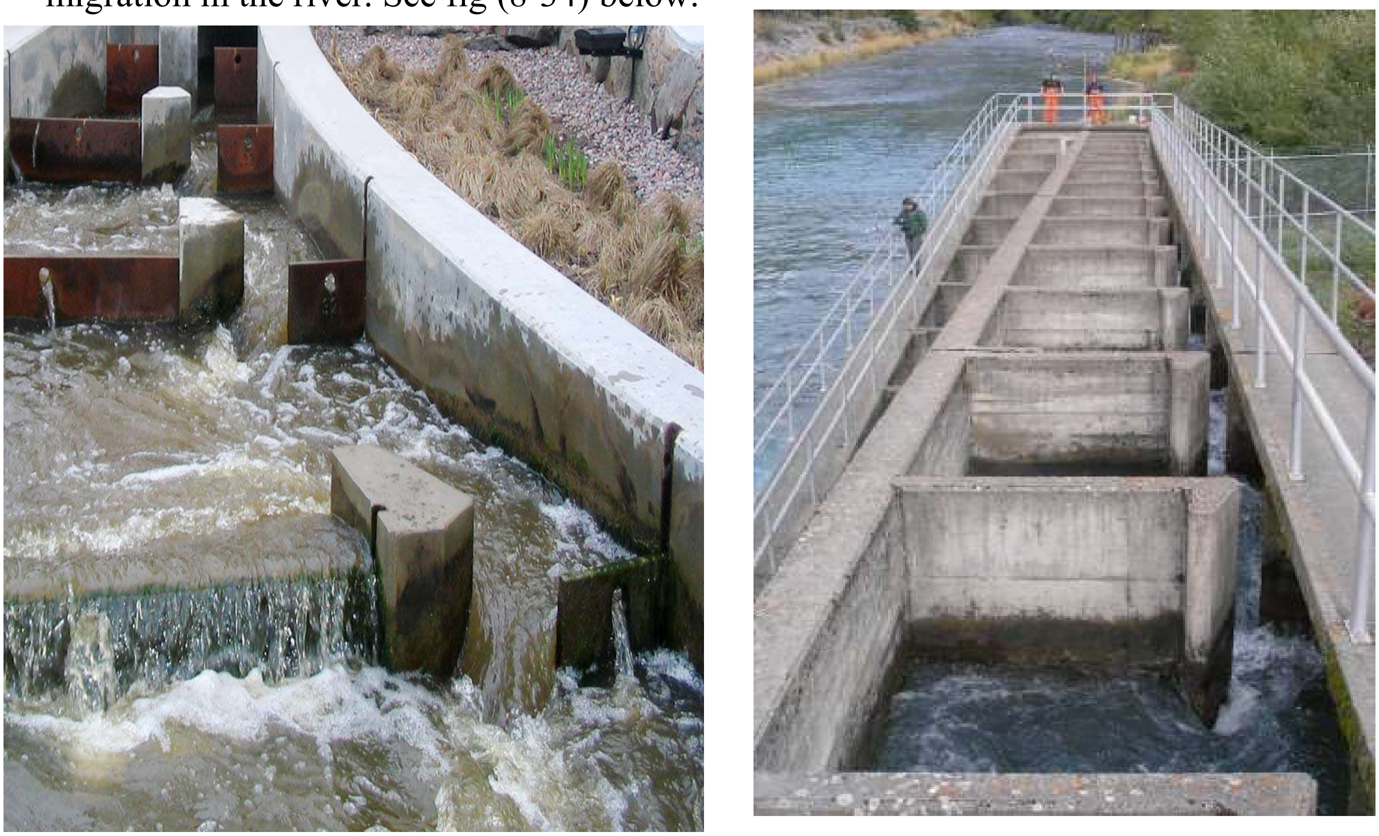


Fig (8-34) Vertical Slot Fish Ladder

4. Baffle fish-way

Is another type of fish-way, which is in the form of a rectangular channel with a series of equally spaced baffles perpendicular to the direction of flow. Generally, in the case of baffle fish-ways as shown in figs (8-35), (8-36) and (8-37) below, water flows continuously without resting unlike in pool type fish ladders. However, pools can also be provided in between baffle walls if needed.

Baffle fish-ways are of different types, namely:

- Denil Fish-way (classic type)
- Larinier Fish-way
- Alaskan Fish pass (used for steeper slope)
- Chevron Fish-way

Fig (8-35) Baffle Fish-way

Fig (8-36) Larinier Fish-way

Fig (8-37) Denil Fish-way

Denil Fish-way is classic baffle fish-way in which baffles are provided on the sides and floor of rectangular pass. While in the case of Larinier fish-way and chevron fish-way baffles are provided only on the floor of the fish pass. Alaskan fish passes are recommended for steeper slopes.

5. Rock ramp Fish-way

Rock ramp fish-ways are built using large size rocks and timber logs. In this type of fish-way, a rock ramp is prepared directly over the obstruction across its width with some slope. Pools and falls are created by these large rocks in such a way that the fishes can easily pass over them. Rock ramp fish-ways are well suitable for low height obstructions, where upstream water level control is not essential and some barriers suitable for rock ramp fish-way are low height weirs, short waterfalls, channel stabilization structures, as shown in figs (8-38) below.

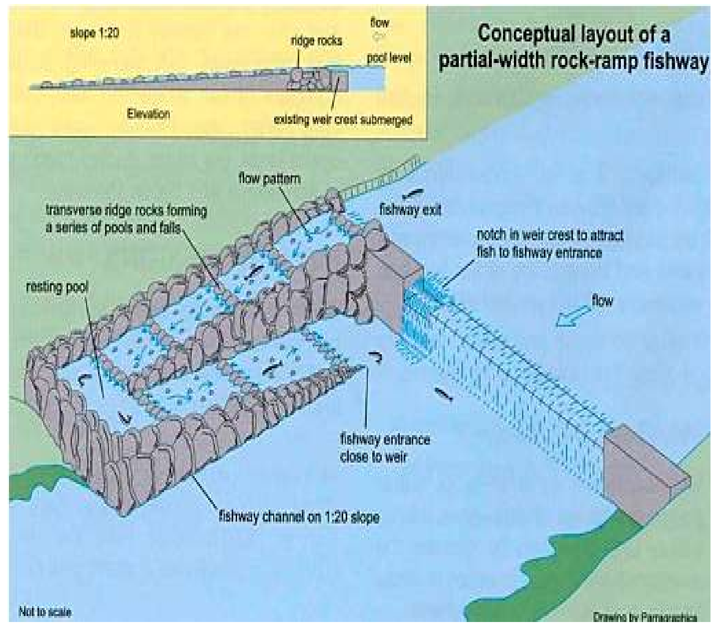


Fig (8-38) Rock Ramp Fish-way

6. Fish elevator

Fish elevators also called as fish lifts, they another type of fish-ways in which fish are lifted by water-filled chamber from downstream to the upstream. Fish lifts are well suitable for tall obstructions such as arch dams, high weirs. The fish elevator facilitates a huge amount of fish migration at a given time. Some large-sized species may not travel to upstream in ladder-type passes due to the small size of openings, poor swimming capabilities. For such type of species, Fish elevators are best suitable as shown in fig (8-39) and fig (8-40) below.

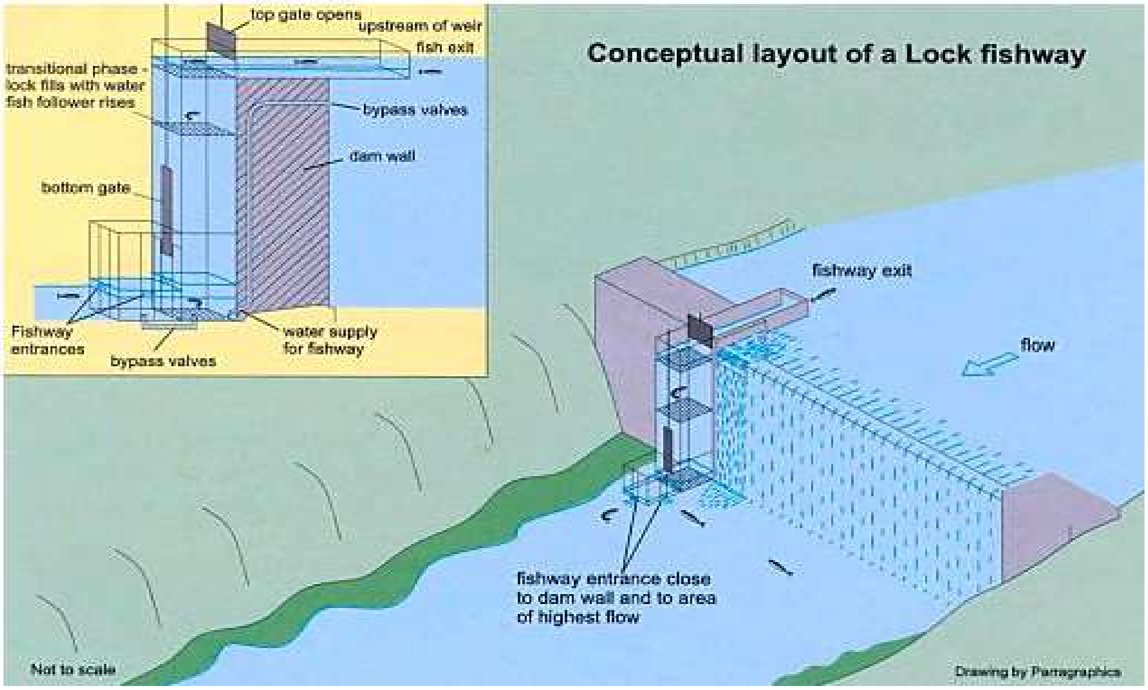


Fig (8-39) Fish Lift Structure

Fig (8-40) Fish Lifting Procedure, Holyoke Dam, USA

7. **Siphon Fish-way**

Fish siphon or siphon fish-way is a closed fish pass, which is provided between two watercourses. The fish enters into the siphon tube, which is partially filled with water, and the flow rate in this tube is controlled by siphon effect. Fish siphon allows all sizes of species through it and it is laid with small gradients. It is best suitable for fish to migrate during flood periods. See fig (8-41) below.

Fig (8-41) siphon fish-way

8. Bypass Fish-way

Bypass fish-ways are low-gradient earthen or rocky channels that mimic the structure of natural streams and are often described as ‘nature-like’ fish-ways. and it's may provide a cheaper alternative to the more technical fish-way designs as shown in fig (8-42) below.

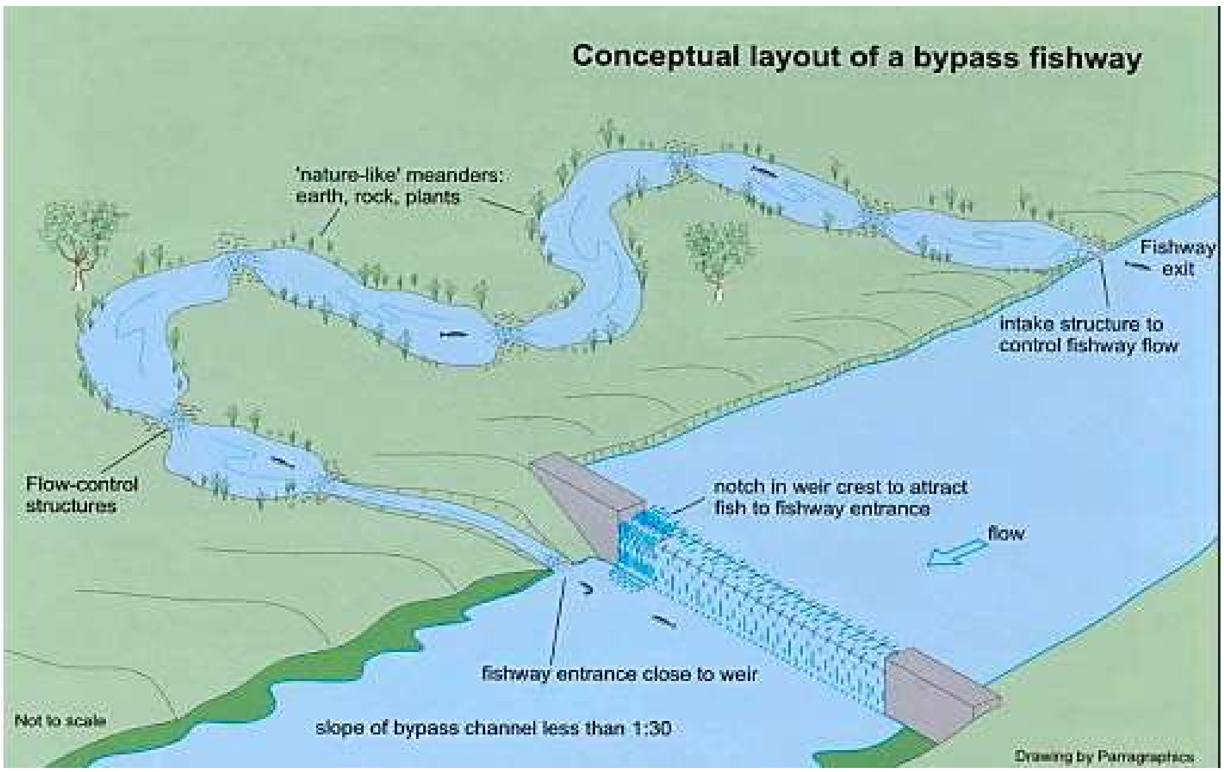


Fig (8-42) Bypass fish-way

8-2-2 Design of pool passage of fishes

8-2-2-1 Pool pass

The principle of a pool pass consists in dividing a channel leading from the headwater to the tail water by installing cross-walls to form a succession of stepped pools. The discharge is usually passed through openings (orifices) in the cross-walls and the potential energy of the water is dissipated systematic in the pools.

8-2-2-1-1 Plan view of pool pass

The design of pool passes is usually straight from headwater to tail water and sometimes take curved path, as illustrated in fig (8-43) and (8-44) below.

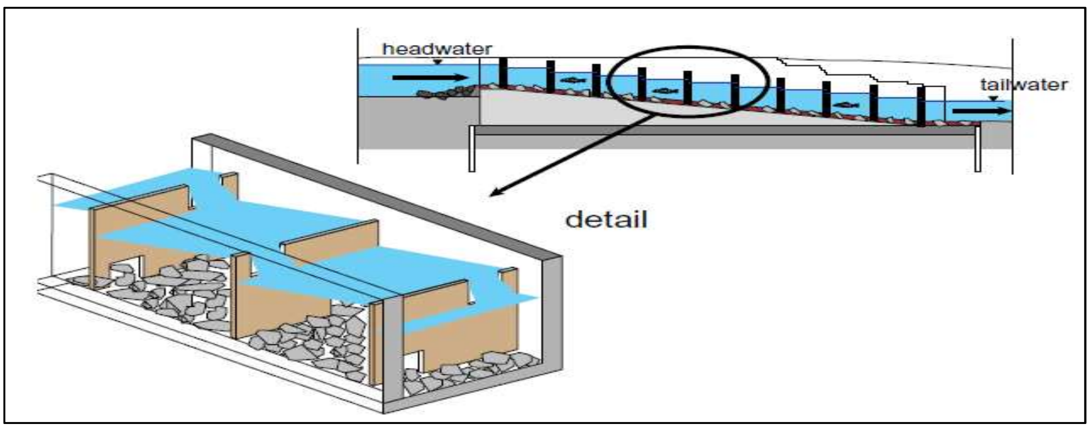


Fig (8-43) Simple pool pass profile

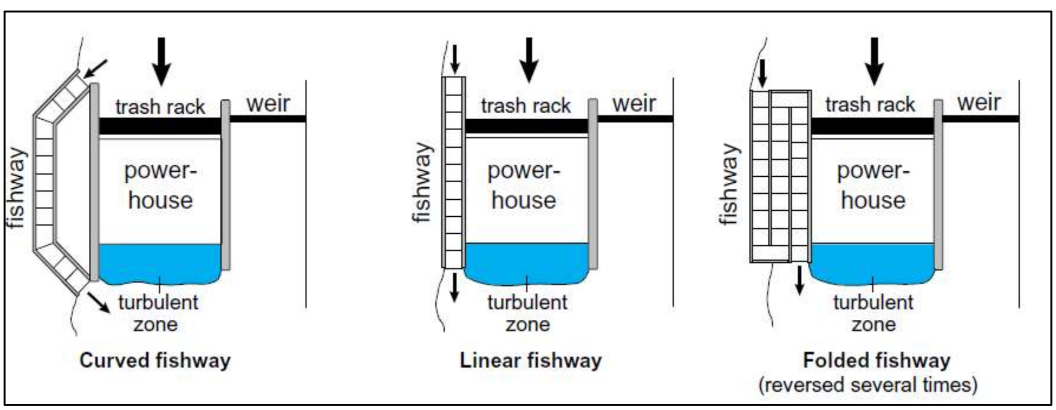


Fig (8-44) Type of pool paths

8-2-2-1-2 longitudinal section of pool pass

Differences in water level between individual pools govern the maximum flow velocities. They are therefore a limiting factor for the ease with which fish can negotiate the pass. In the worst case, the difference in water level (Δh) must not exceed 0.2 m; however, differences in level of (Δh) = 0.15 m at the normal filling level of the reservoir are more suitable, See fig (8-45) & fig (8-46).

The ideal slope for a pool pass is calculated from the difference in water level and length of the pools (l_b):

$$I = (\Delta h) / l_b \qquad \textbf{... (17)}$$

The values of ($I = 1{:}7$) to ($I = 1{:}15$) are obtained for the slopes if the value l_b ranges from 1.0 m to 2.25 m. Steeper slopes can make considerable turbulence in the pools and should be avoided if possible.

The number of pools needed (n) is obtained from the total head to be overcome (h_{tot}) and the permissible difference in water level between two pools (Δh) as in equation **(18)** below and as shown in fig (8-45) later.

$$n = \frac{h_{tot}}{\Delta h} - 1 \qquad \textbf{... (18)}$$

h_{tot} is obtained from the difference between the maximum filling level of the reservoir (maximum height) and the lowest tail water level.

8-2-2-1-3 Pool dimensions

Pool pass channels are generally built from concrete or natural stone. The partition elements (partition cross-walls) can consist of wood or prefabricated concrete.

The pool dimensions must be selected in such a way that the ascending fish have adequate space to move, the energy contained in the water is dissipated with low turbulence and the flow velocity must not be reduced to the extent that the pools silt up. A volumetric dissipated power of (150 W/m^3 - 200 W/m^3) is permissible in the salmonid zone.

The bottom of the pools must always have a rough surface in order to reduce the flow velocity in the vicinity of the bottom and make it easier for the benthic fauna and small fish to ascend. Embedding stones in concrete base can produce a rough surface. Table (8-16) recommend specific dimension for pool passes and figs (8-45) & (8-46) below are illustrate that in term of the terminology of pool profile and longitudinal section.

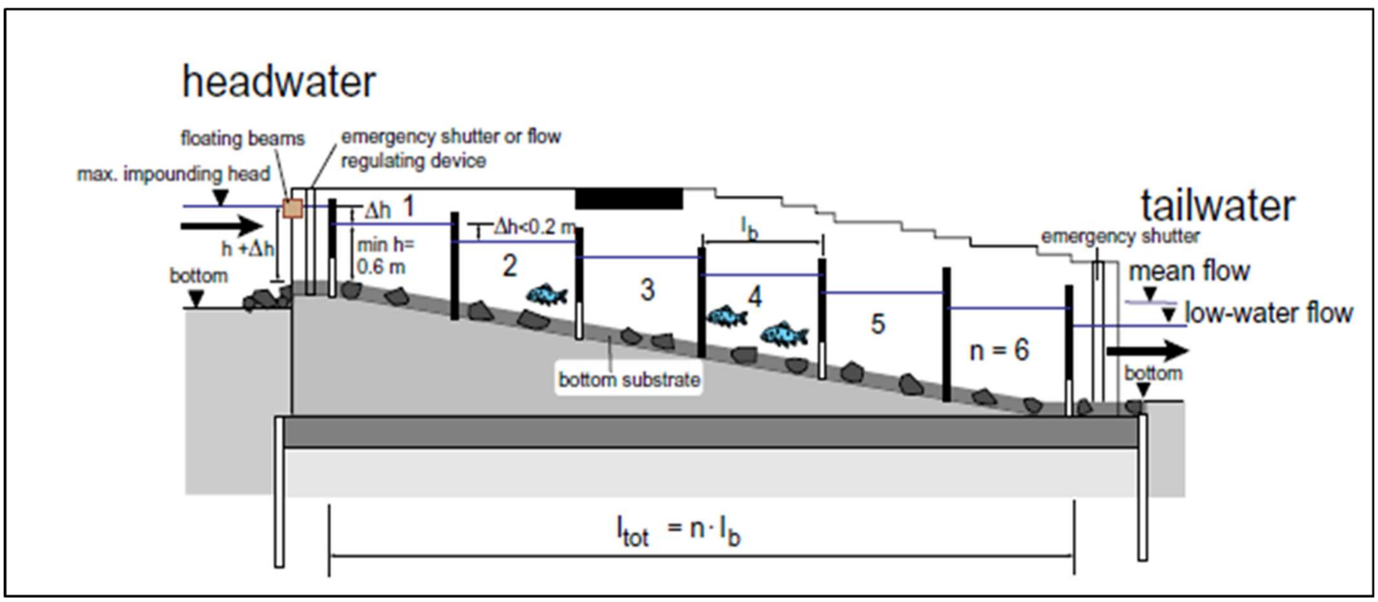


Fig (8-45) longitudinal section through a pool pass

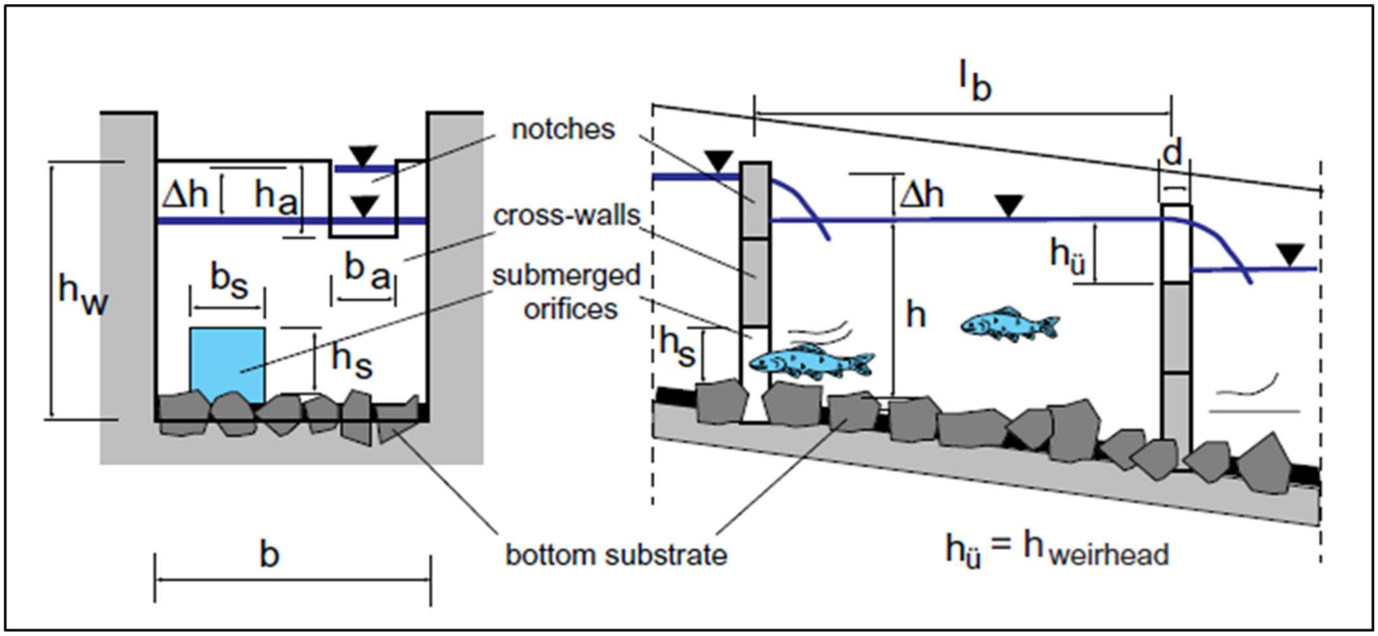


Fig (8-46) Pool-pass terminology

8-2-2-1-4 Cross-wall structures

Conventional pool passes are characterized by vertical cross-walls that stand at right angles to the pool axis (Sometimes the cross-walls are arranged obliquely to the pool axis 45 to 60° and obliquely to the pass bottom by 60°, then the whole pool pass is called Rhomboid pass).

The cross- wall may be solid (concrete or masonry) or sometimes from wood.

The cross-walls have submerged openings (orifices) that are arranged in alternating formation at the bottom of the cross-wall (dimensions as in Table 8-16) through which fish can ascend by swimming into the next pool. The openings reach to the bottom of the pool and allow creating a continuous rough-surfaced bottom where the substrate is put in. Usually the cross wall is provided with surface openings (notches), that are also allow to swim up to the next pool, but their lower edge should still be submerged by the water level of the downstream pool in order to avoid plunging flows and thus allow fish to swim over the obstacle.

In general, submergence of cross-walls should be avoided wherever possible so that water flows only through the orifices (or surface notches).

Table (8-16) Recommended dimensions for pool passes

Fish species to be considered	Pool dimensions[1)] in m			Dimensions of submerged orifices in m		Dimensions of the notches[3)] in m		Discharge[4)] through the fish pass m^3/s	Max. difference in water level[6)] Δh in m
	length l_b	width b	water depth h	width b_S	height h_S[2)]	width b_a	height h_a		
Sturgeon[5)]	5 – 6	2.5 – 3	1.5 – 2	1.5	1	-	-	2.5	0.20
Salmon, Sea trout, Huchen	2.5 – 3	1.6 – 2	0.8 – 1.0	0.4 – 0.5	0.3 – 0.4	0.3	0.3	0.2 – 0.5	0.20
Grayling, Chub, Bream, others	1.4 – 2	1.0 – 1.5	0.6 – 0.8	0.25 – 0.35	0.25 – 0.35	0.25	0.25	0.08 – 0.2	0.20
upper trout zone	> 1.0	> 0.8	> 0.6	0.2	0.2	0.2	0.2	0.05 – 0.1	0.20

8-2-2-1-5 Hydraulic design

The following parameters are crucial and must be respected in pool pass designing.

- Flow velocities in the orifices, which must not exceed v_{max} = 2 m/sec.
- Discharge in the fish pass
- Volumetric power dissipation should not exceed E = 150 W/m3 in general, or E = 200 W/m3 within the salmonid region, in order to ensure low-turbulence flows in the pools.

Flow velocities occur within the orifices can be calculated from equation **(19)** below:

$$v_s = \sqrt{2g\Delta h} \qquad \textbf{... (19)}$$

The discharge through **submerged orifice** can be calculated from equation **(20):**

$$Q_s = \psi A_s \sqrt{2g\Delta h} \qquad \textbf{... (20)}$$

where $A_s = h_s b_s$

The discharge coefficient is define by the following range $\psi = 0.65$ to 0.85.

The discharge over the **top notches** can be calculated from equation **(21)** below:

$$Q_a = \frac{2}{3}\,\mu\,\sigma\,b_a\sqrt{2g}\,h_{weirhead}^{3/2} \qquad \textbf{... (21)}$$

$h_{weirhead}$ is the difference in the water level between headwater and tail water.

μ is the discharge coefficient ($\mu \approx 0.6$), and

σ is the drowned-flow reduction factor.

$$\sigma = \left\{1 - \left[1 - \frac{\Delta h}{h_{weirhead}}\right]^{1.5}\right\}^{0.385} \qquad \textbf{... (22)}$$

which is valid for the range: $0 \le \frac{\Delta h}{h_{weirhead}} \le 1$,

for $\Delta h > h_{weirhead}$, $\sigma = 1$

The maximum velocities of the jet coming from top notches can calculated from equation

$$v_s = \sqrt{2g\Delta h} \qquad \textbf{... (19)}$$

(19)

To ensure a flow with low turbulence and adequate energy conversion within the pools, the

Volumetric dissipated power should not exceed E = 150 to 200 W/m3. The power density can be estimated from the following formula:

$$E = \frac{\rho g \Delta h Q}{b h_m (l_b - d)} \qquad \text{... (23)}$$

in which the total discharge $Q = Q_s + Q_a$ must be entered for Q.

8-2-2-1-6 Example of calculation for a conventional pool pass

Calculations are to be made for a conventional pool pass, The water level differences between headwater and tail water fluctuate between h_{tot} = 1.6 m and h_{tot} = 1.2 m for the discharges that have to be used as basis for the design, Large salmonids such as sea trout or salmon are not anticipated. See fig (8-47) below.

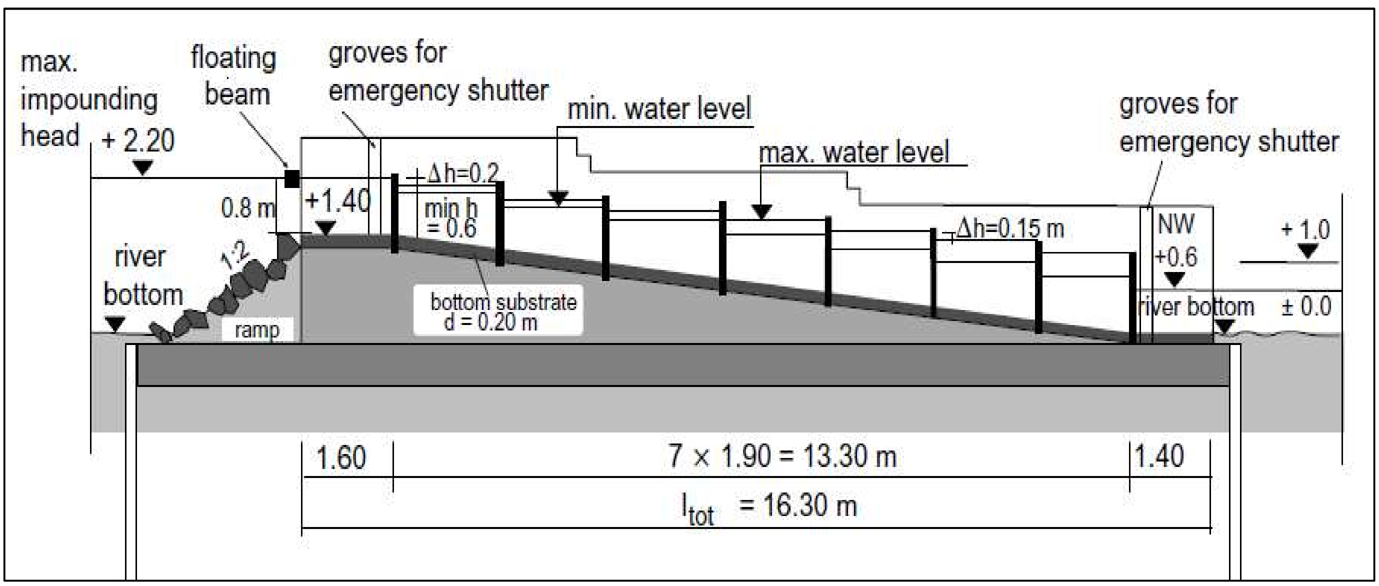


Fig (8-47) longitudinal section through pool pass

The pool dimensions are selected from **Table (8-16)** as follows:

Pool width b = 1.4 m

Minimum water depth h = 0.6 m.

The surface of the pool bottoms is roughened using river boulders as shown in Fig (8-48).

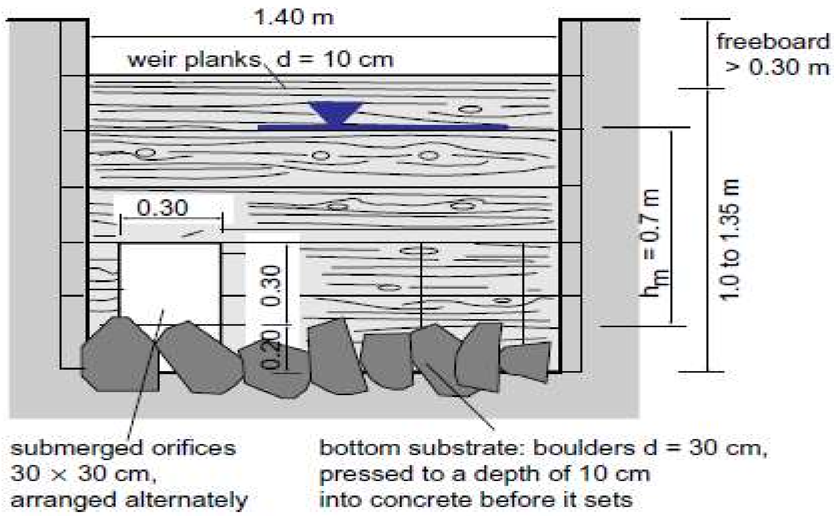


Fig (8-48).

The cross-walls are to have only bottom orifices with a clear orifice span of $b_s = h_s = 0.3$ m.

Top notches are not planned for.

The maximum water level difference must not exceed $\Delta h_{max} = 0.2$ m.

So that the number of pools will be from equation **(18)**:

$$n = \frac{h_{tot}}{\Delta h} - 1 = \frac{1.6}{0.2} - 1 = 7 \text{ pools.}$$

With higher tail water levels, the water-level difference falls to

$$\Delta h_{min} = \frac{1.2}{8} = 0.15 \text{ m.}$$

For Δh = 0.2 m, flow velocity is from equation **(19):**

$$v_s = \sqrt{19.62 \cdot 0.2} = 1.98 \text{ m/s}$$

And for Δh = 0.15 m, flow velocity is from equation **(19)** again**:**

$$v_s = \sqrt{19.62 \cdot 0.15} = 1.71 \text{ m/s;}$$

The flow velocity is thus always lower than the permissible maximum of v_{max} = 2.0 m/s.

With an assumed coefficient of discharge equal to 0.75, the discharge from equation **(20)**:

$$Q_{s.max} = \psi A_s \sqrt{2g\Delta h} = 0.75 \cdot 0.3^2 \cdot 1.98$$
$$= 0.134 \text{ m}^3/\text{s}$$

$$Q_{s,\,min} = 0.75 \cdot 0.3^2 \cdot 1.71 = 0.115 \text{ m}^3/\text{s.}$$

By assuming that value of E = 150 W/m3

And minimal mean water depth of $h_m = h + \Delta h/2 = 0.6 + 0.2/2 = 0.7$ m

And a plank thickness of d = 0.1 m

Pool length will be from equation **(23)**

$$(l_b - d) = \frac{\rho g \Delta h Q}{E b h_m} = \frac{9.81 \cdot 1000 \cdot 0.134 \cdot 0.20}{150 \cdot 1.40 \cdot 0.7}$$

$$l_b = 1.89 \approx 1.90 \text{ m}$$

To find the height of cross wall:

At a water depth of 1.0 m, a bottom substrate layer of 20 cm and Δh = 0.15 m, the height of the downstream cross-wall is

$$h_w = 1.0 + 0.20 + 0.15 = 1.35 \text{ m}$$

And the height of the upstream wall $h_w = 0.8 + 0.20 = 1.0$ m.

The height of the intermediate cross-walls is stepped down by 5 cm each.

The total length is equal to (7 × 1.90) + 1.4+1.6 = 16.3 m.

8-2-2-1-7 Real example of pool pass

POOL PASS AT KOBLENZ			
Details of the dam		**Details of the fish pass**	
River:	Moselle, Rhineland-Palatinate	Pool width:	b = 1.80 m
Use:	Water power generation, navigation (shipping)	Pool length:	$l_b \sim 2.60$ m
		Number of pools:	n = 24
Flows:	$NQ_{1971/80} = 20\ m^3/s$	Water depth:	h = 1.0 m
	$MQ_{1931/90} = 313\ m^3/s$	Total length:	$l_{tot} = 102$ m
	$HQ_{1993} = 4165\ m^3/s$	Slope:	$I \sim 1:12$
Fall head:	$h_F = 5.30$ m	Cross-walls:	Concrete walls with top notches and bottom orifices 30 × 30 cm
Year of construction:	1945-54		
Responsible:	Federal Waterway Authorities/ Moselle Hydroelectric Company		

Fig (8-49) The Coblenz/Moselle fish pass (view from tailwater)

8-2-2-1-8 The biggest fish ladder in the world

Fig (8-50) biggest fish ladder in the Elbe River near Hamburg

The 550 meter-long ladder, which consists of nearly 50 linked shallow pools to allow on average 40,000 fish pass through every day.

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