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Abstract – In this report, structural and civil engineers are introduced to the world of very large floating structures (VLFS) that have been gradually appearing in the waters off developed coastal cities (and countries with coastlines). Their presence is largely due to a severe shortage of land and the sky-rocketing land costs in recent times. After providing a description of a VLFS and highlighting its advantages (under certain conditions) over the traditional land reclamation in creating space from the sea, the authors bring to attention the early, the present and future applications of VLFS. The input design data, hydroelastic analysis and design considerations for very large floating structures are discussed, albeit in the most basic forms.

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1. INTRODUCTION

As population and urban development expand in land-scare island countries (or countries with long coastlines), city planners and engineers resort to land reclamation to ease the pressure on existing heavily-used land and underground spaces. Using fill materials from seabed, hills, deep underground excavations, and even construction debris, engineers are able to create relatively vast and valuable land from the sea. Countries such as the Netherlands, Singapore and Japan, have expanded their land areas significantly through aggressive land reclamation programmes. Probably the first large scale and systematic land reclamation work was carried out by Kiyomori Taira off Kobe's coastal waters in the 12th Century. However, land reclamation has its limitation. It is suitable when the water depth is shallow (less than 20 m). When the water depth is large and the seabed is extremely soft, land reclamation is no longer cost effective or even feasible. Moreover, land reclamation destroys the marine habitat and may even lead to the disturbance of toxic sediments. When faced with these natural conditions and environmental consequences, very large floating structures may offer an attractive alternative solution for birthing land from the sea.

There are basically two types of very large floating structures (VLFSs), namely the semi-submersible-type and the pontoon-type. Semi-submersible type floating structures are raised above the sea level using column tubes or ballast structural elements to minimize the effects of waves while maintaining a constant buoyancy force. Thus they can reduce the wave-induced motions and are therefore suitably deployed in high seas with large waves. Floating oil drilling platforms used for drilling for and production of oil and gas are typical examples of semi-submersible-type VLFSs. When these semi-submersibles are attached to the seabed using vertical tethers with high pretension as provided by additional buoyancy of the structure, they are referred to as tension-leg platforms. In contrast, pontoon-type floating structures lie on the sea level like a giant plate floating on water. Pontoon-type floating structures are suitable for use in only calm waters, often inside a cove or a lagoon and near the shoreline. Large pontoon-type floating structures have been termed Mega-Floats by Japanese engineers. As a general rule of thumb, Mega-Floats are floating structures with at least one of its length dimensions greater than 60 m. Referring to Fig. 1, a Mega-Float system consists of a (a) very large pontoon floating structure, (b) mooring facility to keep the floating structure in place, (c) an access bridge or floating road to get to the floating structure from shore, and (d) a breakwater (usually needed if the significant wave height is greater than 4 m) for reducing wave forces impacting the floating structure.

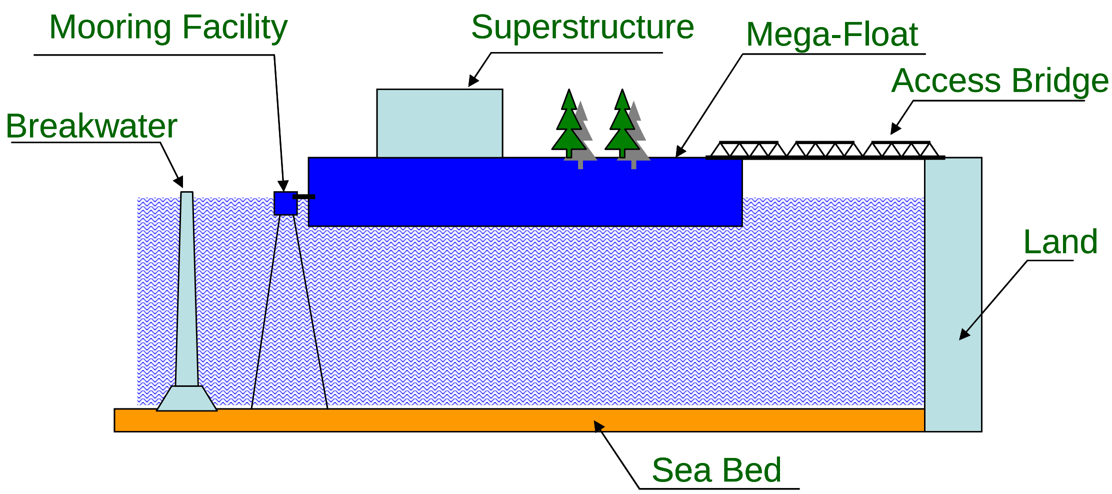

Fig. 1. Components of a Mega-Float System

These Mega-Floats have advantages over the traditional land reclamation solution for space creation in the following respects:

- they are cost effective when the water depth is large (note that the cost of imported sand for land reclamation in some countries has risen significantly and it may come a time that sand may not be even available from neighbouring countries),
- environmental friendly as they do not damage the marine eco-system, or silt-up deep harbours or disrupt the tidal/ocean currents,
- they are easy and fast to construct (components may be made at different shipyards and then brought to the site for assembling) and therefore sea-space can be speedily exploited,
- they can be easily removed (if the sea space is needed in future) or expanded (since they are of a modular form),
- the facilities and structures on Mega-Floats are protected from seismic shocks since they are inherently base isolated,
- they do not suffer from differential settlement due to reclaimed soil consolidation,
- their positions with respect to the water surface are constant and thus facilitate small boats and ship to come alongside when used as piers and berths.
- their location in coastal waters provide scenic body of water all around, making them suitable for developments associated with leisure and water sport activities.

In the sequel, the readers are introduced to the applications of pontoon-type VLFSs, the input data, analysis and design considerations for these VLFSs. For detail information on the design specifications for floating structures, the reader may refer to the *Very Large Floating Structures: Technical Standard and Commentary* produced by the Coastal Development Institute of Technology and the Mega-Float Technological Research Association of Japan (now merged into the Shipbuilding Research Centre of Japan) in 2001.

2. FROM EARLY TO FUTURE APPLICATIONS OF VLFS

Very large floating structures have been used for a variety of purposes. Below, we highlight their applications from early times to present times as well as their applications in the near future.

2.1 Floating Bridges

This section summarizes the large floating bridges pointed out by Watanabe and Utsunomiya (2003). Early applications of very large floating structures take the form of floating boat bridges over rivers that date back to antiquity (Brown 1993). About 480 BC, King Xerxes of Persia led his army across the Hellespont, now called the Dardanelles, using two rows of floating bridges, each consisting of about 300 boats laid side by side as shown in Fig. 2 (Study Group of World Cities, 1988).

Fig. 2 King Xerxes' Floating Boat Bridge across the Hellespont

Fig. 3 Hood Canal Floating Bridge, USA

In 1874, a 124-m long floating wooden railroad bridge was constructed over the Mississippi River in Wisconsin and it was repeatedly rebuilt and finally abandoned. Brookfield Floating Bridge is still in service and it is the seventh replacement structure of a 98-m long wooden floating bridge (Lwin 2000). In 1912, the Galata steel floating bridge was built across Istanbul's Golden Horn where the water depth is 41 m. The 457-m long bridge consists of 50 steel pontoons connected to each other by hinges. However, in 1992, soon after a new bridge was erected just beside the original bridge, a fire broke out and the old Galata floating bridge was burned down (Maruyama *et al.* 1998). The sunken bridge is placed upstream after having been raised from the seabed. The lesson that one can learn from this steel bridge is its amazing resilience against the corrosive sea environment, contrary to engineers' perception that corrosion would pose a serious problem to such floating steel structures.

Other floating bridges include Seattle's three Lake Washington Bridges, i.e. (i) the 2018-m long Lacey V. Murrow Bridge which uses concrete pontoon girders and opened in 1940, (ii) the 2310-m long Evergreen Point Bridge completed in 1963, and (iii) the 1771-m long Homer Hadley Bridge in 1989; the 1988-m long Hood Canal Bridge built in 1963 (see Fig. 3); the Canadian 640-m long Kelowna Floating (concrete) Bridge which was opened to traffic in 1958, the Hawaiian's 457-m long Ford Island Bridge which was completed in 1998.

More recent floating bridges built from 1990s include the two famous Norwegian floating bridges: 845-m long Bergsoysund Floating Bridge built in 1992 near Kristiansund over a fjord depth of 320 m and the 1246-m long Nordhordland Floating Bridge built in 1994 at Salhus over a fjord depth of 500 m (see Fig. 4). Both bridges are horizontally curved (in the form of funicular curves) to better resist the wave, the water current and wind forces. An interesting pedestrian floating bridge is the 94-m long West India Quay Footbridge which was constructed in 1997 (see Fig. 5). This bridge resembles a giant pond skater.

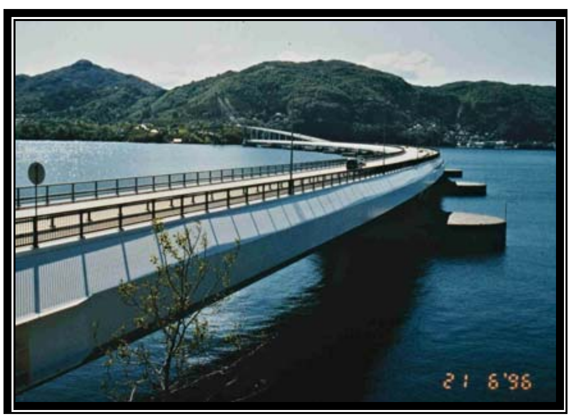

Fig. 4 Nordhordland Floating Bridge, Norway

Fig. 5 West India Quay Footbridge, United Kingdom

An outstanding floating bridge that was built at the turn of the millennium is the 410-m long Yumemai Bridge (see Fig. 6). The bridge is constructed across a water channel, and it floats on two hollow steel pontoons (each of dimensions 58 m x 58 m x 8 m). The bridge can be swung around a pivot axis near one end of the girder when a passage way for very large ships in the channel is needed (for more details of this fascinating bridge, the reader may refer to the paper by Watanabe *et al.* 2001).

It is worth noting that many armies have in their possession floating bridges and floating causeways. Army engineers assemble the floating modules rapidly to form floating bridges for soldiers and vehicles to cross rivers and lakes. Long floating causeways are used by the navy to transport soldiers and equipment from ships to shore (Ertekin and Riggs 2003).

Fig. 6 Yumemai Bridge, Japan

2.2 Floating Entertainment Facilities

As the waterfront and the sea appeal to the general public, VLFSs have been constructed to house entertainment facilities with a scenic 360 degrees view of the surrounding water body. There is a very large Floating Island (130 m x 40 m x 5 m) at Onomichi, Hiroshima. Designed to resemble the Parthenon of Greece, this amusement facility has a 3D visual image theatre, an aquarium and a marina (see Fig. 7). Another floating amusement facility is the Estrayer (128 m x 38 m), shaped like a ship, which is moored at the leisure pier in Kure, Hiroshima Prefecture, Japan. The top deck is used as an event plaza while its deck below houses a movie theatre, restaurants and a game centre.

The first floating hotel in Australia was located at the Great Barrier Reef. It was built in Singapore and is seven storey high, 90 m long and 27 m wide. In case of a cyclone, one mooring end was disconnected and the wind would blow it around in a circle after everyone has evacuated. The floating heliport, tennis courts and pool may be disconnected and towed some distance from the hotel to ride out the storm. After one year of operation, the hotel was towed to Ho-Chi-Minh, Vietnam. It is now located in North Korea.

Hong Kong boasts of having a famous floating restaurant called Jumbo Restaurant. In 1991, Japan built a floating restaurant (on a 24 m x 24 m x 3.2 m pontoon) in Yokohoma (see Fig. 8). The pier, next to the restaurant, is also a floating structure. Very large floating structures are also used as fishing piers. For example, the 101.5 m x 60 m x 3 m floating fishing pier at Awaji Island.

Fig. 7 Floating Island at Onomichi, Hiroshima, Japan

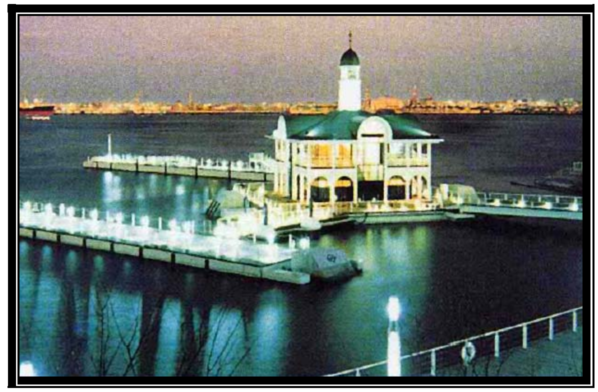

Fig. 8 Floating Restaurant in Yokohoma, Japan

2.3 Floating Storage Facilities

Very large floating structures have been used for storing fuel. Constructed like flat tankers (box-shaped) parked side by side, they form an ideal oil storage facility, keeping the explosive, inflammable fluid from populated areas on land. Japan has two major floating oil storage systems. One oil storage facility is located in Shirashima (see Fig. 9) with a capacity of 5.6 million kilolitres while the other is at Kamigoto (see Fig. 10) with a capacity of 4.4 million kilolitres.

Fig. 9. Shirashima Floating Oil Storage Base, Japan (Photo courtesy of Shirashima Oil Storage Co Ltd)

Fig. 10 Kamigoto Floating Oil Storage Base, Nagasaki Prefecture, Japan

2.4 Floating Emergency Bases

As floating structures are inherently base isolated from earthquakes, they are ideal for applications as floating emergency rescue bases in earthquake prone countries. Japan has a number of such floating rescue bases parked in the Tokyo Bay, Ise Bay and Osaka Bay. Table 1 shows their specifications (Takahashi 2003) and Figs. 11 and 12 show the emergency rescue bases at Tokyo bay and Osaka bay, respectively.

Fig. 11 Emergency Rescue Base
In Tokyo Bay

Fig. 12 Emergency Rescue Base
in Osaka Bay

Table 1 Specifications of Emergency Bases at Tokyo Bay, Ise Bay and Osaka Bay

Specification Item		Tokyo Bay	Ise Bay	Osaka Bay
Structure of floating body		Steel structure	RC hybrid structure	PC hybrid structure
Length x Width x Height (m)		80 x 25 x 4	40 x 40 x 3.8 (Caisson A) 40 x 20 x 3.8 (Caisson B)	80 x 40 x 4
Free Board (m)	During Normal Times	0.7	1.0 to 1.5	1.0
	During Emergencies	1.86		1.5
Regular Mooring Place		Next to earthquake-resistant berth for inland trade, Minato Mirai 21 District, Yokohama Bay	Caisson A: Kinjo Wharf, Nagoya Bay Caisson B: Garden Wharf, Nagoya Bay	Sakura-jima, Konohana District, Osaka Bay

2.5 Floating Plants

A floating structure consisting of two sections was constructed in 1978 in Brazil. One section of the structure is built for a pulp plant (230 m x 45 m x 14.5 m) while the other section is for a power plant (220 m x 45 m x 14.5 m). It was towed to its site at Munguba as a floating structure but was installed in its location on piled foundations.

In 1979, Bangladesh purchased from Japan a 60.4 m x 46.6 m x 4 m floating power plant. The power plant is located at Khulna, Bangladesh. In 1981, Saudi Arabia built a 70 m x 40 m x 20.5 m floating desalination plant and towed to its site where it was sunk into position and rests on the seabed. In 1981, Argentina constructed a 89 m x 22.5 m x 6 m floating polyethylene plant at Bahia Blance. In 1985, Jamaica acquired a 45 m x 30.4 m x 10 m floating power plant. This plant was built in Japanese shipyards and towed to Jamaica and moored by a dolphin-rubber fender system. Studies are already underway to use floating structures for wind farms (see Fig. 13), sewage treatment plant and power plant in Japan.

Fig. 13 Concept Design of a Clean Energy Plant by Floating Structure Association of Japan

2.6 Floating docks, piers, berths and container terminals

There are in existence many floating docks, piers and wharves. For example, the 124 m x 109 m floating dock in Texas Shipyard built by Bethlehem Marine Construction Group in 1985. Floating structures are ideal for piers and wharves as the ships can come alongside them since their positions are constant with respect to the waterline. An example of a floating pier is the one located at Ujina Port, Hiroshima (see Fig. 14). The floating pier is 150 m x 30 m x 4 m. Vancouver has also a floating pier designed for car ferries. Car ferry piers must allow smooth loading and unloading of cars and the equal tidal rise and fall of the pier and ferries is indeed advantageous for this purpose. A floating type pier was also designed for berthing the 50000 ton container ships at Valdez, Alaska. The floating structure was adopted due to the great water depth.

Fig. 14 Floating Pier at Ujina, Japan

2.7 Floating Airports and Mobile Offshore Base

In circa 1920, Edward Armstrong proposed the concept of a *seadrome* (an aerodrome in the sea) as stepping stones for aircrafts flying across the oceans. At that time, the planes could not travel long distances and needed refueling. In 1943, US Navy Civil Engineers Corps constructed a floating airfield (1810 ft x 272 ft) consisting of 10,920 pontoons. It has a flight deck and a parking area. However, the enthusiasm for building these floating airfields was dampened by the extraordinary non-stop flight of Charles Lindbergh from New York to Paris in 1927.

In more recent times, a different sort of problem arose. Land costs in major cities have risen considerably and city planners are considering the possibility of using the coastal waters for urban developments including having floating airports. As the sea and the land near the water edge is usually flat, landings and take-offs of aircrafts are safer. In this respect, Canada has a floating heliport in a small bay in Vancouver. Moreover, this busy traffic heliport is built for convenience as well as noise attenuation. Japan has made great progress by constructing a large airport in the sea. Kansai International Airport at Osaka is an example of an airport constructed in the sea, albeit on a reclaimed island. The first sizeable floating runway is the one-km long Mega-Float test model built in 1998 in the Tokyo bay (see Fig. 15). This floating runway was awarded the world's largest man-made floating island in the Guinness book of records in 1999. Studies on the test model include the investigation of facilities and equipment for floating airport, development of simulation technology of functions of airport, instruments for landing, landing and taking off tests on a floating runway, effects on the environment and verification of construction technologies of a floating airport. The Mega-Float is a precursor to a 3.6-km floating runway which will augment Haneda airport facilities. The decision to proceed building this ultra-large floating Haneda runway (see Fig. 16) will be known by the March 2005.

Fig. 15 Mega-Float in Tokyo Bay, Japan (Photo courtesy of SRCJ)

Fig. 16 Proposed Floating Runway at Tokyo International Airport (Haneda)

The Office of Naval Research, US, has been conducting studies on the technical feasibility and costs of building a mobile offshore base (Taylor 2003). A mobile offshore base is a self-propelled, modular, floating platform that could be assembled into lengths on the order of one mile to provide logistic support of US military operations where fixed bases are not available. We may be seeing these huge mobile offshore bases in the oceans in the future.

2.8 Floating Cities

Perhaps in this 21st Century, floating cities may become a reality with the advancing technology in construction and the shortage of land. Architects and engineers have already made design sketches of how such floating cities could look like. Figures 17-20 show artist impressions of some floating cities that are proposed by various Japanese corporations. Focus A and Focus B are named after their proposed locations at the two foci of the elliptical Osaka Bay.

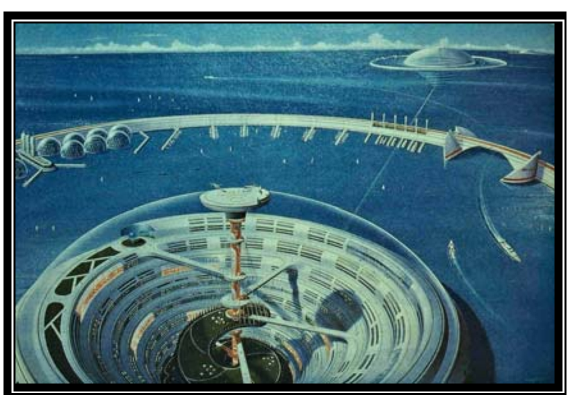

Fig. 17 Marine Uranus by Nishimatsu Corporation

Fig. 18 Pearl Shell by Shimizu Corporation

Fig. 19 Osaka Focus A by Japanese Society of Steel Construction

Fig. 20 Osaka Focus B by Japanese Society of Steel Construction

3. ANALYSIS AND DESIGN OF VLFS

The analysis and design of floating structures need to account for some special characteristics (Clauss *et al.* 1992, Moan 2004) when compared to land-based structures; namely:

- Horizontal forces due to waves are in general several times greater than the (non-seismic) horizontal loads on land-based structures and the effect of such loads depends upon how the structure is connected to the seafloor. It is distinguished between a rigid and compliant connection. A rigid connection virtually prevents the horizontal motion while a compliant mooring will allow maximum horizontal motions of a floating structure of the order of the wave amplitude.
- In framed, tower-like structures which are piled to the seafloor, the horizontal wave forces produce extreme bending and overturning moments as the wave forces act near the water surface. In this case the structure and the pile system need to carry virtually all the vertical loads due to selfweight and payload as well as the wave, wind and current loads.
- In a floating structure the static vertical selfweight and payloads are carried by buoyancy. If a floating structure has got a compliant mooring system, consisting for instance of catenary chain mooring lines, the horizontal wave forces are balanced by inertia forces. Moreover, if the horizontal size of the structure is larger than the wave length, the resultant horizontal forces will be reduced due to the fact that wave forces on different structural parts will have different phase (direction and size). The forces in the mooring system will then be small relative to the total wave forces. The main purpose of the mooring system is then to prevent drift-off due to steady current and wind forces as well as possible steady and slow-drift wave forces which are usually more than an order of magnitude less than the first order wave forces.
- A particular type of structural system, denoted tension-leg system, is achieved if a highly pretensioned mooring system is applied. Additional buoyancy is then required to ensure the pretension. If this mooring system consists of vertical lines the system is still horizontally compliant but is vertically quite stiff. Also, the mooring forces will increase due to the high pretension and the vertical wave loading. If the mooring lines form an angle with the vertical line, the horizontal stiffness and the forces

increase. However, a main disadvantage with this system is that it will be difficult to design the system such that slack of leeward mooring lines are avoided. A possible slack could be followed by a sudden increase in tension that involves dynamic amplification and possible failure. For this reason such systems have never been implemented for offshore structures.

- Sizing of the floating structure and its mooring system depends on its function and also on the environmental conditions in terms of waves, current and wind. The design may be dominated either by peak loading due to permanent and variable loads or by fatigue strength due to cyclic wave loading. Moreover, it is important to consider possible accidental events such as ship impacts and ensure that the overall safety is not threatened by a possible progressive failure induced by such damage.
- Unlike land-based constructions with their associated foundations poured in place, very large floating structures are usually constructed at shore-based building sites remote from the deepwater installation area and without extensive preparation of the foundation. Each module must be capable of floating so that they can be floated to the site and assembled in the sea.
- Owing to the corrosive sea environment, floating structures have to be provided with a good corrosion protection system.
- Possible degradation due to corrosion or crack growth (fatigue) requires a proper system for inspection, monitoring, maintenance and repair during use.

3.1 Loads and Load Effects

Design Loads

In the design of VLFSs, the following loads must be considered: dead load, hydrostatic pressure (including buoyancy), live load, abnormal loads (such as impact loads due to collision of ships with the floating structure), earth pressure on mooring system such as dolphins, wind load, effects of waves (including swell), effects of earthquakes (including dynamic water pressure), effects of temperature change, effects of water current, effects of tidal change, effects of seabed movement, effects of movements of bearings, snow load, effects of tsunamis, effects of storm surges, ship waves, seaquake, brake load, erection load, effects of drift ice and ice pressure, effects of drifting bodies, and effects of marine growths (corrosion and friction).

Buoyancy, waves, current and wind

The buoyancy is computed by the integration of hydrostatic pressure. The specific weight of seawater may be taken to be 10.09 kN/m^3 or 1.03 t/m^3. In the design of very large pontoon floating structures, the change in water level due to tide, tsunami and storm surge may dominate the design loads when the structure is designed with a fixed vertical position relative to the seafloor. Since the point of action of buoyancy depends on the tide and water level, the most unfavourable case will be considered.

Surface water waves may be generated by wind, tidal bore, earthquakes or landslides. The focus here is an oscillatory wind-generated surface waves. Waves developed in an area may endure after the wind cease and propagate to another area; as swell with decaying intensity and slowly changing form. Long period swell travels a very long distance as long-crested waves.

Wind-generated waves consist of a large number of wavelets of different heights, periods and directions superimposed on one another. Although regular waves are not found in real seas they can closely model some swell conditions. They also provide the basic components in irregular waves and are commonly used to establish wave conditions for design. Regular waves are characterised by the wave period and height. The kinematics and hydrodynamic pressure within a regular wave are described by the wave potential as described subsequently.

During a suitably short period of time (from half an hour to some hours) the *sea surface elevation,* ς is commonly assumed to be a zero mean, stationary and ergodic Gaussian process (e.g. Kinsman, 1965). The Gaussian process is completely specified in terms of the wave spectral density, $S_{\varsigma}\left(f_i\right)$ for long-crested waves. In the *time domain* the wave elevation may be described by a sum of long-crested waves specified by linear theory, with different amplitudes a_i, frequencies f_i and phase angles ε_i which are uniformly distributed over $(-\pi,\pi)$. The amplitude a_i may be expressed by the wave spectrum: $a_i = \sqrt{2S_{\varsigma}\left(f_i\right)\Delta f}$.

According to the linear wave theory, the wave kinematics in irregular waves is obtained by superimposing the kinematics of the regular waves constituting the irregular sea. Various analytical formulations for the wave spectrum are applied as parameterized by the significant wave height and period as well as possible other parameters. The significant wave height $H_{1/3}$ (i.e. the average wave height of the highest one-third of all waves) and the peak period T_p or significant wave period $T_{1/3}$ (i.e. the average wave period of the highest one-third of all waves) are used to define the wave spectrum.

In developing seas the JONSWAP spectrum (Hasselman *et al.*, 1973) is recommended and frequently used. Based on hindcasting Nagai *et al.* (1990) established data for JONSWAP wave spectra for the Tokyo Bay. For fully developed seas, the Pierson-Moskowitz spectrum is relevant. Wind sea and swell have different peak periods and a combined sea state may have a two-peaked spectrum, as proposed e.g. by Torsethaugen (1996). It should be noted that much of the wave energy is concentrated in a narrow frequency band close to the peak(s) of the spectrum. Moreover there is a significant difference in the spectral amplitudes for high frequencies, implied by different models.

The variation of the sea state in a long-term period, i.e. of some years duration, can be described as a sequence of short-term sea states, each completely described by the spectral density. For a given analytical model of the spectrum (JONSWAP, Pierson-Moskowitz), the spectral parameters $H_{1/3}$, T_p , etc. completely specify the sea state. By expressing the magnitude of these parameters and direction in probabilistic measures, the long term process is described. For extra tropical regions, like the North Sea, the joint probability density of the parameters is applied towards this aim (see e.g. ISSC, 1973). For tropical areas subjected to hurricanes, the long-term wave climate described by storms arriving in a sequence may be used (e.g. Jahns and Wheeler, 1972).

Data for the long-term model of the waves can be generated (i) by direct observation of wave condition; (II) hindcasting based on wind data.

In addition to wave conditions, current and wind need to be characterized. Besides generating waves, the wind generates a surface current as well as contribute to wind loads. These effects of the wind depend on its velocity, direction and duration, the coastline topography and the depth of the sea. The design wind speed may be specified as a 10-min average at a height of 10 m above the sea surface. Natural load effects such as wind may become critical in some cases.

The current velocity, in general, is composed of two components, namely, wind driven (v_{cwi}) and tide driven (v_{ct}) components. In addition, coastal and ocean currents may occur. Also, eddy currents, currents generated over steep slopes, currents caused by storm surge and internal waves, should be considered. Very little information about their surface velocity and velocity distribution is generally available and measurements are necessary.

The long-term model of wave, wind and current conditions form the basis for identifying the relevant environmental conditions for determining loads for design. In case of design for ultimate strength, the sea loads with an average occurrence period of, say, 100 years or an annual exceedance probability of 10^{-2} is relevant. The wave load pattern may be described by a relevant sea state or even a representative regular wave. When significant structural dynamics effects influence the wave load effects, design based on a sea state rather than a (calibrated) regular wave should be used. Load effects for fatigue analysis should be determined by considering all sea states that might be experienced by the structure.

3.2 Basic Assumptions, Equations and Boundary Conditions for Hydroelastic Analysis of VLFS

The fluid-structure system and the coordinate system are shown in Fig. 21. The origin of the coordinate system is on the undisturbed free surface. The z-axis is pointing upwards, and the sea-bed is assumed to be flat at $z = -h$. The VLFS has a maximum length of $2a$ in the x-direction, a maximum width of $2b$ in the y-direction, and a draft d in the z-direction. The problem at hand is to determine the response of the VLFS under the action of wave forces.

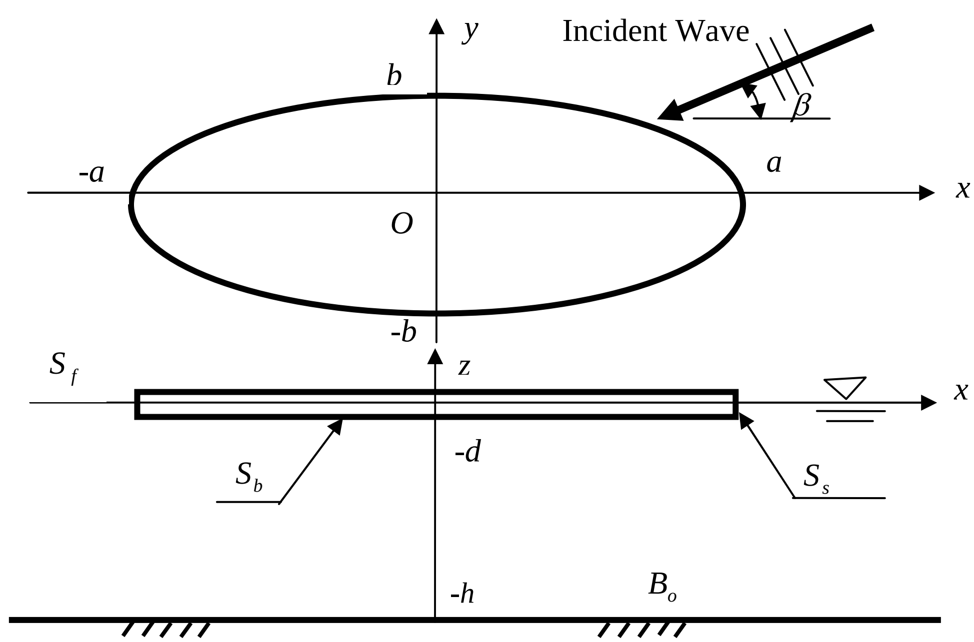

Fig 21 Pontoon-type VLFS under Wave Action

In a basic hydroelastic analysis of pontoon-type VLFSs, the following assumptions are usually made:

- The VLFS is modeled as an elastic (isotropic/orthotropic) thin plate with free edges
- The fluid is incompressible, inviscid and its motion is irrotational so that a velocity potential exists.
- The amplitude of the incident wave and the motions of the VLFS are both small and only the vertical motion of the structure is considered (i.e. we constrained the plate from moving horizontally in the analysis).
- There are no gaps between the VLFS and the free fluid surface.

The analysis may be carried out in the frequency domain or in the time domain. Most hydroelastic analyses are carried out in the frequency-domain, being the simpler of the two. However, for transient responses and for nonlinear equations of motion due to the effects of a mooring system or nonlinear wave (as in a severe wave condition), it is necessary to perform the analysis in the time-domain. Below, we present the governing equations, boundary conditions and briefly describe the commonly used methods for the analysis in the frequency-domain and in the time-domain.

Frequency-Domain Analysis

Considering time-harmonic motions with the complex time dependence $e^{i\sigma t}$ being applied to all first-order oscillatory quantities, where i represents the imaginary unit, σ the angular frequency (which is obtained from a given wave period) and t the time, the complex velocity potential $\phi(x, y, z)$ is governed by the Laplace's equation in the fluid domain,

$$\nabla^2\phi(x, y, z) = 0\,, \tag{1}$$

The velocity potential $\phi(x, y, z)$ must satisfy the boundary conditions on the free surface, S_f, on the sea-bed, B_0, and on the wetted surfaces of the floating body, S_b (bottom surface) and S_S (side surface):

$$\frac{\partial\phi(x, y, z)}{\partial z} = \frac{\sigma^2}{g}\phi(x, y, z) \quad \text{on } S_f\,, \tag{2}$$

$$\frac{\partial\phi(x, y, z)}{\partial z} = 0 \quad \text{on } B_0\,, \tag{3}$$

$$\frac{\partial\phi(x, y, -d)}{\partial n} = i\sigma w(x, y) \quad \text{on } S_b\,, \tag{4}$$

$$\frac{\partial\phi(x, y, z)}{\partial n} = 0 \quad \text{on } S_S\,, \tag{5}$$

where $w(x, y)$ is the vertical complex displacement of the plate, d the draft of the floating structure, g the gravitational acceleration and n the unit normal vector pointing from the fluid domain into the body. The radiation condition for the scattering and radiation potential is also applied at infinity,

$$\lim_{r\to\infty}\sqrt{r}\left[\frac{\partial(\phi-\phi_I)}{\partial r}+ik(\phi-\phi_I)\right]=0 \quad \text{on } S_\infty, \tag{6}$$

where r is the radial coordinate measured from the centre of the VLFS, k the wave number that obeys the dispersion relation $k\tanh(kh)=\sigma^2/g$ for a finite water depth and ϕ_I the potential representing the undisturbed incident wave and it is given by

$$\phi_I=\frac{gA}{i\sigma}\frac{\cosh(k(z+h))}{\cosh(kh)}\exp[ik(x\cos\beta+y\sin\beta)] \tag{7}$$

where A is the amplitude of the incident wave (obtained from the wave spectrum for a given frequency or period) and β the angle of incident wave (see Fig. 21).

By assuming the VLFS as an elastic, isotropic, thin plate, the motion of the floating body is governed by the equation of a thin plate resting on a uniform elastic foundation:

$$D\nabla^4 w(x,y)-\sigma^2\gamma w(x,y)+\rho g w(x,y)=p(x,y), \tag{8}$$

where D is the plate rigidity, γ the mass per unit area of the plate, ρ the density of the fluid and $p(x,y)$ the dynamic pressure on the bottom surface of the plate. Based on the linearized Bernoulli equation, the dynamic pressure $p(x,y)$ is related to the velocity potential $\phi(x,y,z)$ by

$$p(x,y)=-i\rho\sigma\phi(x,y,-d). \tag{9}$$

The floating body, with no constraints in the vertical direction along its edges, must satisfy the zero effective shear force and zero bending moment conditions for a free edge:

$$\frac{\partial^3 w(x,y)}{\partial n^3}+(2-\nu)\frac{\partial^3 w(x,y)}{\partial n\partial s^2}=0, \tag{10a}$$

$$\frac{\partial^2 w(x,y)}{\partial n^2}+\nu\frac{\partial^2 w(x,y)}{\partial s^2}=0, \tag{10b}$$

where n and s denote the normal and tangential directions and ν is the Poisson ratio.

The commonly-used approaches for the analysis of VLFS in the frequency domain are the *modal expansion method* and the *direct method*. In the *modal expansion method*, the interaction problem of the fluid motion and the plate response [given by Eqs. (8) and (9)] is decoupled into a hydrodynamic problem in terms of the velocity potential $\phi(x,y,z)$ and the mechanical problem of a freely vibrating plate with free edges. For the latter, the motion of the plate $w(x,y)$ is expanded by modal functions that can be arbitrarily chosen.

$$w(x,y)=\sum_i\zeta_i w_i(x,y) \tag{11}$$

where ζ_i is the amplitude of the *i*-th mode of freely vibrating plate and these amplitudes are the unknowns that are to be determined. For the modal functions, researchers have used products of free-free beam modes (Maeda *et al.* 1995, Wu *et al.* 1995, 1996, 1997, Kashiwagi 1998a, Nagata *et al.* 1998, Utsunomiya *et al.* 1998, Ohmatsu 1998a); B-spline functions (Lin and Takaki 1998), Green functions (Eatock Taylor and Ohkusu 2000), two-dimensional polynomial functions (Wang *et al.* 2001) and finite element solutions of freely vibrating plates (Takaki and Gu 1996a).

These modal functions are then used in the hydrodynamic analysis as shown below. Based on the linear theory, the velocity potential $\phi(x, y, z)$ can be expressed as the sum of the incident potential ϕ_I, the diffraction potential ϕ_D and radiation potential ϕ_R by using the same modal amplitudes (Newman 1994):

$$\phi(x, y, z) = \phi_I(x, y, z) + \phi_D(x, y, z) + \sum_i \zeta_i \phi_{iR}(x, y, z) \tag{12}$$

where ϕ_{iR} is the radiation potential corresponding to unit amplitude motion of the *i*-th modal function.

In view of Eqs. (11) and (12), the boundary conditions on the surface, i.e. Eqs. (4) and (5), become

$$\frac{\partial \phi_{iR}(x, y, z)}{\partial n} = i\sigma w_i(x, y) \quad \text{on } S_b \tag{13}$$

$$\frac{\partial \phi_{iR}(x, y, z)}{\partial n} = 0 \quad \text{on } S_s \tag{14}$$

and

$$\frac{\partial \phi_D(x, y, z)}{\partial n} = -\frac{\partial \phi_I(x, y, z)}{\partial n} \quad \text{on } S_b \text{ and } S_s \tag{15}$$

Equations (1), (12) and the boundary conditions, given in Eqs. (13) to (15), are then solved for the velocity potential $\phi(x, y, z)$. Apart from a circular fluid domain associated with a circular floating body where closed form solution for the velocity potential may be obtained (see Watanabe *et al.* 2003), numerical methods (such as the boundary element method) have to be employed for determining the velocity potential. After having obtained the velocity potential, the Galerkin's method (by which the governing equation of the plate is approximately satisfied) is then used to calculate the modal amplitudes ζ_i. Using Eq. (11), the modal responses are summed up to obtain the total response. For more details of the hydroelastic analysis using the modal expansion method, readers may refer to the papers by Utsunomiya *et al.* (1998) and Watanabe *et al.* (2003).

In *the direct method*, the deflection of the VLFS is determined by directly solving the motion of equation without any help of eigenmodes. Mamidipudi and Webster (1994) pioneered this direct method for a VLFS. In their solution procedure, the potentials of diffraction and radiation problems were established first, and the deflection of VLFS was determined by solving the combined hydroelastic equation via the finite difference scheme. Their method was modified by Yago and Endo (1996) who applied the pressure distribution method and the equation of motion was solved using the finite element method.

Ohkusu and Namba (1996) proposed a different type of direct method which does away with the commonly used two-step modal expansion approach. Their approach is based on the idea that the thin plate is part of the water surface but with different physical characteristics than those of the free surface of the water. The problem is considered as a boundary value problem in hydrodynamics rather than the determination of the elastic response of the body to hydrodynamic action. This approach was used to analyze a similar problem of two dimensional ice floe dynamics by Meylan and Squire (1994). Ohkusu and Namba (1998) treated the VLFS as a plate of infinite length and the velocity potential was solved directly from a combined hydroelastic 6^{th}-order differential equation. The deflections are estimated from the resultant velocity potential. The advantage of this method is that a closed form solution may be obtained in the case of shallow waters.

In Kashiwagi's direct method (1998b), the pressure distribution method was applied and the deflection was solved from the vibration equation of the structure. In order to achieve a high level of accuracy in very short wavelength regime as well as short computational times and fewer unknowns, he uses bi-cubic B-spline functions to represent the unknown pressure and a Galerkin method to satisfy the body boundary conditions. His method for obtaining accurate results in the short wavelength regime is a significant improvement over the numerical techniques proposed by other researchers (Yago 1995, Wang *et al.* 1997) who have also employed the pressure distribution method.

In sum, the principal difference between the modal superposition method and the direct method lies in the treatment of the radiation motion for determining the radiation pressure. For example, we observed that Takaki and Gu (1996a, 1996b) used the shape function of dry eigen-modes of a plate with free edges while Yago and Endo (1996) employed the shape function of a constant panel for the unknown pressure. The shortcoming of the constant panel method is that it is very difficult to deal with short incident waves that are important in VLFS analysis. In order to cater for the short wave case, Lin and Takaki (1998) proposed the method be based on high-order B-spline panels.

Recently, acceleration techniques for the hydrodynamic analysis using free-surface Green's function method have been developed, and applied very successfully for the hydroelastic analysis of VLFSs (Kring *et al.* 2000, Utsunomiya *et al.* 2001a, 2001b, 2003).

Time-Domain Analysis

The commonly-used approaches for the time-domain analysis of VLFS are the direct time integration method (Watanabe and Utsunomiya 1996, Watanabe *et al.* 1998) and the method that uses the Fourier transform (Miao *et al.* 1996, Endo *et al.* 1998, Ohmatsu 1998b, Endo 2000, Kashiwagi 2000). In the direct time integration method, the equations of motion are discretized for both the structure and the fluid domain. In the Fourier transform method, we first obtain the frequency domain solutions for the fluid domain and then Fourier transform the results for substitution into the differential equations for elastic motions. The equations are then solved directly in the time domain analysis by using the finite element method or other suitable computational methods.

Load Effect and Design Checks

For given floating body (modeled as a plate) dimensions, depth of sea, wave frequency (or period), wave height, and amplitude of the incident wave, the hydroelastic analysis yields the velocity potential and the vertical deflection of the floating body. The vertical deflections may then be used in the computation of the stress resultants and stresses. The deflections are checked against serviceability requirements while the stresses are checked against strength requirements.

Local Structural Analysis

In this paper we have focused on the global analysis of a pontoon type VLFS. To accomplish structural design the load effects in the various components of the structure, such as stiffened panels of deck and bottom and bulkheads, girders, stiffeners and plates are required. A particular challenge is associated with determining load effects for fatigue design checks, for which local (hot spot) stresses are required. In general a hierarchy of finite element models would be used for this purpose.

3.3 Design Considerations for Floating Body

The design of the floating structure must meet the operating conditions, strength and serviceability requirements, safety requirements, durability, visually pleasing to the environment and cost-effective. An appropriate design service life is prescribed depending on the importance of the structure and the return period of natural loads. Its service life is generally expected to be as long as 50 to 100 years with preferably a low maintenance cost.

Materials

The materials used for the floating body may be steel, or concrete or steel-concrete composite and the relevant specifications should be followed. Since watertightness of concrete is important to avoid or limit corrosion of the reinforcement, either watertight concrete or offshore concrete should be used. High-performance concrete containing fly ash and silica fume is most suitable for floating structures. The effects of creep and shrinkage are considered only when the pontoon are dry, and hence not considered once the pontoon are launched in the sea. Steel used for floating structures shall satisfy the appropriate standard specifications (such as the Technological Standard and Commentary of Port and Harbour Facilities 1999).

Configurations

As mentioned above, the main configurations of VLFS are the pontoon or barge and the semi-submersible type structures. Various special features may be envisaged for the pontoon type. For instance it may be connected to a submerged plate and skirt like structures which reduces motions (Ohta *et al.* 1999), or it might be attached to a floating breakwater to reduce wave excitation on the VLFS itself, as discussed in Section 3.5. Yet another feature could be the design of the edges of the pontoon type VLFS. By proper choice of edge layout the propagation of the incident waves into the main part of the structure is reduced by efficient scattering or reflection of the incident waves on the weather side. Finally, possible use of air-cushion to reduce excitation of vertical motions should be mentioned (Ikoma *et al.* 2003). Some of these innovative design features are promising, but commonly there are disadvantages as well, and they need to be assessed.

Design Criteria

Adequate performance of offshore structures is ensured by designing them to comply with serviceability and safety requirements for a service life of 100 years or more. Serviceability criteria are introduced to ensure that the structure fulfils the function required, and are specified by the owner. Typical serviceability requirements relate to motions and structural deformations. Motion characteristics might not only include displacements, velocities and accelerations. It is noted that criteria in terms of the third order time derivative of the displacement are also considered for floating bridges.

Safety requirements are imposed to avoid ultimate consequences such as fatalities, environmental damage or property damage. Depending upon the regulatory regime, separate acceptance criteria for these consequences are established. Property damage is measured in economic terms. But, fatalities and pollution obviously have economic implications. While fatalities caused by structural failures would be related to a major structural damage, smaller damage may result in property damage which is expensive to repair e.g. for an underwater structure. An important design issue regarding safety of personnel is evacuation and rescue. An effective safety measure in this connection could be to provide a safe place where people can survive on board after an accident some time before safe escape can be made.

In principle the global failure modes of floating structures include capsizing, sinking, global structural failure and drift-off. A broad risk analysis approach needs to be carried out to identify possible accident scenarios and their likelihood. However, overall stability of floating structures is considered in terms of overturning moment by wind only, and uprighting moment due to hydrostatics of the inclined body. However, due to the large horizontal dimensions of VLFS, stability of the intact VLFS structure is not a problem. Even damage to a few compartments does not seem to impose a stability problem. Sinking could be caused by (excessive) flooding or structural failure. Hence, global failure of the structure and mooring system are, therefore, major failure modes.

Modern safety criteria for marine structures are expressed by limit states as indicated in Table 2, and are briefly outlined in the following.

Table 2 Safety Criteria (e.g. ISO 19900 1994, Moan 2004)

Limit states	Description	Remarks
Ultimate (ULS)	- Overall "rigid body" stability - Ultimate strength of structure, - Ultimate strength of mooring system	(Not relevant for VLFS) Component design check
Fatigue (FLS)	- Failure of joint – normally welded joints in hull and mooring system	Component design check depending on residual system strength after fatigue failure
Accidental collapse (ALS)	- Ultimate capacity[1)] of damaged structure (due to fabrication defects or accidental loads) or operational error	System design check

[1)] Capacity to resist "rigid body" instability or total structural failure

ULS and FLS criteria for structural components have been developed for the relevant failure modes dependent upon geometry and load conditions. The relevant criteria follow the same principles as established for ships and especially for offshore structures, which are based on first principles. However, the implicit safety level aimed at should be carefully considered in view of the potential consequences of failure. The safety level implied by ULS and FLS requirements is determined by the chosen definition of characteristic values of loads and strength and the safety factors in ULS criteria and safety margin in FLS.

The fatigue life of the structure is estimated by comparing the long-term cyclic loading in a structural detail with the resistance of that detail to fatigue damage. The main design approach for determining fatigue damage is based on the S-N data. This approach uses an S-N curve which gives the number of cycles to failure for a specific structural detail or material as a function of constant stress range, based on experimental results. The long-term stress distribution is used to calculate the cumulative fatigue damage ratio, D, given by

$$D = \sum_i \frac{n_i}{N_i} \tag{16}$$

where n_i is the number of cycles within stress range interval i and N_i the number of cycles to failure at stress range i as given by the appropriate SN curve. The allowable damage is often taken to be 1.0 for ships, while for offshore structures it varies between 1.0 and 0.1. It can be shown that with an allowable D of 1.0, the probability of fatigue failure in the service life of the structure will be 10% (Moan, 2004). This estimate has been validated for offshore structures and ships by service experiences. With a large number of welded joints fully utilized according to this criterion, many fatigue cracks should be expected in a structure with thousands of welded joints. On the other hand, the definition of fatigue failure implicit in SN curves is typically through thickness crack. Hence, in monocoque structures made of stiffened steel panels, there will be a significant period of crack growth before the cracks really become critical from a strength point of view. From a safety point of view, a fatigue criterion with D = 1.0 would be acceptable, but the maintenance and repair efforts implied may imply so large expenditure that a more restrictive fatigue design criterion would be more optimal based on cost-benefit considerations.

Accidental Collapse Limit State (ALS) requirements are motivated by the design philosophy that "small damage, which inevitably occur, should not cause disproportionate consequences". Since the purpose of this criterion is to prevent progressive development of failure, the criterion was initially denoted Progressive Limit State criterion. A quantitative ALS criterion was introduced for offshore structures in Norway in 1984 (e.g. NORSOK N-001 2000).

The initial damage according to the NORSOK N-001 should correspond to events which are exceeded with an annual probability of 10^{-4}, e.g. due to ship impacts or fires, as identified by risk analyses. The (local) damage, or permanent deformations or rupture of components need to be estimated by accounting for nonlinear effects. Relevant initial damage for the mooring system should also be assessed. For floating offshore structures, it is required that one of the mooring lines has failed. Obviously for application of such a criterion to VLFSs, the probability level that defines the initial damage condition should be judged in view of the target safety level aimed at. For VLFSs, the relevant damage due to ship impact would involve structural damage and loss of buoyancy due to possible flooding.

The structure and mooring system are required to survive the various damage conditions as mentioned above without global failure. Compliance with this requirement for the hull can in some cases be demonstrated by removing the damaged parts, and then accomplishing a conventional ULS design check, based on a global linear analysis and component design checks using truly ultimate strength formulations. However, such methods may be very conservative and more accurate nonlinear analysis methods should be applied.

Corrosion Protection

The corrosion protection system includes coatings, cathodic protection, corrosion allowance and corrosion monitoring. Overprotection which may cause hydrogen embrittlement should be avoided. In areas where marine organisms are active, antifouling coatings may be considered to reduce marine growth.

The steel should be protected from corrosion using a corrosion protection system that is in accordance to specifications such as NACE Standard RP-01-76. Care should be given to parts just beneath the mean low water level (MLWL) where severe local corrosion occurs. For such parts, cathodic protection is generally applied while coating methods are applied for parts shallower than the depth of 1 m below the low water level (LWL). The coating methods include painting, titanium-clad lining, stainless steel lining, thermal spraying with zinc, aluminium and aluminium alloy.

Table 3 presents the standard values of the rate of corrosion and Fig. 22 shows a sketch of the distribution of corrosion according to depth of water and seabed. The splash zone is the most severe with regard to corrosive environment and its upper limit zone is determined according to the installation of the structure. The ebb and flow zone corresponds to the next most severe environment but this zone does not exist for floating structures since they conform to the changing water level. Special attention should be given to the region immediately below LWL. In the seawater zone, the environment becomes milder but marine growths and water current may some times accelerate the corrosion. The environment in the soil layer beneath the seabed is even milder, although it depends on the salt density and the degree of contamination.

Table 3 Rates of Corrosion

	Corrosive Environment	Corrosion Rate (mm/year)
Offshore side	Above HWL	0.3
	HWL to – 1 m below LWL	0.1-0.3
	1 m below LWL to seabed	0.1-0.2
	Mud layer beneath seabed	0.03
Onshore side	Air	0.1
	Earth above water level	0.03
	Earth below waer level	0.02

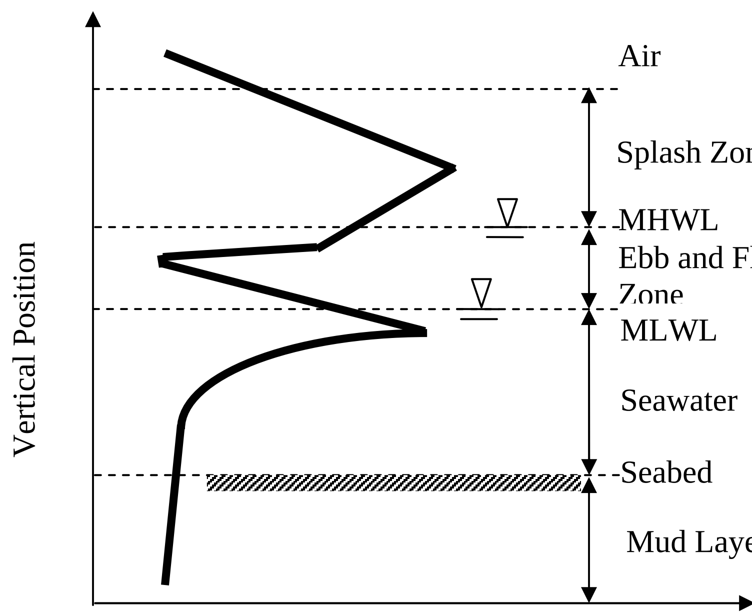

Fig. 22 Distribution of corrosion rate of steel in air, seawater and seabed

3.4 Design Considerations for Mooring System

The mooring system must be well designed as it ensures that the very large floating structure is kept in position so that the facilities installed on the floating structure can be reliably operated and to prevent the structure from drifting away under critical sea conditions and storms. A freely drifting very large floating structure may lead to not only damage to the surrounding facilities but also the loss of human life if it collides with ships. Note that there are a number of mooring systems such as the dolphin-guideframe system, mooring by cable and chain, tension leg method and pier/quay wall method (see Fig. 23).

The design procedure for a mooring system may take the following steps: we first select the mooring method, the shock absorbing material, the quantity and layout of devices to meet the environmental conditions and the operating conditions and requirements. The layout of the mooring dolphins for example is such that the horizontal displacement of the floating structure is adequately controlled and the mooring forces are appropriately distributed. The behaviour of the floating structure under various loading conditions is examined. The layout and quantity of the devices are adjusted so that the displacement of floating structure and the mooring forces do not exceed the allowable values. Finally, devices such as dolphins and guide frames are designed by applying the design load based on the calculated mooring forces.

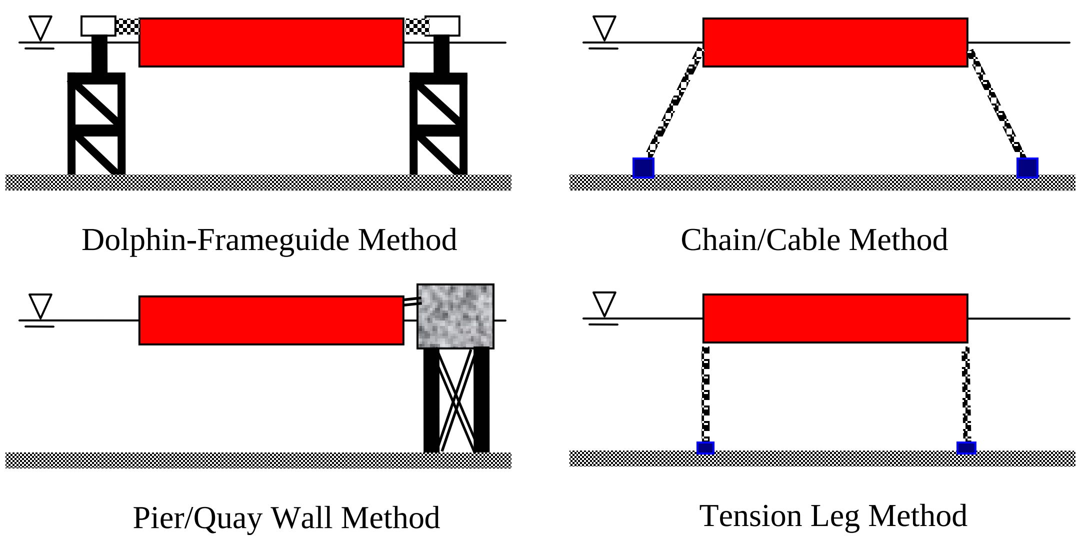

Fig. 23 Various Types of Mooring Systems

The materials for the mooring system shall be selected according to the purpose, environment, durability and economy.

Research studies on the analysis of VLFS with the allowance for a mooring system were carried by Maeda *et al.* (2000), Shimada and Miyajima (2002), Ookubo *et al.* (2002) and Shiraishi *et al.* (2002).

3.5 Design Considerations for Breakwaters

In order to reduce the wave amplitude impacting the VLFS, breakwaters are constructed nearby. A general rule of thumb is to have a breakwater if the significant wave height is greater than 4 m.

Breakwaters are usually of the gravity-type (or bottom-mounted). For methods of analysis of VLFS in waves with a breakwater, one may refer to the following references: Nagata *et al.* (1998, 2003), Utsunomiya *et al.* (1998), and Seto and Ochi (1998).

Although conventional bottom-mounted breakwaters provide the best wave-breaking performance, they however cut off water flow around the VLFS and thus they cancel the ecological friendly merit. Moreover, the construction costs for these breakwaters may be high when the installation depth is deep. In view to reduce costs as well as to maintain the environmental friendly space, breakwaters which allow water to flow through openings at their bottom have been proposed. Ohmatsu *et al.* (2001) and Maeda *et al.* (2001) considered various kinds of breakwaters such as the Oscillating Water Column (OWC) type and structure embedded by OWC type breakwater. Takaki *et al.* (2002) proposed a system consisting of a floating breakwater using submerged plate. Hong *et al.* (2002) treated vertical barriers floating or fixed types and studied the hydroelastic responses of VLFS by varying the gap between the bottom of the breakwaters and the seabed. They concluded that the hydroelastic response of VLFS may be reduced by more than 70% by using single surface-piercing vertical wave barrier with 50% under water opening ratio and for double layer barriers, the additional effect is only expected when the same size barriers are deployed. The performance of multi-layered wave barriers is mainly governed by the barrier with the largest blockage ratio and additional submerged barriers have little effect.

CONCLUDING REMARKS

The definition, applications, analysis and design of very large floating structures have been presented. For details of analysis and design on pontoon-type VLFS, the reader may refer to a large body of references given in a recent literature survey paper by Watanabe *et al.* (2004). It is hoped that this report will create an awareness and interest in structural and civil engineers on the subject of very large floating structures and to exploit their special characteristics in conditions that are favourable for their applications.

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REFERENCES

Brown, D.J. (1993). *Bridges: Three Thousand Years of Defying Nature*, Reed International, London.

Clauss, G., Lehmann, E. and Ostergaard, C. (1992). *Offshore Structures, Vol 1 Conceptual Design and Hydromechanics*, Springer-Verlag, Berlin, 1992. (Translated by M.J. Shields).

Eatock Taylor, R. and Ohkusu, M. (2000). "Green functions for hydroelastic analysis of vibrating free-free beams and plates," *Applied Ocean Research*, **22,** pp. 295-314.

Endo, H., Yago, K. and Chiaki, S. (1998). "Elastic responses of a floating platform stimulated by dynamic load," In: *Proc. 14th Ocean Engrg. Symp.*, Soc. Nav. Arch. Japan, 411-416.

Endo, H. (2000). "The behaviour of a VLFS and an airplane during takeoff/landing run in wave condition," *Marine Structures*, **13**:477-491.

Ertekin, R.C. and Riggs, H.R. (2003). "Static and dynamic analyses of a moored causeway," *Proceedings of the International Symposium on Ocean Space Utilization Technology*, NRMI, pp. 111-120, January 28-31, Tokyo, Japan.

Hasselman, K. *et al.* (1973). 'Measurements of the wind-wave growth and swell decay during the Joint North Sea Wave Project (JONSWAP)', *Erganzungsheft zur Deutschen Hydrographischen Zeitschrift*, Reihe A, No. 12, Hamburg, Germany.

Hong, S.Y., Choi, Y.R. and Hong, S.W. (2002). "Analysis of hydroelastic response of pontoon-type VLFS coupled with floating breakwaters using a higher-order boundary element method," In: *Proc. of the 12th Int. Offshore and Polar Engrg. Conf.,* 313-318, Kitakyushu, Japan, 26-31 May 2002.

Ikoma, T., Maeda, H., Masuda, K. And Rheem, C.K. (2003). "Effects of the air-chambers on the hydroelastic response reduction," *Proceedings of International Symposium on Ocean Space Utilization Technology*, 180-188, Tokyo, Japan, 28-31 January 2003.

ISO 19900, (1994). 'Petroleum and Natural Gas Industries – Offshore Structures – Part 1: General Requirements', (1994) Int. Standardization Organization, London

ISSC (1973). "Report of ISSC Committee I.1. Environmental Conditions", *Proc. 5th International Ship Structures Congress*, Hamburg.

Jahns, H.O. and Wheeler, J.D. (1972) 'Long Term Wave Probabilities based on Hindcasting of Severe storms', Paper No. 1590, *Offshore Technology Conf.*, Houston.

Kashiwagi, M. (1998a). "A B-spline Galerkin scheme for calculating the hydroelastic response of a very large floating structure waves," *J. Mar. Sci. Tech.*, **3**(1):37-49.

Kashiwagi, M. (1998b) "A new solution method for hydroelastic problems of a very large floating structure in waves," In: *Proc. of the 17th Int. Conf. Offshore Mechanics and Artic Engrg.*, ASME, Lisbon, Portugal, OMAE98-4332, July 8.

Kashiwagi, M. (2000). "A time-domain mode-expansion method for calculating transient elastic responses of a pontoon-type VLFS," *Journal of Marine Science and Technology*, **5**:89-100.

Kinsman, B. (1965) "*Wind Waves*", Prentice Hall, Englewood Cliffs, New Jersey.

Kring, D., Korsmeyer, T., Singer, J. and White, J. (2000). "Analyzing mobile offshore bases using accelerated boundary element methods," *Marine Structures*, **13**:301-303.

Lin, X. and Takaki, M. (1998). "On B-spline element methods for predicting hydroelastic responses of a very large floating structure in waves," In: M Kashiwagi et al. (Eds)., In: *Proc. of the 2nd Hydroelasticity in Marine Technology,* Kyushu University, Fukuoka, Japan, December 1-3, 219-228.

Lwin, M.M. (2000). Floating Bridges, in: Chen W.F. and Lian Duan (eds) Bridge Engineering Handbook, 22, pp. 1-23, CRC Press.

Maeda, H., Masuda, K., Miyajima, S. and Ikoma, T. (1995). "Hydroelastic responses of pontoon-type very large floating offshore structure," *J. Soc. Naval Arch. Japan*, **178**, 203-212. (in Japanese).

Maeda, H., Ikoma, T., Masuda, K. and Rheem, C.K. (2000). "Time-domain analyses of elastic response and second-order mooring force on a very large floating structure in irregular waves," *Marine Structures*, **13**:279-299.

Maeda, H., Rheem, C.K., Washio, Y., Osawa, H., Nagata, Y., Ikoma, T., Fujita, N. and Arita, M. (2001). "Reduction effects of hydroelastic responses on a very large floating structure with wave energy absorption devices using OWC system," In: *Proc. of 20th OMAE Conf.*, OMAE01/OSU-5013, Rio de Janeiro, Brazil.

Mamidipudi, P. and Webster, W.C. (1994). "The motions performance of a mat-like floating airport," In: *Proc. Int. Conf. on Hydroelasticity in Marine Technology*, Trondheim, Norway, 25-27 May 1994, Edited by O Faltinsen et al., AA. Balkema, Rotterdam, 363-375.

Maruyama, T., Watanabe, E. and Tanaka, H. (1998). "Floating swing bridge with a 280 m span, Osaka," *Structural Engineering International* (IABSE), 8(3), pp. 174-175.

Meylan, M. and Squire, V.A. (1994). "The response of ice floe to ocean waves," *Journal of Geophysical Research*, **99**.

Miao, Q., Du, S., Dong, S. and Wu, Y. (1996). "Hydrodynamic analysis of a moored very large floating structure," In: Y Watanabe (Ed.), In: *Proc. Int. Workshop on Very Large Floating Structures*, Hayama, Kanagawa, Japan, November 25-28:201-208

Moan, T. (2004). "Safety of floating offshore structures" *Proc. 9th PRADS Conference*, Keynote lecture, PRADS Conference, Luebeck-Travemuende, Germany, September 12-17, 2004.

Nagai, T, Goto, C. and Kobune, K. (1990) "Wave hindcasting model using the hybrid-parameter method (1st report) – Application to Tokyo Bay" *Report of the Port and Harbour Research Institute*, Vol. 29. No. 4, 85-118.

Nagata, S., Yoshida, H., Fujita, T. and Isshiki, H. (1998). "Reduction of the motion of an elastic floating plate in waves by breakwaters," In: M Kashiwagi *et al.* (Eds), In: *Proc. of the 2nd Hydroelasticity in Marine Technology,* Kyushu University, Fukuoka, Japan, pp. 229-238., December 1-3, 1998.

Nagata, S., Niizato, H., Yoshida, H., Ohkawa, Y. and Kobayashi, K. (2003). "Effects of breakwaters on motions of an elastic floating plate in waves," *International Journal of Offshore and Polar Engineering*, 13(1), 43-51.

Newman, J.N. (1994). "Wave effects on deformable bodies," *Applied Ocean Research*, **16**, 47-59.

NORSOK N-001(2000). "Structural Design (of Offshore Structures)" Standard Norge, Oslo.

Ohkusu, M. and Namba, Y. (1996). "Analysis of hydroelastic behaviour of a large floating platform of thin plate configuration in waves," In: Y Watanabe (Ed.), In: *Proc. Int. Workshop on Very Large Floating Structures*, Hayama, Kanagawa, Japan, November 25-28:143-148.

Ohkusu, M. and Namba, Y. (1998). "Hydroelastic behaviour of a large floating platform of elongated form on head waves in shallow water," In: M. Kashiwagi et al. (Eds)., In: *Proc. of the 2nd Hydroelasticity in Marine Technology,* Kyushu University, Fukuoka, Japan, December 1-3: 177-183.

Ohmatsu, S. (1998a). "Numerical calculation of hydroelastic behaviour of pontoon type VLFS in waves," In: *Proc. of 17th Int. Conf. on Offshore Mechanics and Artic Engrg.*, ASME, OMAE98-4333.

Ohmatsu, S. (1998b). "Numerical calculation of hydroelastic behaviour of VLFS in time domain," In: *Proc. 2nd Int. Conf. Hydroelastic in Marine Tech.*, Fukuoka, 89-97.

Ohmatsu, S., Kato, S., Namba, Y., Maeda, K., Kobayashi, M. and Nakagawa, H. (2001). "Study on floating breakwater for eco-float mooring system," In: *Proc. of 20th OMAE Conf.*, OMAE01-5012, Rio de Janeiro, Brazil.

Ohta, H., Torii, T., Hayashi, N., Watanabe, E., Utsunomiya, Y., Sekita, K. and Sunahara, H. (1999). "Effect of attachment of a horizontal/vertical plate on the wave response of a VLFS," *Proceedings of the 3rd International Workshop on Very Large Floating Structures,* Vol. 1, pp. 265-274.

Ookubo, H., Yamane, M. and Torii, T. (2002). "A study on the mooring of large floating structures inside a reef," In: *Proc. of the 12th Int. Offshore and Polar Engrg. Conf.,* 229-234, Kitakyushu, Japan, 26-31 May 2002.

Seto, H. and Ochi, M. (1998). "A hybrid element approach to hydroelastic behaviour of a very large floating structure in regular waves," In: M. Kashiwagi et al. (Eds)., In: *Proc. of the 2nd Hydroelasticity in Marine Technology,* Kyushu University, Fukuoka, Japan, December 1-3: 185-193.

Shimada, K. and Miyajima, S. (2002). "Comparison of mooring simulations and measurement for Mega-Float Phase II model," In: *Proc. of the 12th Int. Offshore and Polar Engrg. Conf.,* 261-266, Kitakyushu, Japan, 26-31 May 2002.

Shiraishi, S., Iijima, K., Yoneyama, H. and Harasaki, K. (2002). "Elastic response of a very large floating structure in waves moored inside a coastal reef," In: *Proc. of the 12th Int. Offshore and Polar Engrg. Conf.,* 327-334, Kitakyushu, Japan, 26-31 May 2002.

Study Group of World Cities (1988). Seto Bridges Memorial Museum, Kurashiki City, Japan Photo Company (in Japanese).

Takahashi, Y. (2003). "Application and advantages of ocean floating structure technology," *Proceedings of the International Symposium on Ocean Space Utilization Technology*, NRMI, pp. 427-435, January 28-31, Tokyo, Japan.

Takaki, M. and Gu, X. (1996a). "Motions of a floating elastic plate in waves*," J. Soc. of Naval Arch. of Japan*, **180**:331-339.

Takaki, M. and Gu, X. (1996b). "On motion performance of a huge floating structure in waves," In: Y Watanabe (Ed.), In: *Proc. Int. Workshop on Very Large Floating Structures*, Hayama, Kanagawa, Japan, November 25-28:157-164.

Takaki, M., Imai, Y. and Lee, S.M. (2002). "Reduction effect of a submerged-plate toward wave exciting forces acting on a very large floating structure," In: *Proc. of the 12th Int. Offshore and Polar Engrg. Conf.,* 391-398, Kitakyushu, Japan, 26-31 May 2002.

Taylor, R. (2003). "MOB project summary and technology spin-offs," *Proceedings of the International Symposium on Ocean Space Utilization Technology*, NRMI, pp. 29-36, January 28-31, Tokyo, Japan.

Torsethaugen, K. (1996). 'Model for a Double Peaked Wave Spectrum', SINTEF Civil and Environmental Engineering, Rep. No., STF22 A96204, Trondheim.

Utsunomiya, T., Watanabe, E. and Eatock Taylor, R. (1998). "Wave response analysis of a box-like VLFS close to a breakwater," *17th Int. Conf. on Offshore Mechanics and Artic Engineering*, OMAE98-4331, 1-8.

Utsunomiya, T., Watanabe, E. and Nishimura, N. (2001a). "Fast multipole method for hydrodynamic analysis of very large floating structures," *Proc. 16th Int. Workshop on Water Waves & Floating Bodies*, 161-164.

Utsunomiya, T., Watanabe, E. and Nishimura, N. (2002b). "Fast multipole algorithm for wave diffraction/radiation problems and its application to VLFS in variable water depth and topography," *20th Int. Conf. on Offshore Mechanics and Artic Engineering*, OMAE01-5202, 1-8.

Utsunomiya, T. and Watanabe, E. (2003). "Wave response analysis of hybrid-type VLFS by accelerated BEM," *Hydroelasticity in Marine Technology*, Oxford, UK, 297-303.

Wang, C.M., Xiang, Y., Utsunomiya, T. and Watanabe, E. (2001). "Evaluation of modal stress resultants in freely vibrating plates," *Int. J. Solids and Struct.*, **38**(36-37):6525-6558.

Wang, S., Ertekin, R.C. and Riggs, H.R. (1997). "Computationally efficient techniques in the hydroelasticity analysis of very large floating structures," *Comput. Struct.*, **62**:603-610.

Watanabe, E. and Utsunomiya, T. (1996). "Transient response analysis of a VLFS at airplane landing," In: Y Watanabe (Ed.), In: *Proc. Int. Workshop on Very Large Floating Structures*, Hayama, Kanagawa, Japan, November 25-28:243-247.

Watanabe, E., Utsunomiya, T. and Tanigaki, S. (1998). "A transient response analysis of a very large floating structure by finite element method," *Structural Engrg/Earthquake Engrg, JSCE*, **15**(2):155s-163s.

Watanabe, E., Maruyama, T., Kawamura, T. and Tanaka, H. (2001). "A new millennium steel pontoon swing arch bridge of Osaka," *International Journal of Steel Structures*, **1**, 55-61.

Watanabe, E. and Utsunomiya, T. (2003). "Analysis and design of floating bridges," *Progress in Structural Engineering and Materials*, **5**, 127-144.

Watanabe, E., Utsunomiya, T., Wang, C.M. and Xiang, Y. (2003). "Hydroelastic analysis of pontoon-type circular VLFS," *Proceedings of the 13th International Offshore and Polar Engineering Conference*, Honolulu, Hawaii, USA, May 25-30, 2003, pp. 93-99.

Watanabe, E., Utsunomiya, T. and Wang, C.M. (2004). "Hydroelastic analysis of pontoon-type VLFS: A literature survey," *Engineering Structures*, **26**(2), 245-256.

Wheeler, J.D. (1970). "Method for calculating forces produced by irregular waves", *Journal of Petroleum Engineering*, 359-367.

Wu, C., Watanabe, E., and Utsunomiya, T. (1995). "An eigenfunction expansion-matching method for analysing the wave-induced responses of an elastic floating plate," *Applied Ocean Research*, **17**, 301-310.

Wu, C., Utsunomiya, T, and Watanabe, E. (1996). "Application of Galerkin's method in wave response analysis of flexible floating plates," In: *Proc. of the 6th Int. Offshore and Polar Engrg,* Los Angeles, USA, pp. 307-314, May 26-31, 1996.

Wu, C., Utsunomiya, T., and Watanabe, E. (1997). "Harmonic wave response analysis of elastic floating plates by modal superposition method," *Structural Engrg./Earthquake Engrg.*, JSCE, **14**(1), 1s-10s.

Yago, K. and Endo, H. (1996). "On the hydroelastic response of box-shaped floating structure with shallow draft," *J. of Soc. of Naval Arch. of Japan*, **180**:341-352 (in Japanese).

Yago, K. (1995). "Forced oscillation test of flexible of flexible floating structure and hydrodynamic pressure distributions," In: *Proc. of the 13th Ocean Engrg Symp.*, Soc. Nav. Arch. Japan, 313-320. (in Japanese).