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COMMITTEE VI.2 VERY LARGE FLOATING STRUCTURES

COMMITTEE MANDATE

Concern for analysis and design procedures developed for very large floating structures whose characteristics, especially size and flexibility, require special considerations in design, analysis, construction, assembly, and operation. Due consideration shall be given to the use of hydroelasticity theory to determine the structural response. Simple and more refined procedures shall be compared, and their applicability and accuracy shall be judged through experimental data when available. Design criteria for safety and serviceability shall be discussed. Recommendations shall be made for standard analysis and design procedures.

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KEYWORDS

Very Large Floating Structure (VLFS), Hydroelasticity, Mega-Float, MOB, Analysis, Design, Failure Mode, Reliability.

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1. INTRODUCTION

Researches on Very Large Floating Structure have made significant progress in the last decade. A Very Large Floating Structure (VLFS) is a unique concept of ocean structures primarily because of its unprecedented length, displacement and associated hydroelastic response, analysis and design. Considering the nature and task of the Special Task Committee on VLFS, review of researches is not limited to the preceding three years but significant earlier contributions are also referenced.

This report first provides a brief overview of VLFS to give the concept for readers new to the subject. History, application and uniqueness with regard to engineering implication are presented. The Mobile Offshore Base (MOB) and Mega-Float, which are typical VLFS projects that have been investigated in detail and are aimed to be realized in the near future, are introduced. Differences of behavior of VLFS from conventional ships and offshore structures are described. The engineering challenges associated with behavior, design procedure, environment, and the structural analysis of VLFS are introduced and compared with conventional ships and offshore structures. A comparative study of hydroelastic analysis tools that were independently developed for MOB and Mega-Float is made in terms of accuracy of global behavior. The effect of structural modeling on the accuracy of stress analysis is also discussed. VLFS entails innovative design methods and procedure. Development of design criteria and design procedures are documented and application of reliability-based approaches are documented and discussed.

The conclusions and recommendations point the way forward for future work.

2. OVERVIEW OF VLFS

Very large floating structures, such as the Mobile Offshore Base (MOB) concept and the Mega-Float concept, may be thought of as potential megaprojects given their anticipated length scales, displacements and estimated costs (i.e., $10^3 - 10^4$ m, $10^6 - 10^7$ tons, and 5B-15B \$US, respectively). These concepts are illustrated in Figures 1 and 2, respectively.

Megaprojects are defined as the largest of their kind in some respect, though this is necessarily both a subjective and temporal definition. Some examples are listed in Table 1. The megaprojects tabulated here are either in construction or completed. They typically involve massive investment of resources. As a result, program managers generally avoid risk so that leading edge technology is not necessarily a characteristic of megaprojects. Megaprojects are also typically designed with flexible capacity to further reduce risk of investment, and their design life spans range anywhere from 10 to 100 years. They are most

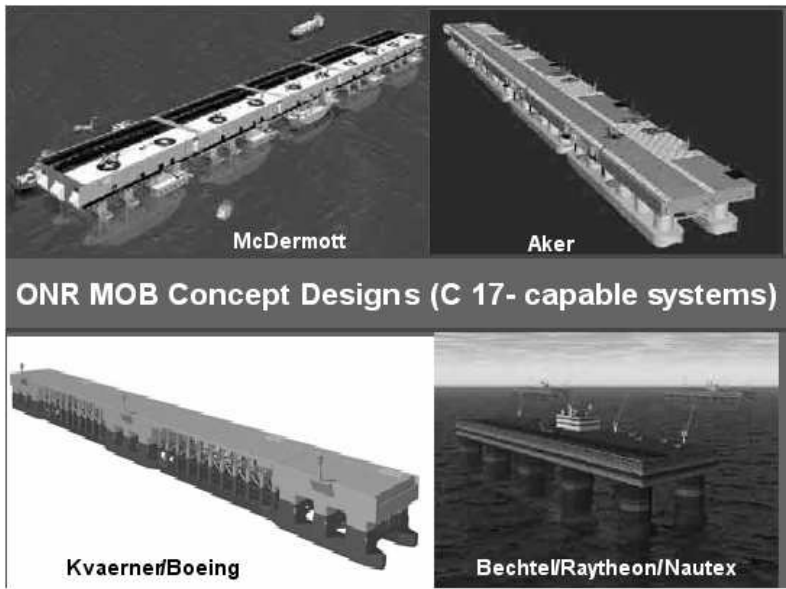


Figure 1 Mobile Offshore Base Concepts (Taylor, 2003)

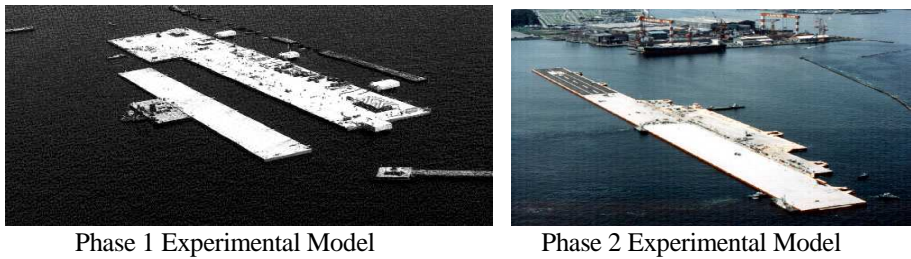


Figure 2 Mega-Float Concepts (TRAM, 1999a-2002)

often government-sponsored projects, but ownership is also sometimes international in scope. Countries in Asia in particular with their high growth economies have recently shown more political will and enthusiasm for megaprojects. It is also clear that megaprojects are temporal; today’s megaprojects are likely to be upstaged given the historical pace of technology.

Table 1 Megaprojects

Country	Name	Cost \$US B	Owner	Ref.
Malaysia	Petronas Twin Towers	1	Private	Sci. Amer. (1997) Dooling (1995)
UK, US and Japan	Fiber-optic Link Around the Globe (FLAG)	1.5	Private	Dooling (1995)
Japan	Akashi Kaikyo Bridge	4	Gov.	Sci. Amer. (1997)
US	Boeing 777 Jetliner Development	4	Private	Dooling (1995)
Switzerland	Swiss Rail Gottard Tunnel	9	Gov.	Dooling (1995)
US	Big Dig	15	Gov.	Wikimedia (2004)
China	Chek Lap Kok Airport	21	Gov.	Sci. Amer. (1997) Dooling (1995)
China	Three Gorges Dam	25	Gov.	Wikimedia (2005)
US, Russia, EU, Japan, Canada	International Space Station (ISS)	100	Gov.	Dooling (1995)

Borrowing from this description of megaprojects, it may be concluded that VLFS are megaprojects characterized by: (1) largest-ever construction of their type, (2) massive costs, labor and resources, (3) technology that is risk averse, (4) modularity or flexible configuration, and (5) long design lives (50-100 years).

Two basic hull types characterize VLFS: (1) a complex of pontoon hulls designed for operation in protected waters, and (2) a complex of semisubmersible hulls designed for operation in deeper water and/or open ocean. The semisubmersible hull may be an array of columns, or it may be a system of columns resting on submerged pontoons. A VLFS is further characterized by hydroelastic behavior. Moreover, due to the unprecedented length scales and the geometrical configurations involved, marine designers must expect to contend with unfamiliar potential global failure modes that are due to coherence issues involving the physical ocean environment. Implicit in all these VLFS characteristics is engineering challenge.

An unresolved issue remains as to whether a VLFS is characterized as a vessel, an offshore facility, or something else. The legal jurisdiction of various national and international regulating bodies, particularly those concerned with environmental compliance regulation, will hinge on such a determination along with the nature of the ownership and the mission of a VLFS.

2.1 *Applications of VLFS*

The first concept of VLFS that appeared in the modern world after the industrial revolution was the Floating Island described by the 19th century French novelist Jules Verne, one of the founders of science fiction. The first VLFS promoted in earnest was the Armstrong Seadrome. It was proposed initially to enable airline routes across the world's oceans (Armstrong, 1924). Its stability was demonstrated in tank tests, and various other related platforms were promoted until Armstrong's death in 1955 (Nelson, 2001).

Interest in utilising the space afforded by the seas surrounding a nation, for purposes other than conventional shipping or ocean resource extraction, has increased as coastal population densities have increased. Until the potential of modern shipbuilding technology became apparent in the 1950s the only manner in which this ocean space could be exploited on a large scale was through land reclamation. This limited such exploitation to shallow regions of the continental shelf. In 1950s architects were drawn to the idea of floating cities and such a concept was demonstrated in part at the Okinawa International Ocean Exhibition in 1975 with a semi-submersible unit of such a city. In a similar manner, a floating airport was proposed for the new Kansai International Airport in 1973. Since the early 1970s the technology for very large floating structures has developed continually, while changing societal needs have resulted in many different applications of the technology being considered.

2.1.1 *Airports*

Proposals to use floating structures for take-off and landing of aircraft were first considered in the 1920s to enable airline routes across the world's oceans. These concepts were investigated more seriously for military applications by the US in the 1940s and a demonstration project was built and tested successfully in 1943.

With vast improvements in technology having been made, a floating airport was proposed for the new Kansai International Airport in Japan in 1973. Although the initial phase was not built as a floating structure, interest in the concept remains strong. This is perhaps the area of VLFS research that has received the most attention, due in large part to the efforts of the Technological Research Association of Mega-Float (TRAM) active in Japan from 1995 to 2001. This association studied the fundamental design and construction needs for a floating airport to be realised. Numerical analysis tools were developed alongside an experimental programme that resulted in the construction of a 1000m technology demonstrator. The regulatory regime and environmental impact of such structures were also considered in depth and design guidelines produced. Although a floating airport has yet to be approved for construction interest remains strong, as evidenced by proposals for the extension to Haneda International Airport in Japan.

2.1.2 *Mobile Offshore Base*

In the post-Cold War era, with smaller-scale conflicts seen in geographically disparate regions, the importance of strategic sealift is heightened. One solution considered is that of a mobile offshore base (MOB), a very large floating structure consisting of several elements

that are maneuvered into position and then joined to form a single base. From this base large fixed-wing aircraft and ocean-going ships may support littoral combat operations from a secure position without reliance on achieving political support in other nations. The US Navy supported a substantial research effort throughout the 1990s to develop the design methodologies suitable for this concept. This work focused on verifying suitable numerical methods and on conducting experimental trials. A similar concept may be envisaged for a mobile emergency rescue base to operate in support of humanitarian relief operations worldwide. A fixed Sea Base Facility (SBF) was also considered as an alternative to land reclamation for relocation of the US Marine Corps Station Futenma offshore Okinawa, Japan.

2.1.3 Offshore port facilities

Just as a floating airport is an attractive proposition in regions where suitable land close to urban centres is limited, so offshore port facilities are being considered. Proposals have been produced for offshore container terminals to service large ocean-going vessels and supply the immediate hinterland with feeder container ships. It may also be beneficial to site termini for potentially hazardous vessels, such as LNG carriers, offshore.

2.1.4 Offshore storage and waste disposal facilities

The potential of a very large floating structure as a storage facility is demonstrated in Japan where two of the nation's ten national oil stockpile bases consist of floating units, constructed in 1988 and 1996. In increasingly densely populated coastal regions, the ability to site storage facilities (of any kind), together with waste processing and treatment plants, out of sight of land is an attractive one and such a facility may also incorporate power generation capability.

2.1.5 Energy islands and food production

In an extension to merely generating power from waste disposal, an offshore facility may be considered for siting a range of sustainable energy technologies. Depending on the prevailing climate such a structure may include some, or all, of wind turbines, wave power generators, tidal current turbines and ocean thermal energy conversion units. Such a structure may also be a natural host to environmental research activities and food production through aquaculture and marine biomass plantations. Variations on this application of very large floating structure technology are being actively considered in South Korea with plans to install a wind and current turbine plant in the Yellow Sea within 5 years, together with proposed installations from Japan, France, United Kingdom and USA.

2.1.6 Habitats

As perhaps the original idea for a very large floating structure, it is perhaps surprising that more plans for offshore floating cities have not been developed over the years, although there are current proposals for offshore sports facilities and theme parks in Japan and South Korea. However, with ever increasing pressure on coastal zones from increasing urban populations and the threat of environmental change, it is likely that such ideas will re-

surface in the coming years, taking advantage of the technology already developed and providing impetus for future research.

2.2 *MOB and Mega-Float*

US research on VLFS was mainly for military application. The US Navy Civil Engineering Corps developed a floating pontoon flight deck in the early 1940's for use by Great Britain. It was constructed of many hinged arrays of pontoons (called strings), measured $552\text{ m} \times 83\text{ m} \times 1.5\text{ m}$ with a 0.5 m draft, and was deployed in protected water. Takeoff and landings were successfully demonstrated in 1943 (Laycock, 1943). In October of 1963 a C-130 transport aircraft successfully conducted 21 landings and takeoffs from the deck of the USS Forrestal aircraft carrier (nominally 305 m in length) using no arresting gear or catapult. However, reported head winds of up to 40 knots aided this effort. The idea was eventually deemed too risky. There was a flurry of activity in the 60's and 70's principally at US Navy laboratories and universities to devise concepts for mobile offshore basing structures and offshore ports and cities. Though constructability needed to be verified, the material of choice was concrete for these semi-submersible type concepts. A military interest in sea basing continued in 1980's as a means to compensate for the loss of overseas bases. Also in the late 1980's and early 1990's, the US National Science Foundation was sponsoring work on VLFS and it was out of this work that the term 'very large floating structure' and the acronym VLFS sprung (VLFS, 1991). DARPA's Maritime Platform Technology Program was executed from 1993 to 1996 with a budget of about \$40M (McAllister, 1996). ONR's MOB Science and Technology Program followed, costing \$35M-\$40M and executed from 1997-2000 (Taylor, 2003). MOB concepts consist of from three to five semi-submersible modules or single base units assembled end-to-end using various inter-module connector configurations, as depicted in Figure 1. The modules are designed to transit, assemble/disassemble and operate in the open ocean. When configured for C-17 transport capability, their flight decks are about 1800 m in length. Much of the technological development culminated in advanced analysis and design methodologies, most of which is reflected in the classification document prepared for MOB (ABS, 1999a, 1999b).

Across the Pacific Ocean, the concept of VLFS for commercial application was evolving in Japan. Shipbuilding technology had attracted the attention of Japanese architects in the late 1950's, and there was movement in architecture and urban design to utilize ocean space and expand human habitation onto the ocean surface (Kikutake, 1994). Aquapolis, a large semisubmersible, portrayed a unit of a floating city concept, and was constructed for the Okinawa International Ocean Exhibition held in 1975. Kansai International Airport was slated for Osaka Bay to reduce noise pollution. In 1973, a floating airport was initially proposed for Phase 1 construction. Though the proposal was ultimately declined in favor of land reclamation, industry had formally commenced research on VLFS technology. Based on lessons learned during the oil crises of 1973 and 1979, Japan later constructed ten national oil stockpiles. Two are floating stockpiles sited off the islands of Kamigoto (in 1988) and Shirashima (in 1996). Each is constructed of several units. Floating stockpile units measure about $400\text{ m} \times 80\text{ m}$ to $100\text{ m} \times 25\text{ m}$ to 27 m . The Technological Research Association of Mega-Float (TRAM) was founded in 1995 and conducted research

on Mega-Float until 2001. Mega-Float is a pontoon type VLFS, which includes both mooring and access systems. It is intended for deployment in protected waters. Fundamental design and construction technologies were developed with a budget of \$172 million (TRAM, 1999a-2002). Insitu experiments were conducted to demonstrate soundness of the technology with a Phase 1 demonstration platform measuring $300\text{ m} \times 60\text{ m} \times 2\text{ m}$, and a Phase 2 demonstration platform measuring $1000\text{ m} \times 60\text{ m}$ to $120\text{ m} \times 3\text{ m}$, as shown in Figure 2. Takeoff and landings were successfully demonstrated in the Phase 2 experiment. The activities of the Association were followed by activities of the Shipbuilding Research Center of Japan that promoted a new floating runway for Haneda International Airport. The Shipbuilders' Association of Japan conceived this concept (Sato, 2003).

Milestones in the development of VLFS are listed in Table 2. Although both the MOB and Mega-Float programs were independently initiated and executed, core science and technology goals were similarly structured and included similar research objectives. This is particularly true with regard to operational requirements and advanced analysis and design methodologies. Mega-Float interests placed added emphasis on construction and environmental impact as a result of having actually installed demonstration platforms.

Table 2 Milestones Developments in VLFS Technology

In the United States	
1924-1955	Armstrong Seadrome and Related Concepts
1942-1944	US Navy Civil Engineering Corps Flight Deck - Project SOCK
1963	C-130 Landing and Takeoff Demonstrations on USS Forrestal
1960's-1970's	Navy Laboratory/University Research
1989-1996	NSF Sponsored Research
1991	First International Workshop of VLFS - University of Hawaii
1993-1996	DARPA Maritime Platform Technology Program
1997-2000	ONR Mobile Offshore Base Science & Technology Program
In Japan	
1950's	Floating City concepts in architecture and urban design
1960's	Puppet drama "Hykkori Hyoutan Jima"
1973-1974	Proposal of Floating Airport for Kansai International Airport Phase 1 construction, semisubmersible type floating structure
1975	Okinawa International Ocean Exhibition - Aquapolis
1988	Kamigoto Oil Stockpile $390\text{m} \times 97\text{m} \times 27.6\text{m} \times 5$ Units
1994	Proposal of Floating Runway for Kansai International Airport Phase 2 construction, pontoon type floating structure
1995	Technological Research Association of Mega-Float (TRAM, 1999a)
1995-1996	TRAM Phase1 Experiment $300\text{m} \times 60\text{m} \times 2\text{m}$ (TRAM, 2001)
1996	Shirashima Oil Stockpile $397\text{m} \times 82\text{m} \times 25.1\text{m} \times 8$ Units
1997-2001	TRAM Phase 2 Experiment $1000\text{m} \times 60\text{-}120\text{m} \times 3\text{m}$ Landing & Takeoff Experiments (TRAM, 2002)
2001-2005	R&D by Shipbuilding Research Center. Proposed Haneda International Airport Runway; Pontoon/Semisub Combination Hull

2.3 *Uniqueness of VLFS*

2.3.1 *Uniqueness*

VLFS concepts such as MOB and Mega-Float are unique ocean structures primarily because of unprecedented lengths and displacements that vary between 10^3 and 10^4 m and between 10^6 and 10^7 tons, respectively. These values are at least one order magnitude longer and twice heavier than existing floating structures. Profound challenges for engineers follow from this. Different from a ship, which has evolved with accumulation of experience and with progress in analysis technology, a VLFS is an unprecedented floating structure not only in terms of size and displacement but also in cost and design life, \$5B to \$15B and 50 to 100 years, respectively. The following considerations characterize further the uniqueness of VLFS.

(1) *Large Size*

VLFS is an unprecedented large and flexible floating structure. Consequently, hydroelastic response becomes dominant and has driven supporting research and development in global analytical methods for VLFS. Global failure modes are key issues. To compensate for lack of experience, a first principles approach has been a constant theme. Evolutionary trial and error type development is not acceptable because of huge socio-economic implications. It also follows that VLFS technology must be averse to risk.

(2) *Environment Condition*

Physical environmental conditions in which a VLFS must operate may not be simply considered spatially uniform in the sense that coherence of environmental conditions such as wind, wave and current must be considered. Conventional methods for describing environmental conditions were in fact insufficient for the design of Mega-Float and MOB. Spatial coherence of waves was a key issue in the MOB project and was extensively investigated (Borgman 1999).

(3) *Design and Analysis*

For purposes of global analysis, a ship is generally considered a rigid structure and its response to wave loading is evaluated by solving a rigid body hydrodynamics problem. A VLFS is considered an elastic structure wherein interaction between elastic response and fluid response is considered. Once external loading is resolved, internal force effects are evaluated in the same manner as for a ship structure but accuracy is largely dependent on modeling internal elastic and inertia forces at the local level. Many analysis tools with different levels of modeling and complexities have been developed for hydroelastic analysis in MOB project, Mega-Float project and related researches. Innovative design methodology is desired in the presence of uncertainty and risk engendered by VLFS. It is therefore considered reasonable to pursue a probabilistic and risk based design approach. This approach is adopted in the safety guideline of MOB and the safety guideline of Mega-Float (ABS 1999b; TRAM 1999b).

(4) *Connection at Sea*

A VLFS is assembled or constructed by joining base units either with flexible or rigid connections. Mega-Float was constructed from many pontoon base units welded together in largely protected waters at sea. Control of thermal deformation and alignment of units were key issues in the construction (Yamashita et al., 2003). In contrast to permanently connecting base units with Mega-Float, base unit modules of MOB are connected so as to provide for assembly/disassembly at sea. Strength and fatigue of the intermodule connectors are key issues in the design of MOB (Ramsamooj and Shugar, 2002).

(5) Positioning

Station keeping of MOB is accomplished by DPS, and positioning of Mega-Float is accomplished with dolphin moorings. Reliability of the station keeping system is crucial to the operational availability of MOB, and to the prevention of drifting, which is an especially important global failure mode for Mega-Float. Mega-Float mooring capacity and breakwater capacity are design tradeoffs, and reliability with respect to seismic effects was extensively investigated (TRAM 1998).

(6) Design Life and Other Unique Criteria

Design criteria follow from functionality. Design life is typically 50 years and 100 years for MOB and Mega-Float, respectively. These criteria are substantially greater than those for conventional ships and offshore structures. For example, fatigue design criteria including corrosion-fatigue are much more onerous. Relative rotation of the hinges in intermodule connectors in the MOB flight deck is limited to just a few degrees, but otherwise depends on module length and operational sea state. When employed as an airfield, Mega-Float's runway must be equivalent with and conform to codes for land-based runways. One of the most difficult of these criteria is that the minimum radius of curvature for the runway is 30,000 m as reported by Sato (2003). Special criteria are indicated not only for structural design but also for inspection and maintenance, bearing in mind that normal dry-docking for maintenance and repair is generally not possible.

2.3.2 Difference between conventional ship and offshore structures and VLFS

(1) Behavior

Different from ships and floating offshore structures, elastic response is dominant for VLFS. A comparison of static response under a concentrated load is illustrated schematically in Figure 3. As a rational measure to distinguish VLFS from the conventional ship and floating offshore structures in terms of global response, a *characteristic length* λ_c , Eq. (1), has been proposed by Suzuki and Yoshida (1996).

$$\lambda_c = 2\pi \left(\frac{EI}{k_c} \right)^{\frac{1}{4}} \quad (1)$$

λ_c is analytically derived from a uniform beam model on an elastic foundation; EI is the bending stiffness of beam and k_c the spring constant of hydrostatic restoring force.

λ_c corresponds to the length of locally deflected region by a static concentrated load, as shown in Figure 3. This indicates that the influence of an applied load on the elastic deformation is limited within the region of the length λ_c . Accordingly, if the length of structure is smaller than the characteristic length, the response is dominated by rigid-body motions, whereas if it is larger than the characteristic length, as typically in VLFS, the response is dominated by elastic deformations. The relationship between the wavelength and the characteristic length is another important factor on the global response of floating structures. If the wavelength is smaller than the characteristic length, the wave exciting forces alternates in the range of the length λ_c and the load effect cancels each other, resulting in smaller global response, whereas if it is larger than the characteristic length, the global response becomes significant. These characteristics of the global response with respect to the characteristic length are summarised in the map of Figure 4.

In summary, VLFS can be characterized by its huge structural size compared not only to the wave length but also to the characteristic length λ_c , Eq. (1), and this can be regarded as a definition of VLFS in a mechanical sense.

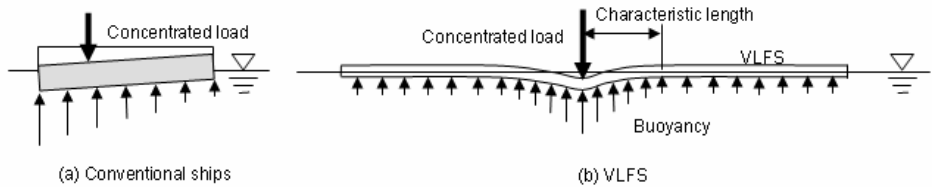


Figure 3 Global response under a static load

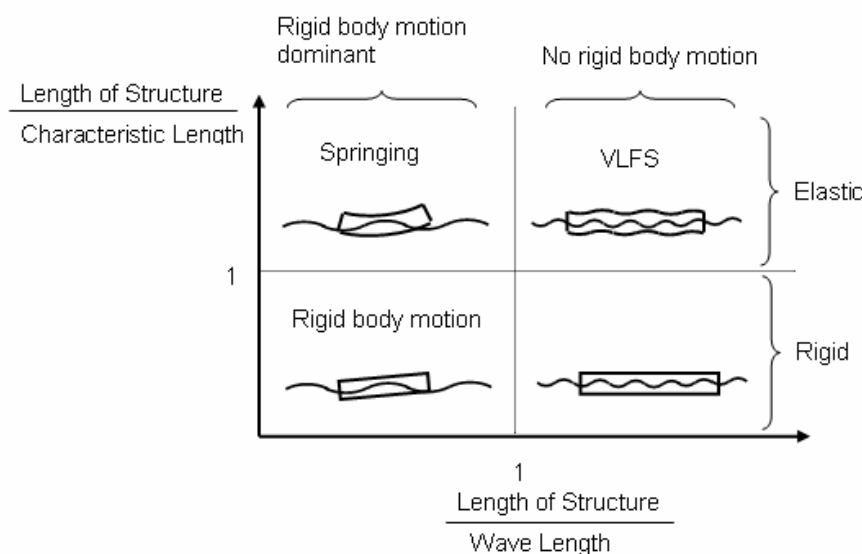


Figure 4 Mapping of global response of floating structures

(2) Design Procedure

In the structural design of conventional ships and floating offshore structures, the external load and major load effects, such as cross sectional forces, are determined from the rigid-body motions. The dimensions of structural members and arrangement are subsequently determined so that the structure has sufficient strength and stiffness against the given loads and load effects.

On the other hand, in VLFS, the global responses including the deflection, load and load effects, are determined by elastic responses. Structural stiffness is therefore a governing parameter for the design of VLFS. The relationship between structural stiffness and global elastic responses is generally complex as a result of fluid-structure interaction effects. For instance, an increase in structural stiffness can lead to an increase in cross-sectional forces and stresses, and hydroelastic response is again affected by the change. Accordingly, hydroelastic response analysis must be performed at every structural design stage in order to consider the effects of design changes on structural stiffness and responses.

Figure 5 shows a typical design flow proposed for VLFS which considers the characteristics of VLFS and developed from the design practices of Mega-Float (SNAJ, 2004). As shown, the design flow can be divided into three basic stages.

During the first stage, a relatively simple method of hydroelastic response analysis is employed, and the global stiffness and the corresponding basic design variables, such as structural depth, primary-members' arrangement, and size are determined. The characteristic length and frequency derived by Suzuki and Yoshida (1996) are referenced during this process. The hydroelastic response analyses that assume the uniform rectangular plate model (Kashiwagi, 1996; Ohmatsu, 1997; Ertekin and Kim, 1999; Sun et al, 2002a ; Song et al, 2002b) are generally employed. A combination of a plate FE model and a modal approach (Okada et al, 1999) is also applied when a more refined structural modeling (e.g., a variable flexural stiffness) is needed. Other factors may also be considered at this stage, (Song et al, 2005). These factors are, but not limited to, the variation of water depth, structural and wave nonlinearities (Sun et al, 2003a; 2003b; Chen et al, 2003a, 2003b, 2003c).

During the second stage, detailed design for actual structural configurations that have variable structural depth, variable planar shape, opening in bulkheads for the usage of internal space, and so on is performed. The 3D detail method of hydroelastic response analysis, developed by Seto et al (2003), is generally applied at this stage. The modeling of fluid domain implemented in this method can address an uneven body boundary, a variable sea depth, and the presence of breakwaters and seashores. From the results of global response analysis, the local stress response under combined load effects is evaluated using a zooming technique. Through the evaluation of strength and serviceability limit states, both the size and arrangement of structural members are then determined.

It is to be noted in Figure 5 that both the initial (first) and detail (second) design stages have a design loop that includes the hydroelastic global response analysis. This is a typical feature of the structural design procedure of VLFS.

During the third stage of the design flow, the structural safety assessments for the system levels are performed. One safety assessment examines partially damaged conditions and the other investigates system collapse behavior under abnormal load effects. The safety assessment during this stage generally needs a nonlinear progressive collapse analysis.

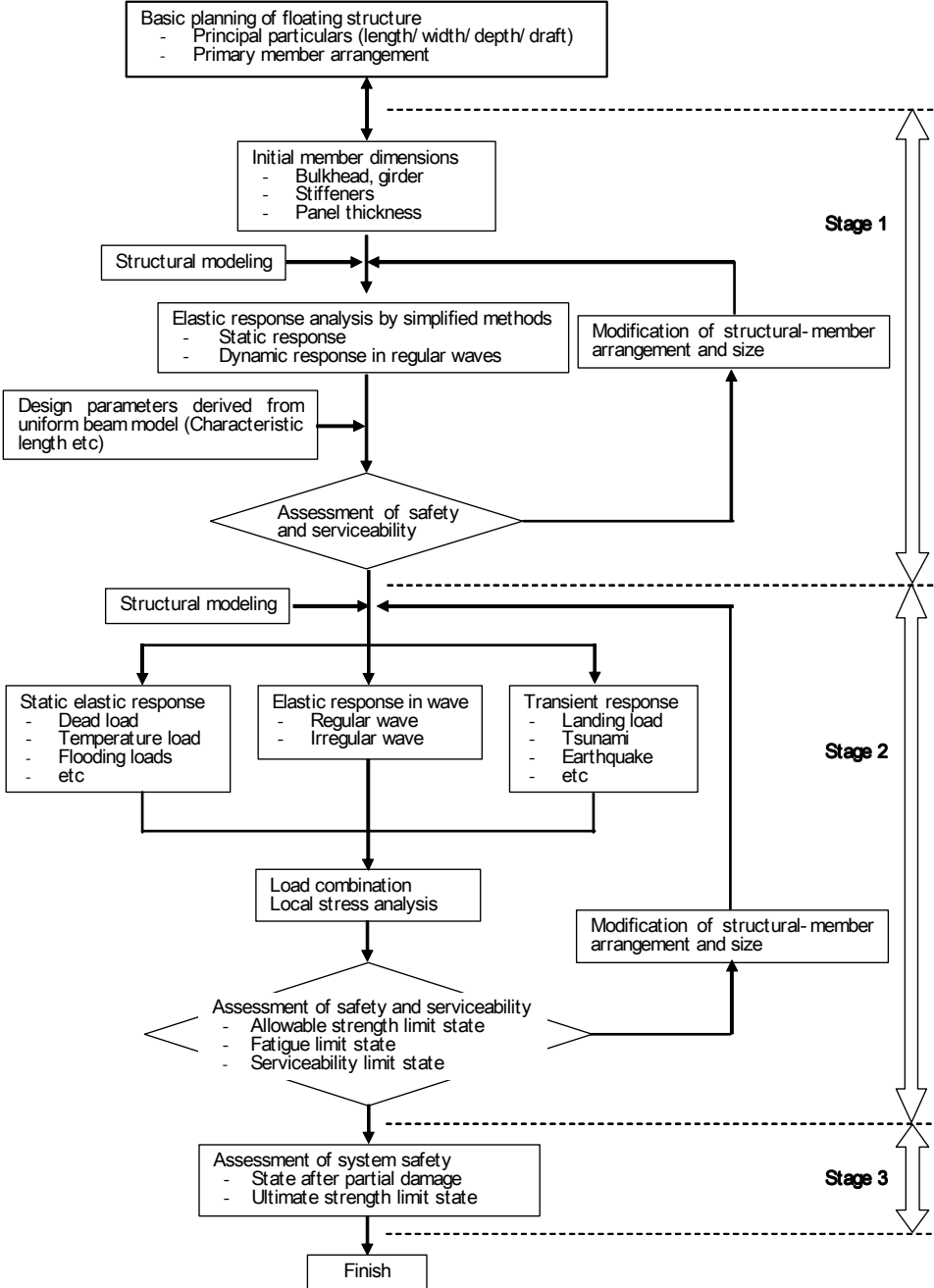


Figure 5 Typical structural design flow of Mega-Float VLFS

3. ENVIRONMENT

The unprecedented size and configuration of VLFS, such as Mega-Float or MOB, are unlike any ship ever built. There is no experience upon which to base estimation of risk. This created a requirement for improved understanding of the ocean environment, and development of new environmental design criteria. Research, investigation, measurement, assessment, and modeling of the marine environment have been conducted.

3.1 *Measurements Supporting Environmental Compliance*

Murakami (1996) described environmental parameters to be investigated to estimate the environmental impact of VLFS. Takata (1996) measured several kinds of data, such as current flow pattern, water quality, bottom material and ecological system data before and after installation to conduct an environmental assessment. A checklist was presented by Champ et al (1998) for assessing the environmental risks of a VLFS that encompassed the design and operation phases. A numerical simulation of the tidal flow and ecosystem in the sea around a Mega-Float was presented by Kyojuka et al (1999) and Nakagawa et al (1999). The model consists of two parts, a hydrodynamic model and a marine ecosystem. Tidal currents, water temperature, salinity and water density are calculated in a bay with/without a Mega-Float using a hydrodynamic model. A marine ecosystem model including nutrients, phytoplankton, zooplankton and organic matter was developed. Kobayashi and Sato (1999) outlined the insitu experiment and the measurement system configuration for 1000m Mega-Float in Tokyo Bay. Wind, wave, and other measurement results were also briefly described. Fujino et al (2001) investigated the marine environment surrounding the VLFS model. Continuous monitoring of the water column for temperature, salinity, dissolved oxygen and chlorophyll-a profiles, etc., were conducted at several fixed stations below and around the floating structure. Irie et al (1999) made an observation of vertical heat transfer through a Mega-Float model in Tokyo bay. Recordings of temperature, humidity and heat flux on the pontoon-type floating structure were conducted in summer and winter. Kokubun et al (2000) used an experimental study to predict the pressure distribution of underwater acoustics around the VLFS.

3.2 *Environment Specification and Typhoon Database Supporting Structural Design*

MOB is intended for wide-ranging operation in the open ocean while Mega-Float is largely aimed at site-specific operation in protected waters. However, the physical environmental specifications developed for MOB are applicable to any VLFS intended for open ocean waters. It is comprised of an environmental specification and environmental effects emphasizing wave coherence as follows:

3.2.1 *Physical environmental specification*

A realistic metocean specification for wind, wave, and current was developed for inclusion in the MOB Preliminary Design Guide. The specification package consists of two data sets. The first set is a comprehensive report summarizing all existing theoretical and measured information regarding phenomena such as wind gusting, wave spatial spreading, joint

distributions of parameters such as significant wave period and height, and internal wave and soliton models. The second set comprises two representative hindcast databases of vector descriptions of wind, wave, and current: one for long-term, and one for short-term within extratropical storms. The first is an interactive database of joint/wave/current hindcast descriptors at 23 sites, averaged over 6 hours/15 mile intervals, for 20 years. The second is hindcast data for 25 large Northwest pacific typhoons simulated over a much finer 1 hour/1 mile moving grid (Pawsey and Manetis, 1999).

3.2.2 Wave coherence measurement

The spatial coherence of ocean wave crest lengths at scales up to 2 km was investigated. This information is necessary for accurate numerical simulations of hydroelastic behavior for very long platforms. The sponsored studies had two other complementary thrusts but a “quick-look” study was most forthcoming. It quantified wave coherence based on direct analysis of available data. The emphasis was on basic information using “reasonable” criteria and analysis techniques. A key advancement was a Scanning Radar Altimeter (SRA) data set from NASA of instantaneous surface measurements in seven of eight octants in Hurricane Bonnie wave field. It constitutes the first ever measurement of a complete representative wave field in a tropical storm. All seven panels shown in Figure 6 measure approximately 6 km by 1 km. The significance of this set to VLFS was that the crest in the largest wave had a uniform height of 18 m and a straight length of at least 1.5 km. The expectation was that a hurricane wave field would be short-crested due to the continually changing direction of the wind field. The finding of such a long, coherent crest length reinforced the fact that VLFS designers required more information on large-scale spatial wave characteristics (Borgman et al, 1999).

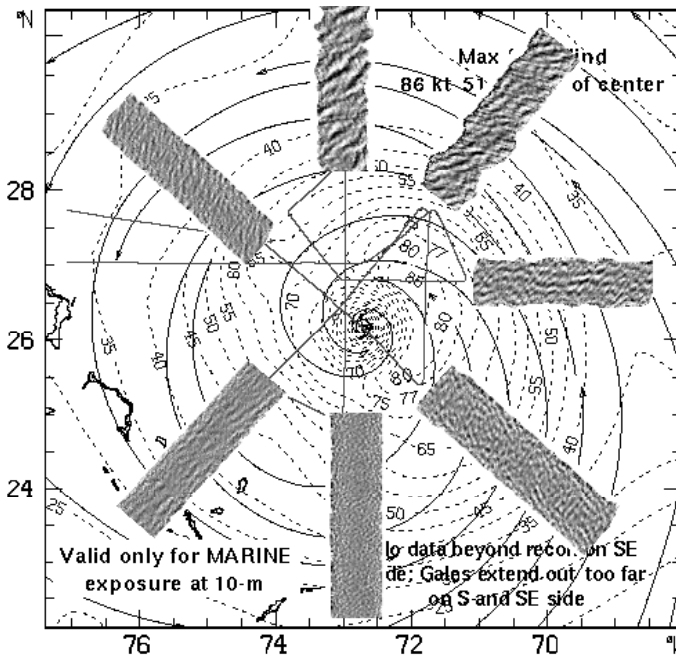


Figure 6 Storm Waves Topography in Hurricane Bonnie

4. STRENGTH AND FAILURE BEHAVIOR

4.1 *Components*

In order to ensure the structural integrity and stability of VLFS, failure modes and associated design limit states are to be adequately identified from both component and system levels. Table 3 summarizes the limit states defined in the safety guideline of MOB (ABS 1999b) and that of Mega-Float (TRAM 1999b). There exist slight differences in the terminology of limit states between MOB and Mega-Float, but the basic philosophy and definitions are almost the same.

The fatigue and strength limit states in the MOB definitions, Table 3, are essentially component-level limit states (element/assembly or subsystem level), while the last one pertains to the entire system. The component-level limit states are those normally checked in the design of large steel-plated structures, and thus the existing criteria for yielding, buckling and fatigue strengths can be generally applied. Some specific features in relation to the component design of VLFS are described herein.

Inter-module connectors are a critical component in the MOB design. An order magnitude increase in transmitted forces beyond those for FPSO connectors can be anticipated for MOB inter-module connectors. Riggs et al. (1999) and Weybrant, et al. (1999) showed clearly that MOB connector forces are dependent on connector stiffness, and that connector stiffness must be tuned with respect to the natural frequencies of hydroelastically behaving VLFS. The fracture of inter-module connectors can immediately lead to the catastrophic failure of a global system, and thus the precise prediction of fatigue life is one of the key issues to realize the MOB. Ramsamooj and Shugar (2001 and 2002) studied both the fatigue life and its reliability for a rigid MOB connector concept. It is believed that fatigue design of MOB connectors is feasible based on fracture mechanic approach, but the reliability level required for classification will not be satisfied unless hot spot stress levels are reduced to about 135 MPa, at least for a rigid connector design. Further, corrosion-fatigue effects cannot be included with confidence in the absence of experimentally validated corresponding fatigue crack growth rate models. Articulated connector concepts are subjected to a unique form of cyclic loading that involves very large forces spread over relatively small contact/impact bearing surfaces. Selection of specially designed sleeve bearings lined with high strength polymers has been recommended for articulated connector components (Ferguson and Patterson 1999).

Mega-Float type VLFS is characterized by its thin mat-like configuration. Unlike the ships that can be regarded as a beam, the deck and bottom panels in Mega-Float are generally subjected to combined biaxial and shear loads of similar order of magnitude. When a longitudinal framing system is employed, the loading conditions on deck and bottom panels are severer in the transverse direction than in the longitudinal direction in general as reported by Fujikubo (2003). Because of a relatively very small depth of the structure, shear strength of bulkheads is to be carefully examined, particularly when a large opening for the usage of the internal space of structure is present in the bulkheads. In this case, shear bending effects in the deck and bottom girders may become significant (Inoue, 2003).

4.2 *System*

Two types of system-level limit state checks are undertaken for VLFS. One is a progressive collapse analysis of an intact structural system under abnormal load effects and the other a residual strength analysis of a structural system in a damaged condition. From the novel nature of VLFS, these system-level checks require an identification of failure scenarios and an evaluation of associated risks, which can give a quantitative measure of the safety.

Typical hazards that may cause substantial damage to MOB include weapons effects, explosions, terrorism and so on. The semi-submersible form, typical for MOB units, was developed for offshore exploitation, but the robustness of this type of structure against the above-mentioned hazards was never addressed. The basic procedures of system-level limit state checks for MOB are described in ABS (1999a).

Mega-Float must be safeguarded against catastrophic failure, such as sinking, drifting, and catastrophic collapse of the floating structure. Major global failure modes of Mega-Float were investigated. The model employed in the studies had an assumed deck area of 500 ha and was moored by more than 30 dolphins. Fujikubo et al (2005) investigated the progressive collapse behavior of the floating structure under extreme wave conditions and demonstrated little possibility of the progressive collapse in the short crested irregular waves. Kato et al. (2002) performed a quantitative risk analysis of multiple mooring dolphins for a pontoon-type VLFS. Time domain numerical simulations of the VLFS-dolphin system in short crested irregular waves were performed varying winds and currents and taking non-linear characteristics of fenders and dolphins into account. The result indicated the probability of drifting of a VLFS moored e.g. by 50 dolphin units was 10^{-6} . The optimum number of dolphins was also discussed based on the expected total life-time cost. Regarding the damaged condition, the damage due to ship collisions was analyzed. The effects of compartment size on the damage extent and the residual strength after the damage and flooding were investigated. Based on the results, the requirements for compartmental division were determined. The effect of airplane crash onto the VLFS was also investigated.

Technical Guideline of Mega-Float (TRAM 1999b) requires an overall safety evaluation of a whole Mega-Float system that consists of a floating structure, a topside facility, a mooring facility, a wave control facility (if required) and an access from shore. Its objective is to extract the most probable worst failure scenario and examine if the Mega-Float satisfies the corresponding acceptable risk levels as a whole system. Fujikubo et al (2003) performed a structural safety assessment of a pontoon-type VLFS in extreme waves with consideration of the damage to the breakwater and showed that the probability of failure satisfied a target safety level.

Table 3 Limit States for MOB and Mega-Float

Limit State			Definition
MOB		Mega-Float	
Serviceability/operability		Serviceability	Disruption of normal use due to excessive deformation, motion or vibration
Fatigue		Fatigue	Critical level of cumulative fatigue damage or critical crack size
Strength		Allowable Strength	Failure of structural element (e.g. panel or stiffener), assembly (e.g. stiffened panel or bulkhead) and subsystem (e.g. deck or column) including inter-module connector
Global failure/ Survivability	(i) Progressive collapse	Ultimate Strength	Loss or failure of entire structure (e.g. capsizing, sinking, drifting, global collapse) (i) as a result of progressive collapse, or (ii) in damaged condition
	(ii) Damaged condition	State after Partial Damage	

5. ANALYSIS

5.1 Methodology

It is one of the essential works in the design stage to estimate dynamic responses of VLFSs in waves. Structural safety and serviceability can be guaranteed by estimating levels of motions, deformations and stresses in VLFSs in waves. In design of floating structures, linearity is usually assumed for the response, although ocean waves are to some extent nonlinear. And responses of VLFSs are also estimated by linear hydroelastic analysis for regular waves and those superposition. In other words, linear structural responses and linear ocean waves are assumed for the estimation in design stage. VLFSs are horizontally so large that dynamical elastic deformations are dominant compared with rigid motions. Interaction between hydrodynamic pressure and elastic deformation are essential for the dynamic response. That is the reason why hydroelastic numerical analysis is needed for structural design of VLFSs.

In this section, various kinds of numerical analysis methods for hydroelastic responses of pontoon-type VLFSs shown in Figure 7 are described. Discussion is limited to frequency

domain analysis of hydroelastic response analysis of VLFSS. Time domain analysis, or transient response due to impulsive load is not discussed here.

Table 4 Analysis levels of hydro-elastic analysis for pontoon type VLFSS

Dim. of Structure and Fluid	Structure model	Fluid model (Potential fluid)	Note
1D-2D	Beam model	2-D fluid	Structural response is overestimated because radiation of waves from side edges is neglected. (Yamashita)
1.5D-2.5D	Semi-infinite plate without side edges	Oblique wave	Incident angle of wave can be considered. Excitation on side edges cannot be considered. (Okusu, Tsubogo)
2D-3D	Plate with 4 free edges (isotropic, orthotropic)	3-D fluid (shallow draft, zero draft)	Useful for initial design stage. Non-uniformity of structure cannot be considered enough. (Yago, Yasuzawa, Omatsu, Okada)
3D-3D	Plate + grillage, Shell structure	3-D fluid (non-uniform draft)	Detailed hydroelastic analysis. Non-uniformity or structural discontinuity can be considered. (Seto)

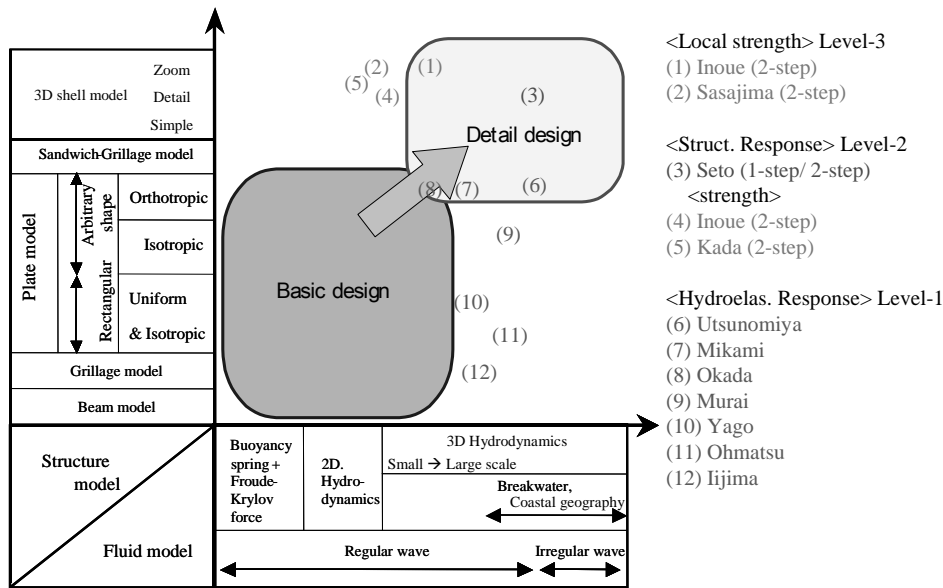


Figure 7 Level of hydroelastic & structural (response) analyses of Mega-Float

5.1.1 Analysis levels for hydro-elastic analysis for pontoon type VLFS

Many numerical tools have been developed for hydroelastic response analysis of pontoon type VLFSs in waves. In case of modeling the hydroelastic problem, we can select the spatial dimension for the structure and surrounding fluid as shown in Table 4. The simplest one is modeled by 1-D structure model and 2-D fluid domain. The most detailed analysis is modeled by 3-D structure model and 3-D fluid model. But hydroelastic analysis is generally time-consuming calculation, because it must be solved considering interaction between fluid and structure simultaneously. Various options for modeling for numerical analysis of hydroelastic response of VLFS due to waves in frequency domain are shown in the followings.

5.1.2 Modeling and formulations for structure - fluid coupled problem

Even in 3D models (2D-3D, 3D-3D in Table 4), various options for fluid and structure regions are given as follows.

(1) Modeling of fluid region

Ocean waves or incident waves are expressed by superposition of regular waves. Dynamically deforming VLFSs produce diffraction and radiation waves. In the analysis surrounding water is modeled as infinite, irrotational, and incompressible ideal fluid without viscosity. Therefore potential theory can be applied for the hydro-dynamics. Dynamic pressure acting on the outer shell of the structure is composed of quasi-static pressure due to change of draft and dynamic pressure due to waves. The quasi-static pressure is proportional to vertical deflection which can be formulated by buoyancy spring. Dynamic pressure due to waves becomes inertia force by added mass of water and wave-making damping.

Firstly numerical methods for fluid region or surrounding sea water region are introduced. Hydrodynamic problem to be treated is the water wave problem of the sea with finite water depth and horizontal infinity. The fluid can be assumed as a potential fluid. Then governing equation becomes Laplace equation and some boundary conditions. This problem is solved by using Green function method, domain decomposition method, or combined method. For 3-D fluid region, the following methods can be used.

1. 3-D Boundary Element Method
 - 1) Direct method
 - 2) Indirect method or distributed source method
2. Domain Decomposition Method (Method of Eigen function expansion)

Fluid region is treated as open sea or protected sea with breakwater or harbor. Direct BEM in which velocity potential and normal derivative of velocity potential at boundary nodes are unknown variables (Yasuzawa, 1997)). Okada (1999) used indirect BEM. Ohmatsu (2000) used a hybrid BEM of domain decomposition type. Seto(1998) used a hybrid finite/infinite element method of domain decomposition type. Two fictitious boundaries were introduced there: an outer boundary which includes all the irregularities inside, and an

inner boundary which is just below the VLFS. Then, the overall domain is divided into three domains, where an outer infinite domain which is discretized by hybrid infinite elements, and the inner domains which are discretized by hybrid finite elements, where each of them is represented by planar finite elements and the orthogonal eigen-function expansions in the vertical direction. Each domain solution has to be matched each other on the respective interfaces so that the corresponding continuity conditions may be satisfied.

As for the treatment of non-uniform water depth, Seto(2003) and Utsunomiya(2001) demonstrated analysis for distributed water depth like step functions or uneven linearly slanted seabed under the structure.

(2) Modeling of floating structure

VLFSs are generally steel or concrete structures. They are modeled as linear elastic plate and shell structures to solve structural response. Generally finite element methods (FEMs) for the structural analysis can be applied for spatial discretization. Therefore the governing equation is generally expressed like excited structural vibration problems as follows.

$$\mathbf{M}_s \ddot{\mathbf{x}} + \mathbf{C}_s \dot{\mathbf{x}} + \mathbf{K}_s \mathbf{x} = \mathbf{f} \quad (2)$$

where \mathbf{M}_s , \mathbf{C}_s , and \mathbf{K}_s are structural mass, damping, and stiffness matrices respectively. And \mathbf{x} is the nodal displacement vector. \mathbf{f} is the equivalent nodal force vector due to dynamic fluid pressure. Structural damping is usually neglected in discussing hydroelasticity due to wave excitation because its effect is much lower than that of wave-making damping in the significant wave period. And when harmonic motion, $\mathbf{x} = \mathbf{x}_0 \exp(-j\omega t)$, due to a certain regular wave with angular frequency ω is assumed, Eq.2 becomes,

$$(-\omega^2 \mathbf{M}_s + \mathbf{K}_s) \mathbf{x}_0 = \mathbf{f} \quad (3)$$

For modeling the structure, the following models can be used.

1. Plate model (isotropic or orthotropic)
2. Sandwich-grillage model
3. 3-D structural model

Plate model is based on plate bending theory. Sandwich-grillage model is made of grillage with upper and bottom plates. Upper and bottom plates are composed of membrane elements. Grillage is made of beam elements. 3-D structural model is the most detailed model which can express realistic distribution of global bending, shear, and torsional stiffness and mass. This detailed model may show more exact estimation than other models. But detailed analysis needs more information of scantlings and a lot of time for modeling and calculation. Therefore grillage model or plate model should be used at initial design stage in which optimal design or design loop is needed, detailed model may be used for second or third stage.

(3) *Direct method and modal method considering fluid–structure coupled problem*

Fluid-structure interaction may be considered by both kinematical and dynamical relations between the above two regions. In case of pontoon type VLFS, the draft is very small compared with the horizontal size. Therefore the interaction is usually considered only on the bottom surface of the structure. And zero draft approximation can be applied for simple formulation approximately.

In considering interaction between fluid and structure parts, coupled problem can be solved by either direct method or modal methods. As pressure acting on the bottom surface is expressed by buoyancy spring and wave forces induced by diffraction and radiation waves, the coupled problem can be expressed as a vibrating structure induced by wave exciting force as follows,

$$((\mathbf{K}_s + \mathbf{K}_w) + \omega^2(\mathbf{M}_s + \mathbf{M}^*))\mathbf{x} = \mathbf{f}^* \quad (4)$$

where \mathbf{K}_w is buoyancy spring due to hydrostatic pressure and \mathbf{M}^* is a complex mass matrix due to added mass of water and wave making damping dependent on ω . And \mathbf{f}^* is an equivalent exciting force vector induced by water waves which is expressed by incident and diffraction wave potentials. This vibration equation is solved direct or by modal analysis.

Direct method is the method to solve Eq.(4) directly. The method is straightforward but must solve a large-scale matrix equation with complex coefficient matrix because significant ocean wavelength are much smaller than structure size. That means a large amount of finite elements are needed for VLFSs. Therefore large memory space and CPU time are demanded for calculation.

On the other hand, dimension to solve can be reduced by modal coordinates. When modal analysis is applied, dry modes obtained by solving eigenvalue problem with being both \mathbf{M}^* and \mathbf{f}^* equal to zero in Eq.(4) is to be used properly by making use of the orthogonality in terms of $(\mathbf{K}_s + \mathbf{K}_w)$ and \mathbf{M}_s matrices. Modal coordinates are used for the formulation in the same way as modal vibration analysis. Generally natural dry modes corresponding to natural frequencies of the specific structure with buoyancy spring obtained from the eigenvalue problem. In modal analysis, D.O.F. can be reduced by modal matrix, as follows.

$$\mathbf{x}_0 = \mathbf{U}\mathbf{y}_0 \quad (5)$$

where \mathbf{U} ($m \times n$) and \mathbf{y}_0 are a modal matrix and modal displacement vector respectively. Generally dimension of \mathbf{y}_0 , n , is selected much smaller than that of \mathbf{x}_0 , m , due to mode truncation.

Velocity potential, gradient of potential, and source vector are reduced by use of the above modal matrix as well. Then final matrix equation is expressed in the form as,

$$(-\omega^2(\overline{\mathbf{M}}_s + \overline{\mathbf{M}}_F) + (\overline{\mathbf{K}}_s + \overline{\mathbf{K}}_w))\mathbf{y}_0 = \overline{\mathbf{f}}_F \quad (6)$$

Generally direct coupling solution of Eq. (4) needs a large memory and computing time to solve, compared with modal analysis Eq. (6). But it is important to judge the effect of mode truncation in case of modal analysis.

(4) Stress estimation

VLFSs are too large to estimate dynamic wave-induced stresses in local structural members with one step calculation. Even in case of ship structural analysis, it is practically impossible to estimate primary stress in structural members directly by one step calculation considering hydroelastic behavior. Generally in direct calculation of ship structural analysis, global response analysis, hull module analysis, and zooming analysis are performed. Therefore it may be said that multi-step calculation is needed for detailed stress estimation in VLFS as well.

Let us show one of the estimation steps based on the approach for ship structure.

1st step:

Global response should be estimated fully considering hydroelasticity. Then we can get stress resultants (bending moments, torsional moment, and shear force), deformation, and hydrodynamic pressure on the bottom surface of VLFS. In this step, VLFSs may be modeled as large plates usually.

2nd step:

Significant structural module is cut out and module analysis is performed using stress resultants and external loading obtained from the 1st step calculation. Here the module is a partial box girder structure composed of stiffened plates and bulkheads.

3rd step:

Zooming analysis at the part of stress concentration may be performed considering welding line configuration for fatigue and crack. Holes in plates are considered in the 2nd or 3rd step.

Inoue's method for estimating stresses in the inner structure around large openings is similar to 1st and 2nd steps above. There is a feature in the paper that a new idea is incorporated to combine coarse mesh for global response and fine mesh for noticed part in the 2nd step calculation.

Kada et al. also proposed a new 2nd step calculation making advantage of the characteristics of hydrodynamic load distribution. The problem to solve is treated as vibration analysis which can be done by commercial FE software like NASTRAN. But the method is not guaranteed for other wave conditions than following waves in the longitudinal direction.

Seto's approach (Seto, 2001) seems to be the most direct one step calculation instead of 1st and 2nd step calculations because

- 1) Distribution of non-uniform vertical inertia force
- 2) Distribution of non-uniform rotary inertia force due to bending deformation
- 3) Distribution of non-uniform vertical shear force and the corresponding deformation
- 4) Distribution of non-uniform torsional rigidity

are directly considered into the analysis.

5.2 *Comparative Study*

5.2.1 *Comparison of 3D hydroelastic analysis tools*

In this section the wave-induced global response, in the form of RAOs, obtained from four computer programs is compared for a benchmark problem. A simplified model, the same for both pontoon-type and semisubmersible-type VLFS, is considered.

The model is a rectangular, pontoon structure 500 m long and 100 m wide. It is 2 m high, the draft is 1 m, and it is in 20 m of seawater. Transverse and longitudinal bulkheads are spaced at 50 m and 20 m, respectively. The top deck, bottom hull, bulkheads, and sides are steel plates with a nominal thickness of 20 mm. To include the additional bending stiffness associated with stiffeners that are not modeled, the plate bending thickness is 150 mm for all plates. The mass density of steel is 7850 kg/m^3 , and the structural mass is based on the nominal steel volume given the 20 mm plate thickness. The modulus of elasticity for steel is 200 GPa. To model the non-structural mass, the top and bottom plates are assigned an additional mass density of $17,131 \text{ kg/m}^3$. As a result, the CG is located at the geometric center of the cross section. Note that all dimensions are midplane dimensions consistent with a shell finite element model of the structure. The density of seawater is 1025 kg/m^3 , and gravitational acceleration is 9.81 m/s^2 . Structural damping is not included in the dynamic analysis. The seawater is assumed unbounded horizontally and the seabed is flat.

The model has been designed to have significant flexible response under waves. Of interest herein is the global response, and in particular displacements and stresses. Results are reported for wave periods between 5 and 30 sec, and wave heading of 0° (head seas). Oblique waves are also of significance, but space limitations do not allow their inclusion here.

The five computer programs that were used to obtain the dynamic response are: HYDRAN (OCI 2005), KU-VLFS (Yasuzawa et al. 1997), MEGA (Seto et al. 2003), VODAC (Iijima et al. 1997) and LGN (Ertekin and Kim 1999). In the first four programs, the fluid model is based on linear, 3-D potential theory. In the last program, the fluid model is based on the linear Green-Naghdi equations for long waves. HYDRAN uses a traditional constant panel Green function formulation for the fluid and a 3-D shell finite element model for the structure. KU-VLFS uses bi-linear direct BEM formulation and an equivalent plate finite element model. MEGA uses a hybrid finite/infinite element fluid model with vertical modal expansion of the wave field and an equivalent plate finite element structural model. VODAC uses a traditional constant panel Green function formulation for the fluid and a 3-D grillage model for the structure. LGN uses the Green-Nagdhi equations in the fluid domain, as mentioned, and a linear Kirchhoff plate model; the governing equations are matched at the juncture boundaries and they are solved by the boundary-integral equation method.

The HYDRAN model used 8,480 quadrilateral fluid panels and 19,280 five-node quadrilateral shell elements; there was a 1-to-1 correspondence between panels and shell elements on the wetted surface. HYDRAN used the first 30 dry normal modes of vibration. The first deformation mode corresponded to vertical bending in the longitudinal direction

and had a natural period of 24.6 sec, while the second bending mode had a period of 8.96 sec and the third bending mode had a period of 4.62 sec. The first bending mode in the transverse direction had a natural period of 1.32 sec.

The KU-VLFS model used 2,000 quadrilateral fluid panels and 2,000 quadrilateral four-node plate elements; there was also 1-to-1 correspondence between fluid element and plate elements on the wetted surface. KU-VLFS uses direct method without using normal modes.

The MEGA model used 2000 orthotropic shell elements for the structure and 28,000 prism elements with 3 terms for the vertical modal expansion in the fluid domain.

The VODAC model used 22,000 quadrilateral fluid panels and 400 beams in a plane grillage model.

The LGN method used 480 line elements along the edges of the plate. No other discretization is necessary because the model uses analytical models for both the fluid and the plate.

The maximum displacements along the centerline induced by a wave with a period of 10 s and an incidence angle of 0° (propagating in the $-x$ direction) is shown in Figure 8. The ‘shape’ is strongly dependent on the wave angle, as can be seen from Figure 9.

These results show generally good comparisons between the programs, especially considering the different numerical models that were used to obtain the results. However, the results in oblique waves for KU-VLFS differ substantially from the other three programs; the reason for this is unclear. However, the bottom two curves in Figure 9 correspond to results based on a plate model for the structure.

The programs also agree well for the RAOs for the vertical displacement at the center and bow of the structure (again along the centerline) in head seas; see Figure 10 and Figure 11.

RAOs of the longitudinal stresses obtained from four of the programs also show good agreement; see Figure 12 and Figure 13. These results are also along the centerline and are for head seas only.

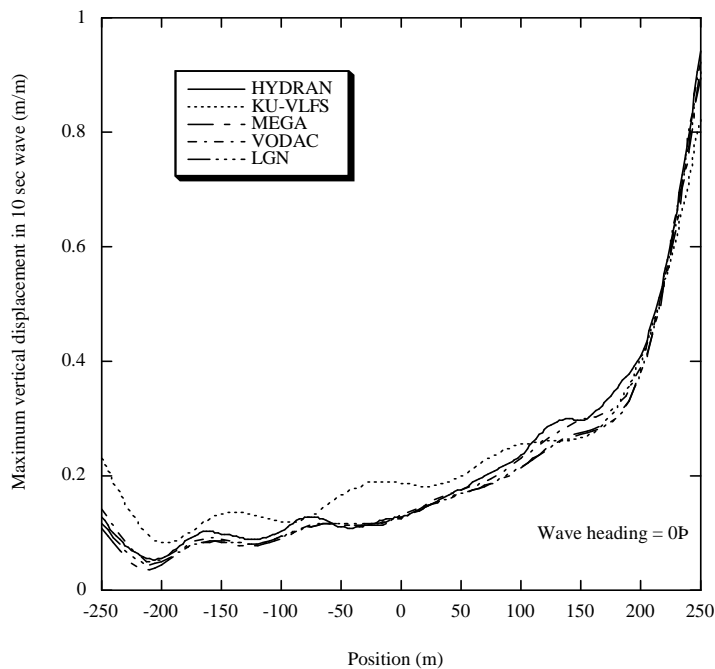


Figure 8 Maximum vertical displacement in 10 s head sea

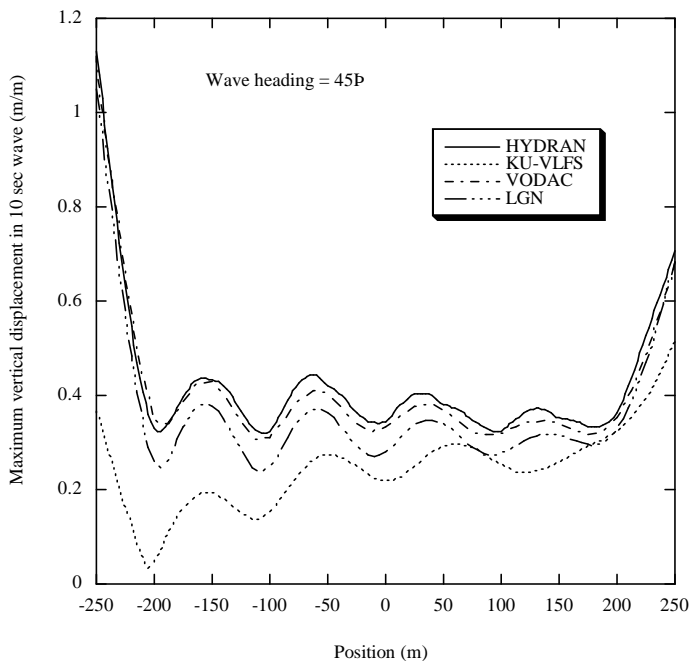


Figure 9 Maximum vertical displacements in 10 s oblique sea

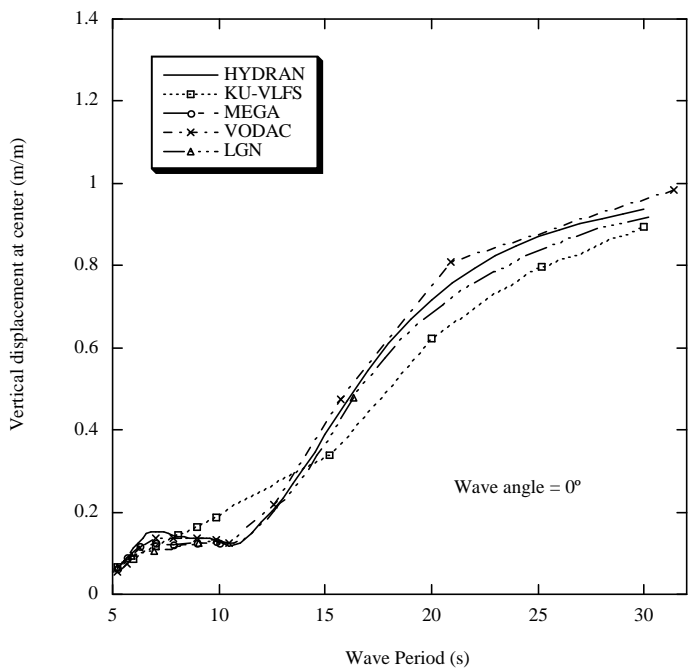


Figure 10 RAO of vertical displacement at center

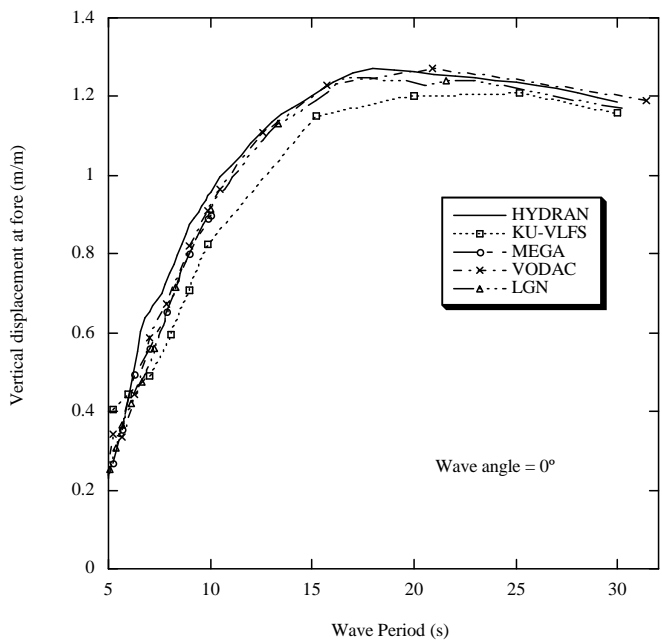


Figure 11 RAO of vertical displacement at bow

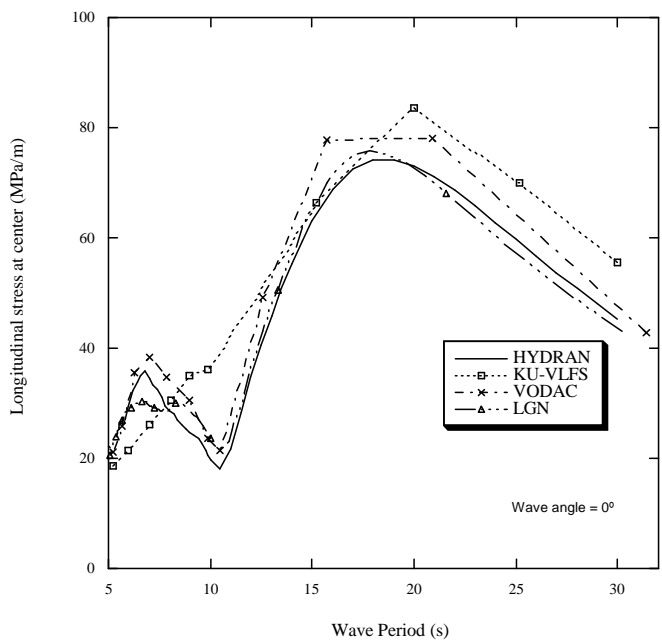


Figure 12 Longitudinal stress at center

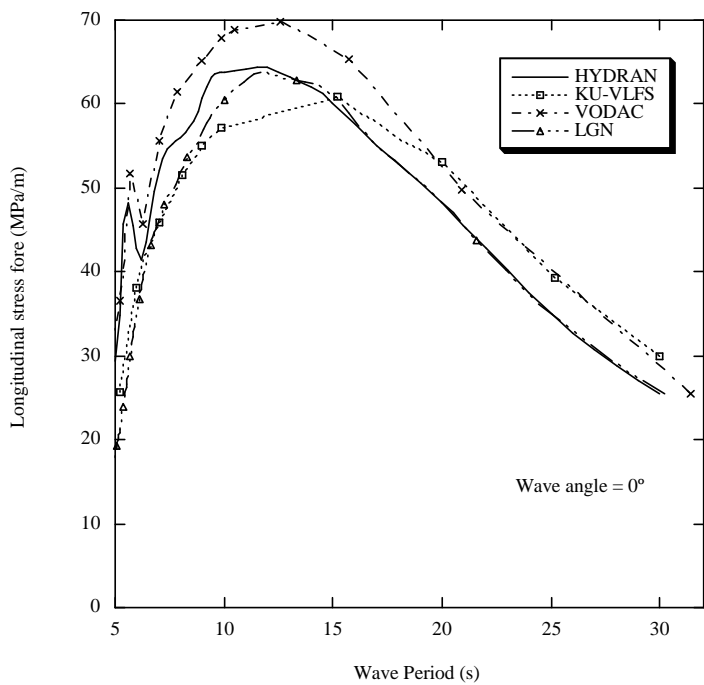


Figure 13 Longitudinal stress 50 m back from bow

5.2.2 *Computation of structural analysis models for pontoon type*

In this section, available stress analysis tools for pontoon-type VLFS are discussed. Then, difference between the computed results for three different structural models of a VLFS is discussed using the same analysis method.

(1) Analysis Tools for Pontoon Type VLFS

The detailed design of VLFS necessitates the following three levels of analyses:

- Level 1: hydroelastic response analysis in the narrow sense, which includes the global flexural vibration of the shell plating of VLFS and the wave loads on it;
- Level 2: structural (response) analysis, which calculates inner forces such as cross-sectional forces and member forces, or stresses in the sub-blocks of the VLFS; and
- Level 3: local stress analysis, which calculates and checks the stress concentration around openings in the structural members.

Because of limited experience and restricted model tests, it is recommended that these have to be done by using not only realistic hydrodynamic and structural models, but also the actual irregular water area and wave conditions as much as possible, although they result in very large-scale computations.

Many simulation methods have been proposed for the hydroelastic analyses of VLFS in the literature. But many of them were limited to simplified level-1 analyses for the initial design of VLFS, which study its global hydroelastic characteristics by using a simple elastic plate floating in the open sea with constant depth. Only a few make possible the practical level-1 analyses for the realistic VLFS in protected sea. Mapping of the methods is shown in Figure 7.

In the Mega-Float project, the level-2 analyses were exclusively done by semi-empirical two-step methods, where the 2nd-step structural analysis for the detailed model were conducted based on the outputs to the simplified equivalent by the 1st-step hydroelastic analyses (Level 1) in Table 5. The “equivalent load” method (Inoue,2001) corresponds to the static finite element analysis of the global model that undergoes the calculated wave loads, and the “equivalent vibration” method (Kada et al.,2003) corresponds to the finite element vibration analysis of the global model with the roughly approximated added mass and wave exciting forces, respectively. Special care should be paid in choosing the equivalent plate (or grillage) model in the 1st-step calculations, how to introduce the 1st-step hydrodynamic outputs rationally into 2nd-step analysis, and how to minimize the resulting unbalanced forces, in order to obtain accurate and reliable results.

An integrated one-step analysis method, MEGA was developed for Level 2 analysis by Seto et al. (1998,2003). They used NASTRAN for the structural discretization and the corresponding eigenmodes computations and made it possible to calculate not only the modal elastic responses of the actual 3D structures caused by regular waves in the open or protected sea with uneven water depth, but also the global structural strength at once. Specifically, they developed a regular BEM-like FEM, termed the hybrid finite/infinite

element method of domain decomposition for fast and accurate computations of very large flow fields of interest. 2D equivalent plate models and 3D-2D hybrid structural models have been examined by MEGA for efficient structural analyses for larger VLFS. Also MEGA has been successfully applied for the detailed design of the actual VLFS such as the Mega-Float Phase-II model 1km long, and the Mega-Float Tokyo Bay model approximately 5km long. The estimated significant value for irregular waves compared satisfactorily with the in situ experimentation data (Miyajima et al.,2003).

The level-3 analysis to Mega-Float was the “direct load” method (Sasajima,1999), which conducted the finite element zooming analysis to local structures subject to the estimated wave loads and sectional forces obtained by the 1st-step hydroelastic computation. Inoue et al. (2003) proposed an improved zooming scheme for local strength estimation based on the equivalent load method. Yet an alternative rational approach has to be formulated for the establishment of the detailed design of VLFS with limited experience (Seto et al.,2004).

(2) Comparison study of structural modeling

Three structural models of a pontoon-type VLFS are compared herein. The analysis code used for this study is MEGA. The model is a very large box-shaped pontoon structure 1,200 m long and 240 m wide. It is 3 m high and the draft is 1 m. In order to conduct the level-2 simulation for the actual 3D structure, transverse and longitudinal bulkheads are spaced 4 m and 12 m, respectively. The top deck and bottom hull are steel plates 15 mm thick, and the bulkheads and sides are steel plates 9 mm thick. The modulus of elasticity for steel is 206 GPa and Poisson’s ratio is 0.3. The open sea with flat bottom 20 m deep is assumed for ease of comparison. The density of seawater is 1025 kg/m³. Results are reported for two wave headings of 0° and 30°, and for two regular wave periods of 5.3 and 9.9 sec, which correspond to the actual short wave conditions to which pontoon-type VLFS becomes realistic and operable, because of the resulting small responses.

Three structural finite element models are used. One is the 3D structural model composed of 16 identical box-shaped units with bulkheads as shown in Figure 14. It is composed of 61,705 nodes and 91,361 shell elements including 12,000 shell elements with average size 4 m × 6 m on the bottom hull. The others are the equivalent orthotropic plate models with and without shear deflection, whose mesh is identical to that on the bottom of the 3D model.

The fluid domain was hierarchically discretized into 75,600 prism elements that included 18,000 elements just below VLFS, and 1,200 hybrid infinite sectorial prism elements, both of them assumed planar finite element subdivision with average size 4 m × 4m, and 3-term vertical orthogonal eigenfunction expansions.

The comparison of the hydroelastic responses of VLFS will be done appropriately by the following steps: first, the comparison of the overall response distribution of VLFS by bird’s-eye view, second, the comparison of spatial distribution of the responses on some specified lines, and then the comparison of RAOs at some specified points on VLFS.

Figure 15 shows a bird’s-eye view of vertical displacement amplitude of a bottom hull of the 1200 m 3D structural model for the wave period $T_w=9.9$ sec and the wave heading

$\beta=0^\circ$. Figure 16 and 17 show the comparison of vertical displacements on the bottom centreline and longitudinal bending stresses near the centreline by three models for the wave period $T_w=5.3$ sec and the wave heading $\beta=0^\circ, 30^\circ$, respectively. For short wave period $T_w=5.3$ sec, the calculated responses are relatively small for $\beta=0^\circ$ and much smaller for $\beta=30^\circ$. For longer wave period $T_w=9.9$ sec, they become larger for head and oblique seas. The responses by 3D model with smaller stiffness agrees well with those by the equivalent plate models, except the responses by equivalent plate model without shear deflection for the short wave period $T_w=5.3$ sec and head sea $\beta=0^\circ$. Here the stiffness of equivalent orthotropic plates is estimated following the procedure to the cross-stiffened plates by Terazawa (1974) and the structural division of the ocean engineering committee of SNAJ(2004).

In Figure 17, $\{\sigma\}_{3D}$ and $\{\sigma\}_{2D}$ are the calculated longitudinal bending stresses on the bottom of the 3D model and the 2D equivalent plate which is as high as the 3D model, respectively. Note that the former is approximated by the stresses of the corresponding cross-stiffened plate model, $\alpha \times \{\sigma\}_{2D}$, where α can be derived approximately by taking the equal curvature of the cross-stiffened and the equivalent plates, and the respective longitudinal bending stress-curvature relations to the bottom plates into consideration.

Conclusion is that the equivalent orthotropic plate model with shear deflection can offer a good estimation for the hydroelastic response analysis for box-shaped structures which have equally arranged longitudinal and transverse identical bulkheads, even in the short wave periods and in head sea condition, though additional tests have to be done for the actual VLFS with complicated structure. It is recommended that the stress estimation has to be done safely by the detailed 3D finite element model, though the bending stresses on the bottom can well be estimated by the conversion of calculated 2D output.

Table 5 Hydroelastic Response Analysis Methods of Pontoon-Type VLFS

Level of Analysis	Authors	Model of structure		Model of fluid	
		Model type	Nonuniformities	Waves & sea	Irregularities
Level- 1 & Level- 2 <Hydroelas. & Structural Response Analysis>	Seto, Ochi, Ohta & Kawakado <1998, 2003>	3D structural model 2D equiv. Models Orthotropic plate Grillage model Sandwich grillage Hybrid model	Assembly of box- shaped units <stepped draft & water plane> Variable mass Variable stiffness	Regular waves Finite water depth Open/ protected sea	Stepped seabed Coast & breakwaters
	<Modal FSI>	<FEM/ NASTRAN>	<quasi- 2D Hybrid FEM + DD>		
	Applied to: 5km class VLFS (Design: the Mega-Float Phase-II, the Haneda expansions)				
	Structural response analysis: 1- step & 2- step analysis (global units/ members, local)				
Level- 1 <Hydroelas. Response Analysis>	Utsunomiya & Watanabe <1998, 2001>	Orthotropic plate Grillage model	Arbitrary hull <arbitrary draft & water plane>	Regular waves Finite water depth Open/ protected sea	Uneven seabed coast & breakwaters
	<Modal FSI>	<Conv. Ritz meth>	<BEM+Fast Multipole Meth. >		
	Applied to: 5km class VLFS				
	Okada, Shibuta, Nagayama & Okamura <1998>	Orthotropic plate	Non- rect. Plate Zero draft Variable mass Variable stiffness	Regular waves Const. wat. depth Open sea	
	<Modal FSI>	<FEM/ NASTRAN>	<BEM+Modal Expansions>		
	Applied to: 1km class VLFS (Initial design: the Mega-Float Phase-II)				
	Mikami, Kobayashi, Shimada, Ohkusu & Kashiwagi <1998>	Orthotropic plate	Non- rect. Plate Zero draft Uniform mass Uniform stiffness	Regular waves Const. wat. depth Open sea	
	<Ritz FSI>	<Conv. Ritz meth>	<B- spline BEM + Galerkin>		
	Applied to: 5km class VLFS				
	Ohmatsu <2000>	Isotropic plate	Rect. Plate Zero draft Uniform mass Uniform stiffness	Regular waves Const. wat. depth Open/ protected sea	Breakwaters
	<Ritz FSI>	<Conv. Ritz meth>	<quasi- 1D Hybrid BEM + DD>		
	Applied to: 5km class VLFS (Initial design: the Mega-Float Phase-II)				
	Yago & Endo <1998>	Orthotropic plate	Rect. Plate Zero draft Uniform mass Uniform stiffness	Regular waves Const. wat. depth Open sea	
	<Direct FSI>	<FEM- plate>	<BEM (direct formulation)>		
	Applied to: 1km class VLFS (Initial design: the Mega-Float Phase-II)				
	Iijima, Yoshida & Suzuki <1997> <Direct FSI>	Grillage with rigid floating units	Grillage + Arbitrary hull <arbitrary draft & water plane>	Regular waves Finite water depth Open sea	
		<FEM- frame>	<BEM +Multi Interact >		
	Applied to: 5km class VLFS				
	Murai & Kaeemoto <2001> <Direct FSI>	Isotropic plate w/ rigid box- shaped units	Assembly of plate units Uniform mass Uniform stiffness	Regular waves Finite water depth Open/ protected sea	Stepped seabed Breakwaters
		<FDM- plate>	<Eigenfunc. Exp +Multi Interact >		
	Applied to: 5km class VLFS				

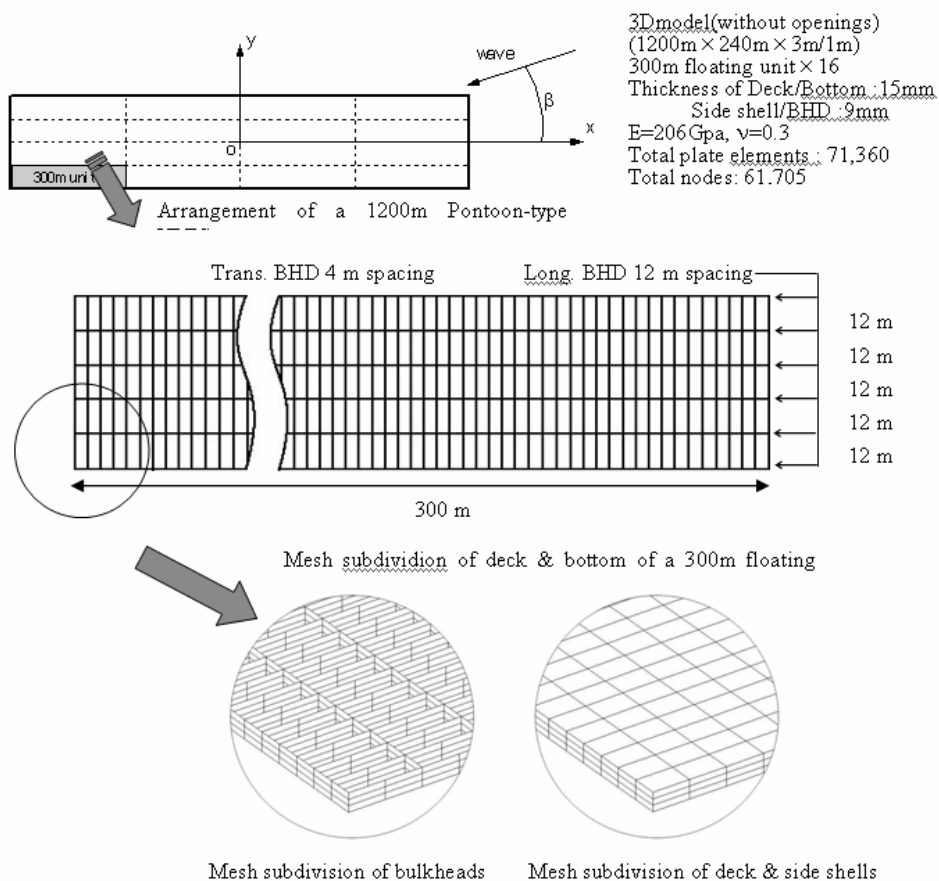


Figure 14 3D structural model of a 1200 m Pontoon-type VLFS

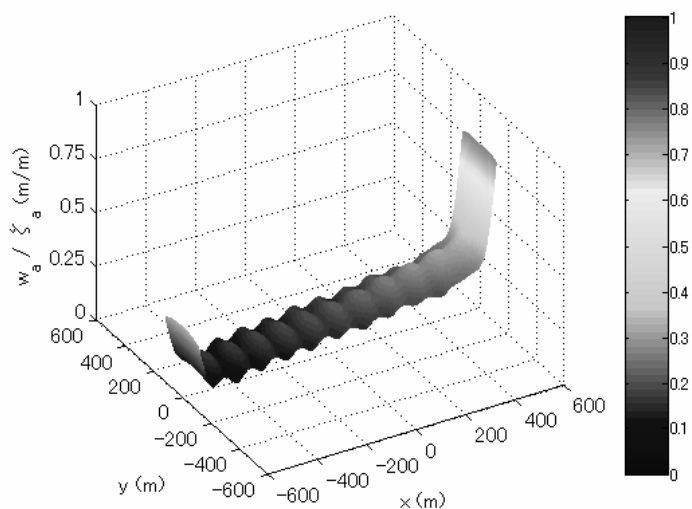


Figure 15 Vertical displacement amplitude of a bottom hull of the 3D VLFS model (1200 m \times 240 m \times 3 m/1 m; $T_w=9.9$ sec, $\beta=0^\circ$)

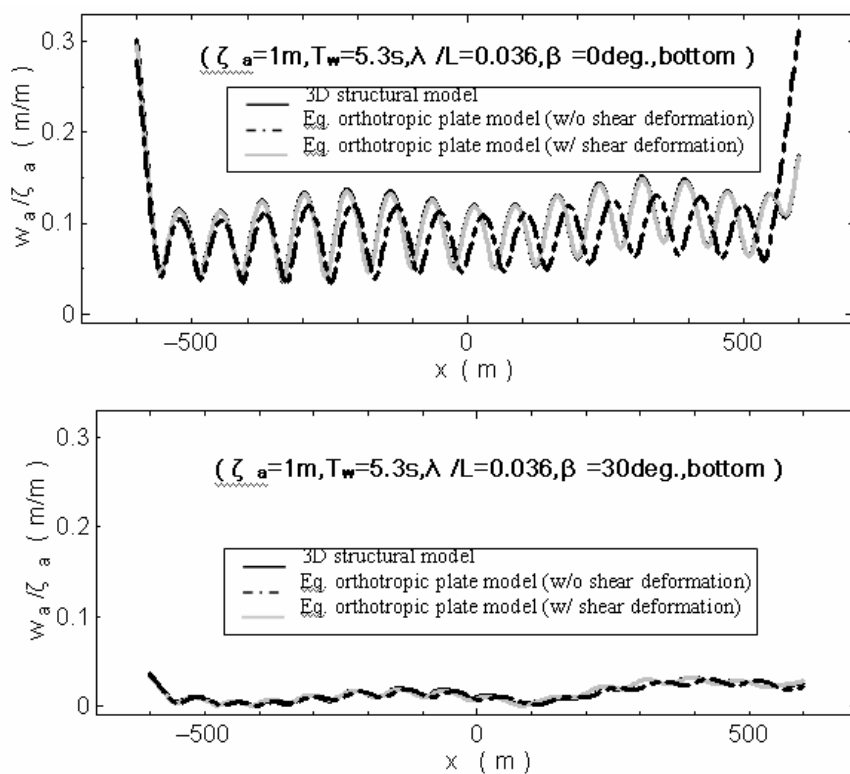


Figure 16 Comparison of vertical displacements on the centerline of a VLFS

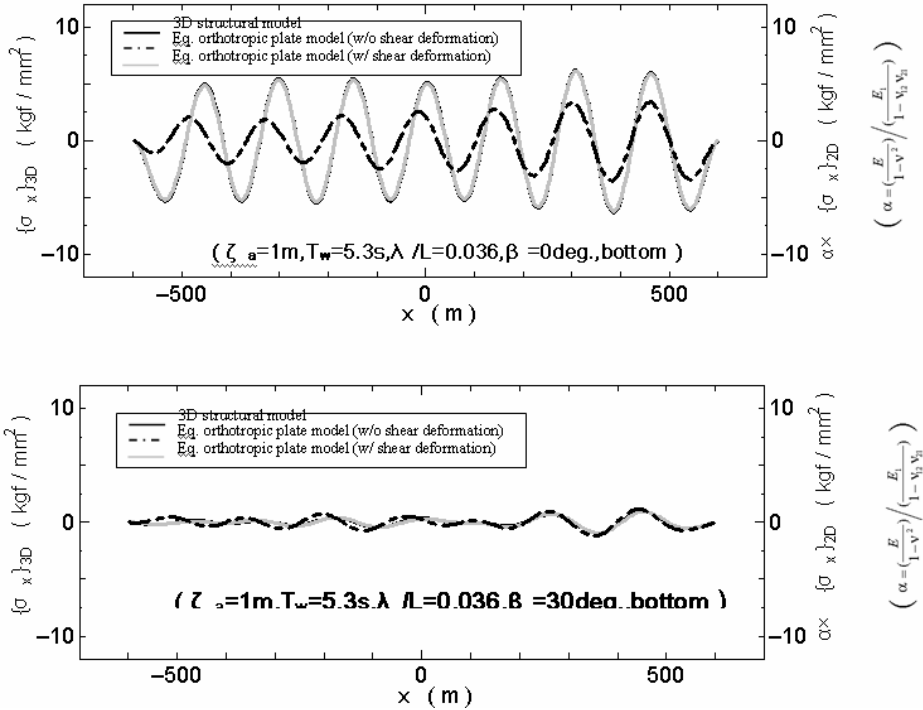


Figure 17 Comparison of longitudinal bending stresses near the centerline of a VLFS

6. DESIGN METHODOLOGY

By definition an innovative marine structure has little or no history of past performance. Additional factors include: (1) incompletely defined or unprecedented loading, (2) new operating environment, (3) lack of verified design criteria, (4) new materials with different properties of strength, fatigue, or corrosion, (5) new fabrication method, and (6) new structure with fewer load paths or greater sensitivity to flaws. The National Research Council (NRC 1993) promotes reliability-based design for such structures.

A VLFS is an innovative structure by most any definition. The Preliminary MOB Classification Guide (ABS 1999a and 1999b) requires and the Technical Guideline of Mega-Float (TRAM 1999b) recommends a risk-based evaluation of safety for VLFS. The risk diagram shown in Figure 18 is based on large-scale engineering projects (Whitman 1984) and was instrumental in clarifying the annual target probability of failure P_f for VLFS. The ordinate is a historical probability of failure rate while the abscissa is consequence in \$'s and associated fatalities. Assuming that the cost of a VLFS is approximately \$10B, the corresponding annual P_f should be roughly 10^{-6} according to these data.

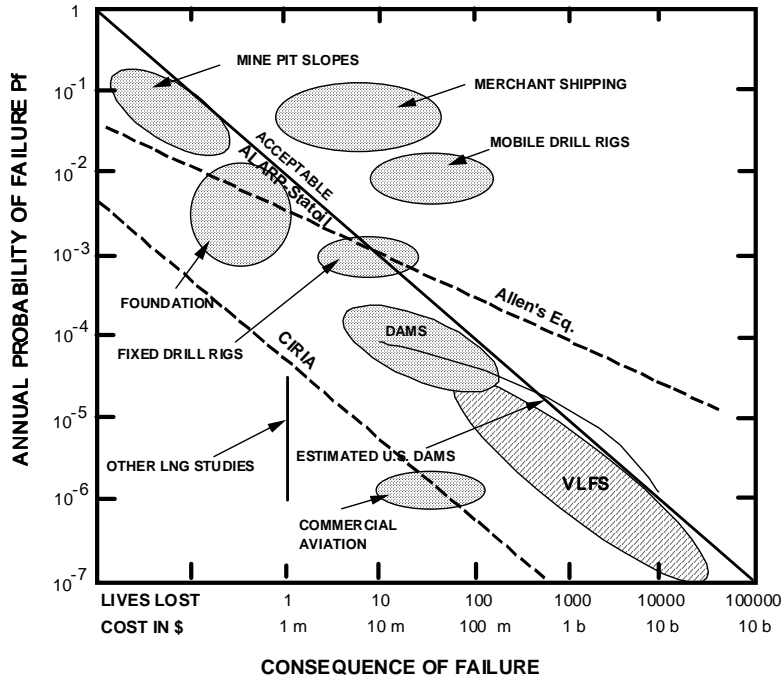


Figure 18 VLFS Risk Diagram

While engineers explicate target reliability level and its relationship to cost, it is for policymakers finally to decide upon the target reliability. An example is found in the Canadian government's setting the target reliability for the Prince Edward Island Bridge project (Pirie 1997 and MacGregor et al. 1997).

6.1 Design Criteria

Risk is the expected consequence from an undesirable event of random magnitude. In the simplest case of a binary outcome, risk is the product of the probability of a failure event and the consequence of the failure event. A design's probability of failure must be less than or equal to an agreed upon target probability of failure. Probability of failure is based upon a set of performance functions involving the load and resistance variables of a system and is the result of an engineering calculation. In this calculation the probability of human error (e.g., a blunder) is usually disregarded. Yet a large percentage of engineering system failures are due to human error. It is therefore not advisable to regard reliability as an absolute quantity, even though, for example, one could partially compensate for human error by increasing the calculated probability of failure by a certain factor. Indeed, sometimes the word notional is used to qualify probability of failure. In a risk approach to calculating a level of safety it is well to keep in mind that safety is relative freedom from risk and so is partly a psychological concept, while structural reliability is quantitative.

6.1.1 Acceptable risk

The target probability of failure is based on acceptable risk (among other factors) as indicated in Table 6 (Bhattacharya et al. 2001). This table clarifies meaning of P_f levels by translating them in terms of common notions of acceptability.

It is also important to distinguish between voluntary and involuntary risk. Here the human element enters again. Voluntary risk is a risk that has been considered, understood, and generally accepted. Involuntary risk is an unknown risk, has not been considered, and where fatalities or casualties may be regarded as innocents.

Table 6. Annual Target Reliability Values and Acceptability

Table 6: Annual Target Reliability Values and Acceptability						
Annual Target P_f				Degree of Acceptability		
$10^{-1} - 10^{-3}$				Unacceptable		
10^{-4}				Control with public expenditure		
10^{-5}				Rare, but voluntary risk (e.g., drowning)		
10^{-6}				Acceptable (e.g., act of God)		
Alternative Measure of Reliability - Reliability Index β						
P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}
β	1.28	2.32	3.09	3.71	4.25	4.75

As an example, acceptable involuntary risk has been taken as 10^{-3} times acceptable voluntary risk in risk analysis of a Mega-Float international airport concept (Suzuki, 2002). The safety level of several activities in Japan, such as transportation, accidents and natural disasters, was examined and expressed in terms of Fatal Accidental Rate (FAR, the number of victims per 100 million man-hours of activity). Then, considering the particular features of VLFS, such as public usage, the involuntary nature of risk, and gross errors, the value of 1×10^{-6} was derived as an allowable annual failure probability of the VLFS. Comparing with the present level of life-loss risk investigated by ISSC2000 (2000), the proposed value almost coincides with the safety level of a hospital, school and other public facility (Fujikubo et al. 2005). The same value for annual target reliability was arrived at independently for MOB (ABS, 1999a).

TABLE 7 TARGET RELIABILITIES FOR VLFS

Class Of Limit State (L.S.)	Consequence	Scale of Participating Structure	Design/ Assessment Methodology	Minimum Permissible Life-time P_f	Minimum Permissible Availability
Serviceability / Operability	minor	Structural element or assembly	LRFD or Direct Reliability	$10^5 q^*$	NA
	moderate	Inter-module connector, structural sub-system, module or fully connected MOB	LRFD or Direct Reliability	$\min(10^4 q^*, 1 - A_1)$	A_1
Fatigue	minor	Welded joints, structural element/ assembly (redundant & accessible)	Direct Reliability	$10^5 q^*$	NA
	moderate	Welded joints, structural element/ assembly (redundant but inaccessible)	Direct Reliability	$10^4 q^*$	NA
	serious	Welded joints, structural element/assembly (non-redundant)	Direct Reliability	$10^3 q^{**}$	NA
	critical	Structural element/assembly (non-redundant) or subsystem	Direct Reliability	$10^2 q^{**}$	NA
Strength	serious	Inter-module (redundant design)	Direct Reliability	$\min(10^3 q^*, 1 - A_1)$	A_2
	critical	Inter-module (non-redundant design)	Direct Reliability	$\min(10^4 q^*, 1 - A_1)$	A_2
	moderate	Structural element (redundant)	LRFD or Direct Reliability	$10^4 q^*$	NA
	serious	Structural element/ assembly (redundant)	LRFD or Direct Reliability	$10^3 q^*$	NA
	critical	Structural element/ assembly (non-redundant) or sub-system	LRFD or Direct Reliability	$10^2 q^*$	NA
	serious	Inter-module connector (redundant design)	Direct Reliability	$\min(10^3 q^*, 1 - A_1)$	A_2
	critical	Inter-module connector (non-redundant design)	Direct Reliability	$\min(10^4 q^*, 1 - A_1)$	A_2
	critical	Structural sub-system subject to weapons effect / accident	LRFD or Direct Reliability	$10^4 q^*$ (conditional)	NA
Progressive Collapse	catastrophic: loss of VLFS	system	Direct Reliability	q^*	NA

6.1.2 *Target reliabilities*

The VLFS target reliability levels listed in Table 7 are recommended for various combinations of limit states and failure consequences. These are specified to achieve a uniform level of risk throughout the structure (Bhattacharya et al. 2001). A recommended

consensus value $P_f = q^*$ for system target failure probability is the starting point. This lifetime target failure probability for the VLFS is assumed to be $q^* < 1 \times 10^{-5}$. Remaining target reliability levels for various limit states are set according to relative safety attached to the limit state and failure consequences, as determined by engineering consensus. Policymakers, for whom these engineering recommendations are made, make the final

decision on system target reliability. In this case, the availability values A_1 and A_2 pertain to a VLFS with intermodule connectors. They are the minimum permissible availability of one module and of the connector, respectively. Policymakers would also provide these values with guidance provided by operational availability models for the VLFS developed during engineering planning.

6.2 *Design Procedures*

Design by analysis is the basic design philosophy for VLFS, and first principle approaches to analysis are naturally indicated for innovative structures. Reliability analysis cannot overcome an important mode of system failure missed by an over-simplified structural model; it is only as good as hydroelastic/structural analysis modeling. Therefore, the modeling must include all important failure modes and be able to realistically account for structural behaviour near global failure including material/geometric nonlinearities, load redistribution etc. as appropriate. This is clearly an engineering challenge of the first magnitude.

The assurance of safety of VLFS structures will therefore require the use of state-of-the-art (SoA) design and analysis tools. However, these tools must have been experimentally validated or must be considered generally accepted tools in keeping with the philosophy of risk-averse technology for VLFS. Application of SoA tools does not lessen the responsibility placed on the engineer for imagination and insight. The intelligent and reliable application of advanced finite element structural and hydroelastic analysis tools, for example, requires a correspondingly advanced level of understanding of the principles underlying these tools, along with experience in their application.

6.2.1 *Limit states*

The four classes of structural limit states in Table 8 should at least be considered in the design loop for VLFS. In this case, it is assumed that the VLFS has intermodule connectors, but these limit states are very similar to the limit states specified for Mega-Float, with return periods for environmental loads appended (TRAM 1999a).

Table 8 VLFS Structural Limit States

Limit State	Scale of Structural Participation	Definition of Limit State
Serviceability/ Operability	Elements/assemblies, sometimes subsystem and system	Disruption of normal use (including military operations) due to excessive deflection, deformation, motion or vibration
Fatigue	Element/assemblies; also inter-module connectors	Critical level of cumulative fatigue damage or critical crack size determined by functional considerations, fracture toughness, and/or static strength
Strength	Element/assemblies; also inter-module connectors	Local failure such as rupture, instability, plastic mechanism, and buckling.
Survivability/ Global Failure	Entire system (fully connected MOB, or single modules when disconnected)	Loss of entire structure (i.e., loss of stability leading to capsizing, sinking, loss of stationkeeping) (i) as a result of <i>progressive collapse</i> , or (ii) in <i>damaged condition</i> after sustaining severe damage in a sub-system

6.2.2 Load types and combinations

For each limit state that is relevant to a given structural element/assembly, sub-system or system, all load types and load combinations should be considered. The following load types should be included in the design loop where applicable:

- a. *Permanent or dead loads.* Permanent loads include self-weight, weight of connectors, fixed equipment, permanent ballast, etc.
- b. *Variable or live loads.* Variable loads include weight of temporary/movable equipment, people, cargo (dry and liquid), variable ballast (dry and liquid), etc.
- c. *Frequent and Rare Environmental loads.* Environmental loads can arise from wind, current, waves and earthquakes. Frequent environmental load represents annual maximum load, which is a random variable. Its nominal value is its one-year return period value. Rare environmental load represents lifetime maximum load, which is a random variable; it is nominally its 100-year return period value.
- d. *Operation loads.* Operational loads include aircraft landing loads, crane loads, berthing/docking loads, connection/ disconnection loads, etc.
- e. *Cyclic loads.* Cyclic loads include repetitive components of environmental and operational loads.
- f. *Accidental loads.* Accidental loads include on-board explosions, fires, and abrupt changes in buoyancy, inter-module collision, crash of aircraft, grounding etc.

In addition, any load associated with construction, fabrication and installation shall also be considered. Load combination schemes considered for the structural design of the VLFS should include all significant sets of simultaneously acting loads for each limit

state under consideration. All realistic scenarios should be investigated; suggestions are provided in Table 9.

Table 9 Load Combination Schemes

Limit States	Load Types							
	Permanent	Variable	Environmental		Operational	Cyclic	Accidental	Installation
			Frequent	Rare				
Serviceability	x	x	x		x			
Fatigue						x		
Strength	x	x		x				
	x	x	x		x			
	x	x	x				x	
	x	x	x					
	x							x
Global Failure: (a) Progressive Collapse	x	x		x				
(b) Damaged Condition	x	x	x					

6.3 *Applicability of Structural Reliability Methods*

Structural reliability can most generally be defined as the probability that the entire spatial domain of the structure will perform satisfactorily during a specified service life under given operation conditions. Application of structural reliability analysis is a particularly important consideration when designing any innovative structure most particularly a VLFS due to its large socio-economic impact. It also promotes economy in structural designs that are safe enough against all failure modes by ensuring that they are uniformly just safe enough in terms of risk. However, the level of safety always remains selectable; it is not a means to greater safety necessarily.

The main issue in the application of reliability methods is that of striking a balance between a rational approach to safety and a practical approach to economical performance. The ability of reliability methods to offer a more rational and systematic certification framework for safety and performance is weighed against the reality of key impediments and concomitant costs of applying reliability methods.

Application of structural reliability methods to VLFS requires knowledge of the safety certification process for a VLFS. The certification goal must be clear at the outset because

engineers must then perform difficult calculations to achieve it. This requires a highly educated engineering labor force, and can imply increased design cost.

In a *direct reliability*-based design, the computed structural reliability has to satisfy explicitly the target reliability for each relevant failure mode. If the reliability is lower (or unnecessarily higher) than the target for the relevant failure mode, the design is modified until an acceptable reliability is obtained. Formulation of limit states, characterization of random variables and model uncertainties, computation of failure probability, and meeting target reliabilities are steps performed explicitly in a framework of direct reliability-based design.

In an *indirect reliability*-based method, such as *load and resistance factor design* (LRFD), the structure implicitly satisfies the target reliability within a certain tolerance. The above framework steps are not performed explicitly in LRFD; rather, calibrated partial safety factors of the design equations (usually given in a safety checking format) are optimized so that the target reliability is satisfied in an average sense for each class of structure and each type of limit state.

Significant uncertainties, reducible or intrinsic, may be associated with some or all of the parameters (e.g., the geometry, material, loading etc.) that govern the performance of a structure. It is most rational to treat such parameters, denoted as basic variables of the problem, as random variables or random processes/fields depending on whether they are temporally/spatially invariant or not. The main uncertainties are in the *modeling* of environmental demands and the global response that are unique to VLFS. Existing uncertainty data for marine structures may be relevant to VLFS (e.g., Nikolaidis and Kaplan, 1991), and where appropriate should be adopted to save design cost.

7. CONCLUSIONS AND RECOMMENDATIONS

Very Large Floating Structure (VLFS) is a unique concept of ocean structures primary because of their unprecedented length, displacement cost and associated hydroelastic response. VLFS requires massive investment of resources and is considered a megaproject. Technology utilized in the project must be well proven or reliable to reduce risk of investment. VLFS has little or no history of performance. Researchers and engineers have improved their understanding of hydroelastic response and its implementation in the design method. Detailed and concentrated efforts were undertaken in the MOB and Mega-Float projects. Considerable development of analysis programs occurred. Design methodology was formalized and the design flow resulted from the effort. To realize reliability, a risk-based evaluation of safety for VLFS was required by the Preliminary MOB Classification Guide and recommended by the Technical Guideline of Mega-Float. MOB and Mega-Float are significantly different in terms of objective and configuration. R&D efforts of MOB and Mega-Float were almost independently carried out but both programs shared similar R&D objectives in many technical aspects. The following conclusions and recommendations are obtained through comparison and evaluation of the two projects, review of related research and comparative study.

(1) Design by analysis and first principle approaches to analysis are natural for VLFS. A consensus that reliability and risk based approaches should be adopted was reached. Design methodology and design flow that incorporates hydroelastic behavior was developed. Efficient design methodology that combines reliability and risk based approach with hydroelastic behavior is an area for further research.

(2) Analysis codes independently developed for MOB and Mega-Float were compared with respect to global behavior. Good comparisons between the programs were observed. Hydroelastic response tends to be discussed from the viewpoint of global behavior especially in academia. From the structural design viewpoint, influence of structural modeling on the detailed stress response was important and was discussed in the comparative study. Recommendations were made for modeling of the structure.

(3) Coherence issues in describing the physical ocean environment and interpretation in design remains an area for further research.

Leading edge technology were developed in many areas of VLFS research particularly in hydroelastic analysis, fatigue design and construction. These results are beneficial not just for VLFS but also for a variety of marine structures. They should be considered for technology transfer.

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