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Open ocean fish cage and mooring system dynamics

David Wayne Fredriksson

University of New Hampshire, Durham

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Open ocean fish cage and mooring system dynamics

Abstract
To satisfy the global need for seafood, marine aquaculture is expected to play an increasing role as wild fish stocks decline. The expansion of near-shore aquaculture is becoming more difficult because of multi-use issues and environmental impact concerns. As a result, a national objective has been initiated to establish an open ocean aquaculture industry. To design and evaluate fish cages and moorings for the energetic open ocean requires a systematic approach utilizing physical and numerical modeling techniques. Using these methods, two robust fish cage and mooring systems were designed, deployed and have survived two New England winters. Prior to the second winter, the north system was refurbished and deployed with nine, 89 kN capacity load cells on the mooring and a six degree of freedom accelerometer motion package inside the fish cage. During the redeployment of the cage, open ocean drag tests were performed. A buoy was also deployed to measure the surface wave forcing. Assuming a linear system, a stochastic approach was used to analyze the load response of critical mooring lines and the motion response characteristics of the fish cage in heave, surge and pitch. Transfer functions were calculated for northeast storms. These normalized functions were compared with results of multiple physical and numerical model tests. The comparisons were used to validate the methods and to understand the dynamics of the deployed system so that future fish cage and moorings are designed and evaluated accurately to assist a new aquaculture industry to become economically feasible.

Keywords
Engineering, System Science, Engineering, Marine and Ocean

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OPEN OCEAN FISH CAGE AND MOORING SYSTEM DYNAMICS

BY

DAVID W. FREDRIKSSON

B.S., Massachusetts Maritime Academy, 1991
M.S., University of New Hampshire, 1993

DISSERTATION

Submitted to the University of New Hampshire
in Partial Fulfillment of
the Requirements for the Degree of

Doctor of Philosophy
in
Engineering: Systems Design

September 2001
This Dissertation has been examined and approved.

M. Robinson Swift
Dissertation Director
Professor of Mechanical and Ocean Engineering

Kenneth C. Baldwin
Professor of Mechanical and Ocean Engineering

Thomas Ballestero
Associate Professor of Civil Engineering

Barbaros Celikkol
Professor of Mechanical and Ocean Engineering

Igor Tsukrov
Assistant Professor of Mechanical Engineering

July 20, 2001
Date
To my wife Julie
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Finally, I send my love to my entire family, especially my wife Julie who has unconditionally supported this effort. In the past six months, she and our son Alek would visit me almost every day. What more would a husband and dad want?
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<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A:</td>
<td>Perpendicular cross-sectional area</td>
</tr>
<tr>
<td>a_m, a_t:</td>
<td>Wave acceleration vector components used in the Morison equation</td>
</tr>
<tr>
<td>A_j:</td>
<td>Wave amplitude used in random phase summation</td>
</tr>
<tr>
<td>A_m:</td>
<td>Cross-sectional area of the net (model-scale)</td>
</tr>
<tr>
<td>A_mod:</td>
<td>Modified cross-sectional area the net (model-scale)</td>
</tr>
<tr>
<td>C:</td>
<td>General coefficient of drag</td>
</tr>
<tr>
<td>C(t):</td>
<td>Constant of integration</td>
</tr>
<tr>
<td>C_D(Re):</td>
<td>Coefficient of drag as a function of Froude and Reynolds number</td>
</tr>
<tr>
<td>C_Re):</td>
<td>Frictional component of the coefficient of drag</td>
</tr>
<tr>
<td>C_w(Fr):</td>
<td>Wave making component of the coefficient of drag</td>
</tr>
<tr>
<td>C_m:</td>
<td>Inertial coefficient</td>
</tr>
<tr>
<td>C_n:</td>
<td>Normal drag coefficient used in the Morison equation</td>
</tr>
<tr>
<td>C_t:</td>
<td>Tangential drag coefficient used in the Morison equation</td>
</tr>
<tr>
<td>C_{xy}(f):</td>
<td>General coincident spectral density as a function (co-spectrum)</td>
</tr>
<tr>
<td>D:</td>
<td>Characteristic diameter</td>
</tr>
<tr>
<td>D_m, D_p:</td>
<td>Model and prototype characteristic diameter</td>
</tr>
<tr>
<td>D_eq:</td>
<td>Equivalent net diameter</td>
</tr>
<tr>
<td>d:</td>
<td>Water depth</td>
</tr>
<tr>
<td>d_H:</td>
<td>The horizontal distance between the anchor and the grid float</td>
</tr>
<tr>
<td>d_v:</td>
<td>Grid line height of the ocean bottom</td>
</tr>
<tr>
<td>f:</td>
<td>Frequency</td>
</tr>
<tr>
<td>F_{drag}:</td>
<td>Force due to drag</td>
</tr>
<tr>
<td>F_{max}:</td>
<td>Maximum mooring line force</td>
</tr>
<tr>
<td>F_{nb}:</td>
<td>Net buoyant force</td>
</tr>
<tr>
<td>F_{ss}:</td>
<td>Steady-state force</td>
</tr>
<tr>
<td>F_w:</td>
<td>Wave induced load</td>
</tr>
<tr>
<td>Fr:</td>
<td>Froude number</td>
</tr>
<tr>
<td>g:</td>
<td>Gravitational constant</td>
</tr>
<tr>
<td>h(t):</td>
<td>Time domain weighting function</td>
</tr>
<tr>
<td>h_eq:</td>
<td>Equivalent net truss elements</td>
</tr>
<tr>
<td>H(f):</td>
<td>General frequency domain transfer function</td>
</tr>
<tr>
<td>H:</td>
<td>Wave height</td>
</tr>
<tr>
<td>H_{mo}:</td>
<td>Energy based significant wave height</td>
</tr>
<tr>
<td>H_{rms}:</td>
<td>Root mean square wave height</td>
</tr>
<tr>
<td>H_{1/3}:</td>
<td>Significant wave height or the average of the top 1/3 waves</td>
</tr>
<tr>
<td>H_{10}:</td>
<td>Average of the top 10 percent of the wave heights</td>
</tr>
<tr>
<td>k:</td>
<td>Wave number</td>
</tr>
<tr>
<td>L:</td>
<td>Wavelength</td>
</tr>
<tr>
<td>L_d:</td>
<td>Deep water wavelength</td>
</tr>
<tr>
<td>l_b:</td>
<td>Length of chain in contact with ocean bottom (static condition)</td>
</tr>
<tr>
<td>l_c:</td>
<td>Length of chain in catenary (static condition)</td>
</tr>
<tr>
<td>l_{a}:</td>
<td>Length of anchor line</td>
</tr>
<tr>
<td>L_{pendulum}:</td>
<td>Length of a swing pendulum</td>
</tr>
<tr>
<td>m_j:</td>
<td>j^{th} degree moment of the spectral curve</td>
</tr>
<tr>
<td>N:</td>
<td>Number of wave frequencies</td>
</tr>
</tbody>
</table>
NC(f): Noise correction function
\( \mathbf{n}_n \): The unit vector in the normal direction
\( \mathbf{n}_t \): The unit vector in the tangential direction
P: Pressure
P_{xx}(f): General two-sided auto-spectral density as a function of frequency
P_{xy}(f): General two-sided cross-spectral density as a function of frequency
Re: Reynolds number
R_{xx}(\tau): Auto-correlation as a function of time displacement
R_{xy}(\tau): Cross-correlation as a function of time displacement
Q_{xy}(f): General quadrature spectral density as a function of frequency (quad-spectrum)
S_{sec}: General acceleration spectrum – actual
S_{sec,m}: General acceleration spectrum – measured
S_{disp}: General displacement spectrum
S_{xx}(f): General one-sided auto-spectral density as a function of frequency
S_{xy}(f): General one-sided cross-spectral density as a function of frequency
T: Period
t: Time
T_p: Dominant wave period
T_{p1}, T_{p2}: The dominant periods in the NH typical spectrum
T_A: Static tension in the anchor line
T_h: The horizontal component of the anchor line tension
T_v: The vertical component of the anchor line tension
T_{rms}: Root mean square tension
T_{1/3}: Average of the top 1/3 tension amplitudes
T_{10}: Average of the 10 percent tension amplitudes
U: Normal (relative) water velocity
U_p, U_m: Prototype- and model-scale velocity
u: Horizontal velocity component of the wave
u: Wave velocity vector used in the Morison equation
V_s: Steady-state velocity
V_w: Maximum wave induced velocity
w: Vertical velocity component of the wave
w_c: Wet weight of stud link chain
x(t): General input time series (actual)
x, \dot{x}, \ddot{x}: Displacement, velocity and acceleration
x: Displacement vector used in the Morison equation
x_c: The x-component in the chain catenary
X(f): Fourier Transform of the general input time series
X(f,T): Finite-range version of the Fourier Transform of the input time series
y(t): General output time series (actual)
Y(f): Fourier Transform of the general output time series
Y(f,T): Finite-range version of the Fourier Transform of the output time series
y_c: The y-component in the chain catenary
z: Vertical direction upwards
\alpha: Time shift variable
\alpha_1, \alpha_2: Shaping parameters for the NH typical spectrum
\beta: Time shift variable
\varepsilon: Random phase used in random phase summation
\gamma_1, \gamma_2: Spectra shaping parameters
\gamma_{xy}(f): General coherency-squared function
$\xi(t)$: Horizontal wave particle trajectory amplitude
$\zeta(t)$: Vertical particle trajectory amplitude
$\lambda$: Modeling scale factor
$\phi(x,z,t)$: Velocity potential
$\rho$: Mass density of a fluid
$\rho_{eq}$: Equivalent mass density of net
$\rho_w$: Mass density of water
$\rho_{sw}$: Mass density of seawater
$\theta_{xy}(f)$: Phase angle between the input forcing and the system response
$\theta_b$: Angle between the ocean bottom and anchor line (static condition)
$\eta(t)$: Wave elevation time series
$\tau$: Time displacement between two measurements in the auto-correlation function
$\mu$: Dynamic viscosity
$\nu$: Kinematic viscosity
$\omega$: Wave radian frequency
ABSTRACT

OPEN OCEAN FISH CAGE AND MOORING SYSTEM DYNAMICS

By

David W. Fredriksson

University of New Hampshire, September, 2001

To satisfy the global need for seafood, marine aquaculture is expected to play an increasing role as wild fish stocks decline. The expansion of near-shore aquaculture is becoming more difficult because of multi-use issues and environmental impact concerns. As a result, a national objective has been initiated to establish an open ocean aquaculture industry. To design and evaluate fish cages and moorings for the energetic open ocean requires a systematic approach utilizing physical and numerical modeling techniques. Using these methods, two robust fish cage and mooring systems were designed, deployed and have survived two New England winters. Prior to the second winter, the north system was refurbished and deployed with nine, 89 kN capacity load cells on the mooring and a six degree of freedom accelerometer motion package inside the fish cage. During the redeployment of the cage, open ocean drag tests were performed. A buoy was also deployed to measure the surface wave forcing. Assuming a linear system, a stochastic approach was used to analyze the load response of critical mooring lines and the motion response characteristics of the fish cage in heave, surge and pitch. Transfer functions were calculated for northeast storms. These normalized functions were compared with results of multiple physical and numerical model tests. The comparisons were used to validate the methods and to understand the dynamics of the deployed system so that
future fish cage and moorings are designed and evaluated accurately to assist a new aquaculture industry to become economically feasible.
CHAPTER 1

INTRODUCTION

1. Background

The global demand for fishery products will become more severe if the world’s population continues to expand and fish stocks decline. In New England, the wild harvest of traditional fisheries, such as cod, haddock, and flounders have reached record low levels (Bucklin and Howell, 1998). Marine aquaculture is expected to play an increasing role in supplying these seafood demands and for providing future economic opportunities. Near-shore aquaculture facilities, like the salmon farms in the Gulf of Maine, have grown tremendously in the last 20 years. Recently, these fish farmers have been plagued by pressure from local and national environmental groups. Environmentalists are concerned about the introduction of escaped fish diluting natural occurring stock species, which is a violation of the Endangered Species Act (O’Connor et al., 2000). They are also concerned about the failure of satisfying the National Pollutant Elimination Discharge (NPDES) requirements with regard to the impact of excess feed and fecal matter in the surrounding water column and benthic communities.

The expansion of near-shore aquaculture is limited, however, not only because of these recent issues, but also because of the resistance to the development of future near-shore sites already used for recreation, commercial fishing and shipping. These problems are not specific to this region but have resonated worldwide initiating interest into moving aquaculture into the open ocean (McElwee, 1996). Open ocean aquaculture in
the U.S.A is defined by Goldberg et al. (1996) as aquaculture taking place in federal waters in the Exclusive Economic Zone (EEZ) and beyond state waters (extending 3 miles from shore). Open ocean aquaculture has many advantages including minimizing multi-use issues, access to better water quality and efficient dispersion of wastes. In New England, finfish open ocean aquaculture is a brand new industry requiring a supporting infrastructure foundation. Technologies such as mooring systems, fish cages and feeding systems need to be designed to withstand the high-energy environment. Many of these system designs are also species specific. This introduces another unknown. What is the optimal species suitable for the exposed conditions of the Gulf of Maine? And how does one optimize a system for a particular species without biological criteria? At the same time, it is difficult to investigate a candidate species for growout in these systems if broodstock and the supporting hatchery infrastructure are nonexistent.

Initiating an open ocean aquaculture industry in New England is a unique multi-disciplinary problem involving the talents of biologists, engineers, environmental scientists, economists and policy makers (among others) to find a common optimized solution. Each requires input from the other to be successful. The task to initiate this type of research in the United States is subsidized by the federal government because new technologies typically take nearly 20 years to develop (McVey, 1996), and the private sector, most likely, could not afford the initial capital costs. The National Oceanic and Atmospheric Administration (NOAA) division of the Department of Commerce has funded demonstration projects to investigate and solve the inter-disciplinary problems associated with finfish aquaculture in the open ocean. One of these sites, called the open ocean aquaculture (OOA) demonstration site is currently permitted and in operation.
through researchers at the University of New Hampshire (UNH). The site is in 50-55 meters of water approximately 6 miles from the coast of N.H (Figure 1.1).

![Map of the UNH open ocean aquaculture demonstration site.](Figure1.1)

**Figure 1.1: The UNH open ocean aquaculture demonstration site.**

### 2. Previous Work

One of the primary engineering problems to be solved is the design of an economic fish cage and mooring system built to withstand the environmental loads of the open ocean. These loads are most commonly due to water currents and surface waves. The design process often relies heavily upon the use of physical models tested in wave/tow basin facilities. For example, towing tests were conducted at the Norwegian facility MARINTEK to investigate the current forces, net deflection and velocity blockage due to the net on a fish cage (Aarnses et al., 1990). In the same facility, tests were conducted using a model of a gravity type, high density polyethylene (HDPE)
surface cage examining the mooring line forces and stress/strain characteristics in the flotation components in varying sea-states (Slaattelid, 1990). At the Heriot-Watt University in Scotland, Linfoot and Hall (1985) and Reville et al. (1995), performed an extensive number of physical model tests using various fish cage designs. These researchers utilized frequency domain analysis techniques to evaluate system loads and motion response from a stochastic sense. In the United States, physical model tests for open ocean aquaculture has taken place at the Massachusetts Institute of Technology (MIT) by Best et al. (1996), investigating a potential commercial product in currents and waves. Another modeling effort was performed by Goudey (1998) using a 1:10 scale model of a novel ocean drifter cage in the David Taylor Model basin in Bethesda Maryland, investigating motion response and internal cage loads. At the University of New Hampshire, Swift et al. (1996) performed a series of wave tests used in the development of a noninvasive optical device that measures fish cage motions in the UNH tow/wave basin.

Recently, numerical models have been developed and applied specifically for open ocean aquaculture applications. Most of these models utilized the finite element analysis (FEA) approach with a Morison equation formulation to calculate loads due to currents and waves (Morison et al., 1950). One such model was developed at UNH by Gosz et al. (1996). This model uses simple truss elements and an updated Lagrangian Formulation to account for large displacements and rotations. Other researchers have applied “off the shelf” products to model fish cage dynamics. Gignoux and Messier (1999) use the latest versions of ABAQUS™ with the module ABAQUS/Aqua. Their model uses beam elements exclusively.
Numerical methods have long been used as design tools in the offshore oil industry, described extensively in Zienkiewicz (1978). This industry has also utilized physical model tests along with the numerical approach efforts and many of these techniques are provided in Chakrabarti (1994). The use of physical and numerical model tools each play a complementary role in the effective design of these structures. The development of systems for open ocean aquaculture must adopt many of the ideas and approaches used in the mature offshore oil industry, but in a more economical fashion.

3. Problem Definition

Unlike the offshore oil industry, open ocean aquaculture engineering, is still in its infancy. Even though many research engineers, over the past two decades have studied specific components such as physical modeling techniques, numerical modeling methods and cage and mooring design, few have performed a comprehensive study examining the dynamics of a cage and mooring system exposed to the open ocean. In this investigation, physical and numerical modeling techniques are validated using field measurements in both ocean currents and waves. One of the most important design parameters examined are the motion response characteristics of a surface cage in the heave (vertical), surge (horizontal) and pitch (rotation) degrees of freedom and the load response of critical mooring components. The motion and load dynamic response is primarily due to surface waves, which have random characteristics. To perform comparisons with physical and numerical modeling methods, frequency analysis, used by systems engineers, is utilized. Normalized motion and load transfer functions are calculated for this comparison purpose.

A comprehensive study examining the dynamics of an open ocean cage and
mooring system will enable engineers to better understand and utilize physical and numerical modeling tools. Proper evaluation and upgrading of these methods will render them more valuable in the design so safety factors can be used with a higher degree of confidence. The purpose is to put physical and numerical modeling on a sound scientific basis so the open ocean mooring design can be done reliably and at a minimum expense. This will in turn, hopefully improve the economics of moving aquaculture into the open ocean not only to help support the increasing global need for seafood but also help initiate a new industry in the United States.

4. Objectives

The objectives of this research are to:

- Working with a team of engineers, design and deploy an operational open ocean aquaculture facility.
- Apply physical models for analyzing open ocean fish cage and mooring system dynamics.
- Utilize numerical modeling techniques in conjunction with the physical model study for design and evaluation purposes.
- Conduct a field program to collect environmental forcing and fish cage/mooring system response data (i.e. motion and loads) to assess the physical and numerical modeling tools.
- Evaluate the dynamics of the fish cage and mooring system deployed at the Isles of Shoals demonstration site.

5. Method of Approach

This comprehensive study begins with a clean slate. Prior to this work, an operational open ocean aquaculture facility was non-existent in the Gulf of Maine. A permitted demonstration site was obtained through federal funds and candidate fish cage and mooring systems were investigated. The first species to be stocked in the cages was
summer flounder (*Paralichthys Dentatus*). This choice was based on the experience of UNH and local biologists (Johns et al., 1981; Howell, 1983; King et al., 1998), a ready supply of juveniles from a local hatchery (Nardi, 1998) and market value at the time of the decision.

The design process was initiated using a series of physical model tests using candidate fish cage and mooring systems. These tests were performed along with complementary FEA simulations using the Gosz, et al. (1996) model. Worst-case environmental conditions for the site were estimated including a design wave height, wave period and current velocity magnitude. Results of the physical and numerical model tests established a nominal design load specification for mooring system components. Selected cage and mooring system designs were presented by the UNH engineering team before a review panel of regional experts, and a final design chosen. Using a marine contractor, the systems were deployed using heavy equipment that included a tug, barge and crane.

As a demonstration project, gear was chosen to have relatively high safety factors to help ensure system survival during the first year of deployment. Many parts of the operation were expensive due to the shear size and weight of the components. It became apparent that a more systematic approach was needed to understand the environmental forces and the resulting load and motion response of the fish cage and mooring system deployed. Understanding the dynamics would enable the physical and numerical model tools to be used more effectively.

After the first year of deployment, one of the cage and mooring systems was retrieved, examined for component wear and refurbished. This presented a unique
opportunity to perform a series of tests using the actual fish cage and to instrument the
cage and mooring system. Prior to redeployment, load cells and accelerometers were
installed to measure in-situ the load and motion response.

Fluid dynamic drag tests were performed during the redeployment of the cage.
These tests were then replicated using improved physical and numerical models. This
information was used to optimize model cage construction techniques and numerical
model input parameters. After the full-scale tow tests, the entire mooring system and
cage were redeployed at the demonstration site along with the instrumentation. Also
deployed was a wave rider buoy used to investigate the local wave climatology. The
deployment of the cage and mooring, load cells (and their data recorders), accelerometers
and the wave rider buoy was a process that took nearly six months because of the
complex logistical nature of the operation. During that time, physical and numerical
model tests were conducted using representations of the cage and mooring system in both
regular and random waves. Since the wave rider buoy had not yet been deployed, data
from the National Data Buoy Center (NDBC) Portland, ME and Boston, MA buoys was
examined for typical and extreme events. This information provided an estimate of the
wave characteristics at the demonstration site that were used for the wave maker and FEA
input.

For the regular wave tests, response amplitude operators for heave, surge and
pitch motion response and for the anchor and bridle line loads were calculated for ten
wave frequencies. These values were compared to the frequency domain linear transfer
functions obtained from the random wave tests. Auto- and cross-spectral density
techniques for the physical and numerical model simulations were performed. The
coherency-squared function was used to investigate how well the system fit a linear assumption and a phase calculation was performed to examine the relationship between the input forcing and the system response.

Transfer functions were also calculated using the *in-situ* data collected. Data sets from northeast storms were downloaded from the wave rider buoy, anchor and bridle line load cells and the cage accelerometers. The normalized transfer functions were then compared to those calculated using the physical and numerical modeling methods.
CHAPTER 2

FISH CAGE AND MOORING SYSTEM DESIGN

1. Design Objectives

The first objective of the engineering component of the project included evaluation of commercially available containment structures and the selection of two fish cages for deployment. Assessment included site visits and review of manufacturers’ literature as well as computer and physical modeling studies. Criteria for selection included suitability for the Isles of Shoals site in particular and New England waters in general. The selected cage system was required to be easily modified to include a taut, fabric bottom to accommodate the benthic dwelling flounder (Linfoot et al., 1990). Floor area was specified to be on the order of 180 m² to accommodate 3.5 metric tons of fish. The selected cage should be capable of year-round operation. The ability to be submerged was a highly desirable attribute so the net pen could be lowered beneath wave motion accompanying severe storms.

The second major engineering goal was to design the mooring system for deploying the two cages. In this initial effort at the demonstration site, reliability was given high priority. The mooring design should allow submergence of the cage, and components should be commercially available. The design process also included developing procedures for setting out anchors and gear at the demonstration site.
2. Design Alternatives

2.1 Fish Cage Systems

Many types of commercially available offshore fish cage systems currently exist on the international market (Balchen, 1990; Brittain, 1996; Ben-Enfraim, 1996; Gunnarsson, 1996; Henriksson, 1996). Some of these systems have been successfully used in offshore applications particularly for the growout of round fish such as salmon (McElwee, 1996). The design requirement for this phase of the OOA demonstration project was unique in that the system chosen had to be suitable for the bottom dwelling flatfish, summer flounder. These requirements include a semi-rigid bottom that allows the passage of solid wastes. Since few offshore aquaculture applications involve the growout of flatfish, the options for purchasing an “off the shelf” product was limited to those that could be simply modified for this specific purpose. After an extensive search and talking with industry professionals, two systems were chosen to be evaluated, a gravity-type cage and a semi-rigid central spar cage (Figure 2.1).

Figure 2.1: Gravity-type fish cage and central spar cage.

A gravity-type cage relies upon buoyancy and weight to maintain tension in the containment nets and to hold the shape of the structure. Many types of gravity cages are
available including PolarCirkle, Wavemaster, Bridgestone and Dunlop. The gravity-type cage analyzed as a part of this effort is manufactured by Northern Plastics Inc. This manufacturer was chosen because the product represented a typical design used in Maine and the Canadian Maritimes. The components of this cage can be separated into the float ring assembly, the cage bottom assembly and the nets, which include the fish containment, predator, floor and surface bird nets. The flotation ring is nearly 16 m in diameter and is constructed of two high-density polyethylene (HDPE) pipes, with an OD of 250 mm, an ID of 232 mm and a corresponding wall thickness of 9 mm. The flotation pipes are held together by brackets placed at approximately 2 m intervals. The float pipes not only support the fish containment and predator nets but also provide sufficient flotation so that it can act as a working platform. The cage bottom is suspended from the flotation rings by the nets. The cage bottom structure would consist of a flat, rigid structure with an area of approximately 180 m². The bottom of the Northern Plastics cage serves two purposes: to maintain tension in the nets, and to support the weight of the flatfish.

The second commercially available cage examined was a central spar-type pen manufactured by Ocean Spar Technologies called the SeaStation™ 49F, as shown in Figure 2.1 (Loverich and Goudey, 1996; Loverich and Gace, 1997). This cage is a self-tensioning structure built around a central spar buoy and an octagon shaped rim. This central spar cage has an internal volume of 595 m³ with a floor area of 177 m². The cage is a rigid structure constructed around a center spar buoy (diameter of 0.92 m, a length of 9.14 m and wall thickness of 10 mm) made of galvanized steel. Nets and 24 radial spoke lines (13 mm in diameter) are held in tension between the spar buoy and the octagon.
shaped rim. The rim of the cage has a nominal diameter of 15 m and is made of eight flanged sections of 273 mm diameter steel pipe. Each of the flanged sections are individually sealed and pressurized. Based on the manufacturer’s specifications, the central spar cage is typically ballasted using 3181 kg of concrete suspended below the spar buoy to maintain a freeboard ranging from 0 to 3.5 m. A variable buoyancy chamber, with a length of 3.16 m inside the spar buoy, enables the cage to be positioned at the surface or submerged. The depth of the cage in the submerged configuration is regulated by the length of line that connects the spar to the ballast weight, which sits on the bottom.

2.2 Moorings

Another component of the engineering process included the study of possible mooring system designs. One of the configurations initially considered for deployment at the demonstration site is an adaptation of multiple cage grid systems. Fish farm site visits revealed that this configuration is used by many aquaculture facilities in the northeastern United States and the maritime provinces of Canada (Muller, 1999). Grid systems typically incorporate multi-array anchoring schemes, which inherently provide redundancy should one or more of the anchor legs become disabled. Depending upon the depth at the site, these mooring configurations often require excessive bottom area, which can be costly to lease on a yearly basis (approximately $185,000 U.S. per km² in N.H). Another concept considered for deployment at the site reflects the bottom space concern and consists of a high-tension (single point) mooring incorporating a spar type buoy with heavy chain.

The first concept considered for deployment employed a pre-tensioned subsurface grid with the top of the grid at a depth of 18 m as shown on Figures 2.2a and 2.2b. It is
designed to have 6 anchors and a double rectangular element (6-node) horizontal grid 37 m above the bottom. Lines with compensator buoys connect the two cages to the grid at the six nodal locations, and the cages are located above the center of each grid rectangle. The grid is anchored to the bottom using six mooring legs each incorporating a chain catenary. Tension is maintained by using flotation at the six nodal points at the top of the grid and by setting the anchors to form the required geometry. Crown lines are attached to the anchors to facilitate mooring adjustments (not shown on Figures 2.2a and 2.2b). The system can be adapted to handle additional cages by adding rectangles and anchor legs to the grid. In this investigation, however, a system using only two cages was considered to minimize initial scope and costs.

Figure 2.2: Top (a) and side (b) views of the subsurface grid mooring system.
The single point, high tension mooring, shown in Figure 2.3, was another concept that was considered. The system consisted of a gravity-type cage like the Northern Plastics cage previously described, a central sleeve fixed to the cage, a spar buoy slip-fit inside the sleeve and a single point, high tension mooring to anchor the spar buoy. The system is simple and minimizes footprint area. The cage component of this system can be arranged to submerge to avoid excessive surface loads. This concept has been demonstrated in the UNH study described by Savage et al. (1997).

**Figure 2.3: Full-scale cage and mooring system investigated in the study.**

In this study, it was required to evaluate, design and deploy a net pen and mooring system at the demonstration site in short order (less than two years). An engineering data set from which to make design decisions concerning the gravity-type and central spar cages, along with the mooring system, was needed almost immediately. The use of
physical and numerical models, as described in Fredriksson et al. (1999a, 1999b, 2000) and Tsukrov et al. (1999, 2000), were relied upon to provide this information.

3. Physical Modeling

3.1 Overview

Physical models are often used in the design process of offshore structures to better understand the behavior of prototype systems to environmental loading conditions (Chakrabarti, 1994). In this study, physical models of potential fish cage and mooring system components were Froude-scaled, constructed and tested following methods described by Vassalos (1999). The tests were conducted in the Jere A. Chase Ocean Engineering Laboratory, which includes a 37.5 m x 3.66 m x 2.44 m tow/wave basin filled with fresh water. A cable driven tow carriage system, mounted on rails above the tow/wave tank, was used to perform model towing tests to simulate a range of ocean currents. Waves were generated by means of a computer based control system that commands a hydraulically driven, flap-type wave maker at one end of the tank. In the initial design study, three sets of physical model tests were conducted at the UNH facility to measure the drag forces on the individual cages due to currents, mooring line loads and motion response of the cage bottom at a specified design wave condition.

Force and load measurements obtained during these tests were used to evaluate systems and to size critical components such as attachment lines, mooring cable and ground tackle. Another issue concerning the rearing of flatfish at exposed sites is the response movement of the fish cage bottom to surface incident waves. Cage bottom response measurements made during the regular wave tests were used to compare the performance of the fish cage systems. Additional complimentary tests, not described
here, were also performed using a computer model to analyze situations that could not be modeled physically because of tank size restrictions (Ozbay, 1999; Tsukrov et al., 2000).

3.2 Model Towing Tests

The first series of tests, as described by Fredriksson et al. (1999a), were performed in the tow/wave basin using Froude-scale models of the gravity-type and central spar cages to simulate drag loading in current. The ratio between the tow/wave basin depth (2.44 m) and an estimated demonstration site depth (55 m), equal to 1:22.5, was chosen to be the scale factor for these tests. This yielded cage model diameters of approximately 0.75 m. Figure 2.4 shows the models of the gravity and central spar cages that were used in the physical model tests.

![Physical models](image)

**Figure 2.4: Physical models (1:22.5) of the gravity-type and central spar cages.**

In each test, the individual cage was connected to the tow carriage using a polyester line (Figure 2.5). The line was connected to a vertical post mounted to the tow carriage. A small, submersible load cell was placed inline between the attachment cord and the cage being towed. For the towing tests, the carriage was set to operate at Froude-scaled velocities ranging from approximately 0.1-0.8 m/s (0.5-3.8 m/s full-scale). During each of the tow tests, carriage velocity and horizontal line tension values were measured.
using light gates and the load cell, respectively. At the demonstration site, maximum current velocities are not expected to exceed 1 m/s (Bub, 1998). Since this value is within the experimental range, interpolated drag load at this design current could be used to help make decisions regarding component selection. The higher speeds were also of interest at this time, for comparison purposes, since the wave generated velocities could be higher and the drag component would be an important forcing mechanism.

![Figure 2.5: Tow test arrangement.](image)

Figure 2.6 and Table 2.1 shows the measured tension in the attachment line for each of the towing tests (estimated error is ± 3600 N). Results show that the gravity-type cage attachment line tension values were considerably higher at the lower velocities than the line tension values obtained from the central spar tests. However, at the higher tow velocities, the values were similar. It was evident that during the higher velocity tow tests using the gravity-type cage, the projected area of the non-rigid structure decreased as the cage deformed. The reduction in projected area resulted in decreased drag loads. Using a design current of 1 m/s, the interpolated drag values for the gravity-type and central spar cages were found to be 28.5 kN and 17.8 kN, respectively.
Figure 2.6: Full-scale drag results of the gravity-type and central spar cage tow tests.

Table 2.1: Full-scale tow test results.

<table>
<thead>
<tr>
<th>Cage</th>
<th>Test #</th>
<th>Tow Speed (m/s)</th>
<th>Full-Scale Velocity (m/s)</th>
<th>Line Tension (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity-Type</td>
<td>1</td>
<td>0.097</td>
<td>0.459</td>
<td>12.3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.177</td>
<td>0.840</td>
<td>25.3</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.423</td>
<td>2.007</td>
<td>58.7</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.545</td>
<td>2.590</td>
<td>84.6</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.792</td>
<td>3.760</td>
<td>194</td>
</tr>
<tr>
<td>Central Spar</td>
<td>1</td>
<td>0.095</td>
<td>0.452</td>
<td>6.19</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.174</td>
<td>0.828</td>
<td>13.6</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.298</td>
<td>1.416</td>
<td>30.3</td>
</tr>
<tr>
<td></td>
<td>4</td>
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<td>2.005</td>
<td>58.1</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.545</td>
<td>2.589</td>
<td>89.6</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>0.668</td>
<td>3.172</td>
<td>134</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>0.792</td>
<td>3.763</td>
<td>190</td>
</tr>
</tbody>
</table>
3.3 Submerged Grid Mooring Test

Next, a series of regular wave tests were conducted to investigate the worst-case mooring loads of the submerged grid system shown in Figure 2.2. Due to tank size restrictions, the Froude-scale mooring system used during these tests utilized both of the cage models attached to only one of the anchor legs of the submerged grid system (Figure 2.7). The intent was to estimate the worst-case situation in which all but one of the 6 grid mooring anchors had become disabled. In this test configuration, mooring line tension and the motion response of each cage bottom to the waves was measured.

![Central Spar Cage](image1)
![Compensating Float](image2)
![Gravity-type Cage](image3)
![Load Cell Location](image4)

**Figure 2.7: Model of the disabled mooring configuration used for the wave tests.**

Input parameters for the wave control system were Froude-scaled representations of a design wave having a nominal height and period of 9 m and 8.8 seconds. The design wave choice was based on a survey of U.S. Army Corps of Engineers wave simulations (http://bigfoot.wes.army.mil) and National Data Buoy Center wave observations near the demonstration site (http://www.ndbc.noaa.gov). The significant wave height for the 50-year storm was estimated to be 8.9 meters with a dominant period $T_p$ of 10 seconds using the techniques described in Appendix A. However, in the tow/wave basin facility, a scaled down wave with these characteristics ($H = 0.39$ m, $T = 1.85$ seconds), approaches the operating limits of system. Since the objective of these tests was to measure line tension values associated principally with fluid drag, a steeper wave with similar wave particle velocities was substituted having a smaller wave height and a shorter period.
During the regular wave tests, surface elevation measurements were made with a capacitance wave probe (estimated error ± 0.4 mm), mooring line tension was sensed using a submersible load cell (estimated error ± 0.25 N), and motion response was recorded using a noninvasive optical measurement system (estimated error ± 0.5 mm).

The optical system consisted of a high-resolution, black and white digital camera operating at a rate of 30 frames/sec (calibrated using a quartz timer). A computer based frame grabber captured the video output and stored each of the frames in the computer. Processing software was then used to analyze each of the frames to track the pixel locations of small black dots strategically located on the white-painted cage models (Swift et al., 1998). Figure 2.8 shows the measured wave elevation and line tension time series results (full-scale) obtained from one of the wave tests using a model of the disabled mooring configuration. The peak line tension measured, for all of the disabled mooring tests, was 54 kN.

![Figure 2.8: Full-scale mooring tension tests results for tandem configuration.](image-url)
Motion response in the heave (vertical), surge (horizontal) and pitch (angle) of each of the fish cage bottoms was investigated using the disabled mooring test configuration. Using the wave elevation time series data collected with the wave probe, wave slope and fluid particle horizontal displacement time series were generated using Airy wave theory as described in Dean and Dalrymple (1991). This information was then compared to the heave, pitch and surge motion response measured using the noninvasive optical system, for each of the fish cages attached to the disabled, submerged grid-mooring configuration.

During these motion response tests, a full-scale regular wave with a height (H) of 8.15 m and a period (T) of 7.98 sec was generated in the tow/wave basin. Using Stoke’s second order wave theory, as described in Dean and Dalrymple (1991), it was found that the maximum particle velocity of this wave was similar to the wave of the design condition. These motion response results were normalized and compared with similar tests conducted with the single point, high tension mooring results.

3.4 Single Point, High Tension Mooring Wave Tests

Physical model tests were also performed using a Froude-scaled model of the single point, high tension mooring shown in Figure 2.3. A complete description of these testing procedures can be found in Fredriksson et al. (1999b). For this test, the motion response of the fish cage bottom was measured for a wave with a full-scale height and period of 6.3 m and 7.93 s. These results were also normalized for comparative purposes.

3.5 Cage Motion Comparisons

Evaluating the fish cages for summer flounder suitability involves analyzing the motion response of the cage bottoms where this species is expected to reside most of the time. Cage bottom responses to each of the individual waves for heave, pitch and surge
can be represented by the following normalized parameters:

- Heave Response: heave amplitude/wave amplitude,
- Surge Response: surge amplitude/wave excursion (horizontal) amplitude,
- Pitch Response: pitch amplitude/wave slope amplitude.

Using the wave elevation and the motion response data sets, response parameters were calculated for a wave period of approximately 7.9 s for the three fish cage systems analyzed in this study. The heave, surge and pitch response parameters for the gravity-type and central spar cages in the disabled mooring configuration are provided in Table 2.2. Also provided in the Table is the cage bottom response for the gravity-type cage with the high tension mooring.

**Table 2.2: Wave response for each cage (full-scale).**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Gravity-type (disabled)</th>
<th>Central Spar (disabled)</th>
<th>Gravity-type (high tension)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Incident Wave Height (m)</td>
<td>8.15</td>
<td>8.15</td>
<td>6.90</td>
</tr>
<tr>
<td>Wave Period (s)</td>
<td>7.98</td>
<td>7.98</td>
<td>7.93</td>
</tr>
<tr>
<td>Heave Response (normalized)</td>
<td>0.719</td>
<td>0.620</td>
<td>0.850</td>
</tr>
<tr>
<td>Surge Response (normalized)</td>
<td>0.755</td>
<td>0.684</td>
<td>0.829</td>
</tr>
<tr>
<td>Pitch Response (normalized)</td>
<td>0.681</td>
<td>0.703</td>
<td>0.891</td>
</tr>
</tbody>
</table>

If the value of the response parameter has a value near one, the entire cage structure and the fish move with the motions of the wave. The fish will not sense the wave orbital velocities, but if the inertial motion is excessive, they may experience motion sickness and therefore stress (Linfoot et al., 1990). This condition typically occurs for lower frequency waves. If the response parameter is substantially less than one, the cage remains stationary and the fish sense the wave particle motions. In some sense, this best mimics a natural bottom, which is fixed and subject to wave velocity. In this environment, however, the fish may have to swim to hold their position relative to
the cage bottom. This could cause excessive fatigue that can affect growth rates. On the other hand, if the fish choose not swim, they will move with the particle motions and may be more susceptible to abrasion with cage bottom.

Currently, it is still unknown which condition will affect summer flounder in an extreme manner. This preliminary study shows the need to consider the biological requirements of the species to optimize growth rates. Because of the lack of conclusive biological data, decisions were made from a structural viewpoint. It was assumed that if the movement of the cage is minimized, so will the wave induced loads imparted on the mooring system. From this standpoint, the central spar cage exhibited the best overall motion response characteristics for a wave at the design condition period.

Physical model tests to determine the response for multiple frequencies were performed for only the single spar system (Fredriksson, 1999b). Results of these tests clearly indicated an over-damped system having no resonant frequency. Due to time constraints, physical model tests for the disabled configuration (using both the gravity-type and central spar cage) were performed only at the design wave condition. It was expected that these systems would also show over damped characteristics.

4. Final Design

For the initial open ocean deployment, subject to extensive public scrutiny, it was essential that there be no engineering "failures". It had to be certain the moorings would hold in storms and the cages would not break apart. Additionally, the system must be deployable using existing UNH capabilities, or abilities that could be contracted at moderate cost. The value of conservative, minimal risk decision-making was important in this process.
Therefore, it was decided not to use the taut line, single mooring system. Having one line without backup was of great concern. The physical model tests also indicated more cage motion response and that there could be a problem in restraining the cage from being forced off the top of its spar in extreme conditions. Finally, the single, very large dead-weight anchor was viewed as being too difficult to deploy by either UNH or a small company. In general, it was decided that using moorings employing multiple embedment anchors was the best course at this time.

A lesser, but still important, consideration was cost. A major variable expense was the permit fee, which was proportional to the "footprint area" of the installation. Due to the scope required of embedment anchors, a very large amount of bottom area is needed in comparison with cage size. For example, a four-anchor system using a horizontal to vertical distance ratio of 4:1 in 55 m of water, the ratio of footprint area to flounder cage floor area (177 m²) would be approximately 550:1. Our strategy for reducing permit cost was to use the horizontal, mid-depth grid. There would be liberal scope below the grid to the embedment anchors. While above the grid, minimal "scope" leading to the cages would be used since the lines lead from "fixed" grid node points. The grid essentially acts as a "false bottom" with attachment points. The initial grid concept was to use a single grid, consisting of two back-to-back squares and to deploy both cages as shown in Figure 2.2. The pre-tensioned, 6-node grid was designed to be 37 m above the bottom. Four lines with compensator buoys from the corners of each square would lead to a cage located over the square's center.

At this time, it was believed that it would be useful for comparison purposes to use one cage of each type - a gravity net pen and a central spar cage. The gravity net pen
would be kept on the surface while the central spar cage could be submerged below the grid. The central spar cage was intended to be submersible and had built-in, variable air buoyancy chamber within its central spar. With partial air release, the cage could be submerged until the suspended ballast weight was supported on the bottom.

A conceptual cage/mooring system design was completed based on the physical model testing and the accompanying computer model simulation. Since the demonstration site was to be a regional facility, it was appropriate for the design to be presented before a review panel made up of independent experts from the Woods Hole Oceanographic Institute, Massachusetts Institute of Technology, Cornell University and the University of Maine. The panel members were provided with all engineering documents before meeting at UNH for a design presentation. A discussion period followed where verbal comments from panel members were recorded. Panel members were then kind enough to provide written input within a few weeks of the meeting.

The panel generally found that the engineering was thorough and sound, but members emphasized the severe conditions that could be encountered at the exposed Gulf of Maine location. Recommendations were made to simplify and strengthen the system. In particular, members believed that the deployment of the complex, double element grid would be difficult and that cages on the surface during northeasters and hurricanes would be extremely vulnerable.

In response, the grid was split into two, separately anchored, square units - one for each cage as shown in Figure 2.9 (surface position shown). It was decided not to use the gravity net pen, but rather to employ two submersible central spar cages. Results of the physical model tests indicated that this cage has lower fluid drag and smaller motion
response to incident waves. The self-tensioned frame of the central spar cage was an additional factor in this decision. Though separated, the same gear component sizes were retained, and the new configuration dynamics were checked using the finite element model (Tsukrov et al., 2000).

![Diagram of mooring system](image)

**Figure 2.9:** The final fish cage mooring system used (surface position shown) with an anchor line scope of 4:1 in approximately 55 meters of water.

Components for the grid system illustrated in Figure 2.9 were chosen for their ability to sustain the design loads, commercial availability and for marine mammal concerns (Muller, 1999). The load capacities of the components were compared with a "worst case" design mooring load inferred from the physical model experiments. The design wave mooring load from the tandem cage test (Figure 2.7) served as the basis of the calculation since the system was disabled to a single line and anchor. The maximum measured line load (full-scale value from Figure 2.8) was apportioned between the gravity and central spar cages according to the ratio of tow carriage forces at 1 m/s (full-scale). The estimated maximum wave force on the central spar cage \(F_w\) was then combined with the central spar steady current force \(F_s\) at the very conservative maximum steady current flow speed of 1 m/s. The maximum mooring line force was assumed to be a function of the steady and wave forces as shown by
\[ F_{\text{max}} = f(F_s = CV_s^2, F_w = CV_w^2). \]  

(2.1)

Nonlinear effects were taken into account assuming contributions were entirely velocity-squared dependant processes (again, the worst case assumption),

\[ F_{\text{max}} = C(V_s + V_w)^2, \]  

(2.2)

where \( C \) is the inclusive drag coefficient assumed to be the same in currents or waves, \( V_s \) is the maximum steady current flow and \( V_w \) is the maximum wave particle velocity. Expanding the polynomial,

\[ F_{\text{max}} = CV_s^2 + 2CV_sV_w + CV_w^2, \]  

(2.3)

and substituting \( F_s \) and \( F_w \) results in

\[ F_{\text{max}} = F_s + 2\sqrt{F_sF_w} + F_w, \]  

(2.4)

the maximum mooring line force with a value of 77.0 kN. This specification was then used to size mooring components.

Seaboard 1000 kg Samson anchors (similar in shape to Bruce anchors) were chosen. According to the manufacturer, these anchors will become embedded within their own length. The anchors were attached to 27 m of 57 mm stud link chain to maintain nearly horizontal loads on the anchors. The rest of the anchor line, to the grid corners, was made up of 100 m of 7-strand wire-laid polyester rope. The grid sides and lower bridle consists of co-polymer rope with lengths of 65 and 32 meters, respectively. The upper bridle lines are 11 meters of polyester configured in a v-shaped bridle. Details of the mooring system components are found on Table 2.3.

The anchor line factor of safety was intentionally high since line failure is potentially disastrous and this component is difficult to access by divers. The anchor factor of safety can be lower because anchor drag is not catastrophic, anchors can be reset.
using the crown lines, and some drag may actually be desirable for stress relief purposes. The grid, lower and upper bridle design factor of safety is moderate since these lines can be subject to regular inspection, maintenance and replacement. The design factor of safety values presented here are consistent with those recommended by Lloyds’s Register (1992), U.S. Navy (1985) and Flory, et al. (1997).

Table 2.3: Mooring system component details.

<table>
<thead>
<tr>
<th>Component</th>
<th>Description</th>
<th>Max Load (kN)</th>
<th>Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchors</td>
<td>1000 kg, Seaboard Anchor</td>
<td>180(^a)</td>
<td>2.3</td>
</tr>
<tr>
<td>Anchor Chain</td>
<td>57 mm stud link (weighing 467 N/m)</td>
<td>~1760</td>
<td>23</td>
</tr>
<tr>
<td>Anchor Line</td>
<td>40 mm, 7-strand polyester wire laid rope</td>
<td>580</td>
<td>7.5</td>
</tr>
<tr>
<td>Grid Line</td>
<td>48 mm, 8-strand co-polymer</td>
<td>345</td>
<td>4.5</td>
</tr>
<tr>
<td>Lower Bridle Line</td>
<td>48 mm, 8-strand co-polymer</td>
<td>345</td>
<td>4.5</td>
</tr>
<tr>
<td>Upper Bridle Line</td>
<td>24 mm, 12-plait polyester</td>
<td>112</td>
<td>2.1(^b)</td>
</tr>
<tr>
<td>Crown Line Chain</td>
<td>26 mm, long link</td>
<td>n.a.</td>
<td>n.a.</td>
</tr>
<tr>
<td>Crown Line</td>
<td>48 mm, 8-strand co-polymer</td>
<td>345</td>
<td>4.5</td>
</tr>
</tbody>
</table>

\(^a\) Holding power in sand/gravel sediments
\(^b\) Upper bridle line is configured with a v-shaped bridle, safety factor considers geometry.

In the original design, the grid corners were held up by 0.94 m steel balls (net buoyancy of 3.2 kN), and the bridle line flotation (compensator) buoys were 0.94 m polyethylene balls (net buoyancy of 4.3 kN). The compensator buoys had a hole in bottom so the buoyancy could be adjusted. They were typically maintained half filled with air. After the first year, however, it was discovered that these compensator buoys would often fail due to relative motion abrasion and therefore removed. Crown lines consisted of 2 m of long link chain, 8 m of 50 mm chain and 41 m of copolymer rope. The crown lines lead to spar-type perimeter buoys which tension the rope portion against the heavy chain normally lying on the bottom and acting as deadweight. The grid can be adjusted by towing the crown lines to adjust the anchor position. Line lengths, anchor
positions and flotation buoyancy were specified to maintain a minimum static tension.

The tensioning was intended to inhibit system movement under normal conditions of low to moderate seas, minimize line "snap" and maintain a minimum, correctly directed load on the embedment anchors. Line tensioning was also intended to reduce the possibility of a whale or sea turtle becoming entangled. Static analysis was performed using the catenary equations for the anchor chains, as defined by Faltinsen (1990), and applying equilibrium equations to free body diagrams of intersection points. The definition sketch for this analysis is shown on Figure 2.10.

At the time, the average depth of the water at the site was estimated to be 50-55 meters. In this analysis, a value of 52 meters was used and therefore the grid plane was design to be approximately 33.7 meters off the bottom (defined as \( d_v \) on Figure 2.10). For the design static condition, 20.44 meters chain (\( l_b \)) was chosen to maintain contact with the seafloor with 7 meters incorporated into the catenary (\( l_c \)). Equations (2.5) through (2.10) describes the geometry of the system including the catenary of the chain.

**Figure 2.10: Anchor leg definition sketch.**
component,

\[ d_H = l_b + x_c + l_r \cos \theta_b, \]  
\[ d_v = y_c + l_r \sin \theta_b, \]  
\[ x_c = \frac{T_h}{w_c} \sinh \left( \frac{l_c \cdot w_c}{T_h} \right) \]  
\[ y_c = \frac{T_h}{w_c} \left[ \cosh \left( \frac{w_c \cdot x_c}{T_h} \right) - 1 \right], \]

and

where: \( d_H \) is the horizontal distance between the anchor and the grid float, \( l_b \) is the length of the anchor line, \( T_A \) is the tension in the anchor line, \( l_c \) is the total length of chain, \( x \) is the horizontal component of the chain catenary, \( y \) is the vertical component of the chain catenary, \( T_h \) is the horizontal component of chain tension, \( T_v \) is the vertical component of chain tension and \( w_c \) is the weight per length of chain.

The horizontal \( (T_h) \) and vertical \( (T_v) \) tension components in the anchor are expressed as

\[ T_h = T_A \cos \theta_b \]  
\[ T_v = w_c \cdot l_c = T_A \sin \theta_b. \]  

The static tensions calculated for the mooring without the compensator floats are provided on Table 2.4. Also provided on Table 2.4 is the minimum static tension in the crown lines performed using hydrostatic equations.
Table 2.4: Design static tension for the mooring components.

<table>
<thead>
<tr>
<th>Mooring Component</th>
<th>Design Static Tension (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Line</td>
<td>9.18</td>
</tr>
<tr>
<td>Grid Line</td>
<td>7.31</td>
</tr>
<tr>
<td>Lower Bridle Lines(^a)</td>
<td>~0</td>
</tr>
<tr>
<td>Upper Bridle Lines(^a)</td>
<td>~0</td>
</tr>
<tr>
<td>Anchor Crown Lines(^b)</td>
<td>2.20</td>
</tr>
</tbody>
</table>

\(^a\) Represents the current tension in the line w/o compensator floats  
\(^b\) Anchor crown line tension was calculated using hydrostatics

5. Deployment and Recovery Operations

Two systems were deployed in a north-south orientation in May and June of 1999, as described in Baldwin et al. (2000). Originally, it was planned, for experimental purposes, to keep one cage on the surface while the other was submerged during the summer growout season. Due to the need for keeping the flounder in the warmer surface waters, however, as well as ease in feeding and monitoring fish condition, both cages were normally kept on the surface. The ability to submerge became very useful, though, at the approach of Hurricane Floyd. Both of the cages were lowered and neither suffered any damage. The flounder were harvested during the late fall, 1999. After harvesting, the cages were submerged for the 1999-2000 winter.

In the summer of the second year, the northern fish cage and mooring system was recovered and inspected for damage (Irish et al., 2001). Preparations had been made to instrument the mooring system with nine strategically placed load cells to measure the tension response in storms. The fish cage net was removed and treated with an anti-fouling paint. The clean net was the placed back onto the cage in one of the dry dock facilities in the Portsmouth Naval shipyard and prepared for re-deployment.
CHAPTER 3

FLUID DYNAMIC DRAG MODELING OF THE CENTRAL SPAR CAGE

1. Fish Cage Drag Forces

During the redeployment of the central spar cage, an open ocean tow test was conducted to investigate the drag characteristics of the cage covered with a clean net. The intent of the experiment was to obtain multiple velocity-tension data points, measure the velocity reduction through one panel of clean netting and to compare the results with physical and numerical model tests. Previous physical and numerical model tests described by Fredriksson et al. (2000) and Tsukrov et al. (2000) were performed primarily to develop design loads for the specification of the mooring gear currently deployed. During the modeling efforts, it was discovered that the nets of the cage make up the largest amount of the total surface area. The actual twine of the net consists of knotless nylon having a twine diameter of 2.38 mm and square openings with a side length of 22.23 mm and is approximately 22% of the total surface area that is outlined by the net borders. Since the net has the largest area, it will most likely contribute a large portion of the total cage drag. It was also revealed during these physical and numerical model tests, that the containment netting surrounding the cage was difficult to model. Representing the nets must take into consideration Froude number, Reynolds number, as well as blockage and shadowing characteristics. In this context, “blockage” is defined as the ratio of actual material projected area to the net outline area, while “shadowing” refers to velocity field reduction downstream of a cage component.
2. Open Ocean Drag Test

2.1 Full Scale Line Tension Measurements

The field tow test using the R/V Gulf Challenger was completed using a clean net and with the ballast weight attached to the spar. The ballast weight was close-coupled to the spar to prevent fouling in shallow waters. During this test, a towline length of 55 meters was used to minimize the influence of the towing vessel wake (Figure 3.1). A single 89 kN capacity load cell, developed by engineers at the Woods Hole Oceanographic Institution, was attached just below the A-frame of the R/V Gulf Challenger and connected to the towing line (Irish et al., 2001). It was set to operate continuously recording a point every two seconds. Velocity measurements were made at two locations synchronized using a time clock set at Greenwich Mean Time (GMT). A shipboard Acoustic Doppler Current Profiler (ADCP) was used to collect both ship and current velocity at a depth of 5.12 meters approximately every 30 minutes during the tow. Another set of velocity measurements was obtained from an S4 electromagnetic current meter placed inside the fish cage to investigate net shadowing characteristics. The S4 was strung between the top and bottom net panels 3 meters from the surface and 4 meters behind the rim of the central spar cage (approximately ½ the horizontal distance between the rim and the spar). The S4 was set to measure continuously recording 5 second averages. The Captain of the R/V Gulf Challenger changed engine RPM, when instructed, to obtain velocity and line tension data at different tow velocities.
2.2 Line Tension Processing and Results

Post processing the field data acquired from the tow tests first included organization of the load cell and ADCP data sets with respect to the GMT time stamps. Figure 3.2 shows the load cell time series data (with and without a 20-point filter) and the time locations that the ADCP velocities were acquired, each surrounded by a box on the Figure. The resulting data set was then sorted into 3 velocity bins each containing at least 4 velocity-tension pairings. The velocities and tensions from each bin were averaged to obtain three velocity-load data points (Table 3.1). The error limits provided on Table 3.1 represent ± the maximum standard deviation calculated from the raw line tension data. This is estimated to be the 95.4% confidence level (Taylor, 1982). The error bars for the field test data are rather large giving an indication of the amount of variability that existed in the data. In addition to the load cell measurement variability, at one time during the test, nearly 50 sets of lobster pot gear were fouled in the cage. This could have produced an unsteady tow situation that could have effecteda the velocity measurements and the matching line tension data points.

Figure 3.1: Field tow of the central spar cage with the R/V Gulf Challenger.
2.3 Net Shadowing Measurement Results

The field tow exercise also presented an opportunity to investigate the velocity reduction through the leading net panels of the cage. In addition to the ADCP current velocity measurements taken outside of the cage, an S4 electromagnetic current meter was placed inside the central spar cage to investigate the current speed difference through the net layer. The S4 data was compared to the total relative water velocity measured by
the ADCP at the 5.12 meter depth. The data from each instrument was synchronized based on the GMT time stamps then time averaged for six tests. The percent velocity reductions between the two measurements are provided on Table 3.2.

Table 3.2: Velocity reduction through one net panel.

<table>
<thead>
<tr>
<th>ADCP (m/s)</th>
<th>S4 (m/s)</th>
<th>Percent Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.755</td>
<td>0.593</td>
<td>21.4</td>
</tr>
<tr>
<td>0.740</td>
<td>0.691</td>
<td>6.67</td>
</tr>
<tr>
<td>0.729</td>
<td>0.658</td>
<td>9.70</td>
</tr>
<tr>
<td>0.621</td>
<td>0.544</td>
<td>12.5</td>
</tr>
<tr>
<td>0.589</td>
<td>0.614</td>
<td>-4.23</td>
</tr>
<tr>
<td>0.525</td>
<td>0.455</td>
<td>13.3</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>9.90</td>
</tr>
</tbody>
</table>

3. Physical Modeling

3.1 Scale Modeling Effects and Limitations

After the field tow tests, a physical modeling effort was initiated to replicate the at-sea procedure. To perform physical model tests accurately, prototype components and environmental conditions need to be scaled such that the dominant dynamic processes of the model system are similar. Dimensionless quantities have been developed from the Buckingham PI theorem to represent these dynamic processes. Environmental forces on surface fish cages depend primarily upon Froude and Reynolds numbers (Linfoot and Hall, 1986). The Froude number represents the ratio between inertial and gravitational forces and is typically used in physical modeling at the free surface when wave or wave making forces are dominant in the system. The Reynolds number, on the other hand, is the ratio between inertial and viscous forces and is employed as the basis for dynamic scaling when frictional resistance dominates and the body is fully submerged in currents and or wind. These dimensionless quantities are related to the drag force on a fish cage.
component, such as the nets, by considering,

$$F_{\text{drag}} = \frac{1}{2} \rho_{\text{sw}} AC_D U^2,$$

(3.1)

where $\rho_{\text{sw}}$ is the density of seawater, $A$ is the perpendicular cross sectional area, $U$ is the relative water velocity and $C_D$ is coefficient of drag. Following naval architecture practice (Lewis ed., 1988), it is assumed that wave making and frictional drag are separable. Thus the coefficient of drag,

$$C_D = C_D(\text{Fr}, \text{Re}) = C_w(\text{Fr}) + C_f(\text{Re}),$$

(3.2)

is assumed to be comprised of two primary components, a wave making part $C_w(\text{Fr})$ and a frictional part $C_f(\text{Re})$, which are a function of Froude and Reynolds numbers,

$$\text{Fr} = \left( \frac{U^2}{g D} \right)^{1/2},$$

(3.3)

and

$$\text{Re} = \frac{DU}{\nu},$$

(3.4)

respectively, where $g$ is acceleration due to gravity, $D$ is a characteristic diameter, and $\nu$ is the fluid kinematic viscosity. Therefore, the expression for the total drag force is separated between the wave making and frictional components, as shown below,

$$F_{\text{drag}} = \frac{1}{2} \rho AC_w(\text{Fr})U^3 + \frac{1}{2} \rho AC_f(\text{Re})U^3.$$

(3.5)

In tank testing physical models in water, it is not possible (except at a ratio of 1:1) to match both Froude and Reynolds number because $\text{Fr} \propto U/\sqrt{D}$ and $\text{Re} \propto U \cdot D$. Furthermore, for many hydrodynamic tests, especially surface tow studies, Reynolds scaling is impractical because model tow velocities are larger than the prototype values.
This can be shown by matching Reynolds number at the model and prototype scales, 
\( \text{Re}_m = \text{Re}_p \), and

\[
\left[ \frac{U_m D_m}{\nu_m} \right] = \left[ \frac{U_p D_p}{\nu_p} \right].
\] (3.6)

If the scale factor, \( \lambda = D_p / D_m \), and \( \nu_p / \nu_m = 1 \), the relationship between the velocities at the model and prototype scale would be \( U_m = \lambda U_p \). In addition, it can be shown that model forces using Reynolds scaling are equal to prototype forces, making the construction of the actual model with sufficient integrity difficult. For tests involving surface (gravity) waves, Froude scaling is required for dynamic similitude of wave processes. Some adjustment is made with respect to the frictional contribution, often it is taken to be small, or it is assumed that the Reynolds number dependence is not strong, (that is, \( C_f \) is nearly constant). Since wave drag is generated by surface tows and the drag studies were part of a larger tank-test program, which includes seakeeping experiments (see Chapter 5), Froude scaling was used (thereby avoiding the practical difficulties of Reynolds scaling).

Interpreting the results of these tests, however, must be performed carefully because the Reynolds number at the prototype scale is \( \lambda^{3/2} \) times larger than at the model scale. Systems comprised of mostly nets may have a high Reynolds number dependence that contribute significantly to the overall dynamics. Modeling methods were therefore, adjusted to account for the decrease in Reynolds number at the model scale.

3.2 Experimental Methodology

To examine these fish cage modeling issues, Palcynski (2000) performed an extensive number of UNH tow tank studies investigating net modeling techniques, wave
making drag contributions and net blocking characteristics. One approach used to reduce the Reynolds number discrepancy is to minimize the scale ratio. The cage model used (Figure 3.3) was scaled using a ratio of $\lambda = 15.2$. This model is nearly 33% larger than the one used during the previous tests described in Fredriksson et al. (2000). However, a Reynolds number difference between the prototype and Froude-scale model still existed. A method described by Palczynski (2000), to accommodate the discrepancy involved modifying the projected area of the net.

To summarize this technique, first consider that when using Froude scaling, the total model scale drag force $[F_{\text{drag}}]_M$ and coefficient of drag $[C_D]_M$ can be measured. Just as in the full-scale situation, $[C_D]_M$ has a wave making and frictional component as a function of the Froude and Reynolds numbers, respectively (see equations 3.1 through 3.5). Since, the frictional component of the drag at the model and prototype scales are not equal, $C_f[(Re)_M] \neq C_f[(Re)_p]$ (because of the Reynolds number decrease at the model scale), the total coefficient of drag at the model and prototype scales are not equal. The wave making drag coefficients $C_w$ are, of course, equal at the model and prototype scales because $(Fr)_M = (Fr)_p$. To accommodate these differences, the total projected area of the nets was modified according to,

$$A_{\text{mod}} = \frac{C_f[(Re)_p]}{C_f[(Re)_M]} A_M,$$

where $A_{\text{mod}}$ is the modified net projected area.

Model and prototype Reynolds numbers were calculated using equation (3.4) and values for $C_f[(Re)_M]$ and $C_f[(Re)_p]$ were estimated from empirical data sets for circular cylinders in Roberson and Crowe (1990). A modified projected area, $A_{\text{mod}}$, was then
obtained from equation (3.7).

As described in Palczynski (2000), to fully bracket the range of interest, a relatively low blockage (5.3\%) model net was used (see the specifications in Table 3.3), and tests were conducted with 1-, 2-, 3-, and 4-layers. This allowed blockage effects to be characterized—whether due to prototype net changes or an increase in biological fouling (bio-fouling). Choosing a nominal full-scale velocity of 1 m/s and a representative $v = 9.3 \times 10^{-7} \text{ m}^2/\text{s}$, the full-scale $\text{Re}$ was 3225 which decreased to 105 at the model scale. The corresponding full-scale cylindrical drag coefficient was 0.96, which increased to 1.5 at the model scale. Thus, the geometrically scaled net projected area needed to be decreased by a factor of 0.96/1.50. In terms of blockage, the geometrically scaled model blockage of 22\% needed to be "corrected" to 15\%. Thus, the 3-layer test condition represents experiments with Reynolds number correction for net viscous drag, while 4 layers is nearly the straight, geometrically scaled case and could represent nets with fouling. The other components of the cage were specified according to straight Froude scaling laws (Chakrabarti, 1994). These specifications are provided on Table 3.3, while the model construction details are provided by Palczynski (2000).

![Figure 3.3: Scale model (1:15.2) of the central spar cage without the ballast weight (Palczynski, 2000).](image-url)
### Table 3.3: Central spar cage physical model particulars.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Full-Scale</th>
<th>Model-Scale</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CENTER SPAR</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td>9.14 m</td>
<td>601.6 mm</td>
</tr>
<tr>
<td>Diameter</td>
<td>0.92 m</td>
<td>60.32 mm</td>
</tr>
<tr>
<td>Total Mass</td>
<td>2178 kg</td>
<td>0.624 kg</td>
</tr>
<tr>
<td>Material(^a)</td>
<td>Galvanized Steel</td>
<td>2&quot; PVC Pipe</td>
</tr>
<tr>
<td><strong>RIM SECTIONS (8)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nominal Diameter</td>
<td>14.93 m</td>
<td>1117 mm</td>
</tr>
<tr>
<td>Pipe Diameter</td>
<td>0.273 m</td>
<td>22.24 mm</td>
</tr>
<tr>
<td>Mass</td>
<td>2064 kg</td>
<td>0.508 kg</td>
</tr>
<tr>
<td>Material</td>
<td>Galvanized Steel Pipe Oak Dowels</td>
<td></td>
</tr>
<tr>
<td><strong>HARVEST RING</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thickness</td>
<td>50 mm</td>
<td>3.8 mm</td>
</tr>
<tr>
<td>Diameter</td>
<td>1.88 m</td>
<td>123.8 mm</td>
</tr>
<tr>
<td>Mass</td>
<td>476 kg</td>
<td>0.02 kg</td>
</tr>
<tr>
<td>Material</td>
<td>Galvanized Steel Pipe Oak Dowels</td>
<td></td>
</tr>
<tr>
<td><strong>BALLAST WEIGHT</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Height</td>
<td>0.35 m</td>
<td>23.8 mm</td>
</tr>
<tr>
<td>Diameter</td>
<td>1.98 m</td>
<td>128.6 mm</td>
</tr>
<tr>
<td>Mass</td>
<td>3181 kg</td>
<td>0.825 kg</td>
</tr>
<tr>
<td>Material</td>
<td>Steel reinforced concrete Aluminum</td>
<td></td>
</tr>
<tr>
<td><strong>TOP STAYS</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td>8.8 m</td>
<td>584 mm</td>
</tr>
<tr>
<td>Diameter</td>
<td>13 mm</td>
<td>0.812 mm</td>
</tr>
<tr>
<td>Material</td>
<td>Spectra</td>
<td>Nylon coated wire</td>
</tr>
<tr>
<td><strong>BOTTOM STAYS</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td>7 m</td>
<td>463 mm</td>
</tr>
<tr>
<td>Diameter</td>
<td>13 mm</td>
<td>0.812 mm</td>
</tr>
<tr>
<td>Material</td>
<td>Spectra</td>
<td>Nylon coated wire</td>
</tr>
<tr>
<td><strong>NETTING</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inside Length (knot to knot)</td>
<td>22.23 mm (square)</td>
<td>12.2 mm by 17.3 mm</td>
</tr>
<tr>
<td>Diameter</td>
<td>2.38 mm</td>
<td>0.38 mm</td>
</tr>
<tr>
<td>Material(^b)</td>
<td>Knotless nylon</td>
<td>Monofilament nylon</td>
</tr>
</tbody>
</table>

\(^a\) Spar can be ballasted so that the cage can be at the surface or submerged.

\(^b\) See net construction detail in text.

Using this cage and net model, Palczynski (2000) performed tow tests using layer combinations either to simulate other types of netting or blockage due to bio-fouling. This technique also facilitated the removal of back net panels for comparison with full netting tests. Results of the comparison showed that these back panels are subject to reduced velocities. Tows were conducted at the free surface and submerged. The
submerged tow test results indicated that the wave making drag is a small part of the resistance force.

3.3 **Physical Model Tests to Simulate Field Tow**

Two sets of physical modeling tests were performed using 3- and 4-net layers to simulate the field tow using the clean net with the ballast weight close-coupled to the spar component of the fish cage. The model was attached to the tow carriage, which was operated at speeds ranging between 0.5 and 2.0 m/s (full-scale). The time series data are shown on Figure 3.4. Tow carriage velocities and line tension information was obtained in triplicate and acquired using light gates and a 45 N load cell, respectively. The time-averaged results of the 3- and 4-net layer tow tests are provided in Table 3.4. The error limits, also provided in Table 3.4, represent a bracket equal to ± one standard deviation.

![Figure 3.4: Raw data time series for the physical model tows (full-scale).](image-url)
Table 3.4: Tow test results for the physical model tests (full-scale).

<table>
<thead>
<tr>
<th>3-Panel Tow Vel. (m/s)</th>
<th>Line Tension (kN)</th>
<th>Error (kN)</th>
<th>4-Panel Tow Vel. (m/s)</th>
<th>Line Tension (kN)</th>
<th>Error (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.497</td>
<td>5.491 ± 0.212</td>
<td></td>
<td>0.497</td>
<td>5.619 ± 0.260</td>
<td></td>
</tr>
<tr>
<td>0.745</td>
<td>10.49 ± 0.384</td>
<td></td>
<td>0.745</td>
<td>11.34 ± 0.702</td>
<td></td>
</tr>
<tr>
<td>0.999</td>
<td>18.07 ± 0.730</td>
<td></td>
<td>0.989</td>
<td>18.82 ± 1.005</td>
<td></td>
</tr>
<tr>
<td>1.123</td>
<td>21.92 ± 1.400</td>
<td></td>
<td>1.117</td>
<td>23.27 ± 1.567</td>
<td></td>
</tr>
<tr>
<td>1.483</td>
<td>37.09 ± 1.137</td>
<td></td>
<td>1.483</td>
<td>39.34 ± 1.477</td>
<td></td>
</tr>
<tr>
<td>1.706</td>
<td>47.78 ± 1.927</td>
<td></td>
<td>1.714</td>
<td>50.05 ± 2.340</td>
<td></td>
</tr>
<tr>
<td>1.934</td>
<td>60.06 ± 1.757</td>
<td></td>
<td>1.936</td>
<td>63.66 ± 1.741</td>
<td></td>
</tr>
</tbody>
</table>

4. Numerical Modeling

4.1 The AquaFE Model

Numerical model simulations, using the AquaFE computer program developed at UNH, were also performed in conjunction with the physical model tests. The AquaFE model is based on the Finite Element Analysis Program (FEAP) originally programmed by Professor R.L. Taylor from the Department of Civil Engineering at the University of California, Berkeley. Wave and current loading on truss elements were incorporated into the model using a Morison equation formulation (Morison et al., 1950) for use with aquaculture net pen systems by Gosz et al. (1996). This computer model was successfully used in support of the OOA demonstration project for the design and evaluation of fish cage and mooring systems currently deployed at the site (Ozbay, 1999 and Tsukrov et al., 2000).

The core finite element code is written in FORTRAN, incorporating truss, buoy and massless elements to model various parts of the net pen/grid mooring system. A nonlinear Lagrangian formulation is employed to account for large displacements of structural elements. In addition, the unconditionally stable Newmark direct integration
scheme is adopted to solve the nonlinear equations of motion.

Since component elements are typically circular in cross-section, hydrodynamic forces on the structural elements are calculated using the Morison equation modified to account for relative motion between the structural element and the surrounding fluid. The fluid force vector per unit length can be represented as,

\[ F = \frac{1}{2} \rho C_n D |(u - \dot{x})_n| + C_t (u - \dot{x})_t + \rho \frac{\pi D^2}{4} a_n + \rho (C_M - 1) \frac{\pi D^2}{4} (a - \ddot{x})_n, \]  

where \( \rho \) is the density of the fluid; \( u \) and \( a \) are the wave particle velocity and acceleration vectors according to Airy wave theory described by Dean and Dalrymple (1991); \( \dot{x} \) and \( \ddot{x} \) are the structural component velocity and acceleration vectors; \( D \) and \( A \) are the diameter and the cross-sectional area of the element in the deformed configuration; \( C_n \) and \( C_t \) are the normal and tangential drag coefficients (the subscripts \( n \) and \( t \) denote the normal and tangential components, respectively) and \( C_M \) is the inertia coefficient.

According to Sarpkaya and Isaacson (1981), for irrotational flow, accelerating around a stationary cylinder (or truss element), \( C_M \) is typically equal to two. The numerical procedure has two options for representing the normal and tangential drag coefficients, \( C_n \) and \( C_t \), respectively. The first option is standard user input, which remains constant for each time step, regardless of the Reynolds number. The second option utilizes a method obtained from Choo and Casarella (1971) that updates the drag coefficients based on the Reynolds number defined as

\[
C_n = \begin{cases} 
\frac{8\pi}{Re_n s} \left( 1 - 0.87s^{-2} \right) & (0 < Re_n \leq 1) \\
1.45 + 8.55 Re_n^{-0.90} & (1 < Re_n \leq 30) \\
1.1 + 4 Re_n^{-0.50} & (30 < Re_n \leq 10^5)
\end{cases}
\]

(3.9)
\[ C_r = \pi \mu \left( 0.55 \Re_n^{1/2} + 0.084 \Re_n \right), \quad (3.10) \]

where

\[ s = -0.077215665 + \ln \left( \frac{8}{\Re_n} \right) \quad (3.11) \]

and \( \rho_w \) and \( \mu \) are the mass density and dynamic viscosity of the water. For the simulations performed as part of this study, the Choo and Casarella (1971), method is utilized to obtain \( C_n \) and \( C_t \). Equation (3.8) is known to adequately predict the hydrodynamic force on a submerged element whose diameter is small compared to the length of the wave (Haritos and He, 1992; Webster, 1995).

### 4.2 Net and Cage Modeling

The nets used in the numerical model cage were simulated using the equivalent truss method described in Tsukrov et al. (2000). Using this method, care is taken to consider the dominant processes such as the drag forces, buoyancy, component stiffness and inertia. The nets in each of the 8 upper and lower panels were represented by 6 truss elements (\( h_i \), where \( i = 1 \ldots 6 \)), where the total length for each panel is given by

\[ h_{eq} = \sum_{i=1}^{6} h_i. \quad (3.12) \]

By knowing the actual projected area of the net, an equivalent net diameter (\( D_{eq} \)) can be calculated, so that projected area between the actual and model net are equal. The mass density of the net was also adjusted so that the total buoyant forces remained equal. This equivalent density parameter (\( \rho_{eq} \)), was found using the following expression,

\[ \rho_{eq} = \rho_w \left( \frac{4 F_{nb}}{\pi D_{eq}^2 h_{eq} g} \right), \quad (3.13) \]

where \( \rho_w \) is the mass density of water and \( F_{nb} \) is the actual buoyant force of the net. The
modulus of elasticity of the equivalent material was chosen to have the same stiffness as the net as described in Tsukrov et al. (2000). The inertia of the truss structure is dependant upon the mass density and equivalent volume, the same parameters as the total buoyancy force. Thus, it is impossible to satisfy two characteristic requirements. For these tests, the buoyancy is considered dominant in the overall dynamic behavior of net structures. The inertia of the net is negligible as compared with the inertia of other parts of the fish cage/mooring grid system as described in Tsukrov et al. (2000). Components of the central spar cage were modeled using truss elements that were sized and orientated such that the cross-sectional areas in the vertical planes closely match that of the actual part. The material properties for the central spar cage model are provided in Table 3.5.

Table 3.5: Material properties used in the model simulations.

<table>
<thead>
<tr>
<th>Component</th>
<th>Mass Density (kg/m³)</th>
<th>Modulus of Elasticity (Pa)</th>
<th>Cross Sectional Area (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rim</td>
<td>714</td>
<td>1.817 x 10⁸</td>
<td>5.850 x 10⁻²</td>
</tr>
<tr>
<td>Spoke Lines</td>
<td>960</td>
<td>1.030 x 10⁸</td>
<td>1.327 x 10⁻³</td>
</tr>
<tr>
<td>Spar</td>
<td>708</td>
<td>8.598 x 10⁷</td>
<td>6.647 x 10⁻¹</td>
</tr>
<tr>
<td>Lower Bridle</td>
<td>940</td>
<td>1.830 x 10⁷</td>
<td>1.810 x 10⁻³</td>
</tr>
<tr>
<td>Net</td>
<td>1027</td>
<td>1.030 x 10⁷</td>
<td>1.798 x 10⁻²</td>
</tr>
<tr>
<td>Mass-less</td>
<td>n.a.</td>
<td>1.030 x 10¹¹</td>
<td>1.798 x 10⁻³</td>
</tr>
<tr>
<td>Ballast Weight</td>
<td>2000</td>
<td>2.500 x 10⁶</td>
<td>2.112 x 10⁻¹</td>
</tr>
<tr>
<td>Harvest ring part #1</td>
<td>1025</td>
<td>2.500 x 10⁶</td>
<td>1.340 x 10⁻²</td>
</tr>
<tr>
<td>Harvest ring part #2</td>
<td>1025</td>
<td>2.500 x 10⁶</td>
<td>6.360 x 10⁻¹</td>
</tr>
<tr>
<td>Harvest ring part #3</td>
<td>1025</td>
<td>2.500 x 10⁶</td>
<td>1.070 x 10⁻¹</td>
</tr>
<tr>
<td>Shaded Net</td>
<td>1027</td>
<td>1.030 x 10⁹</td>
<td>1.798 x 10⁻²</td>
</tr>
<tr>
<td>Shaded Spar</td>
<td>708</td>
<td>8.598 x 10⁹</td>
<td>6.647 x 10⁻¹</td>
</tr>
<tr>
<td>Shaded Rim</td>
<td>714</td>
<td>1.817 x 10⁸</td>
<td>5.850 x 10⁻²</td>
</tr>
</tbody>
</table>

4.3 Numerical Model Test to Simulate Field Tow

The field tow configuration was investigated using the AquaFE model program with the ballast weight close-coupled to the spar component of the cage (Figure 3.5). The tow point was placed 3 meters above the surface interface and the towline length was 55 meters. The simulation was performed such that 2 meters of the freeboard existed.
Seven simulations were performed with the current velocity setting ranging between 0.5 and 1.25 m/s.

![Diagram of FEA model test to simulate field tow (initial and steady condition).](image)

**Figure 3.5: FEA model test to simulate field tow (initial and steady condition).**

Steady state tensions and error values are listed in Table 3.6 and the line tension results as a function of time are shown in Figure 3.6 (results are compared with the field tow and physical model tests in the next section). The error for the numerical model test was calculated by subtracting the maximum value in the time series by the steady state mean. This error calculation method was chosen because the most variability of the numerical calculation occurs in transient portion of the simulation.

**Table 3.6: Tow test results for the numerical model tests.**

<table>
<thead>
<tr>
<th>Tow Velocity (m/s)</th>
<th>Line Tension (N)</th>
<th>Error (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.500</td>
<td>8093 ± 1480</td>
<td></td>
</tr>
<tr>
<td>0.547</td>
<td>9677 ± 919</td>
<td></td>
</tr>
<tr>
<td>0.678</td>
<td>14787 ± 14.6</td>
<td></td>
</tr>
<tr>
<td>0.788</td>
<td>19820 ± 5.23</td>
<td></td>
</tr>
<tr>
<td>0.834</td>
<td>22130 ± 4.36</td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td>31412 ± 1.70</td>
<td></td>
</tr>
<tr>
<td>1.25</td>
<td>48115 ± 1.60</td>
<td></td>
</tr>
</tbody>
</table>
5. Discussion of Drag Modeling Results

The results of the physical model and numerical model tests are plotted and compared with the results from the actual field tow on Figure 3.7. In general, the field tow results were somewhat noisy. Future full-scale tow tests should be conducted in a more controlled manner (if possible). Each of the physical model tests, using the 3- and 4-net layer configurations (with Re number adjustment), underpredicted the field results by approximately 30%. This suggests that decreasing net friction in an effort to compensate for Reynolds number effects may not be desirable. It should be noted, however, that physical model data and an extrapolation of the field tow appear to converge at 1 m/s (see the blue dashed curve on Figure 3.7). This speed was taken at the nominal speed for the Reynolds number adjustment calculation. Future work to perform Reynolds number adjustment over a range of tow velocities needs to be investigated.
The numerical model results in this velocity range, on the other hand, cross the field data points but have a steeper slope. This is most likely because the AquaFE model calculates the forces on cage elements without net shadowing considerations. This result is consistent with the previous interpretation drawn from the physical model drag force measurements with and without back net panels removed as described in Palczynski (2000). In addition, an approximate 10% velocity reduction was also found when ADCP measurements outside of the cage were compared with the S4 measurements inside of the cage during the field tow (Table 3.2) providing further evidence of the shadowing effect. These values are also consistent with results from Aarsnes (1990) using nets of similar dimensions.
Since the existing numerical procedure did not calculate differences in velocity due to shadowing, a new element for the AquaFE model was developed to take into consideration the shadowing effects that occur behind up-flow net panels. Using this element, certain components of the cage that are known to be in the wake were chosen such that during a numerical simulation, a reduction in velocity can be applied. A new finite element model of the central spar cage was built that included the capability to shadow downstream components (Figure 3.8). A comparison between model predictions with various levels of shadowing and field data is shown in Figure 3.9. Agreement clearly improves with moderate levels of shadowing within the range of the field data.

Figure 3.8: Central spar cage model with shaded net, spar and rim elements.
Figure 3.9: Numerical model tests using the shaded elements.
1. Theoretical Review

Unlike steady drag loads due to currents, wave forces on a fish cage and mooring system are more complex due to their stochastic nature. According to Goda (2000), surface waves are often the most dominating forcing mechanisms on offshore structures. Since it is difficult to obtain explicit mathematical expressions for these time series, a statistical approach is taken to investigate physical and numerical modeling methods with field results. In this investigation, analysis is performed in the frequency domain using an assumed linear stochastic systems model. Relationships called linear transfer functions are developed between the forcing mechanisms and the system response. The random forcing mechanisms are the surface waves and the response variables consist of the cage heave, surge and pitch motions, as well as the lower bridle and anchor line tensions.

Auto- and cross-density functions are used to investigate system characteristics in the frequency domain. The theoretical development of these tools begins with time series measurements and the auto- and cross-correlation functions. The auto-correlation function \( R_{xx}(\tau) \) can be expressed as the ensemble average of the product of two readings (or measurements) of the same time record separated by a time distance of \( \tau \),

\[
R_{xx}(\tau) = E[x(t) \cdot x(t + \tau)] \tag{4.1}
\]

For a stationary, ergodic process, the auto-correlation may be evaluated by time...
averaging over the record length (from -T to +T) according to

$$R_{xx}(\tau) = \lim_{T \to \infty} \frac{1}{2T} \int_{-T}^{T} x(t) \cdot x(t + \tau) \, dt.$$  \hspace{1cm} (4.2)

A stationary process has statistical properties such that the moments of the process distribution do not change over time. An ergodic process is one in which all of the statistics associated with the ensemble can be obtained from a single time series. Ocean wave forcing and the system response are considered weak, stationary and ergodic processes (Ochi, 1998). A weak stationary process is one where the mean value is constant and the auto-correlation is dependent on the time displacement (\(\tau\)) only, (Bendat and Peirsol, 1985).

The Fourier transform can be utilized to move between the time (t) and frequency (f) domain and is defined by the following expressions

$$X(f) = \int_{-\infty}^{\infty} x(t)e^{-i2\pi ft} \, dt$$  \hspace{1cm} (4.3)

and

$$x(t) = \int_{-\infty}^{\infty} X(f)e^{i2\pi ft} \, df$$  \hspace{1cm} (4.4)

By taking the Fourier transform of equation (4.2), a two sided, auto-spectral density function \(P_{xx}(f)\) continuous for f between \((-\infty\) and \(\infty\)) is obtained such that

$$P_{xx}(f) = \int_{-\infty}^{\infty} R_{xx}(\tau)e^{-i2\pi ft} \, d\tau.$$  \hspace{1cm} (4.5)

Similarly, for two separate time series, \(x(t)\) and \(y(t)\), a cross-correlation expression, \(R_{xy}(\tau)\), can be defined by,

$$R_{xy}(\tau) = E[x(t) \cdot y(t + \tau)].$$  \hspace{1cm} (4.6)
Applying the same weakly stationary and ergodic assumptions,

$$R_{xy}(\tau) = \lim_{T \to \infty} \frac{1}{T} \int_{-T}^{T} x(t) y(t+\tau) dt.$$  \hspace{1cm} (4.7)

Likewise, the Fourier transform of equation (4.7) yields the two sided cross-spectral density function,

$$P_{xy}(f) = \int_{-\infty}^{\infty} R_{xy}(\tau) e^{-j2\pi f \tau} d\tau.$$ \hspace{1cm} (4.8)

In this case, $x(t)$ could represent the ocean surface elevation time series forcing and $y(t)$ the motion and or force/load response of a particular component of the fish cage and mooring system. The two sided auto- and cross-spectral density functions are continuous for all frequencies between $-\infty$ and $\infty$. In practice, however, measurements are made between 0 and $\infty$ and therefore the one sided auto- and cross-spectral density functions are used, where

$$S_{xx}(f) = 2P_{xx}(f) \quad 0 \leq f \leq \infty$$ \hspace{1cm} (4.9)

and

$$S_{xy}(f) = 2P_{xy}(f) \quad 0 \leq f \leq \infty$$ \hspace{1cm} (4.10)

Equations (4.2), (4.4) and (4.9) can be combined for the case when $\tau=0$ to form the variance,

$$\frac{1}{2T} \int_{-T}^{T} x^2(t) dt = \int_{0}^{\infty} S_{xx}(f) df.$$ \hspace{1cm} (4.11)

Unlike the one sided auto spectral density function, $S_{xx}(f)$, the cross spectral density function, $S_{xy}(f)$, has both real and complex components,

$$S_{xy}(f) = C_{xy}(f) - iQ_{xy}(f),$$ \hspace{1cm} (4.12)
called the coincident spectral density function (co-spectrum) and the quadrature spectral density function (quad-spectrum), respectively. The absolute value and the phase angle can then be determined by

\[ |S_{xy}(f)| = \left[ C_{xy}^2(f) + Q_{xy}^2(f) \right]^{1/2} \]  

(4.13)

and

\[ \theta_{xy}(f) = \tan^{-1} \frac{Q_{xy}(f)}{C_{xy}(f)}, \]  

(4.14)

respectively.

2. Input/Output Systems

The relationship between the input and the output of a constant parameter linear system in the time domain can be represented by a weighting function \( h(\tau) \). A constant parameter system is one that has mechanical properties (stiffness, damping, added mass, etc.) that are invariant with time. In the case of the fish cage mooring system time series variables, \( x(t) \) and \( y(t) \) describe the input forcing (waves) and the system response (fish cage motion and mooring line tension), respectively. The weighting function is used in the following convolution integral

\[ y(t) = \int_{0}^{\infty} h(\tau) x(t - \tau) d\tau, \]  

(4.15)

where \( h(\tau) = 0 \) for \( \tau < 0 \). The products \( y(t)y(t + \tau) \) and \( x(t)y(t + \tau) \) yield the following input/output auto- and cross correlation relationships

\[ R_{xx}(\tau) = \int_{0}^{\infty} h(\alpha) h(\beta) R_{xx}(\tau + \beta - \alpha) \alpha d\alpha d\beta \]  

(4.16)

and
\[ R_y(x) = \int_{0}^{x} h(\alpha) R_y(x - \beta) d\alpha. \quad (4.17) \]

where \( \alpha \) and \( \beta \) are separate time shift variables. As described in Bendat and Piersol, (1985), taking the Fourier transform of equations (4.16) and (4.17), and after algebraic manipulation, the following linear transfer function relationships are obtained,

\[ S_{xy}(f) = |H(f)|^2 S_{xx}(f) \quad (4.18) \]

and

\[ S_{xy}(f) = H(f) S_{xx}(f). \quad (4.19) \]

In equations (4.18) and (4.19), the linear transfer function is defined as the Fourier transform of \( h(t) \),

\[ H(f) = \int_{0}^{\infty} h(\tau) e^{-j2\pi f \tau} d\tau. \quad (4.20) \]

In equation (4.18), only the amplitude (gain) portion of the transfer function is maintained. Since \( S_{xy} \) is a complex quantity, while \( S_{xx} \) is real, \( H \) has both an amplitude or gain factor \( |H(f)| \) and a phase factor \( \theta(f) \). Note that in these relationships, \( x(t) \) and \( y(t) \) refer to input and output without the addition of noise.

The one sided auto- and cross-spectral density functions are also used to define the coherence-squared function (coherency),

\[ \gamma_{xy}^2(f) = \frac{|S_{xy}(f)|^2}{S_{xx}(f)S_{yy}(f)}. \quad (4.21) \]

The coherence-squared function, as described by Bendat and Peirsol, (1985), is used to find the fractional component of the variance of the output \( y(t) \), contributed by the input \( x(t) \) at frequency \( f \).
The previous theoretical discussion involves all continuous functions. Analysis procedures, on the other hand, deal with discrete sets of data. Input forcing and response spectra can be defined using a finite-range version of the Fourier transform (equation 4.3), described as

\[ X(f, T) = \int_0^T x(t) e^{-i2\pi ft} dt \]  

and

\[ Y(f, T) = \int_0^T y(t) e^{-i2\pi ft} dt . \]  

respectively. If the forcing and response time series, \( x(t) \) and \( y(t) \), are sampled at increments equal to \( \Delta t \) for a duration of \( N \), the discrete data values are,

\[ x_n = x(n\Delta t) \quad n = 0, 1, 2, \ldots, N-1 , \]  

and

\[ y_n = y(n\Delta t) \quad n = 0, 1, 2, \ldots, N-1 . \]  

Therefore the discrete versions of equations (4.22) and (4.23) are

\[ X(f, T) = \Delta t \sum_{n=0}^{N-1} x_n e^{-i2\pi fn\Delta t} \]  

and

\[ Y(f, T) = \Delta t \sum_{n=0}^{N-1} y_n e^{-i2\pi fn\Delta t} \]  

where computation is performed typically with proven numerical programs called Fast Fourier Transforms (FFTs) using a set of frequencies,

\[ f = f_k = \frac{k}{T} = \frac{k}{N\Delta t} \quad k = 0, 1, 2, \ldots, N-1 . \]
Therefore, the two-sided auto- and cross-spectral densities can be calculated using

\[ P_{xx}(f, T, k) = \frac{1}{N\Delta t} X_k(f, T) X_k^*(f, T), \]  
\[ P_{xy}(f, T, k) = \frac{1}{N\Delta t} Y_k(f, T) Y_k^*(f, T) \]  

and

\[ P_{yy}(f, T, k) = \frac{1}{N\Delta t} X_k(f, T) Y_k^*(f, T), \]

where \( X_k^*(f, T) \) and \( Y_k^*(f, T) \) is the complex conjugate of the FFT of the measured input forcing and response time series. The one sided spectra equivalent for each of these expressions are found using equation (4.9) and (4.10).

**3. Data Processing Techniques**

**3.1 Regular Wave Tests**

Regular wave tests were conducted using both physical and numerical modeling methods at ten deterministic wave frequencies. The characteristics of these waves were approximated using linear (Airy) wave theory as described by Dean and Dalrymple (1991). Linear wave theory uses a velocity potential approach assuming

- waves with small amplitudes, compared to the depth,
- no viscosity with negligible boundary layer effects,
- irrotational flow,
- incompressible fluid,
- two dimensional form and
- a flat bottom.

The velocity potential, \( \phi = \phi(x, z, t) \) is used to obtain the \( u \) (horizontal) and \( w \) (vertical) components of the velocity vector by taking the spatial derivatives,

\[ u = -\frac{\partial \phi}{\partial x} \]
and

$$w = -\frac{\partial \phi}{\partial z}, \quad (4.33)$$

respectively. Using these assumptions, the solution to the velocity potential must satisfy the continuity equation.

$$\frac{\partial u}{\partial x} + \frac{\partial w}{\partial z} = 0 \quad (4.34)$$

so

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0. \quad (4.35)$$

Equation (4.35) is the two-dimensional, incompressible, Laplace equation. From Newton's second law, it can be shown that the equation of motion for inviscid, irrotational flow with a constant density fluid ($\rho$) and pressure ($P$) can be put in the form of the Bernoulli equation,

$$-\frac{\partial \phi}{\partial t} + \frac{1}{2}(u^2 + w^2) + \frac{P}{\rho} + gz = C(t), \quad (4.36)$$

where $z$ is vertical upwards. If the constant of integration, $C(t)$ is taken to be zero and the higher order terms are neglected, the linearized, time dependent Bernoulli equation becomes,

$$-\frac{\partial \phi}{\partial t} + \frac{P}{\rho} + gz = 0, \quad (4.37)$$

which is used with the Laplace equation, subject to the following boundary conditions (summarized),

- Bottom Boundary Condition: no flow through the bottom,
- Dynamic Free Surface Boundary Condition: pressure at the surface is zero (gage),
- Kinematic Free Surface Boundary Condition: no flow through the free surface.
The solution describing progressive waves is obtained by superimposing two standing wave solutions and applying the boundary conditions resulting in the following velocity potential ($\phi$), surface elevation ($\eta$) and the dispersion relation expressions,

$$\phi = -\frac{H}{2} \frac{g}{\omega} \frac{\cosh k(d+z)}{\cosh kd} \sin(\kappa x - \omega t), \quad (4.38)$$

$$\eta = \frac{H}{2} \cos(\kappa x - \omega t) \quad (4.39)$$

and

$$\omega^2 = gk \tanh(kd) \quad (4.40)$$

where:

- $H$ is the wave height,
- $\omega$ is the radian wave frequency equal to $2\pi f$ and $f$ is the frequency,
- $k$ is the wave number equal to $2\pi/L$ and $L$ is the wavelength,
- $d$ is the water depth and
- $g$ is the gravitational constant.

In this study, the linear transfer function magnitude calculated for the deterministic waves are referred to as response amplitude operators (RAOs). The RAOs are obtained by dividing the amplitude of the response by the amplitude of the forcing. The heave, surge, pitch and load response RAOs are defined in the study as the following,

- **Heave RAO**: heave amplitude/wave elevation amplitude,
- **Surge RAO**: surge amplitude/wave excursion amplitude,
- **Pitch RAO**: pitch amplitude/wave slope amplitude,
- **Anchor Line Tension RAO**: anchor line tension amplitude/wave elevation amplitude, and
- **Bridle Line Tension RAO**: bridle line tension amplitude/wave elevation amplitude.

The wave slope amplitude is obtained by taking the partial derivative of equation (4.39) with respect to $x$ and dropping the oscillating term.
Wave Slope Amplitude

\[ Wave\ Slope\ Amplitude = \frac{kH}{2} \] (4.41)

The wave excursion amplitude is defined as the horizontal semi-axis of the ellipse formed by the water particle trajectory (at the surface). It is found by taking the partial derivative of equation (4.38) with respect to \( x \) to obtain the horizontal velocity component. Integrating this result with respect to time (\( t \)), and dropping the oscillating term, gives the wave excursion amplitude,

\[ Wave\ Excursion\ Amplitude = \frac{H}{2} \frac{\cosh kd}{\sinh kd} \] (4.42)

3.2 Random Wave Tests

Physical and numerical model tests were also conducted using random waves, which are more characteristic of actual open ocean waves than the deterministic representation. Random waves (and the system response) are described as a spectrum in the frequency domain in terms of units proportional to energy per frequency band. The wave elevation auto-spectrum is typically described by the significant wave height and the dominant wave period. In a statistical sense, the significant wave height is denoted by \( H_{\text{mo}} \) and is calculated from the zeroth moment of the spectrum,

\[ m_j = \int f S(f) df \quad \text{where} \quad j = 0 \] (4.43)

and \( S(f) \) is the one sided wave elevation auto-spectral density. The zeroth moment of the spectrum is also the area under the spectral curve equal to the variance (from equation 4.11). If the spectrum is narrow banded and the wave heights follow a Rayleigh probability distribution (Ochi, 1998), the \( H_{\text{mo}} \) is obtained from,

\[ H_{\text{mo}} = 4\sqrt{m_2} \] (4.44)
In deep water, the $H_{m0}$ is approximately equal to the $H_{1/3}$, which is the average of the top third wave heights (SPM, 1984). The dominant wave period, $T_p$, is one over the frequency where the maximum energy in a spectrum occurs.

For the random wave tests, linear transfer functions were calculated as a function of frequency using auto- and cross-spectral methods. In the frequency domain, the cage motion and mooring system forcing can be described in terms of energy density $(m^2/Hz)$ as the

- $S_{zz}(f)$: Wave elevation auto-spectrum,
- $S_{xx}(f)$: Wave excursion auto-spectrum and
- $S_{θθ}(f)$: Wave slope auto-spectrum.

The wave elevation auto-spectrum is calculated from the measured times series using equations (4.29) and (4.9). The wave excursion and slope auto-spectra are calculated from the wave elevation auto-spectrum using the following relationships.

$$S_{xx}(f) = S_{zz}(f) \cdot \tanh(kd)^2$$  \hspace{1cm} (4.45)

and

$$S_{θθ}(f) = S_{zz}(f) \cdot (k)^2,$$  \hspace{1cm} (4.46)

respectively.

Likewise, the auto-spectral motion response in heave, surge and pitch, as well as the tension response for anchor and bridle lines time series, are calculated using equations (4.30) and (4.9) as a function of frequency,

- $S_{hh}(f)$: Heave response auto-spectrum $(m^2/Hz)$,
- $S_{ss}(f)$: Surge response auto-spectrum $(m^2/Hz)$,
- $S_{pp}(f)$: Pitch response auto-spectrum $(m^2/Hz)$,
- $Sa_a(f)$: Anchor line tension response auto-spectrum $(kN^2/Hz)$, and
- $Sb_b(f)$: Bridle line tension response auto-spectrum $(kN^2/Hz)$.
The cross-spectral response between the wave elevation and heave, surge, pitch, anchor line tension and bridle line tension time series are found using equations (4.31) and (4.10),

\[ S_{\zeta}\zeta(f) : \text{Heave response to wave elevation cross-spectrum} \quad (m^2/Hz), \]
\[ S_{\xi}\xi(f) : \text{Surge response to wave elevation cross-spectrum} \quad (m^2/Hz), \]
\[ S_{\phi}\phi(f) : \text{Pitch response to wave elevation cross-spectrum} \quad (m^2/Hz), \]
\[ S_{a}a(f) : \text{Anchor tension response to wave elevation cross-spectrum} \quad (m-kN/Hz), \]
\[ S_{b}b(f) : \text{Bridle tension response to wave elevation cross-spectrum} \quad (m-kN/Hz). \]

To obtain the linear transfer function using the auto-spectral technique between the forcing and the response, the following calculations are made.

\[
H_h(f) = \left[ \frac{S_{hh}(f)}{S_{\zeta\zeta}(f)} \right]^{\frac{1}{2}} 
\]

(4.47)

\[
H_s(f) = \left[ \frac{S_{ss}(f)}{S_{\xi\xi}(f)} \right]^{\frac{1}{2}} 
\]

(4.48)

\[
H_p(f) = \left[ \frac{S_{pp}(f)}{S_{\phi\phi}(f)} \right]^{\frac{1}{2}} 
\]

(4.49)

\[
H_a,(f) = \left[ \frac{S_{aa}(f)}{S_{\zeta\zeta}(f)} \right]^{\frac{1}{2}} 
\]

(4.50)

\[
H_b,(f) = \left[ \frac{S_{bb}(f)}{S_{\xi\xi}(f)} \right]^{\frac{1}{2}} 
\]

(4.51)

where:

\[ H_h(f) : \text{Heave transfer function}, \]
\[ H_s(f) : \text{Surge transfer function}, \]
\[ H_p(f) : \text{Pitch transfer function}, \]
\[ H_a,(f) : \text{Anchor line tension auto spectrum and} \]
\[ H_b,(f) : \text{Bridle line tension auto spectrum}. \]
The linear transfer functions were also obtained using cross-spectral density functions of wave elevation input forcing and output tension response. Following equation (4.19), these transfer functions can be evaluated as,

\[
H_a(f) = \frac{S_{y_y}(f)}{S_{yy}(f)}
\]  

(4.52)

\[
H_b(f) = \frac{S_{y_b}(f)}{S_{yy}(f)}
\]  

(4.53)

The advantage of using this method is that the phase between the input and output is maintained. Recall that the cross-spectral density \(S_{xy}\) is made up of real and complex components called the coincident \(C_{xy}\) and the quadrature \(Q_{xy}\) spectral density functions and a phase angle, as described by equation (4.12) and (4.14), respectively. Similarly, the phase information between the forcing and response mechanism described above can be represented as

\[
\theta_y(f): \text{Phase between the wave elevation and heave response.}
\]

\[
\theta_s(f): \text{Phase between the wave elevation and surge response.}
\]

\[
\theta_p(f): \text{Phase between the wave elevation and pitch response.}
\]

\[
\theta_{a_y}(f): \text{Phase between the wave elevation and anchor line tension response and}
\]

\[
\theta_{b_y}(f): \text{Phase between the wave elevation and bridle line tension response.}
\]

The cross-spectral density can also be used to calculate the linear coherency of the forcing and response. The coherency is the ratio between the expected and measured responses (Linfoot and Hall, 1986), as described by equation (4.21). The coherency between the same forcing and response mechanisms is described as

\[
\gamma^2_y(f): \text{Coherency between the wave elevation and heave response,}
\]

\[
\gamma^2_s(f): \text{Coherency between the wave elevation and surge response,}
\]

\[
\gamma^2_p(f): \text{Coherency between the wave elevation and pitch response,}
\]

\[
\gamma^2_a(f): \text{Coherency between the wave elevation and anchor tension response,}
\]

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\( \gamma^2 b_\phi(f) \): Coherency between the wave elevation and bridle tension response.

Coherency values close to one indicate a strong influence between the input forcing and the output response. According to Chakrabarti (1994), reasons for values below one can be attributed to the following:

1. Measurement noise.
2. Resolution bias errors in the spectral estimates.
3. The system relationship between the input and output is non-linear.
4. Other inputs exist.

### 3.3 Field Data Processing

The ocean waves at the demonstration site are obviously stochastic in nature. Data collected as part of the field program, to be further described in Chapter 7, included surface waves, central spar cage motion response in heave, surge and pitch, as well as anchor and bridle line tension. In general, the data processing techniques involved the utilization of the auto-spectral techniques previously described. Cross-spectral density functions, coherency and phase calculations were not performed between the surface wave forcing and the response parameters because wave measurements were collected at a point a considerable distance away from the central spar cage. Spatial randomness changes the wave height time series between the locations so the actual forcing at the cage is not known. The auto-spectral representations are assumed the same and therefore these techniques are applied.
CHAPTER 5

MOTION AND LOAD FREQUENCY RESPONSE USING PHYSICAL MODELS

1. Physical Modeling in the UNH Wave Tank

Designing fish cage and mooring systems, not only to withstand the loads due to waves, but also to optimize the motion response characteristics, is necessary for the survival of mechanical components and the contained fish. Physical model tests can provide an alternative to the high cost and risk of prototype system sea trials to examine mooring system and fish cage response. This method was used extensively as part of the design and evaluation process of potential designs for deployment at the open ocean aquaculture demonstration site (Fredriksson et al., 1999a; Fredriksson et al., 1999b; Fredriksson et al., 2000) as described in Chapter 2. One of the results of these tests was the development of design loads used in the specification of components. Often, certain processes can be observed while using physical models that are not simulated in numerical and analytical approaches. Scale limitations exist, however, such as the Froude and Reynolds number issues discussed in Chapter 3. The physical model tests described in this chapter incorporate some of the lessons learned in past studies and introduce a more comprehensive modeling procedure.

Tests were conducted in the UNH tow/wave basin (Figure 5.1) using a Froude-scaled representation of the northern deployed fish cage and mooring system. The central spar cage model was the same one used as part of the drag tests previously described utilizing the Reynolds number adjustment technique (with three model net panels). The
tests were performed using both regular and random wave conditions. Fish cage motion response in heave, surge and pitch were measured and frequency dependant linear transfer functions calculated. Mooring line tension was also measured at the anchor and bridle lines and transfer functions calculated. Analysis was performed in the frequency domain to examine the system dynamics and for comparison with numerical and in-situ data sets. In addition, resonant conditions were investigated so the use of the configuration currently deployed is optimized.

![Figure 5.1: UNH wave basin facility.](image)

### 2. Physical Modeling Procedure

#### 2.1 Central Spar Cage Physical Model

The models and conditions used during the design investigation were Froude-scaled using a model to prototype ratio of 1:22.5, which represented the ratio between the wave tank depth to the demonstration site depth. This ratio was chosen so that the wave forcing and mooring system geometry would be properly simulated. More recent
experiments have indicated that a significant Reynolds number (ratio of viscous to inertial forces) dependence due to the net component of the cage may exist (Palczynski, 2000). Since both Froude and Reynolds numbers cannot be simultaneously satisfied in water (except at a scale 1:1), measured Froude-scale forces cannot be directly scaled to estimate full size values. The Palczynski (2000) tests employed a larger Froude-scale model (1:15.2) and an adjustment technique (see Chapter 3) to help compensate for the Reynolds number influence. The 1:15.2 scale model is shown on Figure 5.2 through the observation window in the side of the tow/wave basin. The physical model experiments described in this chapter use the 1:15.2 Froude-scale model of the central cage with net panels chosen to minimize Reynolds number differences. The use of this scale ratio, however, introduces additional constraints due to tank dimensions that need to be addressed.

![Figure 5.2: The 1:15.2 Froude-scale model of the central spar cage.](image)

2.2 **Mooring System Physical Model**

Modeling the mooring system required some geometric modifications to accommodate the UNH wave tank size limitations because an exact scale model of the
submerged grid net pen/mooring system would not fit into the tank. Using a scale factor of 15.2 yields an inaccurate site depth representation and the width of the tank constrains the identically modeling of all four of the anchor legs. Therefore, the following modifications were proposed.

- To accommodate the tank depth the mooring lines were shortened by approximately 46%. The chain catenary component was kept identical.
- The model was rotated so two of the four anchor legs are parallel with the wave direction and the mooring lines parallel to the y-z plane (see Figures 5.3 and 5.4) eliminated.
- To accommodate the tank width, two of the grid nodes were eliminated and the other two connected with a diagonal. The original diagonal distance between the in-line nodes of the grid was maintained.
- The pendant line connecting the ballast to the cage was shortened by nearly 70% to fit into the tank to minimize interaction with the grid line diagonal.

Schematics showing the model geometry changes are shown on Figures 5.3 and 5.4.

Figure 5.3: The designed fish cage and mooring system configuration.

Figure 5.4: The modified system for use in the physical model tests.
To justify the shortening of the mooring legs, perturbations about equilibrium of the forward grid point for the full-scale system were examined. The maximum vertical and horizontal excursion of the upstream grid point was estimated by using the maximum particle trajectory of a wave at the design conditions having a waveheight \( H \) of 9 meters and a period \( T \) of 8.8 seconds and a water depth \( d \) equal to 52 meters and a grid point depth \( z \) equal to 18 meters. From small amplitude wave theory (Dean and Dalrymple, 1991), the maximum horizontal and vertical particle trajectories are obtained from

\[
\zeta = -\frac{H \cosh(k(d+z))}{2 \sinh(kd)} 
\]

and

\[
\xi = \frac{H \sinh(k(d+z))}{2 \sinh(kd)},
\]

respectively, where \( k \) is the wave number equal to \( \frac{2\pi}{L} \) and \( L \) is the wavelength. The results for \( \zeta \) and \( \xi \) were found to be -1.808 and 1.709, therefore a nominal value of 1.8 meters was used as the maximum horizontal and vertical excursion of the grid point.

The static, inextensible, catenary equations defined by Faltinsen (1990) and described in Chapter 2 were used to test the differences in anchor line tension at the grid point due to geometry differences resulting from the shorter anchor line lengths (Figure 2.10 defines the components of the anchor leg used in the grid assembly). Using these analytical expressions, a 1.8 meter perturbation was applied in both the vertical and horizontal direction. Equations (2.5) through (2.10) were then solved iteratively for each perturbation condition until the desired geometric condition was satisfied. These calculations were also performed using the shorter anchor line of 54 meters (full-scale). The resulting vertical, horizontal and anchor line tension values are provided in Table
5.1. The maximum percent difference was found to be approximately 3.5% and was considered acceptable.

**Table 5.1: Vertical and horizontal tension comparisons between the 4- and 2-mooring leg configurations.**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>4-Leg x-direction perturbation</th>
<th>2-Leg x-direction perturbation</th>
<th>4-Leg y-direction perturbation</th>
<th>2-Leg y-direction perturbation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Tension (T_v)</td>
<td>4004 N</td>
<td>4013 N</td>
<td>3499 N</td>
<td>3506 N</td>
</tr>
<tr>
<td>Horizontal Tension (T_h)</td>
<td>11785 N</td>
<td>11900 N</td>
<td>9653 N</td>
<td>9270 N</td>
</tr>
<tr>
<td>Anchor Tension (T_a)</td>
<td>12450 N</td>
<td>12558 N</td>
<td>10268 N</td>
<td>9910 N</td>
</tr>
<tr>
<td>Angle (θ_b)</td>
<td>18.77 degrees</td>
<td>18.72 degrees</td>
<td>19.24 degrees</td>
<td>20.72 degrees</td>
</tr>
</tbody>
</table>

Another system modification made at the model scale was the elimination of two of the anchor legs and replaced with a diagonal to accommodate the tank width. The model would be "deployed" in the tank so that the northeast and southwest anchor legs were aligned parallel to the direction of the wave maker. Statically, the 2-leg configuration is identical to the 4-leg setup. In addition, since the waves in the tank are two-dimensional and the setback of the entire mooring system was sufficiently small, the northeast leg would carry nearly the entire wave induced loading. The estimated total drag differences due to the removal of mooring components between the two models were also estimated to be only 3.2%.

Other modifications made at the model scale include the pendant line length. The full-scale pendant line is 33 meters. Scaling this value would result in a model length of 216 cm, which would not fit into the tank. It was decided to size the pendant line not only to accommodate the tank depth, but also to prevent interaction with the grid line diagonal. The resulting model scale pendant line length was 63 cm. Once the justification for modifying the mooring system was made, each mooring system component was Froude-scaled and built for testing in the UNH wave tank. The mooring
system particulars are provided on Table 5.2.

**Table 5.2: Mooring system physical model particulars.**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Full-Scale</th>
<th>Scaled-Prototype</th>
<th>Actual</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>52 m</td>
<td>3.42 m</td>
<td>2.44 m</td>
</tr>
<tr>
<td>Anchor Chain (length)</td>
<td>27.43 m</td>
<td>1.80 m</td>
<td>1.79 m</td>
</tr>
<tr>
<td>Anchor Chain (mass)</td>
<td>705.7 kg/m</td>
<td>2.01 gm/cm</td>
<td>2.08 gm/cm</td>
</tr>
<tr>
<td>Anchor Line (length)</td>
<td>100 m</td>
<td>6.58 m</td>
<td>3.56 m^a</td>
</tr>
<tr>
<td>Anchor Line (material)</td>
<td>Polyester</td>
<td>Polyester</td>
<td>Nylon</td>
</tr>
<tr>
<td>Anchor Line (specific gravity)</td>
<td>1.38</td>
<td>1.38</td>
<td>1.14</td>
</tr>
<tr>
<td>Anchor Line (diameter)</td>
<td>33 mm</td>
<td>2.17 mm</td>
<td>2.16 mm</td>
</tr>
<tr>
<td>Grid Corner Flotation #1 (mass)</td>
<td>136 kg</td>
<td>39 gm</td>
<td>41 gm</td>
</tr>
<tr>
<td>Grid Corner Flotation #1 (diameter)</td>
<td>0.9525 m</td>
<td>6.27 cm</td>
<td>6.19 cm</td>
</tr>
<tr>
<td>Grid Corner Flotation #2 (mass)</td>
<td>136 kg</td>
<td>39 gm</td>
<td>41 gm</td>
</tr>
<tr>
<td>Grid Corner Flotation #2 (diameter)</td>
<td>0.9625 m</td>
<td>6.27 cm</td>
<td>6.19 cm</td>
</tr>
<tr>
<td>Corner Rope Ring/Chain #1 (mass)</td>
<td>61.33 kg</td>
<td>17.46 gm</td>
<td>16 gm</td>
</tr>
<tr>
<td>Corner Rope Ring/Chain #1 (length)</td>
<td>2 m</td>
<td>13.15 cm</td>
<td>14 cm</td>
</tr>
<tr>
<td>Corner Rope Ring/Chain #1 (mass)</td>
<td>61.33 kg</td>
<td>17.46 gm</td>
<td>16 gm</td>
</tr>
<tr>
<td>Corner Rope Ring/Chain #1 (length)</td>
<td>2 m</td>
<td>13.15 cm</td>
<td>14 cm</td>
</tr>
<tr>
<td>Grid Line (length)</td>
<td>91.92 m</td>
<td>6.047 m</td>
<td>6.047 m^b</td>
</tr>
<tr>
<td>Grid Line (material)</td>
<td>Co-polymer</td>
<td>Co-polymer</td>
<td>Polypropylene</td>
</tr>
<tr>
<td>Grid Line (specific gravity)</td>
<td>0.94</td>
<td>0.94</td>
<td>0.91</td>
</tr>
<tr>
<td>Grid Line (diameter)</td>
<td>40 mm</td>
<td>2.63 mm</td>
<td>3.18 mm</td>
</tr>
<tr>
<td>Lower Bridle (length)</td>
<td>32 m</td>
<td>2.17 m</td>
<td>2.17 m</td>
</tr>
<tr>
<td>Lower Bridle (material)</td>
<td>Co-polymer</td>
<td>Co-polymer</td>
<td>Polypropylene</td>
</tr>
<tr>
<td>Lower Bridle (specific gravity)</td>
<td>0.94</td>
<td>0.94</td>
<td>0.91</td>
</tr>
<tr>
<td>Lower Bridle (diameter)</td>
<td>48 mm</td>
<td>3.16 mm</td>
<td>3.18 mm</td>
</tr>
<tr>
<td>Upper Bridle (length)</td>
<td>11 m</td>
<td>0.724 m</td>
<td>0.724</td>
</tr>
<tr>
<td>Upper Bridle (material)</td>
<td>Polyester</td>
<td>Polyester</td>
<td>Nylon</td>
</tr>
<tr>
<td>Upper Bridle (specific gravity)</td>
<td>1.38</td>
<td>1.38</td>
<td>1.14</td>
</tr>
<tr>
<td>Upper (diameter)</td>
<td>25 mm</td>
<td>1.64 mm</td>
<td>2.16 mm</td>
</tr>
<tr>
<td>Pendant Line (length)</td>
<td>33 m</td>
<td>217 cm</td>
<td>63 cm^c</td>
</tr>
</tbody>
</table>

^a Anchor line length was reduced to accommodate tank depth.

^b Grid line diagonal used to accommodate tank width.

^c Pendant line length was reduced to accommodate tank depth and interaction with the grid diagonal.
2.3 Input Wave Scaling

Another concern using the 1:15.2 ratio was the wave characteristic at the model scale. Since the depth of the demonstration site averages 52 meters, the 1:15.2 scale would require the depth for the model tests to be 3.42 meters. The wave basin facility, however, only contains 2.44 meters of water, which scales up to a depth 37 meters. Since the wave characteristics are a function of the depth, more of the higher period waves would be affected by bottom interaction than if the depth was 52 meters. Therefore, a wavelength comparison was made to investigate these differences.

At the demonstration site, wave periods are estimated to be in range from 2 to 12.5 seconds (see Appendix A). The length of the wave in deep water \( L_0 \) and in the depths of 52 and 37 meters associated with 10 periods within this range were calculated using the dispersion relation defined as equation (4.40). Comparisons between the depth dependant wavelength values are provided on Table 5.3. The results show that the maximum percent difference is less than 10%. Therefore, the scaled depth influence on the wave characteristics was considered minor.

<table>
<thead>
<tr>
<th>Period</th>
<th>( L_0 )</th>
<th>Length (d=52 m)</th>
<th>Length (d=37 m)</th>
<th>% Diff.</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.5</td>
<td>244.33</td>
<td>220.43</td>
<td>200.67</td>
<td>8.968</td>
</tr>
<tr>
<td>9.9</td>
<td>155.40</td>
<td>151.39</td>
<td>142.70</td>
<td>5.740</td>
</tr>
<tr>
<td>7.1</td>
<td>79.41</td>
<td>79.46</td>
<td>79.00</td>
<td>0.569</td>
</tr>
<tr>
<td>6.2</td>
<td>60.70</td>
<td>60.73</td>
<td>60.73</td>
<td>0.000</td>
</tr>
<tr>
<td>5.3</td>
<td>43.22</td>
<td>43.23</td>
<td>43.23</td>
<td>0.000</td>
</tr>
<tr>
<td>4.6</td>
<td>32.46</td>
<td>32.47</td>
<td>32.47</td>
<td>0.000</td>
</tr>
<tr>
<td>4.0</td>
<td>25.16</td>
<td>25.17</td>
<td>25.17</td>
<td>0.000</td>
</tr>
<tr>
<td>3.6</td>
<td>20.07</td>
<td>20.08</td>
<td>20.08</td>
<td>0.000</td>
</tr>
<tr>
<td>3.2</td>
<td>16.45</td>
<td>16.47</td>
<td>16.47</td>
<td>0.000</td>
</tr>
<tr>
<td>1.9</td>
<td>5.93</td>
<td>5.93</td>
<td>5.93</td>
<td>0.000</td>
</tr>
</tbody>
</table>
3. **Input Parameters**

3.1 **Regular Wave Tests**

Ten regular wave tests were conducted as part of this study. The tests were performed using the periods shown on Table 5.4. Input into the wave maker software included the scaled wave period and height. The input wave height was used such that the steepness of the wave \((H/L)\) was equal to 1/15. Prior calibration tests indicated that for the three longest periods this wave steepness could not be maintained due to the operating limits of the wavemaker. Wave steepness of the 1.83, 2.56 and 3.21 second period waves was estimated to be 1/32.5, 1/79.3 and 1/158.5, respectively.

<table>
<thead>
<tr>
<th>Period-Full (s)</th>
<th>H-full (m)</th>
<th>Period-Scaled (s)</th>
<th>H-scaled (cm)</th>
<th>Length-Scaled (m)</th>
<th>Slope (H/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.515</td>
<td>1.27</td>
<td>3.21</td>
<td>8.33</td>
<td>13.20</td>
<td>158.5</td>
</tr>
<tr>
<td>9.981</td>
<td>1.80</td>
<td>2.56</td>
<td>11.84</td>
<td>9.39</td>
<td>79.3</td>
</tr>
<tr>
<td>7.135</td>
<td>2.45</td>
<td>1.83</td>
<td>16.09</td>
<td>5.20</td>
<td>32.3</td>
</tr>
<tr>
<td>6.238</td>
<td>4.05</td>
<td>1.6</td>
<td>26.64</td>
<td>4.00</td>
<td>15</td>
</tr>
<tr>
<td>5.263</td>
<td>2.88</td>
<td>1.35</td>
<td>18.96</td>
<td>2.84</td>
<td>15</td>
</tr>
<tr>
<td>4.561</td>
<td>2.16</td>
<td>1.17</td>
<td>14.24</td>
<td>2.14</td>
<td>15</td>
</tr>
<tr>
<td>4.016</td>
<td>1.68</td>
<td>1.03</td>
<td>11.04</td>
<td>1.66</td>
<td>15</td>
</tr>
<tr>
<td>3.587</td>
<td>1.34</td>
<td>0.92</td>
<td>8.81</td>
<td>1.32</td>
<td>15</td>
</tr>
<tr>
<td>3.248</td>
<td>1.10</td>
<td>0.833</td>
<td>7.22</td>
<td>1.08</td>
<td>15</td>
</tr>
<tr>
<td>1.949</td>
<td>0.40</td>
<td>0.5</td>
<td>2.60</td>
<td>0.39</td>
<td>15</td>
</tr>
</tbody>
</table>

3.2 **Random Wave Tests**

Additional physical model experiments were conducted using a random wave condition estimated to be typical of coastal New Hampshire. As described in Chapter 4, random waves are often represented as an auto-spectral density, which describes the relative amount of energy per frequency band. The wave spectral shapes shown on Figure 5.5 were developed using National Data Buoy Center (NDBC) information as provided in...
Appendix A. It was estimated that the average wave condition has an energy based significant wave height ($H_{m0}$) of 1.21 meters and a peak period ($T_p$) of 10 seconds. Input into the wavemaker software used a 1:15.2 Froude-scaled version of this spectral shape and peak period, but with a significant wave height of 2.5 meters. The significant wave height was chosen because prior calibration tests showed that the response using the higher energy wave regime was easier to measure.

![Figure 5.5: Estimated wave spectra for NH (full-scale) where $T_p = 10$ seconds.](image)

3.3 Wave Generation

The generation of regular and random waves in the UNH tow/wave basin is performed using specifically designed software as described in Michelin (2000) to control the electro-hydraulic wave maker at the end of the tank (for system design details see Washburn, 1996). The regular waves are created from simple sinusoidal shapes based on linear wave theory described in Chapter 4. Random waves are generated using
a version of the random phase method described in Chakrabarti (1994) which, in the time
domain, is the superposition of multiple regular waves of the form,

$$\eta(x,t) = \sum_{j=1}^{n} A_j \cos(k_j x - \omega_j t + \epsilon_j), \quad (5.3)$$

where $A_j$ is the amplitude, $k_j$ is the wave number, $\omega_j$ is the wave radian frequency (equal
to $2\pi f$) and $\epsilon_j$ is the random phase. As described in Michelin (2000) and summarized
here, the time series signal is created in the frequency domain from a continuous 1-sided
spectrum $S(f)$. An inverse Fourier Transform is applied to obtain a time series signal
described by equation (5.3). The computer generated digital signal is converted to analog
to drive the electro-hydraulic wave maker.

The wave maker software allows the time series duration to have a number of
discrete data points equal to powers of two to allow efficient computation of numerical
inverse Fourier Transforms. For the random wave simulations used in these physical
model tests, the times series duration was chosen to have 16384 data points using a $\Delta t$ of
0.005 seconds resulting in a total test length of 81.82 seconds. Performing a random
wave test at this duration ensures that all of the wave frequencies used in the
superposition process are present in the signal (estimated to be 72 waves for the scaled
version of the input spectrum shown in Figure 5.5).

4. Measurement Systems

During the physical model tests, two separate measurement systems were used to
acquire both the motion and load response to the wave forcing. Motion measurements
were made of the cage heave, surge, pitch, and wave elevation using a non-invasive
optical measurement system. The optical system consists of a high-resolution, black and
white digital camera, which for these tests, was set to operate at a rate of 10 frames/sec. A computer based frame grabber captured the video output and stored each of the frames in the computer. Processing software is used to analyze each of the frames to track the pixel locations of small black dots strategically located on the white-painted cage (Michelin and Stott, 1996 and Swift et al., 1997). Black dots were placed on the rim and the lower portion of the spar on the model to track its motion. To measure the wave elevation, the top of a white wave follower ball was also painted black and its vertical motion tracked by the processing software.

A second data acquisition system was used simultaneously with the optical system to measure anchor and lower bridle line tension (using two submersible loadcells) and waves using two capacitance probes (wave staffs). As seen on Figure 5.6, the wave staffs were placed directly in front and behind the cage model. This system utilizes a wireless transmission between the analog/digital (A/D) board and the acquisition computer.

![Wave Staff Locations](image)

**Figure 5.6: Experimental setup for the physical model tests.**
5. Data Processing

5.1 Regular Wave Tests

Using the optical measurement system, each of the fish cage scale model response tests was conducted for 20 seconds sampling at 10 Hz. During the same tests, data from the anchor and bridle line load cells and the wave staffs were being collected using the wireless system. The sampling rate using this acquisition system was set at 15 Hz. Due to inefficient transmission in the wireless system, data sampling did not occur at even time increments. The resulting average delta t was about 0.1156 seconds (8.65 Hz).

The time series data sets collected using both acquisition systems were converted to full-scale engineering units using predetermined calibration equations for each instrument. The resulting time series were processed using a routine to identify all of the crests and troughs. Each crest and trough pair was added, divided by two to obtain amplitudes, and the amplitudes averaged. Wave slope and excursion amplitudes were calculated using equations (4.41) and (4.42), respectively. The average amplitudes for each time series were used to obtain the regular RAOs for heave, surge, pitch, anchor line tension and bridle line tension as described in Chapter 4.

5.2 Random Wave Tests

The same data acquisition systems were used during the random wave tests. The optical measurement system was utilized to measure the motion response of the cage and the wave elevation using the wave following ball was set to operate at 10 Hz for a duration of 90 seconds (350 seconds full-scale), which is about 9% longer than the require time. Ten stochastic wave tests, measuring the motion response of the cage, were conducted using different sets of random phases. The individual time series data collected for heave, surge, pitch and wave elevation were converted to full-scale values,
transformed into the frequency domain as auto- and cross-spectral density functions and processed using an 11-point moving average because the measured results closely matched the ideal input spectrum. The auto-spectral density linear transfer function for each motion response parameter was calculated using equations (4.47) through (4.51) and sub-sampled to have a frequency resolution of 0.01 Hz. In addition, the cross-spectra were separated into its real and complex components, the co- and quad-spectra, respectively. The coherence-squared function was then calculated for each test using equation (4.21). These frequency domain function calculations were performed for each of the 10 motion response data sets and the results ensemble averaged. The ensemble averaged co- and quad-spectra functions were then used to calculate the phase response between the wave forcing and the heave, surge and pitch response using equation (4.14).

The data sets collected using the wireless system were processed in a similar manner. However, the discrete time series were not evenly spaced so the full-scale data sets were sub-sampled using a Δt of 0.5 seconds (2 Hz). The individual, sub-sampled time series were used to obtain the same frequency domain functions as the motion response tests. Anchor and bridle line tension response data sets were collect from 13 random wave simulations in which both auto- and cross-spectral linear transfer functions were calculated using wave elevation data from the most forward wave staff. The results were ensemble averaged. The phase between the forcing and line tension response was also determined using the co- and quad-spectra.

6. Physical Model Test Results

6.1 Static Test Results

Before performing the regular and random wave tests, three data sets were
recorded using the small submersible load cells after placing the model in the wave tank to measure the static pre-tension of the system. During the physical model “deployment” process, it was found that the tensions were sensitive to the placement of the anchors. The full-scale anchor line average of the three static tests was found to be 12.28 kN. Since the compensation floats between the bridle lines were removed, the full-scale static tension in the lower bridle was approximately 0 kN.

6.2 Motion Response Tests Results

The motion response of the central spar cage was investigated using both regular and random wave conditions. Response of the cage in heave, surge and pitch along with the wave elevation was measured using the optical measurement system. Ten regular wave tests were conducted using Froude-scaled wave frequencies (or periods) typical of the demonstration site. The time series results for the ten tests are provided in Appendix B. The time series results were processed to find the wave, heave, surge and pitch heights (twice the amplitude) and are provided on Table 5.5. An example of one of these regular wave tests is shown on Figure 5.7. The corresponding Response Amplitude Operators for each of the regular wave frequencies are shown on Figure 5.9.

Table 5.5: Measured results for the regular wave tests (full-scale).

<table>
<thead>
<tr>
<th>Wave Set</th>
<th>Wave Frequency (Hz)</th>
<th>Wave Height (m)</th>
<th>Heave (m)</th>
<th>Surge (m)</th>
<th>Pitch (rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.080</td>
<td>2.16</td>
<td>1.65</td>
<td>2.352</td>
<td>0.120</td>
</tr>
<tr>
<td>2</td>
<td>0.100</td>
<td>2.58</td>
<td>1.83</td>
<td>1.654</td>
<td>0.043</td>
</tr>
<tr>
<td>3</td>
<td>0.140</td>
<td>2.52</td>
<td>1.63</td>
<td>2.02</td>
<td>0.107</td>
</tr>
<tr>
<td>4</td>
<td>0.160</td>
<td>4.60</td>
<td>2.42</td>
<td>3.138</td>
<td>0.254</td>
</tr>
<tr>
<td>5</td>
<td>0.194</td>
<td>2.94</td>
<td>1.33</td>
<td>1.648</td>
<td>0.176</td>
</tr>
<tr>
<td>6</td>
<td>0.220</td>
<td>2.00</td>
<td>0.643</td>
<td>1.0758</td>
<td>0.107</td>
</tr>
<tr>
<td>7</td>
<td>0.250</td>
<td>1.56</td>
<td>0.336</td>
<td>0.65</td>
<td>0.093</td>
</tr>
<tr>
<td>8</td>
<td>0.278</td>
<td>1.28</td>
<td>0.167</td>
<td>0.448</td>
<td>0.055</td>
</tr>
<tr>
<td>9</td>
<td>0.308</td>
<td>1.02</td>
<td>0.076</td>
<td>0.2582</td>
<td>0.035</td>
</tr>
<tr>
<td>10</td>
<td>0.513</td>
<td>0.24</td>
<td>0.026</td>
<td>0.065</td>
<td>0.016</td>
</tr>
</tbody>
</table>
In addition to the regular wave tests, ten random wave tests were conducted as described in Section 5.2. The time series and auto-spectral results for these tests are provided in Appendix D. Wave elevation, excursion and slope spectra were calculated and ensemble averaged from the random time series results. These results are shown on Figure 5.8. The resulting ensemble averaged wave field measured in the tank was found to have an energy based significant wave height (H\textsubscript{mo}) equal 2.2 meters and a peak period between 9 and 11 seconds.
Figure 5.8: Heave, surge and pitch motion response of the cage (full-scale).

The linear transfer functions were then calculated using the auto-spectral density method described as equations (4.47) through (4.49). The random wave linear transfer functions are plotted along with the results from the regular wave tests on Figure 5.9. Next, the co- and quad-spectra of the cross-spectral density function between the wave elevation forcing and motion response were used to calculate the phase between the input and the output using equation (4.14). The phase was also calculated for each of the regular wave tests. Figure 5.10 shows these phase relationships.
Figure 5.9: Heave, surge and pitch response amplitude operators.

Figure 5.10: The phase between the wave elevation and the heave, surge and pitch response.
At the lower frequency waves, the heave motion appears to be in-phase (0 degrees) with the wave elevation. As the frequencies increase, the heave motion of the cage begins to lag the wave elevation for the random wave tests, while the values remain somewhat constant for the regular wave tests. At a frequency of approximately 0.225 Hz, the heave motion is 90 degrees out-of-phase with the wave elevation (random wave tests). The phase of the surge motion response at a frequency of 0.16 Hz is 90 degrees out-of-phase with the wave elevation. The input and output signal become in phase at 0.225 Hz. In the random wave tests, as the wave frequencies increase, so does the surge motion phase lag which peaks at 50 degrees at a frequency approximately 0.3 Hz. The regular wave phase results show a constant 90-degree out-of-phase result for all the frequencies. The pitch results followed a similar pattern, except the values lead the wave slope.

Another useful function that can be utilized in the processing of random data is the coherence-squared function defined by equation (4.21). The coherence-squared function is used to find the fractional component of the variance of the output contributed by the input at a specific frequency (Bendat and Peirsol, 1986). Values close to one indicate a direct relationship. The heave and surge coherence-squared function results approach one for most of the operating frequencies (Figure 5.11). The linear input/output model chosen for these modes are acceptable. The results for the pitch motion response, on the other hand, are not as strong, primarily because the measurements contain more noise. The 95% confidence limits were obtained by Haubrich (1965). In addition, according to Chakrabari (1990), coherency-square values of 0.6 or better are typically considered acceptable for linear systems.
Figure 5.11: The coherency between the wave elevation and the heave, surge and pitch motion response.

6.3 Anchor and Bridle Line Tension Tests

Similar processing techniques were employed between wave elevation time series and the tension time series measured in the model anchor and bridle lines. Frequency analysis was conducted and transfer functions calculated using the regular, auto-spectral, and cross-spectral methods. Anchor and bridle line tension was measured by placing small proof-ring load cells (submersible) in line with the model components. Wave elevation data used in this analysis was obtained from the most forward wave staff. In the static condition, the anchor line is pre-tensioned to a full-scale value between 4.5 to 13.5 kN (depending upon the geometry of the system). The bridle line tension, on the other hand, was in an unloaded state.

The tension line load results for the anchor and bridle lines in regular waves are
provided on Table 5.6. The individual data plot results can be found in Appendix C. The anchor and bridle line transfer functions for the regular wave tests were calculated by dividing the tension amplitude by the wave amplitude. These results are shown on Figure 5.14.

Table 5.6: Mooring load response results (full-scale).

<table>
<thead>
<tr>
<th>Wave Period (sec)</th>
<th>Wave Height (m)</th>
<th>Ave. Anchor Tension (kN)</th>
<th>Ave. Anchor Tension Amplitude (kN)</th>
<th>Maximum Anchor Tension (kN)</th>
<th>Ave. Peak Bridle Tension (kN)</th>
<th>Maximum Bridle Tension (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.5</td>
<td>2.54</td>
<td>11.1</td>
<td>3.78</td>
<td>15.49</td>
<td>11.09</td>
<td>12.1</td>
</tr>
<tr>
<td>9.98</td>
<td>2.62</td>
<td>13.1</td>
<td>2.72</td>
<td>16.94</td>
<td>12.44</td>
<td>14.2</td>
</tr>
<tr>
<td>7.13</td>
<td>2.60</td>
<td>12.6</td>
<td>2.15</td>
<td>16.45</td>
<td>7.21</td>
<td>11.4</td>
</tr>
<tr>
<td>6.24</td>
<td>3.82</td>
<td>10.5</td>
<td>7.23</td>
<td>22.26</td>
<td>7.77</td>
<td>11.2</td>
</tr>
<tr>
<td>5.26</td>
<td>2.68</td>
<td>9.74</td>
<td>4.50</td>
<td>17.42</td>
<td>7.21</td>
<td>9.40</td>
</tr>
<tr>
<td>4.56</td>
<td>1.75</td>
<td>9.60</td>
<td>2.43</td>
<td>15.49</td>
<td>4.25</td>
<td>6.91</td>
</tr>
<tr>
<td>4.02</td>
<td>1.55</td>
<td>8.82</td>
<td>1.86</td>
<td>12.10</td>
<td>3.68</td>
<td>4.67</td>
</tr>
<tr>
<td>3.59</td>
<td>1.29</td>
<td>16.1</td>
<td>1.68</td>
<td>19.36</td>
<td>3.07</td>
<td>4.21</td>
</tr>
<tr>
<td>3.25</td>
<td>0.97</td>
<td>12.6</td>
<td>0.83</td>
<td>15.49</td>
<td>2.30</td>
<td>3.22</td>
</tr>
<tr>
<td>1.95</td>
<td>0.25</td>
<td>14.1</td>
<td>0.53</td>
<td>15.49</td>
<td>1.20</td>
<td>2.50</td>
</tr>
</tbody>
</table>

\(^{a}\) Average amplitude above the mean of the anchor load time series.  
\(^{b}\) Average of the peak bridle loads.

The ensemble averaged auto- and cross-spectral density results of the random wave simulations are shown on Figure 5.12 and Figure 5.13 (the random wave time series plots for each of the tests are provided in Appendix E). The ensemble average of the measured wave spectra for each of the anchor and bridle line wave input yielded an $H_{mo}$ equal to 2.63 meters and a peak period between 9.00 and 11.0 seconds. A clear bi-modal response is evident in both the anchor and bridle line tension auto-spectra occurring at the wave periods of 10.0 and 4.35 seconds (0.10 and 0.23 Hz, respectively). However, the variance of the anchor and bridle line tension response spectra (calculated using equation 4.11), show considerable differences. The variance of the anchor line tension response, or the area under the curve, was found to be $4.82 \text{kN}^2$, while the variance of the bridle
line tension response was only 1.39 kN\(^2\). This 70% difference is primarily a result of the pre-tensioned nature of the anchor line acting as a restoring force, which enables the response load to oscillate around the mean. Since the bridle line is not pre-tensioned, this restoring force does not exist, so the relative energy of the response is less.

Figure 5.12: Auto-spectral density results of the anchor and bridle line tests in random waves.
Using the auto- and cross-spectra results, full-scale linear transfer functions were calculated using equations (4.50) through (4.53). These results are shown along with the regular wave results on Figure 5.14. The relative energy differences in the anchor and bridle line response are evident on Figure 5.14 as the offset between anchor and bridle line transfer function curves. Between the wave frequencies of 0.05 and 0.4 Hz, the anchor and bridle line transfer functions are typically within a nominal upper value of 5 kN/m.

The coherence-squared function was also calculated (equation 4.21) for these input and output signals (Figure 5.15). For the most part, at the operating frequencies, the relationship is linear. However, a non-linear component may also exist along with some measurement noise, and is the most plausible reason why values are not closer to one, especially at frequencies greater than 0.23 Hz.
Figure 5.14: Anchor and bridle line transfer functions (full-scale).

Figure 5.15: The coherency between the wave elevation and the anchor and bridle tension.
As with the motion response random wave tests, the phase relationship between the input wave forcing and the anchor and bridle tension response was also investigated for both the random and regular wave tests (Figure 5.16). Between approximately 0.08 and 0.2 Hz, the phase difference between the signals are nearly zero degrees for the random wave tests. At higher wave frequencies, the anchor line response gradually becomes 90 degrees out of phase with the wave forcing at approximately 0.35 Hz. At these same higher frequencies, it is unclear that the bridle line response is similar for the random wave tests. However, since the anchor and bridle lines are connected at the grid corner of the mooring system, the response is most likely coupled. For the regular wave tests, the phase results ranged typically between 0 and 50 degrees for the anchor line. The bridle line phase results for the regular wave tests were considerably sporadic.

![Phase diagram](image)

**Figure 5.16:** The phase between the wave elevation and the anchor and bridle line tension.
7. Discussion of Physical Model Tests

One of the goals of the physical modeling component of this research was to investigate and compare modeling and data processing techniques. Regular and random wave tests were performed, and motion and load linear transfer functions were calculated. The motion response results, shown on Figure 5.9, show similar trends. The surge results were practically identical. The heave and pitch data, however, did not correspond exactly, but show similar characteristics. One of which is a possible pitch resonant condition at some wave frequency around 0.1 Hz. It is interesting to note that this characteristic was found in the results of both regular and random wave tests. The linear transfer function calculated using the auto-spectral density method represents the ensemble average of ten separate random wave tests. Therefore, a certain degree of confidence is associated with this result. Furthermore, since the regular wave tests showed a similar response, it is likely that some type of pitch resonance exists.

One possible explanation could be a pendulum effect of the ballast weight below the spar while the cage is at the surface. A wave at some frequency around 0.1 Hz could excite the cage system into a modal response where the ballast weight swings beneath the cage. Evidence of this is shown in a photograph taken during the regular wave test # 1 (H = 2.16 meters and T = 12.52 seconds) as shown on Figure 5.17.

To investigate this mode of response, consider the natural period of a simple pendulum,

\[ T_{\text{pendulum}} = 2\pi \sqrt{\frac{L_{\text{pendulum}}}{g}} \]  

(5.4)

where \( L_{\text{pendulum}} \) is the length of the pendulum and \( g \) is the gravitation constant. For the
physical model tests, the full-scale length, $L_{\text{pendulum}}$, is 9.58 meters resulting in a natural period of 6.21 seconds. The deployed system of the north cage has a pendant line length of 33 meters resulting in a natural period of 11.52 seconds. Therefore, it is likely that some type of resonance exists for the actual system at a wave frequency that could be typical of the demonstration site. Deployment of this cage at the surface involves the risk of a resonant condition occurring under the “right” forcing frequency. It is recommended to keep this cage submerged, especially during storm events that include long swells to minimize the possibility of this occurring.

Figure 5.17: Pendulum effect of the pendant line weight in the physical model tests.
CHAPTER 6

MOTION AND LOAD FREQUENCY RESPONSE USING NUMERICAL MODELING METHODS

1. Numerical Modeling Objectives

The primary objective of the numerical modeling component of this research was to validate existing techniques to better approximate fish cage and mooring system dynamics currently deployed at the demonstration site. The first task was to update the material and geometric properties of the cage and mooring system components as described by Muller (1999) and Ozbay (1999). The next issue concerns improvements to the numerical model to better simulate system dynamics. These include incorporating a bottom conditional to model component and ocean floor interaction and random waves to provide more realistic forcing. The use of shaded elements, introduced in Chapter 3, is also investigated as part of a series of tests using the existing design wave and current condition (Fredriksson, 1998). Another goal in the validation process was to perform regular and random wave simulations identical to those performed as part of the physical modeling efforts. Linear transfer functions were calculated to provide a set of normalized parameters to compare with not only the results of the physical model tests, but also in-situ field observations.

2. Model Improvements

The first improvement made to the UNH AquaFE model was to include a bottom conditional in the FORTRAN code. The code changes were made in the truss element

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file. The conditional was set such that as the height, above bottom, of a node of an element is equal or less than zero meters, all buoyant, weight and fluid loading forces were set to zero. Since the external forces were eliminated, the inertial and damping forces were also minimized.

Another improvement incorporated into the numerical procedure is the random waves option. The AquaFE model creates a random wave signal using a superposition of sinusoidal waves using the random phase method (Charkrabarti, 1994) of the form as described by equation (5.3). Prior to operating the numerical procedure, the power spectral density of a particular random wave condition is chosen (described as \( S(f) \)) for a range of frequencies typically between 0.05 and 0.50 Hz. If a superposition of 100 waves, for example, is to be used in the simulation, amplitudes are calculated for a frequency bandwidth \( (\Delta f) \) equal to 0.0045 Hz using,

\[
\frac{1}{2} A_j^2 = S(f) \Delta f.
\]  

The random phase method is actually deterministic over a period equal to \( 1/\Delta f \) when the time series repeats itself. At the end of this time, a new set of random phases needs to be generated to ensure a different wave elevation time series. Currently the AquaFE program does not generate it's own set of random phases; instead it is performed in preprocessing routines. Since all of the wave frequency components are included in the time series, simulations need not be performed for any time longer. However, the error for the results of a single random wave simulation is so large that little confidence is associated with it. Additional random wave simulations were performed using newly generated phases and the results ensemble averaged (in the frequency domain) to ensure smaller error and therefore higher confidence levels.
3. Input Parameters

3.1 Material and Geometric Properties

A finite element model of the north cage/grid system was constructed in the AquaFE model software. The model was based on the one built by Ozbay (2000). The cage/grid model used in this Chapter, however, contained several changes. The model of the central spar cage includes a more detailed ballast weight and harvest ring and shaded elements for simulations performed in currents. The mooring model contained updated material properties and elimination of the compensator floats between the upper and lower bridle lines. Figure 6.1 shows the model used in each of the numerical simulations.

![AquaFE model of the cage and mooring system deployed at demonstration site.](image)

The AquaFE computer program requires that all of the truss elements be described by the component’s mass density (kg/m³), modulus of elasticity (Pa) and cross-sectional area (m²). The program also uses spherical buoy elements, for which the user is required to input the mass density and diameter. Since the software only uses truss elements that are theoretically connected with pins (three degrees of freedom for each
node-displacement in the x, y and z directions), massless elements are often used to maintain structural rigidity. To fabricate the central spar cage model, 108 massless elements were incorporated to render it stiff to minimize local displacements and therefore relative velocities. The material properties for the central spar cage model and mooring system are provided on Table 6.1.

**Table 6.1: Material properties used in the model simulations.**

<table>
<thead>
<tr>
<th>Component</th>
<th>Mass Density (kg/m³)</th>
<th>Young's Modulus (Pa)</th>
<th>Cross Sectional Area (m²)</th>
<th>Diameter (m)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rim</td>
<td>714</td>
<td>1.817 x 10¹⁰</td>
<td>5.850 x 10⁻²</td>
<td>-</td>
</tr>
<tr>
<td>Spoke Lines</td>
<td>960</td>
<td>1.030 x 10¹⁰</td>
<td>1.327 x 10⁻⁴</td>
<td>-</td>
</tr>
<tr>
<td>Spar</td>
<td>708¹</td>
<td>8.598 x 10¹⁰</td>
<td>6.647 x 10⁻¹</td>
<td>-</td>
</tr>
<tr>
<td>Lower Bridle</td>
<td>940</td>
<td>1.830 x 10¹⁰</td>
<td>1.810 x 10⁻¹</td>
<td>-</td>
</tr>
<tr>
<td>Net</td>
<td>1027</td>
<td>1.030 x 10¹¹</td>
<td>1.798 x 10⁻⁴</td>
<td>-</td>
</tr>
<tr>
<td>Mass-less</td>
<td>n.a.</td>
<td>1.030 x 10¹¹</td>
<td>1.798 x 10⁻⁴</td>
<td>-</td>
</tr>
<tr>
<td>Ballast Weight</td>
<td>2000</td>
<td>2.500 x 10¹⁰</td>
<td>2.112 x 10⁻¹</td>
<td>-</td>
</tr>
<tr>
<td>Harvest ring part #1</td>
<td>1025</td>
<td>2.500 x 10¹⁰</td>
<td>1.340 x 10⁻²</td>
<td>-</td>
</tr>
<tr>
<td>Harvest ring part #2</td>
<td>1025</td>
<td>2.500 x 10¹⁰</td>
<td>6.360 x 10⁻¹</td>
<td>-</td>
</tr>
<tr>
<td>Harvest ring part #3</td>
<td>1025</td>
<td>2.500 x 10¹⁰</td>
<td>1.070 x 10⁻¹</td>
<td>-</td>
</tr>
<tr>
<td>Shaded Net</td>
<td>1027</td>
<td>1.030 x 10¹⁰</td>
<td>1.798 x 10⁻²</td>
<td>-</td>
</tr>
<tr>
<td>Shaded Spar</td>
<td>708¹</td>
<td>8.598 x 10¹⁰</td>
<td>6.647 x 10⁻¹</td>
<td>-</td>
</tr>
<tr>
<td>Shaded Rim</td>
<td>714</td>
<td>1.817 x 10¹⁰</td>
<td>5.850 x 10⁻⁴</td>
<td>-</td>
</tr>
<tr>
<td>Anchor Line</td>
<td>1380</td>
<td>8.687 x 10¹⁰</td>
<td>6.583 x 10⁻⁴</td>
<td>-</td>
</tr>
<tr>
<td>Anchor Chain</td>
<td>6610</td>
<td>2.000 x 10¹¹</td>
<td>7.024 x 10⁻³</td>
<td>-</td>
</tr>
<tr>
<td>Upper Bridle</td>
<td>1380</td>
<td>3.580 x 10¹⁰</td>
<td>4.909 x 10⁻³</td>
<td>-</td>
</tr>
<tr>
<td>Grid Float</td>
<td>291</td>
<td>-</td>
<td>-</td>
<td>0.963</td>
</tr>
<tr>
<td>1&quot; Long Link Chain</td>
<td>6920</td>
<td>2.000 x 10¹¹</td>
<td>2.027 x 10⁻²</td>
<td>-</td>
</tr>
<tr>
<td>Grid Line</td>
<td>940</td>
<td>1.830 x 10¹⁰</td>
<td>1.257 x 10⁻³</td>
<td>-</td>
</tr>
</tbody>
</table>

* Diameter only used for flotation elements

### 3.2 Regular Wave Settings

The regular wave settings used for the numerical model simulations were chosen to be identical to the 10 regular physical model tests (full-scale). Two additional regular wave settings, as described by numbers 11 and 12 provided on Table 6.2, were chosen in an effort to bracket a possible resonant condition suspected in the pitch motion of the cage. Wave set numbers 13 and 14 represent the design wave and velocity condition.
input parameters. Wave simulation 13 was performed without the shaded elements “turned on” while simulation 14 was run with a 10% velocity reduction applied to elements that were clearly in the shaded region of the steady current.

Table 6.2: Regular wave input parameters.

<table>
<thead>
<tr>
<th>Wave Set Number</th>
<th>Period (s)</th>
<th>Length (m)</th>
<th>Height (m)</th>
<th>Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.5</td>
<td>220.43</td>
<td>1.27</td>
<td>158.5</td>
</tr>
<tr>
<td>2</td>
<td>9.98</td>
<td>151.39</td>
<td>1.80</td>
<td>79.3</td>
</tr>
<tr>
<td>3</td>
<td>7.14</td>
<td>79.46</td>
<td>2.45</td>
<td>32.3</td>
</tr>
<tr>
<td>4</td>
<td>6.24</td>
<td>60.73</td>
<td>4.05</td>
<td>15</td>
</tr>
<tr>
<td>5</td>
<td>5.26</td>
<td>43.23</td>
<td>2.88</td>
<td>15</td>
</tr>
<tr>
<td>6</td>
<td>4.56</td>
<td>32.47</td>
<td>2.16</td>
<td>15</td>
</tr>
<tr>
<td>7</td>
<td>4.02</td>
<td>25.17</td>
<td>1.68</td>
<td>15</td>
</tr>
<tr>
<td>8</td>
<td>3.59</td>
<td>20.08</td>
<td>1.34</td>
<td>15</td>
</tr>
<tr>
<td>9</td>
<td>3.25</td>
<td>16.47</td>
<td>1.10</td>
<td>15</td>
</tr>
<tr>
<td>10</td>
<td>1.95</td>
<td>5.93</td>
<td>0.40</td>
<td>15</td>
</tr>
<tr>
<td>11</td>
<td>15.0</td>
<td>286</td>
<td>12.00</td>
<td>19</td>
</tr>
<tr>
<td>12</td>
<td>20.0</td>
<td>411</td>
<td>13.70</td>
<td>30</td>
</tr>
<tr>
<td>13^a</td>
<td>8.80</td>
<td>120</td>
<td>9.00</td>
<td>13.3</td>
</tr>
<tr>
<td>14^b</td>
<td>8.80</td>
<td>120</td>
<td>9.00</td>
<td>13.3</td>
</tr>
</tbody>
</table>

^a Included a 1 m/s current in the x-direction without shaded elements
^b Included a 1 m/s current in the x-direction with shaded elements (10% reduction).

3.3 Random wave settings

The random wave input into the numerical software consisted of a typical spectral shape estimated for coastal New Hampshire (see Appendix A) described by

$$S(f) = (H_s)^2 (f^{-p}) \left[ \alpha_1 \beta_1 \left( T_{p1} \right)^{-1} e^{-1.25(T_{p1}/f)^2} + \left[ \alpha_2 \beta_2 \left( T_{p2} \right)^{-1} e^{-1.25(T_{p2}/f)^2} \right] \right]$$

where,

- $H_s = 2.5$ meters,
- $f^p$ = the frequency to the negative power of 4.35,
- $\alpha_1 = 0.142$,
- $\gamma_1 = 6.75$,
- $T_{p1} = 10$ seconds,
- $\alpha_2 = 0.456$,
- $\gamma_2 = 0.50$ and
- $T_{p2} = 5.34$ seconds.

This spectral input data was chosen to be identical to the full-scale version used in the
physical model tests described in Chapter 5. The wave input was generated using the superposition of 100 waves with frequencies ranging between 0.05 ($f_1$) and 0.5 ($f_2$) Hertz using equations (5.3) and (6.1) each with a random phase. The duration of the time series before repeating is equal to $1/\Delta f$, where

$$
\frac{1}{\Delta f} = \left[ \frac{(f_2 - f_1)}{\text{no. of waves}} \right]^{-1},
$$

(6.3)

is equal to 222.22 seconds. The actual duration entered into the numerical software was 250 seconds, chosen to allow time for the transient part of simulation. The dynamic simulations were performed using a time step $\Delta t$ equal to 0.01 seconds. Each of the 100 wave heights, lengths, periods and phases were saved so that frequency analysis could be performed.

**4. Data Processing**

**4.1 Regular Wave Tests**

Data processing for the regular wave simulations was nearly identical to the procedure used for the physical model results. For the regular wave tests, results were processed to obtain motion and load linear transfer functions. The motion transfer functions were calculated using the amplitude of the system response in heave, surge and pitch divided by the amplitude of the wave elevation, excursion and slope, respectively. The anchor and bridle line tension transfer functions were calculated using amplitude of the tension response divided by the amplitude of the surface waves.

**4.2 Random Wave Tests**

For the random wave tests, simulations were performed for a duration of 250 seconds and samples were recorded at a rate of 5 Hz. Auto-spectral density calculations
were performed using the wave elevation, heave, surge and pitch time series. These results were then smoothed using an 11-point moving average. During the processing procedure, it was found that the 11-point moving average produced wave elevation results that closely matched the input spectrum. The wave excursion and slope spectra were calculated using expressions (4.45) and (4.46), respectively, with the smoothed wave elevation spectrum as input. Cross-spectra between the wave elevation and the heave, surge and pitch response was also calculated and separated into the respective co- and quad-spectra. A coherence-squared calculation was also performed using equation (4.21) between the wave elevation input and the heave, surge and pitch motion response. Linear transfer functions were then obtained for each simulation using the auto-spectral techniques described by expressions (4.47) through (4.49). These parameters were ensemble averaged using the results of 10 random wave simulations and sub-sampled to obtain a frequency resolution equal to 0.01 Hz. The ensemble averaged co- and quad-spectra were then used in equation (4.14) to calculate the phase between input forcing and motion response. The random wave motion response transfer functions were then compared with the regular wave simulation results.

The data processing techniques for the anchor and bridle line tension response were nearly identical to those described for the motion response results. Calculations were performed not only to obtain the linear transfer function using the auto-spectral density method, described by expressions (4.50) and (4.51), but also the cross-spectral density method using equations (4.52) and (4.53). The results were ensemble averaged and sub-sampled to obtain a frequency resolution of 0.01 Hz. Coherency and phase response calculations were also performed.
5. Numerical Simulation Results

5.1 Static Test Results

One of the first simulations conducted using the numerical model was a simple static test. In this test, the model of the entire fish cage and mooring system was released and allowed to come to static equilibrium for a period of 30 seconds. Tension in the anchor, grid and bridle lines were measured and the results shown on Figure 6.2.

![Figure 6.2: Anchor and bridle line static tension results.](image)

After the transient portion of the simulation, the anchor and grid line pretension values were calculated to be 9.86 kN and 6.37 kN, respectively. The lower bridle line pretension was calculated to be 0.26 kN. Using equations (2.5) through (2.10), analytical values for the anchor and grid line pretensions were calculated to be 9.2 kN and 7.32 kN, representing a 7.4% and 12% difference, respectively. This result is significant because it validates the bottom conditional code changes to the AquaFE program (see Section 2). Changes were made to better model the chain catenary interaction with the ocean bottom.
Without the conditional, the chain freely hangs in space unsupported, which does not accurately represent the system. These code changes, and validation of the static condition with the analytical calculations, were necessary steps before dynamic simulations could be performed.

5.2 **Regular Wave Results**

Next, linear transfer functions for the regular wave tests using the input parameters provided on Table 5.4 were obtained using the AquaFE model. The motion and tension results are provided on Table 6.3. The time series results for each of the regular wave simulations are provided in Appendix F and G for the motion and load results, respectively. In the normal frequency operating range, heave, surge and pitch results were less than one. The central cage model shows little wave follower characteristics. Therefore, the cage components are exposed to higher relative velocities due to the particle motion of the waves.

<table>
<thead>
<tr>
<th>Wave Set Number</th>
<th>Freq. (Hz)</th>
<th>Heave (m/m)</th>
<th>Surge (m/m)</th>
<th>Pitch (deg/deg)</th>
<th>Anchor Line (kN/m)</th>
<th>Bridle Line (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.080</td>
<td>0.6334</td>
<td>0.3927</td>
<td>0.3438</td>
<td>0.595</td>
<td>1.412</td>
</tr>
<tr>
<td>2</td>
<td>0.010</td>
<td>0.3928</td>
<td>0.3638</td>
<td>0.3298</td>
<td>0.982</td>
<td>1.626</td>
</tr>
<tr>
<td>3</td>
<td>0.140</td>
<td>0.2226</td>
<td>0.3255</td>
<td>0.2240</td>
<td>2.353</td>
<td>2.746</td>
</tr>
<tr>
<td>4</td>
<td>0.161</td>
<td>0.2168</td>
<td>0.2980</td>
<td>0.1507</td>
<td>3.812</td>
<td>4.569</td>
</tr>
<tr>
<td>5</td>
<td>0.190</td>
<td>0.1577</td>
<td>0.2505</td>
<td>0.1185</td>
<td>3.572</td>
<td>4.433</td>
</tr>
<tr>
<td>6</td>
<td>0.219</td>
<td>0.0955</td>
<td>0.2104</td>
<td>0.0836</td>
<td>3.461</td>
<td>4.179</td>
</tr>
<tr>
<td>7</td>
<td>0.249</td>
<td>0.0466</td>
<td>0.1632</td>
<td>0.0458</td>
<td>2.693</td>
<td>3.529</td>
</tr>
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<td>8</td>
<td>0.279</td>
<td>0.0310</td>
<td>0.1326</td>
<td>0.0278</td>
<td>1.741</td>
<td>2.593</td>
</tr>
<tr>
<td>9</td>
<td>0.308</td>
<td>0.0369</td>
<td>0.0872</td>
<td>0.0106</td>
<td>1.228</td>
<td>1.960</td>
</tr>
<tr>
<td>10</td>
<td>0.513</td>
<td>&lt;0.01</td>
<td>&lt;0.01</td>
<td>&lt;0.01</td>
<td>0.379</td>
<td>1.684</td>
</tr>
<tr>
<td>11</td>
<td>0.067</td>
<td>0.3827</td>
<td>0.4666</td>
<td>0.3913</td>
<td>5.300</td>
<td>5.639</td>
</tr>
<tr>
<td>12</td>
<td>0.050</td>
<td>0.4139</td>
<td>0.4759</td>
<td>0.7713</td>
<td>7.770</td>
<td>7.639</td>
</tr>
<tr>
<td>13a</td>
<td>0.113</td>
<td>n.a</td>
<td>n.a</td>
<td>n.a.</td>
<td>n.a.</td>
<td>n.a.</td>
</tr>
<tr>
<td>14b</td>
<td>0.113</td>
<td>n.a</td>
<td>n.a</td>
<td>n.a.</td>
<td>n.a.</td>
<td>n.a.</td>
</tr>
</tbody>
</table>

* Included a 1 m/s current in the x-direction without shaded elements.

* Included a 1 m/s current in the x-direction with shaded elements (10% reduction).

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Another series of tests that was performed (wave set numbers 13 and 14), represent the design wave conditions used in previous model simulations (Ozbay, 1999 and Tsukrov et al., 2000). For these simulations, the design wave input to the model had a wave height and length of 9.0 and 120 meters, respectively, and a current velocity of 1 m/s. These two tests were conducted for comparison with the previous model results. The simulations were performed, with and without shaded elements (using a 10% velocity reduction on the steady velocities only). The results are shown on Figure 6.3.

![Graph of Surface Elevation, Anchor Line Tension, and Bridle Line Tension with and without shaded elements.]

**Figure 6.3: Load response to the design condition with and without shaded elements (10% reduction).**

The maximum peak anchor line tension values were 95 and 100 kN for the tests with and without the shaded elements, respectively. The bridle line tension peak values were 82 and 87 kN. The anchor line tension values, using the shaded elements and the

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code incorporating the bottom contact conditional, were 16% less than results obtain from Ozbay (1999) and Tsukrov et al. (2000). This result suggests that previous model simulations without these code changes may have over predicted mooring design loads.

5.3 Random wave results

Ten random wave simulations were also performed using waves with the spectral characteristics described by equation (6.2) as input. The motion response time series and auto-spectral results from each of the random wave simulations are provided in Appendix H. The ensemble average of the wave elevation, excursion and slope forcing spectra along with the heave, surge and pitch cage motion response are shown on Figure 6.4. The linear transfer functions were calculated using equations (4.47) through (4.49) and the results shown on Figure 6.5. Also shown on Figure 6.5 are the transfer function results from the regular wave tests.

![Figure 6.4: Heave, surge and pitch response using the AquaFE model.](image-url)
In general, the motion response transfer function results using the numerical model shows non-wave following characteristics at frequencies less than 0.45 Hz. The pitch transfer function results may show a resonance because the upward trend begins at 0.1 Hz, but the results are inconclusive. Between the frequencies of 0.1 and 0.3 Hz, the regular wave results correspond with the random wave results.

![Graphs showing heave, surge, and pitch linear transfer functions.](image)

**Figure 6.5: Heave, surge and pitch linear transfer functions.**

The phase relationship between the wave elevation and the heave, surge and pitch motion results was also investigated. Recall that the phase between the two signals is calculated by finding the angle between the co- and quad-spectral components of the cross-spectral density described by equation (4.14). The phase results for both the regular and random wave tests are shown on Figure 6.6. The phase between the wave elevation and the heave response fluctuates between 45° and 90° with the waves leading for both the regular and random wave test results. The surge response is 90° out of phase.
(leading the waves) between wave frequencies of approximately 0.075 and 0.14 Hz for
the random wave tests but lagged in the regular wave tests. At 0.14 Hz the phase
switches by 180°. The pitch response at the low frequency end lags the wave forcing by
approximately 45° for both the regular and random waves. As the frequencies increase,
phase values gradually lag until the response is 90° out of phase.

![Graphs showing phase between wave elevation and motion responses](image)

**Figure 6.6: The phase between the wave elevation and the heave, surge and pitch
motion response results.**

The coherence-squared (coherency) function is also calculated for this set of data.
The coherence results are shown on Figure 6.7. The 95% confidence limits also shown
on Figure 6.7 were calculated using the method described by Haubrich (1965). Between
the frequencies of 0.05 and 0.15 Hz, the coherency values for the heave, surge and pitch
are nearly one indicating that a linear system exists at this frequency range. Beyond 0.15
Hz, however, the coherency values drop to values on the order of 0.2, which indicates a
possible non-linear relationship between the forcing and the response.
Figure 6.7: The coherency between the wave elevation and the heave, surge and pitch motion response.

5.4 Random Wave Anchor and Bridle Line Tension Response

The same numerical model simulation used to investigate the motion of the cage was also used to calculate the load time series of the anchor and lower bridle lines. The individual time series and auto-spectral load results are provided in Appendix I. The ensemble average of anchor and bridle line tension response, along with wave input spectrum is shown on Figure 6.8. The wave input and tension load response time series were also used as input to the cross-spectral density function shown on Figure 6.9. As performed with the physical model random data sets, the auto- and cross-spectral density functions were used to calculate corresponding linear transfer functions using equations (4.50) through (4.53).
Figure 6.8: The auto-spectral density for the wave elevation and anchor and bridle line tension response.

Figure 6.9: The magnitude of the cross-spectral density between the wave elevation and the anchor and bridle line tension response.
The results of the auto- and cross-spectral density linear transfer function calculations are shown on Figure 6.10. It was found that the results using these methods were nearly identical since little measurement noise exists. Also shown in Figure 6.10, are the regular wave linear transfer functions calculated in the previous section. For the wave frequencies between 0.07 and 0.5 Hz, the auto- and cross-spectral transfer function values were approximately 1 kN/m. As the frequency decreased to 0.05 Hz, values approached 8 kN/m. The regular wave results were close to 5 kN/m between the frequencies of 0.15 and 0.25 Hz. At other frequencies, a good correspondence with the random wave tests is evident.

![Graph showing anchor and bridle line transfer functions.](image)

**Figure 6.10: Anchor and bridle line transfer functions.**

The phase relationship between the input wave elevation and the anchor and bridle line tension response for both the regular and random wave tests are shown on Figure 6.11. At the frequency of 0.1 Hz, both anchor and bridle line responses are 90
degrees out of phase with the wave elevation (random wave tests). As the wave frequency increases to 0.2 Hz, the phase steadily decreases to 45 degrees. In general, the results of the regular wave phase tests showed the tensions lagging the waves between 45 and 90 degrees.

![Graph](image)

**Figure 6.11: The phase relationship between the wave elevation and the anchor and bridle line response.**

Using the auto- and cross-spectral results, the coherence-squared function was calculated between the wave elevation forcing and the tension and bridle line response (Figure 6.12). Results show that for frequencies between 0.08 and 0.175 Hz, the input and output signals are approximately coherent. If the measurement error between the results is negligible, the differences at the other frequencies could be the result of a non-linear relationship between the wave elevation and the load responses. Since the AquaFE model uses a Morison equation formulation with both a linear (inertial) and a non-linear (drag) component (see equation 3.8), it is possible that the drag term is influencing the
numerical calculation for these wave frequencies.

![Graph](image)

Figure 6.12: The coherency between the wave elevation and the anchor and bridle line tension.

6. Discussion of Numerical Modeling Tests

One interesting result of the motion response numerical model tests observed was the repeatability. For example, between the frequencies of 0.1 and 0.3 Hz, the motion response linear transfer function data for each the regular and random wave tests were nearly identical. Recall that the transfer functions calculated using random waves were performed using the ensemble average of 10 input forcing and motion response data sets. The regular wave test results were not ensemble averaged because the data originated from individual simulations. To have such a match shows that fewer tests will yield the same results with the same degree of confidence. In other words, the numerical simulations have negligible measurement error. The coherency plots also indicate that
the motion response for cage is, for the most part, linear.

The anchor and bridle line tension response results between the regular and random wave tests did not match at all of the operating frequencies. Since the wave slope values used for the regular wave tests were different (most likely) from the random wave tests, the transfer functions between the wave elevation and the tension responses support the hypothesis of a possible non-linear relationship. This can be further investigated by performing model simulation using more extreme input spectra and comparing the linear transfer function results with those of a moderate sea.
CHAPTER 7

MOTION AND LOAD FREQUENCY RESPONSE FROM FIELD OBSERVATIONS

1. Field Measurement Program

During the fall of 1999 and the winter of 2000, UNH and the Woods Hole Oceanographic Institution (WHOI) collaborated to establish a field measurement protocol to investigate the system characteristics of one of the mooring and central spar cage systems deployed at the demonstration site in the Gulf of Maine. Engineers from WHOI developed specialized instruments to measure the motion response of the cage and the loads in nine critical components of the mooring system. The forces were measured using submersible load cells rated to 89 kN, a specification based on previous numerical and physical modeling efforts (Fredriksson et al., 2000 and Tsukrov et al., 2000). The load cells were placed in-line with the mooring components during the redeployment of a refurbished version of the north cage system during the summer of 2000 by the F/V Nobska operated by Stommel Fisheries, assisted by UNH and WHOI personnel (Irish et al., 2001). The data acquisition systems for the load cells were designed with underwater mateable connectors so they could be removed by divers. The motion package system, which includes six accelerometers, was mounted on the spar of the fish cage submerged to a depth of approximately 5 m. This system was also designed to be serviceable by divers. Wave forcing at the site was measured using a wave rider buoy equipped with three accelerometers that were used to measure buoy movement in the heave and surge directions. The wave rider buoy was designed to transmit data via radio frequency to a
station at the Seacoast Science Center at Odione’s Point on the N.H. coastline. Wave data was also obtained from the National Data Buoy Center (NDBC) web site, which contains wave information from the Portland (Buoy number 44007) and Boston (Buoy number 44013) moored buoy stations. Meteorological data also collected by the NDBC was downloaded from the Coastal-Marine Automated Network (C-MAN) station on White Island, approximately 1 nautical mile north of the demonstration site.

The objective of this portion of the investigation was to obtain environmental forcing, fish cage motion and mooring line response data sets used to analyze the dynamics of northern fish cage and mooring system. Frequency analysis was performed to calculate normalized transfer functions for comparison with physical and numerical modeling test results. Three major sets of data were acquired as part of this investigation:

- Wave elevation from a wave rider buoy,
- Motion response of the central spar fish cage, and
- Load response of the anchor and lower bridle lines of the northeast grid.

2. Data Acquisition Instruments and Parameters

2.1 Wave Rider Buoy

The wave rider buoy was provided by WHOI in support of the UNH-OOA field measurement effort. The wave rider buoy, shown deployed at the demonstration site in Figure 7.1, measures the acceleration of the buoy in the x, y and z directions. The z direction represents the buoy in the heave motion while the x and y components are combined to obtain the buoy surge (horizontal motion).
The buoy was deployed at 9:55 EST on 01/04/01 at a position of 42° 56.724 North latitude and 70° 37.715 West longitude in 51 meters of water nearly due east of the north central spar cage. It was set to burst sample every three hours starting at 0000 Greenwich Mean Time (GMT). Each burst sampled the heave and surge acceleration of the buoy for a duration of 20 minutes at 10 Hz resulting in 11328 counts. This data was stored internally on a flash card in the dry well of the buoy and radio transmitted to the Seacoast Science Center located on Odione’s Point in Portsmouth NH approximately 6 miles north-west of the demonstration site. Originally, the data was to be transmitted in hexadecimal format and stored on a computer located at the science center and publicly available via file transfer protocol (FTP). It was discovered, however, that when seas were greater than 2 meters (significant wave height), the telemetry link worked poorly. Therefore, the entire buoy was retrieved on 03/15/01 to ensure that the data acquired during the winter months was recovered.
2.2 Motion Data Package

To measure the motion response of the cage to wave forcing, a Systron Donner motion package, provided by the WHOI engineers, was mounted onto the spar of the surface cage of the north mooring system. The motion package consists of motion sensors measuring acceleration in and rotation around the x-, y- and z- axis (6 degrees of freedom). This system also includes an anti-alias filter and a PC104 based data logging system. The electronics were housed in a watertight aluminum canister, which was placed in a mounting bracket that was attached to the spar of the cage using band straps (Figure 7.2) at a depth of approximately 5 meters. The device was programmed to sample every three hours starting at 0000 GMT for 18 minutes and 12 seconds (10920 samples) at a rate of 10 Hz. Parameters recorded include 6 degrees-of-freedom motions, two load cells mounted on the cage at the upper bridle lines and the battery voltage. Other data acquisition channels were also designed into the system to incorporate other instrument capabilities for future studies. The motion package was deployed on 01/19/01 and retrieved on 03/27/01.

![Figure 7.2: Motion package canister attached to the spar of the fish cage.](image)
2.3 Mooring System Load Cells

During the refurbishing of the north cage and mooring system, nine load cells (without data loggers) were deployed as integrated components in the mooring. A schematic with all of these locations is shown on Figure 7.3. These locations include each of the mooring anchor legs (load cell numbers 7, 2, 8 and 1 for the N.E., S.E., S.W. and N.W. anchor lines respectively). At the northeast corner, three additional load cells were deployed on each of the grid lines and the lower bridle line, load cell numbers 5, 6 and 3 respectively. The N.E. corner was heavily instrumented because Northeastern storm events are typically responsible for extreme wave conditions in the Gulf of Maine (Pearce and Panchang, 1983). The last two load cells were attached to the upper bridle lines at the central spar cage connection (load cells 9 and 10). Load cell number 4 was not deployed because it was used as a laboratory spare.

![Figure 7.3: Load cell positions in the north cage/grid system.](image-url)
The first deployment of the data recorders onto the mooring system was between 10/24/00 and 01/19/01. The load cell recorders were serviced and redeployed on 01/24/01. Recorder L5 was then retrieved on 03/27/01 and examined for northeast storm data. Recorders L1, L2 and L3 remained on station until July 2001. Table 7.1 provides the load cell and recorder details for these deployments.

**Table 7.1: Location of load cells on the north mooring system.**

<table>
<thead>
<tr>
<th>Load Cell Number</th>
<th>Recorder</th>
<th>Location</th>
<th>Line Measured</th>
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<tbody>
<tr>
<td>001</td>
<td>L1</td>
<td>Northwest</td>
<td>Anchor</td>
</tr>
<tr>
<td>002</td>
<td>L2</td>
<td>Southeast</td>
<td>Anchor</td>
</tr>
<tr>
<td>003</td>
<td>L5</td>
<td>Northeast</td>
<td>L. Bridle</td>
</tr>
<tr>
<td>004</td>
<td>N/A</td>
<td>Lab. Spare</td>
<td>N/A</td>
</tr>
<tr>
<td>005</td>
<td>L5</td>
<td>Northeast</td>
<td>West Grid</td>
</tr>
<tr>
<td>006</td>
<td>L5</td>
<td>Northeast</td>
<td>South Grid</td>
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<td>007</td>
<td>L5</td>
<td>Northeast</td>
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<tr>
<td>008</td>
<td>L3</td>
<td>Southwest</td>
<td>Anchor</td>
</tr>
</tbody>
</table>

2.4 National Data Buoy Center (NDBC) Data

Data sets were also obtained from the National Oceanic and Atmospheric Administration (NOAA), National Data Buoy Center (NDBC) division, which operates environmental data collection systems along the United States coast including the Gulf of Maine. Near the demonstration site, the NDBC maintains two moored buoys and a C-MAN station (Figure 7.4). The moored buoy stations collect oceanographic data including wave elevation information. The buoys are located off the coast of Boston, MA (number 44013) and Portland, ME (number 44007) at the positions 42.35 and 70.69, and 43.53 and 70.14, North latitude and West longitude respectively (see [www.ndbc.noaa.gov](http://www.ndbc.noaa.gov) for a more detailed description). A meteorological C-MAN station is located approximately 1 nautical mile north of the demonstration site on the White Island lighthouse in the Isles of Shoals (station IOSN3). Wind and wave data from these
stations were used in the analysis of the field data collected to estimate the direction of the seas and to compare wave data collected with the OOA operated wave rider buoy with downloaded data sets from the Boston and Portland buoys.

![Map of NDBC moored buoy and C-MAN stations in the northeast](image)

**Figure 7.4:** NDBC moored buoy and C-MAN stations in the northeast (Figure downloaded from NDBC website).

### 3. Data Processing

Time series data, including ocean surface elevation and motion package accelerations, and loads from the anchor and bridle line load cells from the northeast grid corner were processed using similar spectral techniques as the data from the numerical and physical model test results. The time series data was transformed into the frequency domain to obtain auto-spectral density representations. Frequency dependant transfer functions were calculated to obtain normalized parameters for comparison with the numerical and physical model data. Detailed descriptions of the theoretical development of the data processing procedures are provided in Chapter 4.
Load cell data from the northeast anchor and bridle lines (number 7 and 3, respectively), was processed for a N.E. sea for comparison with the physical and numerical model data results. Wind data from the National Data Buoy Center (NDBC) weather station on White Island (C-MAN Station IOSN3 on Figure 7.4) was examined for sustained winds on the order of 15 m/s (30 knots) coming from a direction between 15 and 75 degrees (True) for at least 5 hours. Wave data from the Portland and Boston moored buoy stations was downloaded from the NDBC website and compared with the wave data collected from the OOA wave rider buoy.

The data from the OOA wave rider buoy consisted of a total of 20 minutes of accelerometer information in the x, y and z directions. Sampling at a rate of 10 Hz, 11328 counts for each direction were obtained. The time series acceleration data sets were first filtered using a fifth order, high-pass, Butterworth filter set at 0.05 Hz (Parks and Burrus, 1987 and Oppenheim and Schafer, 1989). These records were processed by segmenting the first 10240 counts (17 minutes, 4 seconds) into 10 sections each 102.4 seconds long. Nine additional sections of data, of the same length, overlapping the previous ten by 52.2 seconds were then taken from the total time series. Each of the sections were demeaned, detrended and weighted using a Blackman type window (Oppenheim and Schafer, 1989) of the form,

\[
\text{window}(m) = 0.42 - 0.5 \cos \left( 2\pi \frac{m - 1}{n - 1} \right) + 0.08 \cos \left( 4\pi \frac{m - 1}{n - 1} \right), \quad m = 1, \ldots, n \quad (7.1)
\]

to reduce sidelobe leakage. A power factor to correct each spectral estimate was not used because overlapping sections were utilized and power differences were attributed to noise and accounted for using a noise correction function.

The sections were then transformed into the frequency domain as energy densities
and ensemble averaged resulting in an acceleration spectrum with a frequency resolution of 0.0098 Hz. This procedure is consistent with that performed by the NDBC, Portland and Boston buoys as described by Steele et al. (1985).

The OOA wave rider buoy used fixed accelerometers placed in the hull to detect buoy response to the surface waves. In the frequency domain, fixed accelerometers produce spurious energy at low frequencies that become unrealistic spikes when converted to displacement spectra. Frequency dependant noise correction functions have been employed using various methods (Earle et al., 1984, Steele et al., 1985 and Lang, 1987) to estimate these energy levels so that the data can be corrected such that,

\[ S_{\text{acc}}(f) = S_{\text{meas}}(f) - NC(f) \]  

(7.2)

where \( S_{\text{acc}}(f) \) is the actual acceleration spectrum, \( S_{\text{meas}}(f) \) is the measured acceleration spectrum and \( NC(f) \) is the noise correction function. For these instruments, the noise correction function was chosen to be characteristic of the sea-state during data acquisition.

The high-pass Butterworth filter set at 0.05 Hz eliminated much of the low frequency noise before the data was transformed into the frequency domain and ensemble averaged. Even after this process, a substantial amount of noise across all of the frequencies existed that appeared to be a function of the sea-state. A noise correction function was therefore chosen specific for each data set based on the acceleration energy at nominal values greater than 0.6 Hz. This high frequency limit was chosen because it is close to the upper bounds where gravity waves exist (Kinsman, 1965). The acceleration energy found at this frequency was set as the NC function and subtracted throughout each band. Figure 7.5 shows the results of this procedure for data collected on March 6 at
0600 GMT. The black curve is the raw heave acceleration spectrum. The red curve is the processed data after applying the Butterworth filter, sectioning and ensemble averaging, and application of the NC function.

![Processing of the wave buoy accelerometer data](image)

**Figure 7.5: Processing of the wave buoy accelerometer data.**

The "corrected" accelerometer spectral data from the OOA wave rider buoy was converted to a heave displacement spectrum using the following expression,

\[ S_{\text{disp}}(f) = \frac{S_{\text{accel}}(f)}{H(f)^2(2\pi f)^4}, \]  

(7.3)

where

- \( S_{\text{disp}}(f) \): Auto-spectral density of the heave motion (m^2/Hz),
- \( S_{\text{accel}}(f) \): Actual auto-spectral density of the heave acceleration \( \left( [m/s^2]^2/Hz \right) \),
- \( H(f) \): Heave linear transfer function (m/m) of the buoy,
- \( f \): wave frequency (Hz).

A similar method is used by the NDBC in their algorithm to find wave elevation using surface buoys (Huang, 1998). The heave linear transfer function for the wave rider
buoy is estimated to have a slight heave resonance at 0.5 Hz (according to a simplified example in Berteaux, 1991). For all other wave frequencies less than 0.5 Hz, it is assumed that the linear transfer function is equal to one.

The accelerometer data from the motion package on the cage was much cleaner with little spurious energy in the low frequencies. These data sets were processed by using the same high pass filter (in the time domain) set at 0.05 Hz. The data was then sectioned and weighted with a Blackman window, converted to the frequency domain and ensemble averaged. The rotational accelerometers on the motion package measure a rate of rotation with the units deg/sec. The rotational responses, pitch (rotation about the east-west axis), roll (rotation about the north-south axis) and yaw (rotation around the vertical axis) were numerically integrated using the trapezoidal rule and converted to radians. A noise correction function was not required to processes these data sets. The load cell data was processed in a similar manner, except integration was not required.

4. Field Data Results

4.1 Winter 2000-2001 in the Gulf of Maine

The winter of 2000-2001 in the southwest corner of the Gulf of Maine contained multiple northeast storms typical of the region. The northeast direction was chosen because one of the north cage grid anchor legs was deployed in this direction and best represents the orientation used in both the numerical and physical models. Classification of these events was performed by analyzing wind data from the C-MAN station on White Island. Data collected during periods when the wind speeds greater than 15 m/s (30 knots) coming from a (nominal) direction between 15 and 75 degrees true were considered for investigation. The northeast storms used in this analysis occurred
December 30 and 31, February 5 and 6, and the most severe of the three on March 5-7. Data sets collected from these events were used in this investigation depending upon which instruments were deployed at the demonstration site. Table 7.2 provides wind speed and direction for these events and the available demonstration site data used in the frequency analysis.

Table 7.2: Winter 2000-2001 storm data sets used in the frequency analysis.

<table>
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<th>Date</th>
<th>Wind Speed (m/s)</th>
<th>Wind Direction</th>
<th>Wave Data</th>
<th>Motion Pack Data</th>
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</table>

\(^a\) All time zones are Greenwich Mean Time (GMT).
\(^b\) Motion pack data is available for the cage without the ballast weight.
4.2 Wave Data Results

Accelerations from the wave rider buoy during these storms were adjusted using the noise correction function procedure and converted to displacement spectra as previously outlined. All of the heave displacement data collected during these events were plotted and compared to spectral wave information from the NDBC Portland and Boston buoys (see Appendix J). Energy based, significant wave heights and peak periods were calculated and the results provided on Table 7.3. Also provided on Table 7.3 are the wave data results acquired from the Portland and Boston buoys.

Table 7.3: Wave data comparison between the OOA, Portland and Boston sites.

<table>
<thead>
<tr>
<th>Date</th>
<th>OOA H&lt;sub&gt;m0&lt;/sub&gt; (m)</th>
<th>OOA T&lt;sub&gt;p&lt;/sub&gt; (sec)</th>
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<td>11.00</td>
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<td>0300</td>
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<td></td>
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<td></td>
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</tr>
</tbody>
</table>

*All time zones are Greenwich Mean Time (GMT).*
The storm event that occurred from March 5 through the 7 was a full-blown Northeaster with sustained winds on the order of 20 m/s (40 knots) for a period of 24 hours. This fetch and duration combination produced nearly fully developed seas. The growth of the sea can be shown by log-log plotting the spectral data (Figure 7.6). Also shown Figure 7.6 is the Pierson-Moskowitz (P-M) spectrum. The P-M spectrum, as described in Pierson and Moskowitz (1964), is one of the classical representations of a local, fully developed wind-driven sea. As with the P-M spectrum, the equilibrium range of the OOA wave spectra is proportional (approximately) to frequency to a power of minus five (between 0.2 and 0.5 Hz), although the OOA spectra appear to have more energy, by an order of magnitude, between the frequencies of 0.15 and 0.3 Hz. This power relationship is consistent with results described in Ochi (1998). It also appears the OOA spectra are bi-modal (two dominant peaks). This characteristic is typical of a building sea in which a local wind-driven component containing higher frequency waves is superimposed onto older, non-local, lower frequency swell (Ochi, 1998). The superposition of two wave fields may be one explanation why the wave conditions at the demonstration site are more energetic than the P-M representation.

The peak of the storm at the demonstration site occurred on March 6 at 0900 GMT \( (H_{mo} = 8.27 \text{ m}, T_p = 10.24 \text{ sec}) \). This is estimated to be a wave system of a 25-year storm (see Appendix A). A time series representation of the sea was calculated by double trapezoidal integration of the high-pass filtered acceleration data (without noise correction) to estimate the surface characteristics (Figure 7.7). Analysis of the time series data indicates that an approximate 17 meter wave could have existed at the demonstration site. Unfortunately, on March 5, the northeast load cell data logger stopped recording, so
mooring line tensions were not collected during this event. The motion package strapped to the spar of the cage did survive and the data was analyzed.

Figure 7.6: Wave energy growth during the March 6, 2001 northeast storm.

Figure 7.7: Estimated wave elevation time series on March 6, 2001 at 0900 GMT.
4.3 Central Spar Cage Motion Response

Using data from 14 data sets, wave forcing and motion response accelerations were obtained to perform linear transfer function calculations. The auto-spectral density results and transfer functions are provided in Appendix K. The variance for the heave, surge and pitch response was calculated from each spectral estimate using equation (4.11) and the results provided on Table 7.4. An example set of the auto-spectral forcing and motion responses is shown on Figure 7.8 for data collected on March 6 at 0600 GMT. Linear transfer functions were calculated for each of the 14 data sets and ensemble averaged. The ensemble-averaged results for each of the 3 degrees of freedom are shown on Figure 7.9.

<table>
<thead>
<tr>
<th>Date</th>
<th>Heave Response Variance (m²)</th>
<th>Surge Response Variance (m²)</th>
<th>Pitch Response Variance (rad²)</th>
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<td></td>
<td></td>
</tr>
<tr>
<td>2100</td>
<td>0.0367</td>
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<td></td>
<td></td>
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<td>0000</td>
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<td>0.0732</td>
<td>0.0019</td>
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<td>0.1195</td>
<td>0.0024</td>
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<td>0.4453</td>
<td>0.1155</td>
<td>0.0026</td>
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<td>0.2269</td>
<td>0.0652</td>
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<td>March 5, 2001</td>
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<td></td>
</tr>
<tr>
<td>2100</td>
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<td></td>
<td></td>
</tr>
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<td>0.1063</td>
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<td>0.0023</td>
</tr>
</tbody>
</table>

* All time zones are Greenwich Mean Time (GMT).
Figure 7.8: Wave forcing and fish cage response for heave, surge and pitch.

Figure 7.9: Ensemble averaged cage transfer functions in heave, surge and pitch.
4.4 Pendant Weight Failure

When the motion package was recovered in late March, it was discovered that the pendant weight attached to the bottom of the spar of the cage had failed during one of the winter storms. Without the pendant weight, the cage floated to the surface tensioning all of the attached bridle lines, as shown on Figure 7.10. Investigation of the heave motion response data sets revealed that the failure occurred at some time after March 6 at 2100 GMT. This was discovered by comparing heave motion response time series for data collected on March 6 at 2100 GMT and March 7 at 0000 GMT (Figure 7.11). Data on the Figure shows that in a similar sea state (see Table 7.3), the standard deviation difference of the motion response was nearly 65%. Furthermore, the dominant wave period during the later stages of this storm approached the natural period of a free-swinging pendulum with a length of 33 meters (11.52 seconds), equal to the length of the pendant line.

![Figure 7.10: Northern cage without the pendant weight.](image)

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4.5 Static Tension Response

The static tension response of the anchor, grid and bridle lines of the fish cage mooring was examined for comparison with the designed analytical results. Load cell data for the northeast grid corner was analyzed during calm seas for a 24 hour period (two tidal cycles) to average tidal current influences (see Table 7.5). Averaging each of the anchor, bridle and grid line tension mean values resulted in 3.85 kN, ~0 kN, and 1.80 kN, respectively. Using equations (2.5) through (2.10), an analytical value for the anchor and grid line pretension was calculated to be 9.2 kN and 7.32 kN. The differences, however, can be accounted for. The static tensions are a function of the geometry of the system. In the deployment process, using vessels larger than 33 m, it is nearly impossible to obtain the ideal geometric condition that the analytical condition uses. Furthermore, it was discovered during the “deployment” of the physical model in the UNH wave tank, a
good deal of static tension sensitivity was discovered by changing the geometry by small amounts.

Table 7.5: Static tension data for the northern mooring system.

<table>
<thead>
<tr>
<th>Date</th>
<th>Anchor Mean (kN)</th>
<th>Anchor STD (kN)</th>
<th>Grid Mean (kN)</th>
<th>Grid STD (kN)</th>
<th>Bridle Mean (kN)</th>
<th>Bridle STD (kN)</th>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>1200</td>
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<tr>
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<tr>
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<td></td>
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<td></td>
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</table>

* All time zones are Greenwich Mean Time (GMT).

4.6 Anchor and Bridle Line Tension Response

Prior to the March storm, the northeast load cell data recorder stopped collecting information. To obtain anchor and bridle line transfer functions, four data sets from the February 6 storm event were analyzed and compared with the wave forcing. The spectral comparisons and transfer functions are provided in Appendix L. An example of one of these comparisons is shown on Figure 7.12. These four anchor and bridle line linear transfer functions were ensemble averaged and the results shown on Figure 7.13. For each frequency, the linear transfer function did not exceed 3 kN/m. At most wave frequencies (greater than 0.12 Hz), tension transfer functions were typically within 1 kN/m. In addition, the anchor and bridle line response variance was calculated from each of the spectral estimates using equation (4.11) and the results provided on Table 7.6.
Figure 7.12: An example of the anchor and bridle line spectral response.

Figure 7.13: Anchor and bridle line transfer function ensemble average.
Table 7.6: The anchor and bridle line response variance for the data sets processed.

<table>
<thead>
<tr>
<th>Date</th>
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<td>0900 5.4416</td>
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<tr>
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<td>0900 2.5181</td>
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</table>

*All time zones are Greenwich Mean Time (GMT).

To analyze the distribution of loads in the mooring system, load cell data from December 31 was also investigated because of the high magnitudes that were measured (see Figure 7.14) even though the motion package and OOA wave rider buoy had not yet been deployed. The anchor and bridle line time series data show a set of peaks greater than 20 kN after the midpoint of the record. The grid line tension, on the other hand, maintained a relatively constant tension. The Boston and Portland buoys recorded energy based significant wave heights of 4.49 and 3.00 meters, respectively, during this time.
5. **Discussion of the Field Data Results**

The field measurement program included the collection of data from multiple instruments deployed at various times throughout the winter of 2000-2001 to obtain information concerning the cage motion and mooring line tension response in waves. As with nearly every field measurement program, 100% data recovery is rare. In this study, the only “failure” was the loss of the anchor and bridle line load cells prior to the March northeast storm. Fortunately, data from a smaller storm in February was utilized to calculate linear anchor and bridle line transfer functions. The motion package on the central spar cage and the wave rider buoy, however, performed exceptionally well allowing the calculation of heave, surge and pitch transfer functions from 14 distinct data sets.
Examining the growth of wave energy from the March northeast storm yielded some interesting findings. In the later stages of the storm, the non-local swell component becomes more mature as the dominating wave periods increase beyond 10 seconds (Table 7.3). It seems coincidental that in seas with longer period waves that the ballast weight attachment line failed. However, as shown with equation (5.4), the natural frequency of a simple pendulum with an attachment line length of 33 meters is equal to 11.52 seconds, while the wave dominant periods prior to failure were on the order of 11 seconds. As shown with the physical model tests, some type of resonance could occur due to the swinging of the ballast weight while the central spar cage is at the surface. At this time, the exact mode of failure is unknown. Nevertheless, future tests should be conducted to investigate the cage response for the submerged case as well. The responses of the cage in heave and surge, on the other hand, show evidence of an over damped system, most likely due to the drag of the nets.

The anchor and bridle line tension response revealed linear transfer function results within an upper limit of 3 kN and typical values within 1 kN/m. Even though the relationship between the wave elevation and tension amplitudes is mostly linear, it is possible that a non-linear component exists so extrapolating these results to extreme wave conditions (50- or 100- year storms) should be performed cautiously. The results can be used for comparison with the physical and numerical model tests because the input wave forcing for all three experiments were moderate in amplitude.

The distribution of loads between the anchor, bridle and grid line was also investigated (Figure 7.14). Tension perturbations of the anchor line fluctuated around the mean while the changes in the bridle line loads were typical in the positive direction.
primarily because of little initial pretension. While, even in high-energy conditions, the
grid line tension values remained somewhat constant. Future design safety factors, for
similar type mooring components should reflect the load distribution results found as part
of this field measurement program.
CHAPTER 8

SYSTEM AND METHOD EVALUATION

1. Gulf of Maine Survival

One of the most significant results of this investigation is that both the northern and southern cage mooring systems survived two Gulf of Maine winters. In the first winter, each of the central spar cages was placed in the submerged configuration. In the second winter, the cage of the refurbished northern system was kept at the surface to collect motion response and mooring line tension engineering data. Both the cage and the mooring survived multiple northeast storms. Furthermore, during the most severe storm occurring on March 6, the maximum $H_{mo}$ was calculated to be 8.27 meters with peak periods over 11 seconds. It is possible that this portion of the storm approached the “25-year storm” for sites near Portsmouth N.H. (see Appendix A). This engineering test showed that the design concept and safety factors developed in the initial stages of the project described in Chapter 2 were not only sufficient but also robust. This design philosophy is necessary in a demonstration project to ensure research continuity. Once the concept is proven, steps can be taken not only to test candidate species and ancillary equipment (i.e. feeding systems) but also to evaluate and optimize the engineering methods necessary to build an economical open ocean aquaculture facility.

2. Central Spar Cage Motion Response

2.1 Transfer Function Results

Seakeeping experiments are typical in the naval architecture and offshore
engineering fields to evaluate the motion response of at-sea ships and structures. Human design criteria for these systems have been established to reflect operational safety conditions (Faltinsen, 1990). Likewise, the successful growout of finfish stocked in a net pen in the open ocean and subject to stochastic forces requires an understanding of the biological criteria and the dynamics of the containment system response. To examine the dynamics, heave, surge and pitch experiments were conducted using physical and numerical models and normalized to obtain linear transfer functions using both regular and random wave forcing. The heave, surge and pitch data sets were then compared with normalized results collected in the field at the OOA demonstration site (Figures 8.1, 8.2, and 8.3, respectively).

![Heave Transfer Function Results](image)

**Figure 8.1: Heave transfer function results comparison.**

In general, the heave motion response for the physical and numerical model tests matched remarkably well. Only the regular wave test results performed in the wave tank
showed conservative differences. Regardless, at wave frequencies greater than 0.09 Hz, an over damped system (normalized values are less than one) in heave exists with values approximately 0.5 at 0.1 Hz. At frequencies less than 0.09 Hz, it is unclear if a resonant condition occurs. The accuracy of all the measurements at frequencies between 0.05 and 0.09 Hz are suspect because the relative amplitudes used in the transfer function calculations are small.

![Surge Transfer Function Results](image_url)

**Figure 8.2: Surge transfer function results comparison.**

Both the physical and numerical model results over predicted the surge response of the cage. One possible explanation is the influence of tidal and other currents. The drag on the cage may set the system back and restrict the horizontal motion. At the OOA site, currents can exceed 40 cm/s (Bub, 2001). This may have influence on a non-linear component of the surge response that was not simulated using the physical and numerical models.
The pitch linear transfer function results revealed some interesting characteristics. Both the physical model tests and the field data showed a resonant condition. The physical model tests results, however, predicted the resonance at a higher frequency. It is suspected that the swinging pendant weight influences the pitch motion. In the physical model tests, the full-scale pendant line length was shorter than the one deployed. As discussed in Chapter 5, a system with a shorter pendant line length will resonate at a higher frequency. The numerical model pitch response, on the other hand, under predicted the transfer functions calculated with the field data and did not show a resonance. The resonant condition appears to occur at wave frequencies less than 0.1 Hz, which can be typical at the site. Submerging the cage may reduce the overall response of the system because wave energy decreases with depth. However, a series of submerged tests should be performed because a (reverse) pendulum may still exist.
2.2 Linear Coherency of the Cage Response

For each of the physical and numerical model motion response tests, linear coherency-squared function calculations were performed to further investigate the system response. For the most part the results of the coherency-squared function indicate a linear system in heave, surge and pitch between the nominal frequencies of 0.05 and 0.2 Hz as shown on Figure 8.4. Higher order response analysis should not be necessary for evaluating systems in waves. However, the influence of tidal and other currents may have a non-linear effect in surge at low frequencies.

![Figure 8.4: Linear coherency-square function comparison between the physical and numerical model results.](image)

2.3 Biological Considerations

Understanding the biological criteria with respect to the design and placement of an open ocean fish cage is difficult because many of the variables are unknown. Optimal temperature and salinity for maximized growing have been investigated for many
candidate species. Some have also investigated the response of fish in currents as described by Grove et al. (1991), Riley (1991) and Kuo and Beveridge (1990). But is there a wave frequency response of fish? Or are there certain cage movements that will cause stress to the fish?

These biological criteria need to be considered to maximize the motion response characteristics of the cage as a function of frequency. The normalized linear transfer functions calculated as part of this study can be used to assess these motions for the deployed fish cage and mooring system. For instance, if the transfer function values are near one, the entire cage structure and the fish move with the motions of the wave. The fish will not sense the wave orbital velocities, but if the motion is excessive, they may experience motion sickness and therefore stress. This condition typically occurs for lower frequency waves. If the values are substantially less than one, the cage remains stationary and the fish may sense the wave particle motions. In this environment, the fish may have to swim to hold their position relative to the cage. This could cause excessive fatigue that can affect growth rates. On the other hand, if the fish choose not to swim, they will move with the particle motions and may be more susceptible to abrasion with cage parts. Results of the physical and numerical model tests, along with the field data information, indicate the central spar cage becomes a wave follower at wave frequencies less than 0.1 Hz. In fair weather (frequencies greater than 0.1 Hz), however, the cage follows the wave motion less and therefore is subject to wave relative velocities and accelerations. Using this cage at the surface in a high-energy environment to contain bottom dwelling flatfish such as flounder or halibut may not be the optimal configuration.

The fish cage response to the March storm provided insight to a possible system
design weakness. In the mature stages of the storm, the pendant weight line attached to the spar of the cage failed, which from an engineering perspective can be considered just a nuisance because the whole of the system remained on location. From a biological standpoint, however, this failure mechanism could have disastrous consequences even in the submerged condition. For example, if the cage was being used to contain a fish with an air bladder such as cod, loss of the ballast weight at some depth causing the cage to rise to the surface may have killed the entire stock due to lack of decompression (Howell, 1996). Even though the submerged dynamics may be different than in the surface case, a similar mode of failure could exist and tests should be conducted to investigate this. Nevertheless, these and other biological motion response criteria need to systematically addressed and quantified.

3. Load Response

3.1 Static Response

In this analysis, data collected to investigate the static characteristics of the anchor and bridle lines was performed in the physical and numerical model tests and compared with data obtained from the field system. These values were then compared with the original static calculation performed using inextensible catenary equations (2.5) through (2.10), as defined by Faltinsen (1990). The results are provided in Table 8.1.

<table>
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<th>Analysis Type</th>
<th>Bridle Line Tension (kN)</th>
<th>Anchor Line Tension (kN)</th>
<th>Grid Line Tension (kN)</th>
</tr>
</thead>
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</tr>
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<td>12.4</td>
<td>n.a.a</td>
</tr>
<tr>
<td>Numerical Model</td>
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</tr>
<tr>
<td>Deployed System</td>
<td>~0</td>
<td>3.85</td>
<td>1.80</td>
</tr>
</tbody>
</table>

*a Grid line tension was not measured during these tests.*
The difference in the deployed system static tensions with the other results is due to the difficulty of achieving the ideal geometry in the field. Even in a controlled environment, such as the UNH tow/wave basin, “deploying” the system was time consuming and very sensitive to geometry. This aspect of the physical modeling testing is invaluable in the practical design process. If a 1:15.2 scale system is difficult to deploy in the laboratory, the full-scale fish cage and mooring system will be orders of magnitude more difficult to deploy in the open ocean. In the past two years, two separate deployment/recovery operations have been performed, as described by Baldwin et al. (2000) and Irish et al. (2001) that reflect these complexities.

The static analysis using the numerical model AquaFE also produced useful results. Changes to the numerical model code were made to more accurately simulate the chain catenary interaction with the ocean bottom. The static line tension is a function of geometry of the system, the placement of flotation elements, and the chain catenary that connects the anchor to the anchor line. Comparison of the numerical and analytical models for the anchor and bridle line tension shows that only a 7.4% and 12% difference exist, therefore validating the code changes.

3.2 Linear Transfer Function Comparison

Linear transfer functions were also calculated between the wave elevation forcing and the anchor and bridle line tension response for the physical and numerical model tests. These results are compared with linear transfer functions calculated using the field data (Figures 8.5 and 8.6). Unlike the motion response of the cage, the anchor and bridle line tension response may have a slightly more of a non-linear relationship with respect to the waves. The transfer functions shown here represent the first-order linear approximation.
In general, all of the linear transfer function results were constant through the nominal frequencies of 0.1 to 0.3 Hz. The anchor and bridle line tension response measured during the physical model tests slightly over-predict the in-situ results. While the random wave results of the numerical model closely resemble the field data calculated linear transfer functions. Tests using the FEA model with regular waves, however, over-predict the field value calculation between the frequencies of 0.15 and 0.3 Hz. Examination of the data indicates that all of the results fall within a nominal upper value of 5 kN/m, which is used in this analysis as a conservative transfer function for the range of operating frequencies.

![Anchor Line Transfer Function Results](image1)

![Bridle Line Transfer Function Results](image2)

**Figure 8.5: Physical model linear transfer comparison with field results.**
3.3 Linear Coherency of Mooring Load Response

The linear coherency-squared function was also calculated for the anchor and bridle line tension for each the physical and numerical model random wave test series. The results are shown on Figure 8.7. The physical model coherency characteristics indicate that a reasonable linear relationship exists. This could be due, in part, to the net modeling technique described in Chapter 3. In an effort to adjust the frictional component at the model scale, which is Reynolds number dependant, a modified cross-sectional area was used on the model based on equation (3.7). This method reduces the frictional drag component, which is proportional to velocity squared and could reduce the anchor and bridle line tension response.
Figure 8.7: Coherency-squared function result for the physical and numerical models tests.

The numerical model coherency also showed some linear trends between 0.08 and 0.175 Hz. The AquaFE model uses a Morison equation formulation with linear and non-linear parts, drag and inertia force, respectively (equation 3.8). If little measurement error exits, results of the coherency calculation can provide insight on which part of the Morison equation is dominating for a specific frequency.

In general, linear coherency is evident for both the physical and numerical model tests results indicating that the original assumption of a linear system fits reasonably well. If a quadratic influence does exist, however, it can be investigated using bispectral analysis, which can be performed to obtain quadratic transfer and coherency-squared functions. This technique has been performed by Linfoot and Hall (1985) in their physical scale model testing of sea-cage systems. They found that mooring loads, in general, show a quadratic trend as the wave height increased. Future physical and
numerical model tests can also be performed with larger wave amplitudes and compared with these results to test if a non-linear relationship exists.

3.4 Design Load Considerations

The design wave condition used in the original analysis had a wave height and period of 9 meters and 8.8 seconds, respectively, with current velocity equal 1 m/s. Physical model tests in waves and currents were performed using the methods described in Chapter 2 and equations (2.1) through (2.4). From the physical model tests, a design load of 77 kN (see Chapter 2) was established for the specification of the mooring components deployed at the OOA site. In a complementary effort, numerical model tests were performed by Ozbay (1999) and Tsukrov (2000) using an older version of the AquaFE model without the use of a bottom contact conditional and shadowed element(s). They performed simulations using the design condition input and obtained anchor and bridle line tension amplitudes of 113 kN and 80.4 kN, respectively. Since the chain catenary interaction with the ocean bottom had not yet been employed, numerical model results were considered excessive. Therefore, the physical model test results were used to calculate design safety factors provided on Table 2.3.

The design safety factors deserve additional consideration in lieu of the experience gained in the past two years. Deployment of the mooring system was costly because of the shear size of the components. Reducing the size of the gear would decrease component and operational costs and perhaps help streamline the economics of open ocean aquaculture. Making this assessment, however, is difficult because failure is catastrophic. One of the goals of this research is to begin to quantify component specification using field data so that the physical and numerical models used are more accurate and experiments/simulations are performed with higher confidence.
The first step was to re-perform the numerical model simulations using the new AquaFE model and the input design criteria. With the bottom conditional and the net shadowing techniques employed, maximum anchor and bridle line amplitudes were calculated to be 95 and 82 kN, respectively (see Chapter 6).

Another method was considered using a stochastic approach and the combined results of this study. A narrow banded spectral representation of an extreme wave condition was developed for sites near Portsmouth, NH using a fitted JONSWAP spectrum (equation A.6). Energy based significant wave height values were extrapolated for return periods of 10-, 25- and 50-year storms and used as input to the fitted spectrum calculations (Figure 8.8). The estimated, nominal linear mooring load transfer function of 5 kN/m was applied in the frequency domain to obtain a mooring load spectra for the 10-, 25- and 50-year storms (Figure 8.8).

If the wave and response spectra are narrow banded and if the amplitudes follow a Raleigh distribution, statistical wave and mooring load response parameters can be calculated. The root-mean-square, the average of the top third, and average of the top 10 wave heights and tension amplitudes can be calculated according to relationships described in the SPM (1984). The 10-, 25- and 50-year storm wave heights and tension amplitude response parameters are provided on Table 8.2. Using a nominal value of the steady drag force equal to 20 kN, obtained from the physical, numerical and field tow results described in Chapter 3, and the $T_{10}$ wave force value for the 50-year storm, a new design load value of 96 kN was calculated using equations (2.1) through (2.4).
Figure 8.8: Estimated 50-, 25-, and 10-year storm and mooring load response.

Table 8.2: Stochastic mooring load tensions.

<table>
<thead>
<tr>
<th></th>
<th>10-Year Storm</th>
<th>25-Year Storm</th>
<th>50-Year Storm</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_{rms} \text{ (m)}$</td>
<td>5.44</td>
<td>5.96</td>
<td>6.30</td>
</tr>
<tr>
<td>$H_{1/3} \text{ (m)}$</td>
<td>7.70</td>
<td>8.43</td>
<td>8.90</td>
</tr>
<tr>
<td>$H_{10} \text{ (m)}$</td>
<td>9.78</td>
<td>10.7</td>
<td>11.3</td>
</tr>
<tr>
<td>$T_{rms} \text{ (kN)}$</td>
<td>13.7</td>
<td>15.0</td>
<td>15.8</td>
</tr>
<tr>
<td>$T_{1/3} \text{ (kN)}$</td>
<td>19.4</td>
<td>21.2</td>
<td>22.4</td>
</tr>
<tr>
<td>$T_{10} \text{ (kN)}$</td>
<td>25.6</td>
<td>26.9</td>
<td>28.4</td>
</tr>
</tbody>
</table>

* Tension values represent amplitudes

This design load matches with the result calculated from the AquaFE numerical model calculations. It also suggests that the design safety factors (see Table 2.3) previously calculated using the original design load of 77 kN are less conservative than expected. It is debatable, however, whether the design current chosen (1 m/s) chosen to
determine the design load is suitable for the demonstration site. Unpublished Acoustic Doppler Current Profiler (ADCP) data from the OOA site exists that needs to be processed to determine how conservative of a value it is. Others may also argue that a 50-year storm load is too conservative for a fish cage and mooring system with components that could be replaced, for example, every five years.

Load transfer function results calculated as part of the field program are linear estimates based on data collected while the central spar cage was at the surface during severe winter weather. If the relationship between the waves and the mooring line loads at some frequencies are a function of velocity squared, quadratic transfer function calculations may be necessary to refine this relationship. Additional physical and numerical model tests can be performed using various sea-state severities to investigate how the loads in the mooring system response from typical to extreme conditions.

4. Wave Forces and Biological Fouling

In Chapter 3, steady forces due to tidal and other currents are discussed and modeling concepts presented. The use of the AquaFE model, using a Morison equation formulation to calculate loads and motions in waves and currents, has also been explored. In the derivation process of Morison’s equation, the width of the structure is assumed much smaller than the length of the wave. In the case of the central spar cage, nearly all components can be broken into small cylinders so the application of Morison’s equation is adequate. It has been observed, however, that heavy bio-fouling, up to 10-12 inches, of the containment nets on the cage can occur within weeks if cleaning is not performed. This changes the flow characteristics around and the time variant force on the cage due to waves.
Dimension analysis is typically used when analyzing the dominant parameters of wave forces on a body. In general, it can be shown that the time variant force \( F \)

\[
F = f(D, H, d, L, \rho, \mu, U_m, T, g),
\]

is a function of the characteristic dimension \( D \), wave height \( H \), water depth, wavelength \( L \), fluid density \( \rho \), dynamic viscosity \( \mu \), maximum horizontal velocity \( U_m \), wave period \( T \) and acceleration due to gravity \( g \). A unitless form of equation (8.1) can be obtain using typical dimensional analysis techniques,

\[
\frac{F}{\rho g H D^2} = f\left(\frac{d}{L}, \frac{H}{L}, \frac{D}{L}, KC, Re\right),
\]

where

\[
\frac{d}{L}: \text{wave-depth parameter},
\]

\[
\frac{H}{L}: \text{wave-steepness},
\]

\[
\frac{D}{L}: \text{diffraction parameter},
\]

\( KC: \) Keulegan-Carpenter number, \( U_m T/D \), and

\( Re: \) characteristic Reynolds number, \( \frac{U_m D \rho}{\mu} \).

In this dimensionless relationship, the wave-depth and wave-steepness parameters describe the incident wave characteristics (Vassalos, 1999; Sarpkaya and Isaacson, 1981). The KC number describes the importance of the viscous scale effects in sinusoidal flow. The diffraction parameter represents the relative size of structure member(s) to particle displacement, which is important when considering the diffraction of waves (Chakrabarti, 1994). The diffraction parameter and the KC number are both a function of the characteristic dimension of the structure, however, the relationships are inversely proportional. High values of the KC number subsequently yield low values of the
diffraction parameter, and vice versa.

The use of these two dimensionless parameters is important when considering the applicability of certain wave force formulations. For small values of the diffraction parameter (D/L < 0.2), the Morison equation is typically used because the wave forces are in the flow separation regime. On the other hand, for large values of the diffraction parameter (D/L > 0.2), the structure is large enough to cause scattering of the incident waves. In this situation wave drag forces become negligible and diffraction theory is typically applied. Chakrabarti, 1994 also notes that many offshore structures are hybrids with members that fall into both categories.

4.1 Wave Force Parameters for the Central Spar Cage

As described in Chapter 3, the central spar cage is built around a central spar buoy and an octagonal rim. Tensioned stays incorporated in the containment net connect these two components forming a semi-rigid structure. Nearly all of the cage components consist of relatively small cylindrical components resulting in small D/L values. When a finite element approach is used to calculate forces and loads on the structure, a Morison's equation formulation representing a flow separation problem is applicable. If heavy biofouling exists, D/L values could change as the object becomes more solid. It is possible that diffraction effects become more important as the whole of the containment nets are covered with growth.

In this extreme case, consider the diameter at the rim of the fish cage, approximately 15.5 meters, as the characteristic dimension (D). Now, the wave force parameters for the net pen/mooring system deployed at the demonstration site can be estimated using this value for D. For instance, the depth (d) at the demonstration site is known to be 52 meters. The wavelength (L) typically found at the site ranges between 6
and 240 meters. Using a wave steepness (H/L) of 1/20, a representative wave height can be determined for each wavelength. The KC number is evaluated by using the expression described in equation (8.2) where $U_m$ is the maximum horizontal wave orbital velocity defined as the following,

$$U_m = \frac{gHk}{2\omega},$$

(8.3)

where $k = \frac{2\pi}{L}$ and $\omega = \frac{2\pi}{T}$ (Dean and Dalrymple, 1991). The wave period $T$ is obtained using the dispersion relation, which is a function of the wavelength ($L$) and the depth ($d$). The results for a heavily fouled, central cage wave force parameters are provided on Table 8.3.

### Table 8.3: Fouled cage wave force parameters

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>Length (m)</th>
<th>Height (m)</th>
<th>d/L</th>
<th>D/L</th>
<th>H/D</th>
<th>KC</th>
<th>Re</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.52</td>
<td>220.43</td>
<td>11.02</td>
<td>0.24</td>
<td>0.07</td>
<td>0.71</td>
<td>2.48</td>
<td>2.07E+06</td>
</tr>
<tr>
<td>9.98</td>
<td>151.39</td>
<td>7.57</td>
<td>0.34</td>
<td>0.10</td>
<td>0.49</td>
<td>1.58</td>
<td>1.65E+06</td>
</tr>
<tr>
<td>7.14</td>
<td>79.46</td>
<td>3.97</td>
<td>0.65</td>
<td>0.20</td>
<td>0.26</td>
<td>0.81</td>
<td>1.18E+06</td>
</tr>
<tr>
<td>6.24</td>
<td>60.73</td>
<td>3.04</td>
<td>0.86</td>
<td>0.26</td>
<td>0.20</td>
<td>0.62</td>
<td>1.03E+06</td>
</tr>
<tr>
<td>5.26</td>
<td>43.23</td>
<td>2.16</td>
<td>1.20</td>
<td>0.36</td>
<td>0.14</td>
<td>0.44</td>
<td>8.69E+05</td>
</tr>
<tr>
<td>4.56</td>
<td>32.47</td>
<td>1.62</td>
<td>1.60</td>
<td>0.48</td>
<td>0.10</td>
<td>0.33</td>
<td>7.53E+05</td>
</tr>
<tr>
<td>4.02</td>
<td>25.17</td>
<td>1.26</td>
<td>2.07</td>
<td>0.62</td>
<td>0.08</td>
<td>0.26</td>
<td>6.63E+05</td>
</tr>
<tr>
<td>3.59</td>
<td>20.08</td>
<td>1.00</td>
<td>2.59</td>
<td>0.77</td>
<td>0.06</td>
<td>0.20</td>
<td>5.93E+05</td>
</tr>
<tr>
<td>3.25</td>
<td>16.45</td>
<td>0.82</td>
<td>3.16</td>
<td>0.94</td>
<td>0.05</td>
<td>0.17</td>
<td>5.37E+05</td>
</tr>
<tr>
<td>1.95</td>
<td>5.93</td>
<td>0.30</td>
<td>8.77</td>
<td>2.61</td>
<td>0.02</td>
<td>0.06</td>
<td>3.22E+05</td>
</tr>
</tbody>
</table>

The regions of validity for force prediction methods of a fixed pile are shown on Figure 8.9. Using this characteristic dimension and the wave force parameters calculated, the D/L and H/D values provided in Table 8.3 are plotted with the regions of validity shown on Figure 8.9. It is clear that the moored central spar cage is not a fixed pile, so the rigorous limit of $D/L = 0.2$ may not be exact. However, the determination of wave
forces using diffraction theory of large floating bodies (Sarpkaya and Isaacson, 1981) and moored floating breakwaters (Sannasiraj et al., 1997) have been applied successfully. The intent of the information provided in Figure 8.9 is to show that diffraction effects may be important when considering the predictions of wave forces on a heavily fouled or larger fish cage systems as wave making drag becomes more important.

![Diagram showing regions of validity for force prediction methods.](image)

**Figure 8.9: Regions of validity for force prediction methods.**

As the push for deploying multiple commercial size cages systems for use in the open ocean increases, utilizing models based on Morison's equation may become less applicable. Development of diffraction model techniques, as used by the offshore oil industry, should be investigated.
CHAPTER 9

CONCLUSION

1. Objective Review

In the open ocean, designing aquaculture facilities to withstand both steady and stochastic forcing requires a set of physical and numerical modeling analysis tools that can be used to produce accurate results. These tools were used in the initial design process to evaluate fish cages and to specify a submerged grid mooring system. It became clear from the preliminary tests that a complete and systematic study was required to understand the physical and numerical modeling methods employed and to investigate the dynamics of the system. Many believed that the gear deployed was excessively robust, but few could quantify this.

A comprehensive study to investigate these issues was therefore performed. Testing of the cage and mooring system in currents and waves using improved physical and numerical methods was conducted and compared with results from an extensive field measurement program. These tests included drag modeling of the central spar cage and mooring line tension and cage response in random waves. System analysis techniques were applied to obtain normalized linear transfer functions and coherency plots interpreted to evaluate the relationship between the forcing and response mechanisms.

2. Drag Modeling in Steady Current

The drag modeling predictions using the AquaFE model performed accurately once the velocity reduction through the netting was quantified. Measurements made
during the open ocean field tow indicated that a 10% velocity reduction through the net existed. The computer model program was modified to facilitate the application of reduced velocities caused by shadowing effects of components known to be in the wake of upstream elements. Since the program uses a Morison equation formulation that models frictional drag as a function of Reynolds number, results using the new model compared well with the field data. This technique, however, can only be used with steady current velocities. An additional improvement to the model code could include a similar velocity reduction applied to the sinusoidal wave particle velocities.

The physical model test results, on the other hand, under predicted the field data by 30% even with the Reynolds number adjustment used for the nets. Future work should include either a more refined Reynolds number adjustment method or some other approach. One method to consider was performed by Zeng (1991). This technique, though time intensive, includes a series of tow tests using a prototype net to obtain a set of full-scale velocity-tension values. These full-scale values would then be Froude-scaled and additional tow tests performed using candidate model nets until the Froude-scaled values matched. Tow tests should also be conducted with different levels of biological fouling to further understand the effects of blockage and shadowing.

3. Motion Response in Waves

The motion response comparisons of the cage, especially heave, using both the physical and numerical models corresponded remarkably well with field data. The numerical model heave results typically under predicted, while the heave response of the physical model cage, over predicted the field data. In general, the clean central spar cage was found to be an over damped system, most likely due to non-linear frictional drag of
the netting. The relationship between the wave forcing and the motion response for each of the physical and numerical modeling methods is linear for most of the wave frequencies as indicated by the coherency-squared calculations and therefore the original assumption of a linear system was appropriate for this analysis.

Using both modeling methods, the normalized surge motion results over predicted the field data because the simulations did not include a steady current that could restrict the cage motion as the cage sets back against the mooring system. To investigate this dynamic response, additional computer model runs could be performed incorporating ADCP data available from the demonstration site. Input to the numerical model could include coincident velocity and wave data sets. Model simulations could then be performed and the results compared with motion response data obtained at the same time as the input forcing. Being able to model waves and currents simultaneously under a wide range of conditions is the primary major advantage of the numerical model.

The numerical technique, however, did not effectively model the swinging motion of the pendant weight beneath the cage. The affect was clear, however, in both the regular and random wave tests performed using the physical model. Furthermore, failure of the pendant line on the deployed system occurring during a storm where the dominant period was estimated to be similar to the a natural period of free swinging pendulum of the pendant line length (33 meters), seems too coincidental to ignore. This possible resonant mode of response due to the pendulum effect is a design consideration that cannot be overlooked for this cage system and others of similar configuration.

4. **Mooring Line Tension Response in Waves**

A static mooring line tension comparison was performed using results calculated
analytically, obtained during the physical and numerical model tests and data collected in-situ. Although some variability existed in the results, the benefits of performing these static tests were valuable. First, numerical modeling code representing the bottom interaction with mooring components was verified. Second, arranging the physical model in the test basin provided insight concerning the complexities of field deployment and static tension sensitivity due to the geometry of the system. Additional work in this area could include investigating the sensitivity relationship between mooring grid static tension and the system dynamic response.

The dynamic load responses of the anchor and bridle line were also compared between the physical and numerical modeling methods and the field data using linear transfer functions. In a moderate wave condition \((2.5 \ m < H_{mo} < 5.0 \ m)\), the linear transfer function values obtained on the deployed structure were typically between 1 and 3 kN/m, and was somewhat consistent across the wave frequencies. The physical model load transfer functions, using the regular wave and auto- and cross-spectral techniques, were slightly conservative. The normalized results from the numerical model tests, calculated using the auto- and cross-spectral methods were nearly identical to the field data, while the regular wave results showed variability.

Coherency plots from physical and numerical model tests indicate that the system relationship is mostly linear and therefore the original assumption is suitable for this analysis. A possible non-linear influence between the wave forcing and mooring line tension may exist so extrapolation of the transfer functions for use with extreme waves should be performed cautiously. To investigate the non-linear relationship between the wave forcing and the mooring line tension response, additional physical and numerical
model tests should be conducted with larger wave amplitudes and compared with these results. Bi-spectral analysis can also be performed to obtain quadratic transfer function and coherency to quantify the non-linear relationship.

5. Future System Design

Consider a nominal upper value for the mooring load transfer function of 5 kN/m and the wave spectra for an estimated 50-year storm. Applying this transfer function, one can obtain a design wave induced tension ($T_{10}$) of 28.4 kN. Combining this value “non-linearly” with a nominal drag load of 20 kN, a new design load of 96 kN was calculated. This value is approximately 24% larger than the original design load of 77 kN. One of the questions that were asked by many during this design process was “Did we over design the submerged grid mooring system?” The answer is “Probably not”.

The result of this calculation shows the uncertainty of the original design process using previous physical and numerical modeling methods, justifying the need for this comprehensive study. The results of the motion and load linear transfer function comparisons calculated as part of this investigation were remarkably close when compared with field data. In general, the physical modeling methods are more conservative than the numerical methods. Each technique, however, has it’s own advantages that compliments the design process. Often the physical modeling approach identifies processes that are not modeled numerically, such as the swinging pendant weight. The numerical model, however, can be used for a wider range of testing scenarios, such as combined wave and current loading or extreme conditions. The fact that each method compared well with the field data provides an increased confidence level that was nonexistent during the initial design phases.
To design future open ocean aquaculture systems, both physical and numerical modeling techniques will need to be utilized in a manner that reflects the strengths of each. As discussed previously, certain aspects of these methods can be further investigated, such as net modeling, system response in combined current and wave loading and the non-linear relationship between forcing and response mechanisms for extrapolation to extreme conditions. Additional field data validation will also be necessary to verify future modeling improvements.

As design concepts change to make open ocean aquaculture more economically viable, mooring systems that minimize bottom footprint areas along with novel anchoring designs will need to be tested. Commercial success will be a function of the economics of scale. Larger systems will need to be deployed. As these facilities become larger, the modeling applications may change as the dominating processes change. Even still, a systematic, scientific approach will be used, but hopefully, with more experience.
REFERENCES


APPENDIX A

EXTREME DESIGN AND STOCHASTIC WAVE CONDITIONS
Introduction

As the coastal and offshore resources of the Gulf of Maine are developed through open ocean aquaculture, coastal community development and other initiatives, the need for engineers to effectively design structures to withstand the environmental forcing of the Gulf of Maine becomes more important. These coastal and offshore structures are typically designed utilizing a combination of numerical and physical modeling techniques as well as analytical approaches. One of the most important components of the design process is developing design criteria, especially the forces and loads that a structure is to withstand.

In the Gulf of Maine, one of the dominant forces is due to surface incident waves. A need exists to understand the wave field in the Gulf of Maine so engineers can more effectively utilize numerical and physical models to design structures, such as open ocean aquaculture facilities, in this energetic location. Design parameters such as the 10-, 25-, 50- and 100-year significant wave height and extreme and typical stochastic sea conditions are used in the design process. Fortunately, wave data sets from oceanographic buoys operated by the National Data Buoy Center (NDBC) are available for site locations near NH (Figure A.1). These data sets can be processed to obtain design wave and random sea condition for design purposes.

The NDBC currently operates three oceanographic buoys near coastal NH. The Boston, MA buoy (number 44013), is at a location of 42.35 North Latitude and 70.69 West Longitude and is in a depth of 55 meters. The Portland, ME buoy (number 44007) is at the location of 43.53 North Latitude and 70.14 West Longitude and is in a depth of 19.2 meters. Data was also considered from the Gulf of Maine buoy (44005), which is at
a location of 42.90 North Latitude and 68.94 West Longitude and is in a depth of 21.9 meters.

![Map of Oceanographic and Meteorological Stations](image)

**Figure A.1:** Oceanographic and Meteorological stations operated by the NDBC in the Northeast (Figure downloaded from the NDBC web site).

Daily wave data available from the NDBC web site for each of these buoys consist of significant wave height ($H_s$), swell height and period, wind wave height and period, wave steepness and average wave period. Historical data is also available at the web site. The archived data sets contain multiple years of hourly meteorological information including $H_s$, dominant wave period and average wave period. The archived data includes hourly spectral information collected during the years of 1996, 1997 and 1998.

The objective is to use the existing NDBC data to estimate extreme design waves (i.e. 50- and 100- significant wave heights) for open ocean locations of New Hampshire. The data is also used to investigate the typical and extreme energy density spectral shapes characteristic of the same geographic locations. Analytical expressions using forms of
the Joint North Sea Wave Project (JONSWAP) spectrum (Hasselman et al., 1973 and Goda, 1985) are then developed as a function of the significant wave height ($H_s$) and dominant wave period ($T_p$), to characterize the extreme and typical random wave conditions. These analytical expressions can then be programmed for use in wave basin operation software and numerical models.

**Extreme Wave Height Analysis**

Extreme wave conditions in the Gulf of Maine are generally created by northeast storms (Pearce and Panchang, 1983). These events are characterized by strong northeast winds that develop because of intense low-pressure regions off the east coast. These extratropical storms can occur any time of the year but are most prevalent during the late fall, winter and spring months. Hurricanes can also influence the wave conditions in the Gulf of Maine. Hurricanes are severe tropical cyclones with sustained winds over 74 mph (64 knots) that form in the southern Atlantic Ocean, Caribbean Sea, and the Gulf of Mexico. Since 1936, New England has been affected by 24 Hurricane events most of which occurred during the months of August and September. Typically, these storms affect southern New England since Cape Cod reduces the intensity of many of these storms, but remnants can make it into the Gulf of Maine.

The design wave height analysis presented here requires that the processes analyzed be of the same population (Pearce and Panchang, 1983). In other words, waves generated by northeast storms should not be mixed with waves generated by other processes such as Hurricanes. As discussed in Muller (1999), the NDBC does not categorize the wave forcing mechanisms but rather presents the meteorological and oceanographic data. In addition, northeast storms can occur during the Hurricane season.
so characterization of the wave forcing process does not boil down to the month of occurrence. Even though Hurricanes can be the cause of severe wave conditions, since they are relatively infrequent compared to northeasters in the Gulf of Maine, it was chosen not to include them in the data analysis.

Review of the available data from the Portland, Boston and Gulf of Maine buoys (Tables A.1 through A.3) indicate that only a few maximum yearly significant wave heights occurred during the typical hurricane season (June through October). One of these storms is the well-known northeast storm called the “Halloween Storm” of 1991 (Cardone et al., 1996). This storm actually combined with the remnants of Hurricane Grace to produce sea states unprecedented in magnitude. Since this storm was characterized as an extratropical storm, it was included in the data set (Cardone et al., 1996). In the same year, the Gulf of Maine buoy recorded a significant height of 6.60 meters (in August) as a result of Hurricane Bob and therefore was excluded. The next highest wave height not associated with Hurricane Bob was found to be in March and was used in the extreme wave height analysis (Table A.3). In October of 1996, both the Portland and Boston buoys recorded extreme waves because of Hurricane Lili. These data points were also replaced by the next highest $H_s$ not associated with the Hurricane (Tables A.1 and A.2). Also given on Tables A.1, A.2 and A.3 is the percent of the data available for the particular year, since some loss of data can occur primarily due to instrumentation problems.
Table A.1: Portland Buoy 44007 Maximum $H_s$

<table>
<thead>
<tr>
<th>Year</th>
<th>Max $H_s$</th>
<th>Month</th>
<th>% Data Available</th>
</tr>
</thead>
<tbody>
<tr>
<td>1982</td>
<td>4.80</td>
<td>April</td>
<td>6.36</td>
</tr>
<tr>
<td>1983</td>
<td>6.80</td>
<td>February</td>
<td>89.43</td>
</tr>
<tr>
<td>1984</td>
<td>6.70</td>
<td>February</td>
<td>67.09</td>
</tr>
<tr>
<td>1985</td>
<td>6.30</td>
<td>February</td>
<td>88.69</td>
</tr>
<tr>
<td>1986</td>
<td>6.00</td>
<td>December</td>
<td>95.33</td>
</tr>
<tr>
<td>1987</td>
<td>6.10</td>
<td>January</td>
<td>91.82</td>
</tr>
<tr>
<td>1988</td>
<td>7.30</td>
<td>February</td>
<td>98.83</td>
</tr>
<tr>
<td>1989</td>
<td>4.10</td>
<td>November</td>
<td>97.25</td>
</tr>
<tr>
<td>1990</td>
<td>5.20</td>
<td>December</td>
<td>99.05</td>
</tr>
<tr>
<td>1991</td>
<td>6.90</td>
<td>October</td>
<td>99.42</td>
</tr>
<tr>
<td>1992</td>
<td>6.80</td>
<td>December</td>
<td>85.06</td>
</tr>
<tr>
<td>1993</td>
<td>7.00</td>
<td>March</td>
<td>93.37</td>
</tr>
<tr>
<td>1994</td>
<td>5.60</td>
<td>January</td>
<td>87.21</td>
</tr>
<tr>
<td>1995</td>
<td>7.30</td>
<td>November</td>
<td>89.01</td>
</tr>
<tr>
<td>1996</td>
<td>7.00</td>
<td>October</td>
<td>99.93</td>
</tr>
<tr>
<td>1996</td>
<td>5.80</td>
<td>April</td>
<td>99.93</td>
</tr>
<tr>
<td>1997</td>
<td>6.14</td>
<td>January</td>
<td>91.28</td>
</tr>
<tr>
<td>1998</td>
<td>5.59</td>
<td>February</td>
<td>97.95</td>
</tr>
</tbody>
</table>

*Hurricane Lili – October 20-22, not included in analysis

Table A.2: Boston Buoy 44013 Maximum $H_s$

<table>
<thead>
<tr>
<th>Year</th>
<th>Max $H_s$</th>
<th>Month</th>
<th>% Data Available</th>
</tr>
</thead>
<tbody>
<tr>
<td>1986</td>
<td>5.00</td>
<td>December</td>
<td>58.0822</td>
</tr>
<tr>
<td>1987</td>
<td>4.70</td>
<td>January</td>
<td>85.7763</td>
</tr>
<tr>
<td>1988</td>
<td>4.70</td>
<td>December</td>
<td>91.5297</td>
</tr>
<tr>
<td>1989</td>
<td>4.20</td>
<td>February</td>
<td>89.0297</td>
</tr>
<tr>
<td>1990</td>
<td>4.00</td>
<td>November</td>
<td>98.7900</td>
</tr>
<tr>
<td>1991</td>
<td>9.10</td>
<td>October</td>
<td>99.4863</td>
</tr>
<tr>
<td>1992</td>
<td>7.30</td>
<td>December</td>
<td>99.1781</td>
</tr>
<tr>
<td>1993</td>
<td>6.10</td>
<td>March</td>
<td>97.8082</td>
</tr>
<tr>
<td>1994</td>
<td>6.70</td>
<td>December</td>
<td>97.5571</td>
</tr>
<tr>
<td>1995</td>
<td>6.40</td>
<td>November</td>
<td>98.7671</td>
</tr>
<tr>
<td>1996</td>
<td>5.83</td>
<td>October</td>
<td>98.4475</td>
</tr>
<tr>
<td>1996</td>
<td>5.82</td>
<td>January</td>
<td>98.4475</td>
</tr>
<tr>
<td>1997</td>
<td>7.57</td>
<td>April</td>
<td>61.2785</td>
</tr>
<tr>
<td>1998</td>
<td>6.16</td>
<td>February</td>
<td>97.9224</td>
</tr>
</tbody>
</table>

*Hurricane Lili – October 20-22, not included in analysis

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Using the data downloaded from the NDBC web site provided in Tables A.1 through A.3, significant wave height statistics were calculated for return periods up to 100 years. To perform the calculations, a function describing the distribution of maximum significant wave heights was implemented so that extrapolation for return periods beyond the limit of the data sets can be conducted. The Weibull distribution functions have been used for design wave studies in the Gulf of Maine as described by Panchang et al. (1990) and Pearce and Panchang (1983). In addition, the U.S. Army Corps of Engineers (USACOE) use the Weibull and Fisher-Tibett Type I distributions as options in wave prediction software available for public use. In this study, one of the Weibull distribution functions was chosen for its previous acceptance.
The conditional form of the Weibull Distribution is given as

\[ F(H_s \leq \bar{H}_s) = 1 - e^{\left(\frac{\bar{H}_s - B}{A}\right)^k} \]  

(A.1)

where \( F(H_s \leq \bar{H}_s) \) is the probability of \( \bar{H} \), not being exceeded, \( H_s \) is the significant wave height, \( B \) is a location parameter, \( A \) is a scale parameter and \( k \) is a shape parameter.

The plotting-position formula can be obtained from equation (A.1) as shown by Goda, (1988),

\[ F(H_s \leq \bar{H}_{sm}) = 1 - \frac{m - 0.20 - 0.27}{N + 0.20 + \frac{0.23}{\sqrt{k}}} \]  

(A.2)

where \( F(H_s \leq \bar{H}_{sm}) \) the probability of \( m^{th} \) significant height not being exceeded, \( H_{sm} \) is the \( m^{th} \) value in the ranked significant wave heights, \( m \) is the ranked significant wave height value = 1, 2, ..., \( N \), and \( N \) is the total number of events during the length of the record. The scale and location parameters, \( A \) and \( B \), are obtained using linear regression analysis described by the following relation,

\[ H_{sm} = Ay_m + B, \ m = 1, 2, ..., N, \]  

(A.3)

where \( y_m \) is the reduced variate defined as,

\[ y_m = \left\{ -\ln(1 - F(H_s \leq H_{sm})) \right\}^{1/k}. \]  

(A.4)

The location and scale parameters are then used to find the return periods (years), \( T_r \) for each of the ordered significant wave heights \( H_{sr} \),

\[ H_{sr} = Ay_r + B \]  

(A.5)

where \( y_r = \left[ \ln(\lambda T_r) \right]^{1/k} \), and \( \lambda \) is the average number of events per year. Using a shape parameter value of \( k = 2 \), the wave height distribution, as a function of the reduced
variante and the return period, is plotted on Figures A.2, A.3, and A.4 for the Portland, Boston and Gulf of Maine buoys, respectively.

Figure A.2: The wave height distribution for Portland buoy 44007.
Figure A.3: The wave height distribution for Boston buoy 44013.

Figure A.4: The wave height distribution for the Gulf of Maine buoy 44005.
Using Figures A.2 through A.4, significant wave heights for various return periods can be obtained. The 10-, 25-, 50- and 100-year significant wave heights for each of the buoys are provided on Table A.4. The Portland and Boston buoy results were then averaged to obtain values for an intermediate location off the coast of Portsmouth NH.

**Table A.4: Significant wave heights for the Gulf of Maine**

<table>
<thead>
<tr>
<th>Return Period</th>
<th>Portland Buoy 44007</th>
<th>Boston Buoy 44013</th>
<th>Gulf of Maine Buoy 44005</th>
<th>Near Portsmouth*</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 year</td>
<td>7.33 m</td>
<td>8.07 m</td>
<td>8.84 m</td>
<td>7.70 m</td>
</tr>
<tr>
<td>25 year</td>
<td>7.86 m</td>
<td>8.99 m</td>
<td>9.58 m</td>
<td>8.43 m</td>
</tr>
<tr>
<td>50 year</td>
<td>8.20 m</td>
<td>9.60 m</td>
<td>10.07 m</td>
<td>8.90 m</td>
</tr>
<tr>
<td>100 yearb</td>
<td>8.53 m</td>
<td>10.15 m</td>
<td>10.52 m</td>
<td>9.34 m</td>
</tr>
<tr>
<td>Data correlation</td>
<td>0.9395</td>
<td>0.9908</td>
<td>0.9903</td>
<td>n.a.</td>
</tr>
</tbody>
</table>

*Open Ocean Portsmouth values obtained by averaging results from the Portland and Boston buoys.

b Extrapolations beyond 3 times the length of the record are considered have a low degree of confidence.

The results using the NDBC information was then compared with wave model simulations performed by Pearce and Panchang (1983) using 22 of the strongest northeast storms occurring in a period between 1944 and 1976. In their model simulations, the Gulf of Maine was covered by an evenly distributed mesh of grid points. At each grid point 50- and 100-year significant wave heights were generated from model results using wind data as input to a deep water, hybrid parametric wave model (Hydraulics Research Station, 1977; Gunther, et al., 1979). The wind field was computed from barometric pressure data. The 50- and 100-year wave height values closest to Portsmouth, New Hampshire was calculated to be 8.88 and 9.70 meters, respectively. The 50-year significant wave height results from the Pearce and Panchang (1983) study are shown on Figure A.5.
Random Sea Analysis

Overview

Two types of random wave conditions are desirable in the design and evaluation of offshore structures. The extreme random wave condition should characterize the sea state during storm conditions. The typical random wave condition should be representative of normal daily sea states. Hourly energy density data downloaded from the NDBC web site for each of the Portland, Boston and Gulf of Maine buoys should provide representative wave information for conditions off the coast of New Hampshire to obtain extreme and typical spectral shapes.

Highest Wave Conditions

To determine the spectral shape for the highest wave condition, the highest hourly
spectrum for each buoy during the years of 1996, 1997 and 1998 was downloaded from the NDBC website. Next, ensemble averages of the three highest yearly spectra for the Portland, Boston and the Gulf of Maine buoys were calculated (Figure A.6).

![Highest spectral ensemble averages for 1996, 1997 and 1998.](image)

An energy based significant wave height (defined as the $H_s$ by the NDBC) is a statistical parameter that can be obtained from the energy density information for the typical ensemble averages. The significant wave height, $H_s$ is obtained by first integrating the area under the energy density curves to find the variance (equation 4.43). The root mean square wave amplitude ($\eta_{rms}$) is then calculated by taking the square root of the variance. Assuming a linear waveform, the wave height root mean square ($H_{rms}$) is estimated to be $2(2)^{1/2} \eta_{rms}$. If the wave heights in the random sea condition follow a Rayleigh distribution, then the significant wave height can be approximated at $1.416 H_{rms}$ (SPM, 1984). The peak period ($T_p$) is found by taking the inverse of the frequency where the highest energy density value occurs. The yearly and three year average results are
Table A.5: Highest wave conditions obtained from the NDBC buoys

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland</td>
<td>$H_s$ (m)</td>
<td>7.02</td>
<td>7.04</td>
<td>5.61</td>
<td>6.58</td>
</tr>
<tr>
<td></td>
<td>$T_p$ (sec)</td>
<td>11</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Boston</td>
<td>$H_s$ (m)</td>
<td>5.83</td>
<td>7.58</td>
<td>6.17</td>
<td>6.57</td>
</tr>
<tr>
<td></td>
<td>$T_p$ (sec)</td>
<td>10</td>
<td>11</td>
<td>11</td>
<td>11</td>
</tr>
<tr>
<td>Gulf of Maine</td>
<td>$H_s$ (m)</td>
<td>6.85</td>
<td>5.91</td>
<td>6.15</td>
<td>6.31</td>
</tr>
<tr>
<td></td>
<td>$T_p$ (sec)</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
</tr>
</tbody>
</table>

The average data from the Portland and Boston buoys provided in Table A.5, were used to obtain the significant wave height ($H_s$) and peak period ($T_p$) for an intermediate location representative in deep water near Portsmouth. Averaging these values, the three year highest wave condition was calculated to have an $H_s$ and $T_p$ equal to 6.58 meters and 10.5 seconds respectively.

**Typical Random Seas**

On any given day in the Gulf of Maine, the sea state is comprised of multiple swell and/or wind wave components. In the frequency domain, the swell wave components will form the low frequency end of the energy density spectra while the wind waves parts will be found at the higher frequencies. Since the typical condition may include multiple frequency components, the representative spectrum should include the energy found through the entire range of frequencies. Therefore, ensemble averages at each frequency were performed for the three-year hourly data sets available for the Portland, Boston and Gulf of Maine buoys.

The significant wave height and the peak period for the Portland, Boston and Gulf of Maine ensemble average data sets for the years of 1996, 1997 and 1998 are provided.
on Table A.6. Also given on Table A.6, is the three year average for each of the respective buoys. The three-year average spectral density results are also shown on Figure A.7. Based on Portland and Boston data sets, the average significant wave height (Hs) and peak period (Tp) for the typical condition offshore Portsmouth, NH was found to be 1.21 meters and 10 seconds respectively.

Table A.6: Typical wave conditions obtained from the NDBC buoys

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland</td>
<td>Hs (m)</td>
<td>1.25</td>
<td>1.13</td>
<td>1.23</td>
<td>1.21</td>
</tr>
<tr>
<td></td>
<td>Tp (sec)</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Boston</td>
<td>Hs (m)</td>
<td>1.20</td>
<td>1.24</td>
<td>1.18</td>
<td>1.21</td>
</tr>
<tr>
<td></td>
<td>Tp (sec)</td>
<td>9</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Gulf of Maine</td>
<td>Hs (m)</td>
<td>1.84</td>
<td>1.63</td>
<td>1.56</td>
<td>1.68</td>
</tr>
<tr>
<td></td>
<td>Tp (sec)</td>
<td>9</td>
<td>9</td>
<td>8.33</td>
<td>9</td>
</tr>
</tbody>
</table>

Figure A.7: Yearly spectral ensemble averages for 1996, 1997 and 1998.
**JONSWAP Representation of the Spectra**

To develop analytical expressions of random sea conditions for this area of the Gulf of Maine, a form of the Joint North Sea Wave Project (JONSWAP) spectrum (Hasselman et al., 1973 and Goda, 1985) was used. The expressions are used as input for the computer software that controls the wave maker in the physical model testing facility and for numerical modeling studies. The input spectra used to represent the random sea condition is a modified version of the JONSWAP spectra described by Goda (1985),

\[ S(f) = \alpha H_s^2 T_p^{-4} f^{-5} \exp\left[-1.25(T_p f)^{-1}\right] Y^\gamma, \]  

(A.6)

where

\[ Y = e^{-\left(\frac{f f_p - i}{2\sigma^2}\right)} \]

\[ \alpha = \frac{0.0624}{0.23 + 0.0336\gamma - 0.185(1.9 + \gamma)^{-1}}, \]

and

\[ \sigma = \begin{cases} \sigma_a &: f \leq f_p \\ \sigma_b &: f \geq f_p \end{cases} \]

and \( f_p \) is the frequency at the spectral peak \( (1/T_p) \). Parameters \( \gamma \) and \( \sigma \) are used to adjust the height and width of the peak of the curve, respectively.

**Highest Wave Spectra**

Using the expressions for the JONSWAP spectrum shown in equation (A.6), values for \( \gamma, \sigma_a, \sigma_b \) and the frequency power were found iteratively until the variance of the respective ensemble average and the JONSWAP representation were within 1\%. The results for the Portland, Boston and Gulf of Maine buoys are provided on Table A.7. The resulting spectral shape is shown on Figure A.8.
Table A.7: JONSWAP parameters for the highest wave spectra.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Portland</th>
<th>Boston</th>
<th>Gulf of Maine</th>
<th>Portsmouth</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_s$ (meters)</td>
<td>6.590</td>
<td>6.570</td>
<td>6.315</td>
<td>6.580</td>
</tr>
<tr>
<td>$T_p$ (seconds)</td>
<td>10</td>
<td>11</td>
<td>11</td>
<td>10.5</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>5.40</td>
<td>2.50</td>
<td>3.90</td>
<td>3.95</td>
</tr>
<tr>
<td>Power of $f$</td>
<td>-4.8</td>
<td>-4.9</td>
<td>-4.85</td>
<td>-4.85</td>
</tr>
<tr>
<td>$\sigma_a$</td>
<td>0.450</td>
<td>0.450</td>
<td>0.130</td>
<td>0.450</td>
</tr>
<tr>
<td>$\sigma_b$</td>
<td>0.155</td>
<td>0.145</td>
<td>0.240</td>
<td>0.150</td>
</tr>
<tr>
<td>Variance</td>
<td>2.703</td>
<td>2.698</td>
<td>2.483</td>
<td>2.749</td>
</tr>
</tbody>
</table>

Figure A.8: Estimated spectral shape for the highest waves at Portsmouth, NH using the JONSWAP spectrum.

**Typical Wave Spectra**

Investigation of the ensemble average spectral data revealed that the shape of the NH spectra could be represented by a superposition of two JONSWAP forms, one of which representing the lower frequencies of the spectrum and the second representing the higher frequencies. Two JONSWAP forms (equation A.6) were superimposed and the parameters chosen iteratively until a fit was made such the variance between the data and
fitted curves were within 1%. The resulting equation was then arranged into the following convenient form,

\[ S(f) = (H_s)^2 (f^{-p}) \left\{ \alpha_1 \gamma_1 \left( \frac{T_{p1}}{T_p} \right)^4 e^{-\frac{1}{2} \left( \frac{f}{f_p} \right)^4} + \alpha_2 \gamma_2 \left( \frac{T_{p2}}{T_p} \right)^4 e^{-\frac{1}{2} \left( \frac{f}{f_p} \right)^4} \right\}, \]  \hspace{1cm} (A.7)

where:
- \( H_s = 1.21 \) meters,
- \( f^p = \) the frequency to the negative power of 4.35,
- \( \gamma_1 = 6.75, \)
- \( T_{p1} = 10 \) seconds,
- \( \gamma_2 = 0.500 \)
- \( T_{p2} = 5.34 \) seconds.

The resulting spectral shape is shown on Figure A.9.

![Figure A.9: Estimated spectral shape for typical conditions Portsmouth, NH using a superposition of two JONSWAP spectra.](image)

**Development of Stochastic Wave Input**

The extreme wave height and spectral energy density results obtained can be used as input into both the AquaFE numerical model and the wave maker control software to
simulate conditions in the open ocean near Portsmouth, NH. Both numerical and physical modeling techniques use a superposition method with random phases of the form,

\[ \eta = \sum_{j} A_j \sin(\omega_j t + \varepsilon_j), \]  

(A.8)

where

\[ \frac{1}{2} A_j^2 = S(f) \Delta f, \]  

(A.9)

\( \omega_j = 2\pi f, \) \( \varepsilon_j \) is the random phase and \( A_j \) is the amplitude of the individual wave components (Chakrabarti, 1994), to generate random waves.

To represent the 50- and 100-year storms stochastically, the extrapolated wave heights could be used as input into the spectral shape described by equation (A.6). The 50- and 100- year storm condition could then be used as input to the appropriate model simulation.
APPENDIX B

PHYSICAL MODEL MOTION RESPONSE IN REGULAR WAVES
Figure B.1: Physical model motion response time series for regular wave set #1.

Figure B.2: Physical model motion response time series for regular wave set #2.
Figure B.3: Physical model motion response time series for regular wave set #3.

Figure B.4: Physical model motion response time series for regular wave set #4.
Figure B.5: Physical model motion response time series for regular wave set #5.

Figure B.6: Physical model motion response time series for regular wave set #6.
Figure B.7: Physical model motion response time series for regular wave set #7.

Figure B.8: Physical model motion response time series for regular wave set #8.
Figure B.9: Physical model motion response time series for regular wave set #9.

Figure B.10: Physical model motion response time series for regular wave Set #10.
APPENDIX C

PHYSICAL MODEL TENSION RESPONSE IN REGULAR WAVES
Figure C.1: Physical model tension response time series for regular wave set #1.

Figure C.2: Physical model tension response time series for regular wave set #2.
Figure C.3: Physical model tension response time series for regular wave set #3.

Figure C.4: Physical model tension response time series for regular wave set #4.
Figure C.5: Physical model tension response time series for regular wave set #5.

Figure C.6: Physical model tension response time series for regular wave set #6.
Figure C.7: Physical model tension response time series for regular wave set #7.

Figure C.8: Physical model tension response time series for regular wave set #8.
Figure C.9: Physical model tension response time series for regular wave set #9.

Figure C.10: Physical model tension response time series for regular wave set #10
Figure D.1: Physical model motion time series results for random wave set #1.

Figure D.2: Physical model motion auto-spectral results for random wave set #1.
Figure D.3: Physical model motion time series results for random wave set #2.

Figure D.4: Physical model motion auto-spectral results for random wave set #2.
Figure D.5: Physical model motion time series results for random wave set #3.

Figure D.6: Physical model motion auto-spectral results for random wave set #3.
Figure D.7: Physical model motion time series results for random wave set #4.

Figure D.8: Physical model motion auto-spectral results for random wave set #4.
Figure D.9: Physical model motion time series results for random wave set #5.

Figure D.10: Physical model motion auto-spectral results for random wave set #5.
Figure D.11: Physical model motion time series results for random wave set #6.

Figure D.12: Physical model motion auto-spectral results for random wave set #6.
Figure D.13: Physical model motion time series results for random wave set #7.

Figure D.14: Physical model motion auto-spectral results for random wave set #7.
Figure D.15: Physical model motion time series results for random wave set #8.

Figure D.16: Physical model motion auto-spectral results for random wave set #8.
Figure D.17: Physical model motion time series results for random wave set #9.

Figure D.18: Physical model motion auto-spectral results for random wave set #9.
Figure D.19: Physical model motion time series results for random wave set #10.

Figure D.20: Physical model motion auto-spectral results for random wave set #10.
Figure E.1: Physical model tension time series results for random wave set #1.

Figure E.2: Physical model tension auto-spectral results for random wave set #1.
Figure E.3: Physical model tension time series results for random wave set #2

Figure E.4: Physical model tension auto-spectral results for random wave set #2.
Figure E.5: Physical model tension time series results for random wave set #3.

Figure E.6: Physical model tension auto-spectral results for random wave set #3.
Figure E.7: Physical model tension time series results for random wave set #4.

Figure E.8: Physical model tension auto-spectral results for random wave set #4.
Figure E.9: Physical model tension time series results for random wave set #5.

Figure E.10: Physical model tension auto-spectral results for random wave set #5.
Figure E.11: Physical model time series tension results for random wave set #6.

Figure E.12: Physical model tension auto-spectral results for random wave set #6.
Figure E.13: Physical model time series tension results for random wave set #7.

Figure E.14: Physical model tension auto-spectral results for random wave set #7.
Figure E.15: Physical model time series tension results for random wave set #8.

Figure E.16: Physical model tension auto-spectral results for random wave set #8.
Figure E.17: Physical model time series tension results for random wave set #9.

Figure E.18: Physical model tension auto-spectral results for random wave set #9.
Figure E.19: Physical model time series tension results for random wave set #10.

Figure E.20: Physical model tension auto-spectral results for random wave set #10.
Figure E.21: Physical model time series tension results for random wave set #11.

Figure E.22: Physical model tension auto-spectral results for random wave set #11.
Figure E.23: Physical model time series tension results for random wave set #12.

Figure E.24: Physical model tension auto-spectral results for random wave set #12.
Figure E.25: Physical model time series tension results for random wave set #13.

Figure E.26: Physical model tension auto-spectral results for random wave set #13.
Figure E.27: Physical model time series tension results for random wave set #14.

Figure E.28: Physical model tension auto-spectral results for random wave set #14.
Figure E.29: Physical model time series tension results for random wave set #15.

Figure E.30: Physical model tension auto-spectral results for random wave set #15.
Figure E.31: Physical model time series tension results for random wave set #16.

Figure E.32: Physical model tension auto-spectral results for random wave set #16.
Figure E.33: Physical model time series tension results for random wave set #17.

Figure E.34: Physical model tension auto-spectral results for random wave set #17.
Figure E.35: Physical model time series tension results for random wave set #18.

Figure E.36: Physical model tension auto-spectral results for random wave set #18.
APPENDIX F

NUMERICAL MODEL MOTION RESPONSE IN REGULAR WAVES
Figure F.1: AquaFE motion response to regular wave set #1.

Figure F.2: AquaFE motion response to regular wave set #2.
Figure F.3: AquaFE motion response to regular wave set #3.

Figure F.4: AquaFE motion response to regular #4.
Figure F.5: AquaFE motion response to regular wave set #5.

Figure F.6: AquaFE motion response to regular wave set #6.
Figure F.7: AquaFE motion response to regular wave set #7.

Figure F.8: AquaFE motion response to regular wave set #8.
Figure F.9: AquaFE motion response to regular wave set #9.

Figure F.10: AquaFE motion response to regular wave set #10.
Figure F.11: AquaFE motion response to regular wave set #11.

Figure F.12: AquaFE motion response to regular wave set #12.
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Figure G.2: AquaFE anchor and bridle tension response to regular wave set #2.
Figure G.3: AquaFE anchor and bridle tension response to regular wave set #3.

Figure G.4: AquaFE anchor and bridle tension response to regular wave set #4.
Figure G.5: AquaFE anchor and bridle tension response to regular wave set #5.

Figure G.6: AquaFE anchor and bridle tension response to regular wave set #6.
Figure G.7: AquaFE anchor and bridle tension response to regular wave set #7.

Figure G.8: AquaFE anchor and bridle tension response to regular wave set #8.
Figure G.9: AquaFE anchor and bridle tension response to regular wave set #9.

Figure G.10: AquaFE anchor and bridle tension response to regular wave set #10.

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Figure G.11: AquaFE anchor and bridle tension response to regular wave set #11

Figure G.12: AquaFE anchor and bridle tension response to regular wave set #12.
Figure G.13: AquaFE anchor and bridle tension response to regular wave set #13.
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NUMERICAL MODEL MOTION RESPONSE IN RANDOM WAVES
Figure H.1: AquaFE motion time series results for random wave set #1.

Figure H.2: AquaFE motion response auto-spectral results for random wave set #1.
Figure H.3: AquaFE motion time series results for random wave set #2.

Figure H.4: AquaFE motion response auto-spectral results for random wave set #2.
Figure H.5: AquaFE motion time series results for random wave set #3.

Figure H.6: AquaFE motion response auto-spectral results for random wave set #3.
Figure H.7: AquaFE motion time series results for random wave set #4.

Figure H.8: AquaFE model response auto-spectral results for random wave set #4.
Figure H.9: AquaFE motion time series results for random wave set #5.

Figure H.10: AquaFE motion response auto-spectral results for random wave set #5.
**Figure H.11:** AquaFE motion time series results for random wave set #6.

**Figure H.12:** AquaFE motion response auto-spectral results for random wave set #6.
Figure H.13: AquaFE motion time series results for random wave set #7.

Figure H.14: AquaFE motion response auto-spectral results for random wave set #7.
Figure H.15: AquaFE motion time series results for random wave set #8.

Figure H.16: AquaFE motion response auto-spectral results for random wave set #8.

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Figure H.17: AquaFE motion time series results for random wave set #9.

Figure H.18: AquaFE motion response auto-spectral results for random wave set #9.
Figure H.19: AquaFE motion time series results for random wave set #10.

Figure H.20: AquaFE motion response auto-spectral results for random wave set 10.
APPENDIX I

NUMERICAL MODEL TENSION RESPONSE IN RANDOM WAVES
Figure I.1: AquaFE tension time series results for random wave set #1.

Figure I.2: AquaFE tension auto-spectral results for random wave set #1.
Figure I.3: AquaFE tension time series results for random wave set #2.

Figure I.4: AquaFE tension auto-spectral results for random wave set #2.
Figure I.5: AquaFE tension time series results for random wave set #3.

Figure I.6: AquaFE tension auto-spectral results for random wave set #3.
Figure 1.7: AquaFE tension time series results for random wave set #4.

Figure 1.8: AquaFE tension auto-spectral results for random wave set #4.
Time Series Results for Wave Set #5

Figure I.9: AquaFE tension time series results for random wave set #5.

Auto Spectral Density Results for Wave Set #5

Figure I.10: AquaFE tension auto-spectral results for random wave set #5.
Figure I.11: AquaFE tension time series results for random wave set #6.

Figure I.12: AquaFE tension auto-spectral results for random wave set #6.
Figure I.13: AquaFE tension time series results for random wave set #7.

Figure I.14: AquaFE tension auto-spectral results for random wave set #7.
Figure I.15: AquaFE tension time series results for random wave set #8.

Figure I.16: AquaFE tension auto-spectral results for random wave set #8.
Figure I.17: AquaFE tension time series results for random wave set #9.

Figure I.18: AquaFE tension auto-spectral results for random wave set #9.
Figure I.19: AquaFE tension time series results for random wave set #10.

Figure I.20: AquaFE tension auto-spectral results for random wave set #10.
APPENDIX J

WAVE SPECTRA COMPARISON BETWEEN THE OOA AND THE PORTLAND AND BOSTON BUOYS
Figure J.1: Buoy wave spectra comparison for Feb. 5, 2001 at 2100 GMT.

Figure J.2: Buoy wave spectra comparison for Feb. 6, 2001 at 0000 GMT.
Figure J.3: Buoy wave spectra comparison for Feb. 6, 2001 at 0300 GMT.

Figure J.4: Buoy wave spectra comparison for Feb. 6, 2001 at 0600 GMT.
Figure J.5: Buoy wave spectra comparison for Feb. 6, 2001 at 0900 GMT.

Figure J.6: Buoy wave spectra comparison for Mar. 5, 2001 at 2100 GMT.
Figure J.7: Buoy wave spectra comparison for Mar. 6, 2001 at 0000 GMT.

Figure J.8: Buoy wave spectra comparison for Mar. 6, 2001 at 0300 GMT.
Figure J.9: Buoy wave spectra comparison for Mar. 6, 2001 at 0600 GMT.

Figure J.10: Buoy wave spectra comparison for Mar. 6, 2001 at 0900 GMT.
Figure J.11: Buoy wave spectra comparison for Mar. 6, 2001 at 1200 GMT.

Figure J.12: Buoy wave spectra comparison for Mar. 6, 2001 at 1500 GMT.
Figure J.13: Buoy wave spectra comparison for Mar. 6, 2001 at 1800 GMT.

Figure J.14: Buoy wave spectra comparison for Mar. 6, 2001 at 2100 GMT.
Figure J.15: Buoy wave spectra comparison for Mar. 7, 2001 at 0000 GMT.
APPENDIX K

IN-SITU DATA RESULTS – MOTION RESPONSE
Figure K.1: Heave, surge and pitch response on Feb. 5, 2001 at 2100 GMT.

Figure K.2: Heave, surge and pitch transfer functions for Feb. 5, 2001 at 2100 GMT.
Figure K.3: Heave, surge and pitch response on Feb. 6, 2001 at 0000 GMT.

Figure K.4: Heave, surge and pitch transfer functions for Feb. 6, 2001 at 0000 GMT.
Figure K.5: Heave, surge and pitch response on Feb. 6, 2001 at 0300 GMT.

Figure K.6: Heave, surge and pitch transfer functions for Feb. 6, 2001 at 0300 GMT.
Figure K.7: Heave, surge and pitch response on Feb. 6, 2001 at 0600 GMT.

Figure K.8: Heave, surge and pitch transfer functions for Feb. 6, 2001 at 0600 GMT.
Figure K.9: Heave, surge and pitch response on Feb. 6, 2001 at 0900 GMT.

Figure K.10: Heave, surge and pitch transfer functions for Feb. 6, 2001 at 0900 GMT.
March 5 2100 GMT

Wave Elevation
Fish Cage Heave

Wave Excursion
Fish Cage Surge

Wave Slope
Fish Cage Pitch

Figure K.11: Heave, surge and pitch response on Mar. 5, 2001 at 2100 GMT.

March 5 2100 GMT

Heave Transfer Function

Surge Transfer Function

Pitch Transfer Function

Figure K.12: Heave, surge and pitch transfer functions for Mar. 5, 2001 at 2100 GMT.

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Figure K.13: Heave, surge and pitch response on March 6, 2001 at 0000 GMT.

Figure K.14: Heave, surge and pitch transfer functions for Mar. 6, 2001 at 0000 GMT.
Figure K.15: Heave, surge and pitch response on Mar. 6, 2001 at 0300 GMT.

Figure K.16: Heave, surge and pitch transfer functions for Mar. 6, 2001 at 0300 GMT.
Figure K.17: Heave, surge and pitch response on Mar. 6, 2001 at 0600 GMT.

Figure K.18: Heave, surge and pitch transfer functions for Mar. 6, 2001 at 0600 GMT.
Figure K.19: Heave, surge and pitch response on Mar. 6, 2001 at 0900 GMT.

Figure K.20: Heave, surge and pitch transfer functions for Mar. 6, 2001 at 0900 GMT.
Figure K.21: Heave, surge and pitch response on Mar. 6, 2001 at 1200 GMT.

Figure K.22: Heave, surge and pitch transfer functions for Mar. 6, 2001 at 1200 GMT.
Figure K.23: Heave, surge and pitch response on Mar. 6, 2001 at 1500 GMT.

Figure K.24: Heave, surge and pitch transfer functions for Mar. 6, 2001 at 1500 GMT.
Figure K.25: Heave, surge and pitch response on Mar. 6, 2001 at 1800 GMT.

Figure K.26: Heave, surge and pitch transfer functions for Mar. 6, 2001 at 1800 GMT.
Figure K.27: Heave, surge and pitch response on Mar. 6, 2001 at 2100 GMT.

Figure K.28: Heave, surge and pitch transfer functions for Mar. 6, 2001 at 2100 GMT.
APPENDIX L

IN-SITU DATA RESULTS – LOAD RESPONSE
Figure L.1: Anchor and Bridle spectral response for Feb. 6, 2001 at 0000 GMT.

Figure L.2: Anchor and Bridle transfer functions for Feb. 6, 2001 at 0000 GMT.
Figure L.3: Anchor and Bridle spectral response for Feb. 6, 2001 at 0300 GMT.

Figure L.4: Anchor and Bridle transfer functions for Feb. 6, 2001 at 0300 GMT.
Figure L.5: Anchor and Bridle spectral response for Feb. 6, 2001 at 0600 GMT.

Figure L.6: Anchor and Bridle transfer functions for Feb. 6, 2001 at 0600 GMT.
February 6, 0900 GMT

Surface Elevation: Portland Buoy
Surface Elevation: Boston Buoy
Surface Elevation: OOA

Figure L.7: Anchor and Bridle spectral response for Feb. 6, 2001 at 0900 GMT.

Figure L.8: Anchor and Bridle transfer functions for Feb. 6, 2001 at 0900 GMT.