THREE-DIMENSIONAL EQUIVALENT STATIC ANALYSIS AND DESIGN METHODOLOGY OF A REINFORCED CONCRETE FLOATING OFFSHORE WIND TURBINE PLATFORM

A Thesis

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Amy Carol Doman

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Amy Carol Doman

Approved by:

_________________________________, Committee Chair
Dr. Benjamin Fell, P.E.

_________________________________, Second Reader
Dr. Matthew Salveson, P.E.

____
Date

Student: Amy Carol Doman

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___________________________, Graduate Coordinator ____________________
Dr. Matthew Salveson, P.E. Date

Department of Civil Engineering

Abstract

of

THREE-DIMENSIONAL EQUIVALENT STATIC ANALYSIS AND DESIGN METHODOLOGY OF A REINFORCED CONCRETE FLOATING OFFSHORE WIND TURBINE PLATFORM

by

Amy Carol Doman

This thesis presents a three-dimensional static analysis and design procedure for a floating offshore wind turbine support platform constructed of reinforced concrete. Results from a global performance analysis, which considers the dynamic interactions of wind and wave loading, from a conceptual case study by the American Bureau of Shipping are used to infer the equivalent static loading applied to the finite element model. Results from the analysis and applicable load combinations obtained from relevant design guidelines are used to design a floating concrete platform structure that includes precast and cast-in-place components as well as post-tensioning. The proposed concrete platform is a tri-floater, semi-submersible structure that consists of a cylindrical central core column to support the wind turbine tower, stabilized by the three buoyancy columns and a rigid pontoon base. The floating reinforced concrete support platform is shown to be an economically viable option when compared to steel platforms. Floating offshore wind turbine technology is a relatively new multi-disciplinary field of engineering where current design concepts primarily utilize steel floating support platforms. Reinforced concrete is advantageous over steel because of the durability of concrete exposed to a marine environment. Construction recommendations are discussed, as are social and economic considerations for floating offshore wind turbines.

________________________, Committee Chair
Dr. Benjamin Fell, P.E.

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Date

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TABLE OF CONTENTS

LIST OF TABLES

Tables Page

LIST OF FIGURES

Figures Page

Chapter 1

INTRODUCTION

1.1 Motivation

The structural analysis and design of a reinforced concrete floating offshore wind turbine (FOWT) platform will be the subject of this thesis. The analysis and design of FOWT platforms is a complicated process that is not well-established. There are significant challenges in determining the loading on the systems as well as generating analysis models that properly reflect the dynamic behavior of wind turbines coupled with floating support structures. The analysis process will be limited for this project by focusing on the linear elastic analysis of a FOWT platform modeled with finite element software, utilizing the results from the analysis to design the concrete structure. The objective of this thesis is to provide an economical and durable design with precast concrete construction, demonstrating that concrete precasting is ideal for mass-producing floating platforms for an offshore wind farm.

The general procedure for the analysis and design of FOWT structures involves four main steps: (1) the establishment of design load cases with environmental conditions including wind and wave parameters specific to each site location; (2) the global performance analysis of a FOWT model that evaluates the dynamic response of the structure considering the coupled behavior of the wind turbine, floating support structure, and stationkeeping system; (3) the finite element analysis of the FOWT structure with resultant loading from the global performance analysis procedure; and (4) the structural design of the floating platform and wind turbine tower based on design guidelines issued by established authorities and results from the analysis processes. This thesis skips to the third step of the analysis and design process by utilizing environmental load cases and global analysis results from case studies performed by the American Bureau of Shipping (ABS). Global loads from an integrated hydrodynamic and aerodynamic analysis will be converted into static loading and applied to a 3D finite element model for linear elastic analysis. The design of the concrete floating platform is based on force results from analysis of the FOWT model. The analysis and design of the wind turbine tower are neglected as the dynamic behavior of such towers is beyond the scope of the present work.

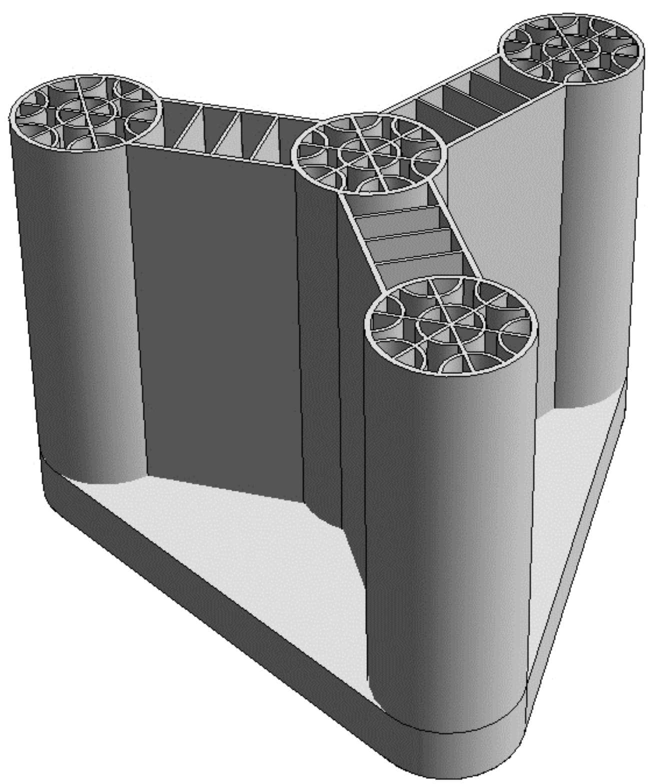


Figure 1.1. Concrete FOWT platform with deck slabs removed from view.

A tri-floater, semisubmersible concrete FOWT support structure was selected for this project. The basic construction concept involves stacking precast concrete rings with hollow honeycomb structures and then post-tensioning each column to resist cracking due to cyclic loading induced by waves. The platform consists of a centralized, cylindrical core column that supports the wind turbine tower, three surrounding buoyancy columns, and a rigid pontoon base. Walls connecting the columns provide bracing and buoyancy for the structure. The platform is arranged to minimize the tensile loading on the components induced by environmental loads such as wind and waves. Platform stability will be provided by a large water plane area and the significant mass of the entire structure. Overturning moments from the wind turbine thrust will be resisted by a water ballasting system in the columns.

FOWT technology is a relatively new field that has only received significant research attention within the last fifteen years. The first FOWT prototype was launched off the coast of Italy in 2007 followed by a full-scale pilot project off the coast of Norway in 2009, and then a FOWT platform installed in the Atlantic Ocean off the coast of Portugal in 2011. In May of last year, the first one-eighth scale FOWT in the United States was deployed by the University of Maine off the coast of the state after six years of research sponsored by public and private funding. Since 2011, the U.S. Department of Energy has allocated $227 million for offshore wind research, development, and demonstration projects (Office of Energy Efficiency & Renewable Energy, 2014). Seattle based Principle Power received a grant in 2012 to build five FOWT platforms off the coast of Oregon as a pilot project. The platform will be their patented WindFloat design, a semisubmersible platform that can accommodate up to 10 MW wind turbines and the same design that has been running successfully off the coast of Portugal since 2011. Resource potential data suggest that more than 4,000,000 MW of wind power capacity could be accessed in waters along the coast of the United States (Office of Energy Efficiency & Renewable Energy, 2014).

Figure 1.2. Artistic rendering of WindFloat offshore wind farm. Reprinted from *Principle Power - Products - WindFloat*, n.d., Retrieved 23 March 2014, from www.principlepowerinc.com/ products/windfloat.html. Copyright 2009-2011 Principle Power, Inc. Reprinted with permission.

Most FOWT platform designs are made of steel like large ships. Steel has many advantages including a high strength for a relatively slender section and the ability to resist tensile forces. The thin steel sections that are high in strength allow for lighter structures that still provide the large water displacement that enables floatation. Steel is prone to corrosion in marine applications and therefore requires significant maintenance to keep structural deterioration under control. Steel structures must be removed from the water in a dry dock for maintenance procedures such as the reapplication of special coatings and cathodic protection treatments. This maintenance is not only costly, but there is also a financial loss the entire time the platform is out of service being towed into shore to be worked on in a dry dock facility due to non-production. Properly constructed marine structures made of concrete will never corrode and require minimal maintenance over their service life. Most required maintenance can be performed while in service, without the need for towing to a dry dock facility. Therefore, in theory, a concrete

FOWT platform will experience minimal downtime over its service life, continuously providing power whenever the wind is flowing.

1.2 Objectives and Scope

The objective of this thesis is to determine whether it is feasible for a FOWT platform structure to be constructed of reinforced concrete. This study examines whether a concrete FOWT platform is capable of resisting the extreme forces of the ocean coupled with the incredible thrust forces generated by a massive wind turbine in operation. This project also investigates whether the construction of a concrete FOWT may be economically viable. The structural analysis of a concrete FOWT concept was performed with SAP2000. This work presents a preliminary reinforced concrete platform design for a floating offshore wind turbine. Recommendations for future action are made.

One of the most complicated steps in establishing a viable analysis model is determining the loading on the system. The loading on floating turbine support structures is complicated due to the coupled dynamic loading from wind and waves. A major challenge of the project was to determine the global loading forces that may be applied to a finite element analysis model of the support structure. Wave and wind pressure loads were based on West Coast data established by ABS. Wind turbine thrust forces were broken into tower base shear and overturning moment applied as static loads to the analysis model. These loads were based on case study results from the integrated dynamic analysis of a global system. The finite element analysis provided forces for the structural design of the platform.

A general goal of the concrete FOWT analysis was for the platform model to experience no concrete cracking due to all applied combinations of loads. The most direct way to avoid rebar corrosion in reinforced concrete exposed to a marine environment is to prevent concrete cracking altogether. As the performance of the structure was expected to remain elastic with linear stress-strain relations, a nonlinear analysis was unnecessary. The plastic behavior that may be assumed with a conventional concrete structure was avoided, resulting in a conservative design.

This project addresses the following topics:

- Economic parameters of FOWTS.
- Principle FOWT design concepts and their mechanisms to achieve stability.
- Research and development of FOWTS and relevant code documents.
- Positive properties of concrete in marine applications.
- Modeling techniques for FOWT support structures.
- Establishment of load cases appropriate for evaluating FOWTS.
- Determination of global loading conditions for a FOWT analysis model.
- Generation of a FOWT platform model with finite element analysis software.
- Performance of finite element analysis on a FOWT platform model and the interpretation of the relevant results.
- Design of a robust concrete FOWT support structure that will survive all evaluated loading conditions.
- Recommendations for concrete floating platform materials and construction procedures.

1.3 Organization and Outline

The report consists of three chapters that will discuss the background of the study, the structural analysis process, and the findings and interpretations of the study.

The background chapter presents a discussion of the economics of FOWTS including their costs and design requirements that conflict with the parameters of economy. Descriptions of the main FOWT platform concepts are provided and hybrid concepts are introduced. The research and development of FOWTs is reviewed with an overview of publications on the subject and a discussion of prototype projects in progress around the world. The recent development of design guidelines for FOWTs is examined with a brief discussion of the authorities developing the code. FOWT modeling techniques, the structural analysis process, and the response characteristics of FOWTS are discussed. Lastly, a concrete FOWT concept is presented along with a discussion of the benefits of concrete versus steel in a marine environment.

The analysis chapter introduces the method of analysis used for this project. The determination of load cases considered for the concrete FOWT structural analysis is then discussed. Next is an explanation of how the considered load cases were applied to the analysis model, followed by a description of analysis model characteristics. An interpretation of the analysis results is given with a discussion of the limitations of modeling and the analysis method.

The findings and interpretations chapter presents the concrete FOWT platform design and proposed construction methods. The steel reinforcement layout will be provided as well as recommendations for post-tensioning the structure. A connection design for the wind turbine tower and floating platform will be presented. A proposed concrete mix design that will achieve high-strength light-weight concrete shall also be included. Finally, feasibility conclusions and recommendations for future work are presented.

Chapter 2

BACKGROUND

2.1 Introduction

The design of floating offshore wind turbine (FOWT) platforms has been developed based on sixty years of research, design and construction experience of designing gas and oil offshore structures (American Bureau of Shipping (ABS), 2012). A variety of FOWT concepts have been proposed over the years, but it was not until recently that advances in technology and government policy have made them an economically viable option for capturing the power of offshore wind. The design of FOWTs is challenging due to the complicated interaction between the wind turbine, floating support structure, and stationkeeping system attached to the ocean floor. To meet these challenges, multiple companies and research institutions are investigating the modeling and design of FOWTS, major standards organizations have developed new code documents, and multiple pilot projects are in progress. This chapter will discuss several of the challenges posed by FOWT construction, along with the main findings and procedures of these recent initiatives in the context of the structural analysis of critical components.

2.2 Economics of FOWTs

A major prohibitive element of implementing FOWT wind farms is their cost. There are many proposed FOWT concepts based on floating oil and gas installations whose technical characteristic have been validated over the years, but there are strong disadvantages to these concepts to counter the advantages. Alternative versions of the proven oil industry designs are considered the most viable options for FOWT support structures, but as their technical performance characteristics have yet to be verified, significant engineering requirements remain, and therefore, cost is uncertain (Henderson, Witcher, & Morgan, 2009). To be economically viable, FOWT wind farms will require mass production of the support platforms. Thus, the design will need to be lighter and leaner than typical floating oil and gas installations (ABS, 2012). Opportunities to apply mass production should drive down the cost of FOWTs considerably and may eventually lead to floating systems that cost less than fixed bottom systems, which must be constructed at sea (Butterfield, Musial, Jonkman, Sclavounos, & Wayman, 2007).

FOWT designs must minimize materials for mass production to be economical. FOWTs are subjected to extreme forces from various environmental loads in addition to the principle loading force from wind turbine thrust. Resisting forces such as strong wave loads requires either large member sections, necessitating large quantities of materials, or high material strength, which may make the production of the platforms uneconomical (Henderson & Vugts, 2001). The thrust force from the wind turbine causes a large overturning moment at the base of the mast. Either a heavy base to counter the overturning moment or large sections to absorb the forces are needed in order to keep the platform stable. Increasing member surface area exposed to wave loading directly amplifies the total wave forces, requiring an even larger capacity to bear the forces. In addition to the direct forces, FOWT platforms are subjected to fatigue damage due to gyroscopic loads from the wind turbine and wave-induced motions, and therefore the platform motion responses must be minimized (Henderson & Vugts, 2001). This is most effectively achieved by larger support structures, which may not be economically feasible (ABS, 2012).

A major expense to implement FOWT wind farms will be the electrical infrastructure required to link the offshore farms to onshore grid systems. Flexible subsea cables installed along the ocean floor will be required to carry power to the shoreline. The construction of subsea cables is incredibly expensive. University of Hawaii Ocean Engineering graduate students conducted a feasibility study for a floating wind farm offshore of the island of Oahu, Hawaii and estimated that the cost to install a power cable will be $56,000,000 (Barnes, Casilio, Frederick, Nolte, & Schwartz, 2012). There are many options being researched that will deliver accumulated offshore power to onshore grids as needed during peak hours of energy consumption. A viable option currently receiving attention from researchers is compressed air or water storage vessels. The systems use the power generated by offshore wind turbines to compress water or air in a container at the bottom of the ocean, storing the potential energy, and then releasing the compressed water or air to a turbine onshore when the energy is needed. The vessels could also serve as anchors for FOWT mooring systems, potentially reducing the high cost of mooring system installation. For the interested reader, Slocum et al. (2013) provided a compelling underwater compressed water storage system design developed specifically for deployment with offshore wind farms.

2.3 FOWT Design Concepts

A FOWT platform must be sufficiently buoyant to support the weight of a wind turbine while restraining the motions of heave, pitch, and roll experienced by floating structures (Butterfield et al., 2007). There are three main technical static stability principles for floating wind turbine support structures that enable them to resist their principle loading, the wind turbine thrust. There are many possible floating platform configurations when considering the variety of ballast options, mooring systems, and means of achieving buoyancy utilized by offshore oil industry platforms. In order to achieve the necessary stability in the water, the forces must be transmitted to the water, and for some FOWT concepts, to the ground (Henderson et al., 2009). The first stability approach, generally referred to as buoyancy concepts, involves resisting the forces with a weighted water plane area, such as a barge or catamaran. These platforms achieve stability through the distribution of buoyancy to create a righting moment (Butterfield et al., 2007). A sub-sector of this genre is column-stabilized floating support structures. These structures consist of an underwater hull connected by columns or caissons which provide floatation and stability (American Bureau of Shipping, 2013). The columns or caissons and topside structure, if present, may be connected by an open space frame or an enclosed hull. The second approach to achieve stability is through high inertial resistance in the form of a heavy counter weight deep under the ocean surface to balance the thrust forces from the turbine, typical referred to as ballast or spar-type design concepts. These structures tend to be cylindrical in shape positioned vertically with a deep draft (ABS, 2013). The third approach utilizes a tensioned mooring system to provide stability for the structure, typically referred to as TLP (tension-leg platform) design concepts. The excess buoyancy of the platform maintains tension in the vertically moored stationkeeping system (ABS, 2013). Each design concept has its own strengths and is considered technically and practically viable. Various versions of each concept are being pursued for research and development around the world (Henderson et al., 2009).

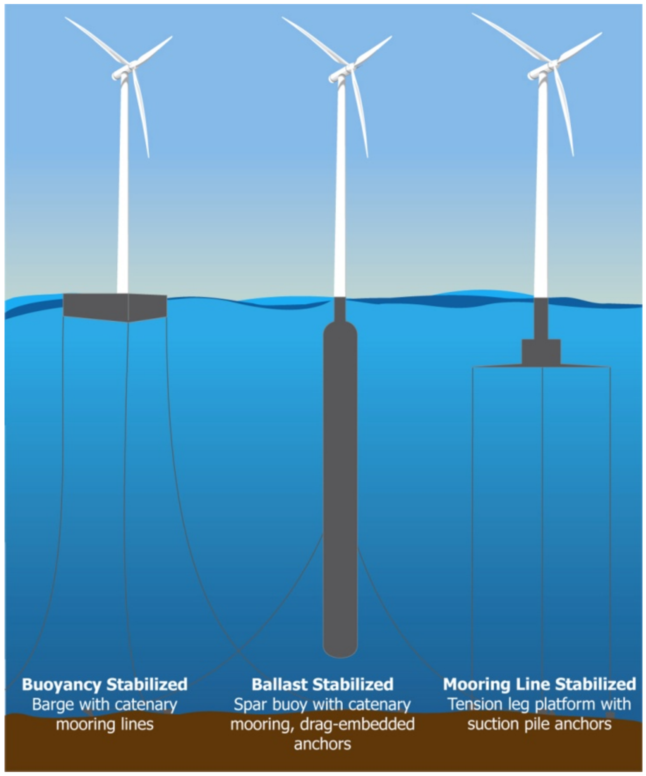


Figure 2.1. FOWT support platform design concepts. Reprinted from *DeepCwind Consortium - Stock Images*, n.d., Retrieved 23 March 2014, from www.deepcwind.org/press-and-media/stock-images. Copyright Advanced Structures and Composites Center. Reprinted with permission.

Each FOWT platform design concept uses some combination of the three primary methods to achieve stability in the water. The advantages and disadvantages of each approach must be considered to reach the most efficient system design (Butterfield et al., 2007). A barge-type structure will have a low production cost due to the simple shape which allows for uncomplicated fabrication techniques. However, since the buoyancy of the barge depends on water plane area, the structure is likely to be large and heavy requiring significant materials for construction. A TLP-type system will have the lowest displacement requirements and will therefore not require the significant material quantities as would a barge. However, TLP systems require mooring legs with high-capacity anchors that are expensive to install. In addition, more complexity is required in the TLP platform design to resist the mooring forces, adding to the system cost (Butterfield et al., 2007). There are various hybrid systems that use a combination of the three methods of stability to achieve an efficient design. An example is the semisubmersible concept that uses a large water plane area combined with a fairly deep draft and ballasting mechanism to provide stability (Robertson & Jonkman, 2011). The pros and cons of each platform concept must be weighed against the economic costs of each design parameter.

2.4 FOWT Research and Development

Thorough surveys of past and current FOWT research and development are provided in various publications available to the public. Perhaps the most comprehensive survey, the American Bureau of Shipping (2012) presented an extensive state of the art review on the major FOWT platform prototypes, design concepts, and their development. The review covers the many FOWT design concepts proposed over the last fifteen years and provides summaries of recent prototype projects installed up to the time of the ABS report release date. The discussions include concept development, model testing, software tools, and numerical simulations used for the analysis process. Wang, Utsunomiya, Wee, and Choo (2010) also provided an in depth literature survey of floating wind turbine research and development. Wang et al. (2010) discussed the conceptual designs and principles of various platforms for floating wind turbines. Recommendations for future work were made based on research that has been performed thus far.

The ABS (2012) state of art review provided a summary on the extensive research efforts at the National Research Energy Laboratory (NREL) and at the Massachusetts Institute of Technology (MIT), where researchers from both organizations invested a large amount of time and effort evaluating the response characteristics of existing FOWT concepts as well as developing and verifying software for the global performance analysis of FOWTs. The author recommends that the interested reader explore the studies performed by Jason Jonkman, an engineer at NREL. Jonkman has produced numerous publications on the dynamics of floating offshore wind turbines and verifying offshore wind system simulation models. Jonkman and Cordle (2011) presented a state of the art review of simulation software available to the offshore wind industry that are capable of performing integrated dynamic analysis for floating offshore wind turbines.

Only a few FOWT prototypes have been installed to-date. A major goal for those prototype studies was to evaluate the predicted dynamic loads and response characteristics of the FOWTs (ABS, 2012). Responses included the motions induced by wind and waves as well as the coupling between the floating support structure and the wind turbine. It was observed that floating platform response motions must be minimized to reduce fatigue damage due to gyroscopic loading and to optimize wind turbine performance (Henderson & Vugts, 2001). The modeling of the coupled system added to the complexity of the design process because analysis methods must account for nonlinear wave loading on the submerged platform and aerodynamic loading on the wind turbine (Henderson et al., 2009). The stability of the platforms was also found to be a key technical challenge, as the thrust from the wind turbine multiplied by the moment arm of the tower results in large overturning moments (Henderson & Vugts, 2001).

The first FOWT prototype was the Italian Blue H tension-leg platform. This scaled prototype was installed off the coast of Italy in 2008 for a six month testing period, not connected to the grid (ABS, 2012). Economic highlights of the Blue H study included: (1) offshore assembly was not required, avoiding the need for expensive marine construction; (2) costly seabed preparation was not required; and (3) the decommissioning cost was low (ABS, 2012). These observations are expected to apply to all future FOWT projects. The petroleum company Statoil introduced

Hywind, the first full-scale prototype, which was installed offshore Norway in 2009 for two years. The spar design consists of a concrete or steel cylinder with ballast and a base draft of 120 meters. The effect of negative damping on the tower vibration was found to be one of the key technical challenges of the design (ABS, 2012).

Figure 2.2. Hywind prototype being towed to installation site off the coast of Norway. Reprinted from *NorSea Group - Wind Power Projects*, n.d., Retrieved 23 March 2014, from www.norseagroup.com/services-and-solutions/project-logistics/other-projects/wind-power-projects.aspx. Copyright 2014 NorSea Group.

The Seattle based company, Principle Power, developed a full-scale prototype of the WindFloat concept with a 2-MW wind turbine that was installed off the coast of Portugal in 2011. A year later, the WindFloat had produced 3 GWh, enough to power 1300 homes (Snieckus, 2012; Expresso Póvoa Weekly, 2012). The WindFloat design consists of a column-stabilized platform with water entrapment plates at the base of each of the three columns connected by a space frame. The triangular structure is constructed entirely out of steel and designed for a 5-MW wind turbine to be positioned on one of the columns. The water entrapment plates and size of the platform facilitate the dynamic stability of the structure while active water ballasts in each column serve to resist changes in overturning moment due to the wind turbine. Principle Power was awarded at $4 million grant from the Department of Energy (DOE) in 2012 and will seek another $47 million in grants to complete a 30-MW wind farm project off the coast of Oregon by the end of 2017 (Davidson, 2013). An in-depth feasibility study of the WindFloat design is available by Roddier, Cermelli, Aubault, and Weinstein (2010). The WindFloat design was a source of inspiration for this project.

Figure 2.3. CAD rendering of Principle Power's WindFloat. Reprinted from *Wikimedia Commons*, 11 February 2011, Retrieved 23 March 2014, from en.wikipedia.org/wiki/File: Diagram_of_Principle_Power%27s_WindFloat.jpg. Copyright 2011 Creative Commons. Reprinted with permission.

Figure 2.4. WindFloat prototype installed approximately 5km offshore of Agucadoura, Portugal. Reprinted from *Wikimedia Commons*, 9 February 2012, Retrieved 23 March 2014, from en.wikipedia.org/wiki/File:Agucadoura_WindFloat_Prototype.jpg. Copyright 2012 Creative Commons. Reprinted with permission.

The VolturnUS project led by the University of Maine and their partners, the DeepCwind Consortium, recently installed the first floating offshore wind turbine in the United States. VolturnUS is a 1:8 scale concrete-composite semisubmersible FOWT prototype installed off the coast of Maine in May of 2013 (Danko, 2013). The concept consists of a tower attached to the top of a main central column surrounded by three pontoon columns that provide flotation and stability. The central column is connected to the flotation columns by smaller diameter pontoons. The system achieves stability with water plane area like a barge, but unlike a barge, the system has a relatively deep draft and utilizes ballasting for further stabilization (Robertson & Jonkman, 2011). The size and weight of this system are significantly larger than other FOWT support structure design concepts. The project received $12 million in funding from the DOE over five years and received an additional $4 million grant from the DOE in order to aid the construction of two full-scale 6-MW models by the end of 2017 with their partners, the DeepCwind Consortium (Danko, 2013). The state of Maine is currently planning to develop a floating offshore wind farm consisting of 80 wind turbines (Danko, 2013).

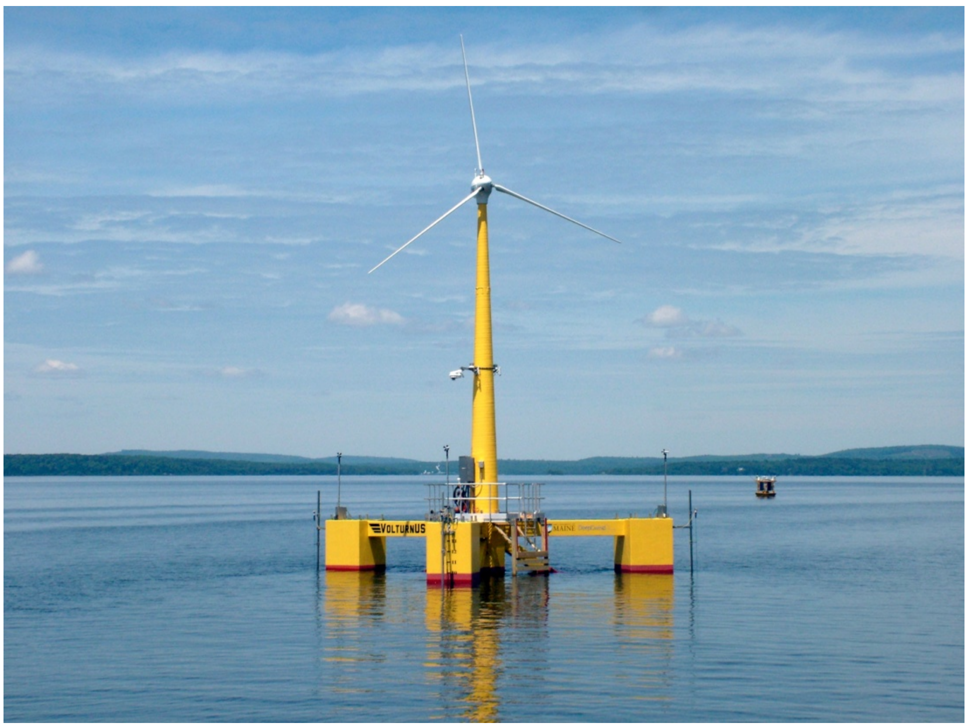


Figure 2.5. VolturnUS prototype installed off the coast of Maine. Andrews, E. (Photographer), Penobscot Bay Pilot, 14 June 2013. Copyright 2013 Ethan Andrews. Reprinted with permission.

The ABS (2012) state of the art review should be referenced as an introduction the development of FOWT concepts with potential application in Japanese waters. The review provides a summary of many publications related to the topic as well as a thorough overview of the projects. The review only discusses Japanese concepts and projects prior to the tsunami in 2011. Since the tsunami and the Fukushima disaster, Japan has been focused on moving away from nuclear energy and is in the process of developing a major wind farm project off the Fukushima coast. The first wind turbine started providing power to the grid in November of 2013 and will soon be followed by two additional 7-MW wind turbines, all three turbines supported by different conceptual FOWT designs (Hsu, 2013). The wind farm will eventually have 140 FOWTs producing an estimated 1 GW of power (Hsu, 2013).

The state of the art review provided in the ABS (2012) report on floating offshore wind turbine technology provides a thorough summary of past and current research and publications on the topic and should be pursued by any interested readers as an introduction to the development of FOWTs. Following the state of the art review, ABS provided three case studies for floating support structure concepts. Findings from the case studies and experience adopted from the offshore oil industry and offshore wind energy sectors were used to develop a design guideline for FOWTs released by ABS in January of 2013. The three major objectives of the case studies were: (1) to gain further insight into the dynamic interactions between the turbine, tower, floating support structure, and stationkeeping system; (2) to assess the applicability of relevant load cases defined in IEC 61400-3 (2009) code document for bottom-founded offshore wind turbines; and (3) to explore potentially critical load cases for FOWTs deployed in hurricane prone areas or during extreme storm events elsewhere (ABS, 2012). The resulting design guideline was the primary source determining design parameters for this project.

2.5 FOWT Design Guidelines

Agencies that determine standards for the marine industry are called classification societies. A structure that is designed and constructed according to the society's established code is said to be classed with the society once the structure passes inspection (ACI Committee 357, 2010). There are several existing classification societies found around the world including the American Bureau of Shipping (ABS) based in Houston, Det Norske Veritas (DNV) based in Oslo, Bureau Veritas (BV) based in Paris, Germanischer Lloyd (GL) based in Hamburg, and Nippon Kaiji Kyokai (NKK) based in Tokyo.

Within the past couple of years, code documents related to the subject of FOWTs have been introduced by major standards organizations including ABS and GL. These design guidelines have been created by merging the experience with fixed-bottom wind turbines, both onshore and offshore, with the experience of oil and gas industry offshore structures and recent FOWT research. These new guidelines focus more on the design of steel FOWT support structures, so there is a lack of information concerning some aspects of concrete support structure design. Particularly, the scantling, general sizing and thickness requirements, is not covered by available code for concrete FOWTS, though it is thoroughly addressed for steel structures. The ABS and GL standards refer to other standards for the design of floating concrete structures such as guidelines issued by the American Concrete Institute (ACI).

ACI (2010) has produced a report, ACI 357.2R-10, which provides practical experience and engineering considerations for the design and construction of floating concrete structures. Recommendations for establishing design loads and criteria are provided, making reference to classification society code documents as well as suggestions for methods of analysis (ACI Committee 357, 2010). Several classification societies have established design guides for various types of floating concrete structures, with the limit state design approach being the most common method for analysis and design. Limit state design examines all the possible conditions that might lead to structural failure or failure to perform the intended service (ACI Committee 357, 2010). A limit state condition is reached if a structure can no longer perform the function for which it was designed or fails to satisfy certain conditions set by the design parameters (ACI Committee 357, 2010). This project incorporates suggestions provided in ACI 357.2R-10 into the design of the floating concrete platform and construction recommendations.

Prior to the release of the GL and ABS design code for FOWT installations, the design of FOWTs was based on code developed for floating oil and gas industry installations as well as for fixed-bottom offshore wind turbines. Examples of such code documents are as follows:

- IEC 61400-3 (2009) – Design Requirements for Offshore Wind Turbines
- ACI 357R-84 (1997) − Guide for the Design and Construction of Fixed Offshore Concrete Structures
- ABS Guide for Building and Classing Offshore LNG Terminals (2010)
- ABS Rules for Building and Classing Mobile Offshore Drilling Units (2001)
- Germanischer Lloyd Guideline for the Certification of Offshore Wind Turbines (2005)
- RP 2A-WSD (2005) – American Petroleum Institute Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms

Both the GL and ABS code documents refer to the International Electrotechnical Commission (IEC) 61400-3 listed above for much of the design criteria related to offshore wind turbines. The code documents are generally consistent with their recommendations for establishing loading parameters and methods of analysis.

The ABS *Guide for Building and Classing Floating Offshore Wind Turbine Installations* released in January of 2013 was the main reference for determining load cases utilized in this project. The guide provides criteria covering the design, construction, installation, and survey activities for permanently sited floating offshore wind turbine installations (ABS, 2013). The guide addresses three main areas: (1) the floating support structure; (2) the station-keeping system; and (3) ancillary equipment and machinery that is not a part of the wind turbine system. Although ABS (2013) addresses the design and construction of both steel and concrete floating support structures, the guide refers to the ABS *Guide for Building and Classing Gravity-Based Offshore LNG Terminals* for general design guidance on the construction of reinforced and prestressed concrete structures. The author chose to follow recommendations presented in ACI 357.2R-10 and ACI 318-11 regarding concrete construction instead of referring to the aforementioned guide. The ABS FOWT guide was used to determine loading conditions and strength design criteria for this project. The load cases selected for investigation include permanent loads combined with realistic operating and environmental conditions (ABS, 2013). Design load conditions are reflected by load combinations established in the guide. The strength design criteria include partial safety factors to be applied to permanent and environmental loads. The ABS guide also recommends protocol for the fatigue assessment of the FOWT support structure, which is beyond the scope of the present work.

2.6 FOWT System Modeling and Analysis

A typical FOWT analysis model is an integrated system consisting of a turbine rotor nacelle assembly (RNA), a floating support structure, and a stationkeeping system. The foremost challenge of FOWT modeling is to predict environmental loads and the dynamic response of a wind turbine coupled with a floating support platform subjected to combined wind and wave loading. Current studies evaluating floating wind turbine concepts involve the generation of analysis models that couple the aerodynamic and hydrodynamic response of the wind turbine and floating platform, utilizing custom software to execute the analysis. The tower structural dynamics of large wind turbines are precisely tuned with advanced control mechanisms to optimize power output while minimizing interactions between the spinning turbine and floating support structure (Cermelli, Aubault, Roddier, & McCoy, 2010). Significant interactions between the wind turbine performance and floating platform motions are expected when the systems are combined (Cermelli et al., 2010).

According to Butterfield et al. (2007), the overall architecture of floating support structures may be determined by first-order static stability analysis that estimates general behavior, although many other critical factors will contribute to the size and characteristics of the final design. The additional complexities in the design process may be addressed by dynamic analyses that provide coupling between the motions induced by the wind turbine and the floating support structure. Coupled aerodynamic and hydrodynamic analysis is performed in the time-domain to capture dynamic nonlinearities. Such analysis involves the use of specialized software that performs advanced numerical simulations developed by organizations like NREL and other private companies. Cordle and Jonkman (2011) provide a summary of the current design tools that simulate floating offshore wind turbines and discuss the testing and validation of the simulation codes. Similar commercially available offshore software is not attuned to the specific complexities of FOWT design. Such software addresses the hydrodynamic loading on structures but neglects the aerodynamic loading experienced by a wind turbine.

The first step in the analysis process is to determine the design load cases applicable to a FOWT. Environmental load cases include wind speeds, wave data, current data, and if applicable, loads from ice and differential temperatures. Other load cases include permanent and variable loads such as the self-weight of the support platform and equipment placed on the deck during maintenance, respectively. The second step is the global performance analysis using a coupled simulation model that evaluates the integrated FOWT system including the wind turbine RNA, the tower, the floating support platform, and the stationkeeping system. Global performance analyses employ various design load cases with time-domain or frequency-domain simulations to evaluate the response characteristics of FOWTs. Both hydrodynamic and aerodynamic loading contribute to the structural response. Global responses include motion in six degrees of freedom and accelerations of the floating support structure, mooring line tensions, bending moments and shear forces at the interface of the tower and hull, and accelerations at the top of the tower (ABS, 2012). Dynamic platform motions include surge, sway, heave, roll, pitch, yaw, offset, and heel. Natural frequencies and modes of the floating support structure are also evaluated along with the resonant response of the various FOWT components. The global performance analysis informs the third analysis step, the finite element analysis of the FOWT support structure. Results from the finite element analysis provide forces or stresses for the structural design of the floating support structure. This project borrows data for the first and second steps above and skips to the third step. The final step in the analysis process is a fatigue assessment of the FOWT structure as recommended by design guideline for steel marine vessels. A fatigue assessment evaluates fatigue induced by long-term cyclic loading by waves and gyroscopic loading due to the wind turbine. A fatigue analysis involves the application of appropriate load spectrum or time series load cases based on accepted theories in accumulated damage (ABS, 2013). As previously stated, this assessment is outside the scope of this project.

2.7 Concrete FOWT Concept

The major reason to select a reinforced concrete FOWT support structure over a steel support structure is the superior performance of concrete in a marine environment. The choice of construction material will affect the lifecycle economy of a FOWT.

The durability of concrete is a critical issue when designing structures for the marine environment. Concerns regarding the deterioration of concrete structures due to rebar corrosion from chloride exposure have been mitigated by imposing limitations to concrete cover over reinforcement and with specialized concrete compositions (Olsen, 2011). Concrete mixes may be designed to be resistant to attacks from the harsh exposure to seawater, which is often achieved with high cement content and low water to cementitious material ratios (ACI Committee 357, 2010). Steel structures must be maintained properly in order to deal with corrosion resulting from continuous exposure to seawater. These maintenance procedures are not only expensive to perform, but time-consuming as well. The FOWTs are usually required to be taken offline and towed into harbor for servicing in a dry dock, losing profits while they are not producing any power. As reported by Butterfield et al. (2007), the operation and maintenance costs associated with offshore wind farm projects are estimated to be nearly a quarter of the lifecycle cost of a FOWT. The magnitude of the operation and maintenance costs may be greatly reduced by using a concrete support structure, although granted, much of the operation and maintenance costs are associated with maintaining the wind turbines themselves. While the potential reduction in costs is difficult to quantify, it does not mitigate the performance advantages of concrete versus steel in marine environments. According to Dr. Alfred Yee (2014), his concrete island drilling structure (CIDS) platform has been in operation for over thirty years with no apparent degradation and has required only minimal maintenance. Despite being subjected to the extreme forces induced by the North Sea over many seasons, the CIDS platform has no cracking in the concrete (Yee, 2014).

A semi-submersible barge-type concept with a ballasting system was selected as the FOWT support structure to be designed for the purposes of this project. Many design iterations were necessary to settle on a final layout for the concrete FOWT model. First, an initial model was generated in SAP2000 with scantling similar to Principle Power's steel-hulled WindFloat design. Various section depths were tested, but the model showed that the design configuration was not appropriate for concrete structures due to the high tensile stresses resulting from the applied loads. A concrete support structure would need to be arranged in a way that minimizes tensile loading by behaving as a rigid unit. When subjected to static loading, stability would be provided by its weighted water plane area just as an idealized barge-type platform. The goal of the design became to maximize the water plane area while maintaining a rigid structure that could be primarily loaded by compression rather than tension. A tri-floater, semisubmersible concept with a central core and large, rigid pontoon base was adopted. When subjected to the overturning thrust forces from the wind turbine and bending induced by wave loading, the core will be braced on all sides by rigid walls connected to floater cylinders, with the whole system behaving monolithically due to the rigid base pontoon. The platform will achieve stability utilizing the water plane area of the pontoon while the concrete buoyancy tanks, also referred to as "floater columns" in this paper, will provide significant mass to resist the overturning moment. Similar to the water entrapment plates of the WindFloat design, the base pontoon will assist in the control of the platform dynamic response characteristics, such as the resonant periods in heave, pitch, and roll motions (Aubault, Cermelli, & Roddier, 2006). The pontoon base should provide a means to dampen the motions of the whole system. Please see Table 2.1 for platform component dimensions. Please see Figure 2.6 below that indicate labels for the major platform components. Table 2.2 gives platform operational characteristics and concrete volume information.

Table 2.1

Concrete Platform Main Dimensions

Platform height	32 m	105.0 ft
Column diameter	10 m	32.8 ft
Column center to center	20 m	65.6 ft
Tower base diameter	8 m	26.2 ft
Length of brace float wall	12 m	39.4 ft
Width of brace float	6 m	19.7 ft
Length of pontoon edge	34.6 m	113.7 ft
Pontoon height	4 m	13.1 ft

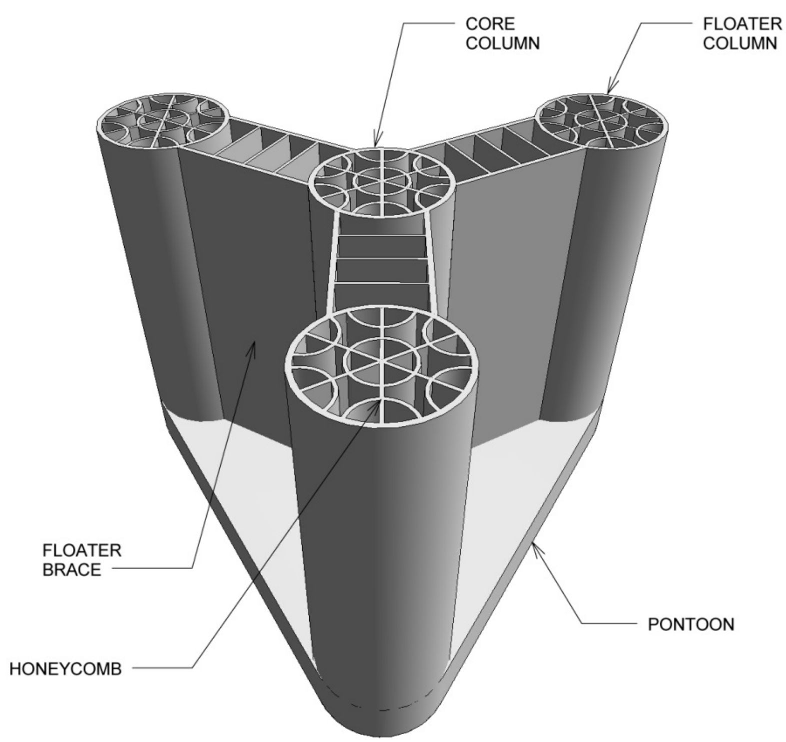


Figure 2.6. Platform component labels (with deck slabs removed from view).

Table 2.2

Platform Operational Characteristics

Draft =	17.6 m =	57.9 ft
Freeboard =	14.4 m =	47.1 ft
50 year wave air gap =	1.75 m =	5.73 ft
Operating draft =	22 m =	72.2 ft
Required water ballast =	4.37 m =	14.3 ft
Displacement at light draft =	13999 m^3 =	18310 yd^3
Displacement at operational draft =	16327 m^3 =	21355 yd^3
Concrete volume =	6748 m^3 =	13495 metric ton
	8825 yd^3 =	12243 short ton

A special feature of the columns is an internal honeycomb skeleton that provides significant stiffness to the structure. The honeycombs consist of six half-cylinders and brace walls rotated 60 degrees from each other. The honeycomb components span the entire height of the columns, acting as vertical diaphragms. The layout of the honeycombs was recommended by Dr. Yee in order to limit the unbraced length of both the honeycomb and column walls (Dr. A. Yee, personal communication, August 14, 2013). The honeycombs serve to brace the columns against external loads and to redistribute forces from weaker to stronger areas of the structure. Please see a plan view of the honeycomb structure below in Figure 2.7.

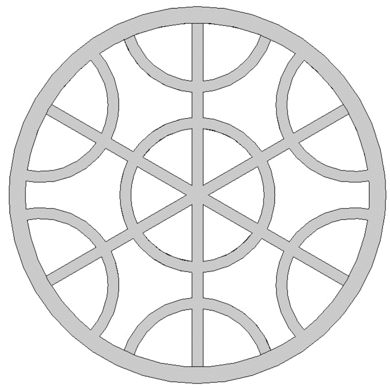


Figure 2.7. Plan view of core column honeycomb structure.

The analysis and design of the wind turbine tower are excluded from this project. Research is currently being conducted by Iowa State University on wind turbine towers constructed of concrete (Crawford, 2013). However, due to the extreme dynamic forces to which a FOWT tower will be subjected, steel towers will likely remain the only viable option. It is assumed these forces will cause huge tensile stresses requiring unreasonable amounts of steel reinforcement for a concrete tower. The towers will not be submersed in seawater, so corrosion will be less of an issue than for the platform. This item requires further investigation not covered by this work. A dynamic analysis will be essential.

The concrete mix for this project will be a high-strength, light-weight concrete, which has only recently become commonly used for floating marine structures. Light-weight concrete is used for marine vessels such as ships and barges, where maximizing payload and minimizing the power required in propelling the vessel are essential (ACI Committee 357, 2010). Normal-weight concrete is typically used for stationary vessels such as floating bridges and docks. Marine concrete employs supplementary cementitious materials, such as silica fume or fly ash, as well as water-reducing admixtures or workability agents to reduce the water-cementitious material ratio. A light-weight concrete may have a unit weight of 120 to 125 lb/ft^3 (1920 to 2000 kg/m^3) as opposed to normal-weight concrete that has a typical unit weight of 150 lb/ft^3 (2400 kg/m^3) (ACI Committee 357, 2010). It is common for normal-weight and light-weight concretes to reach strengths in excess of 12,000 psi (83 MPa) and 9000 psi (62 MPa), respectively (ACI Committee 357, 2010). High-strength, light-weight concrete mixes with great durability have recently become more commonly specified for floating marine structures because they can now be consistently and economically produced in the construction field (ACI Committee 357, 2010). A recommended concrete mix for this project is given later in the report.

In addition to high-strength concrete that is ideal for marine environments, prestressing concrete structures provides the capacity to resist loading typically not suitable for concrete. This adds another element to the seaworthiness of concrete vessels versus steel vessels. Prestressed concrete structures have the ability to control tensile stresses and to close cracks that develop from temporary overload situations, enhancing water tightness and durability (ACI Committee 357, 2010). Concrete cracks may never develop if a marine structure is prestressed correctly, and therefore, the internal rebar will never be exposed to seawater that would cause corrosion, preventing deterioration of the concrete.

Chapter 3

CONCRETE FOWT PLATFORM ANALYSIS MODEL

3.1 Introduction

This chapter presents the three-dimensional equivalent static analysis of a concrete FOWT platform. The general analysis method will be summarized followed by a discussion of the establishment of design load cases. The application of the design load cases to the analysis model will be described with particular attention given to the definition of wave load patterns in SAP2000. The finite element model characteristics are described, including the establishment of appropriately representative boundary conditions. An interpretation of the analysis results and typical force results are given. Maximum forces that govern the platform design are identified and described. The chapter ends with a discussion of the limitations of modeling and the analysis method.

3.2 Analysis Method

SAP2000 Ultimate Version 15.1.0 finite element analysis software was used to perform a first-order elastic analysis on a FOWT platform model. Loading cases accounting for environmental conditions and permanent loads were applied in different combinations and analyzed to find the maximum response of the hull structure as recommended by ABS (2013). Environmental load cases, considering wind and wave loading, were applied to a FOWT platform model as static forces. Wave loads were generated using Airy wave theory and Morison's equation (Computers and Structures, Inc., 2011). Results from the analysis provided forces to design the major platform components and to confirm the sufficiency of the proposed platform scantling. The initial design may be imported into more sophisticated software for integrated aerodynamic and hydrodynamic analysis with site-specific environmental loading criteria. According to Butterfield et al. (2007), linear elastic analysis methods were appropriate for this project as the overall configuration of a FOWT floating support structure may be determined with first-order static stability analysis.

Cermelli et al. (2010) also used SAP2000 finite element analysis software to determine the design stresses for the structural design of the WindFloat platform. However, the forces and stresses applied to their finite element model were determined by running their own numerical simulations of integrated aerodynamic and hydrodynamic models, whereas this project derived its global loading from numerical simulations determined from case studies performed by ABS (2013). The case studies provided global loads from the integrated analysis that were then converted into static loading and applied to the SAP2000 model for linear elastic analysis. The force results from the case studies, particularly the tower base shear and overturning moments, had to be specifically adapted for application to the finite element model. The FEA provided forces for the structural design of the floating reinforced concrete platform members.

3.3 Model Load Cases

Design parameters considered in this project are as follows:

- Return period of extreme environmental conditions
- Turbine operating conditions
- Misaligned loading of wind and wave

Static loads due to wind and wave loading were derived from case studies on conceptual FOWT case studies performed by ABS (2012). The case studies focused on the OC3-Hywind Spar, the MIT/NREL mono-column TLP, and a generic WindFloat semi-submersible concept, each with a NREL 5-MW offshore wind turbine. The case studies applied representative environmental load conditions from the Gulf of Mexico, Northeast Coast, and West Coast on the United States Outer Continental Shelf. The load cases evaluated in the study consider typical operating conditions of a wind turbine combined with environmental conditions. Load cases accounted for both normal weather conditions and extreme weather conditions, represented with 50-year storm and 100-year storm case data. The turbine operational conditions considered were power-production mode, start-up mode, and emergency shut-down. Mean wind, wave, and current conditions are assumed for each operational mode of the wind turbine for the case study simulations. V_{in} is the wind speed at which the wind turbine starts to produce power; V_r is the wind speed at which the rated power output of the wind turbine is achieved; V_{out} is the highest wind speed at which a turbine is designed to produce power (ABS, 2013). Power-production mode typically resulted in the largest thrust forces from the turbine. Emergency shut-down of the turbine occurs during extreme weather events when the high wind speeds would cause damage to an operating wind turbine. The large overturning moments that occurred with this case were largely due to the acceleration of the tower as the platform rides large waves.

Global loading forces used for this project were based on results reported from the case study of a generic WindFloat concept evaluated for West Coast environmental conditions. Site-specific metocean data was derived from the records of a NOAA buoy and water level station located off the coast of Northern California (ABS, 2012). Only load cases applicable to static loading were selected for evaluation from the ABS (2013) FOWT design standard. Load cases considered for this project are given below in Table 3.1. Design environmental conditions include loading due to wind, wave, and ocean currents. A 100-year return period for environmental conditions was assumed with a 5-MW wind turbine loaded by the range wind forces generated within the return period. A conservative approach was taken by combining all extreme load cases within the same return period (ABS, 2012). Partial safety factors of 1.35 were applied to environmental loads as defined in ABS (2013) 7-2/7.5. A partial safety factor of 1.0 or 0.9 was applied to permanent loads, such as the self-weight loads of the tower supporting the RNA and the platform, as per ABS (2013) 7-2/7.5. A 0.9 factor was applied to account for a permanent load that relieves a total load response.

Table 3.1

Summary of Load Cases for Concrete FOWT Analysis

Turbine Operational Mode	Wind Condition	Load Case Label	Directions		Wind Speed at 10 m	Wave Condition			Current Speed	Tower Base Shear	Tower Base OTM
			Wind (deg)	Wave (deg)	(m/s)	H_s (m)	Max (m)	T_p (sec)	(m/s)	(kN)	(kN-m)
Power Production	ETM, V_{in}	1	0	0	2.53	2.01	4.02	11.52	0.6	700	40500
	ETM, V_r	2	0	0	9.17	2.93	5.86	12.31	0.6	2000	130500
	ETM, V_{out}	3	0	0	19.02	5.16	10.32	14.13	0.6	1650	101000
Start-Up	EOG, V_{in}	4	0	0	2.53	2.01	4.02	11.52	0.6	500	20450
	EOG, V_r	5	0	0	9.17	2.93	5.86	12.31	0.6	2175	127000
	EOG, V_{out}	6	0	0	19.02	5.16	10.32	14.13	0.6	1250	80000
Emergency Shut Down	NTM, V_r	7	0	0	9.17	2.93	5.86	12.31	0.6	1750	104500
	NTM, V_{out}	8	0	0	19.02	5.16	10.32	14.13	0.6	1275	90000
(Idling)	10 yr storm	9	0	0	23.49	10.75	21.5	18.53	0.7	1350	70000
	50 yr storm	10	0	0	26.27	12.62	25.24	19.96	0.8	1000	50500
	100 yr storm	11	0	0	27.48	13.43	26.86	20.58	0.8	1150	60000
	100 yr storm	12	30	30	27.48	13.43	26.86	20.58	0.8	1150	60000
	100 yr storm	13	90	90	27.48	13.43	26.86	20.58	0.8	1150	60000
	100 yr storm	14	0	30	27.48	13.43	26.86	20.58	0.8	1150	60000
	100 yr storm	15	0	90	27.48	13.43	26.86	20.58	0.8	1150	60000

Notes:	ETW = extreme turbulence model	V_{in} = cut-in wind speed
	EOG = extreme operating gust	V_r = rated wind speed
	NTW = normal turbulence model	V_{out} = cut-out wind speed
	H_s = significant wave height	T_p = peak period of wave spectrum

Load cases that accounted for aligned and misaligned wind and wave directions were incorporated into the load case definitions for the 100-year storm conditions. The aligned cases applied wind, wave, and current loads at a 0 degree, 30 degree, and 90 degree headings. The misaligned cases assumed a wind and current applied at a 0 degree heading while the wave loads were applied at headings of 30 and 90 degrees. Definitions of the headings are given in Figure 3.1. A total of 60 load combinations applied the load cases at a 0 degree heading and at a 180 degree heading (negative directions of 0 degree cases). A summary of the load combinations may be found in Appendix B.

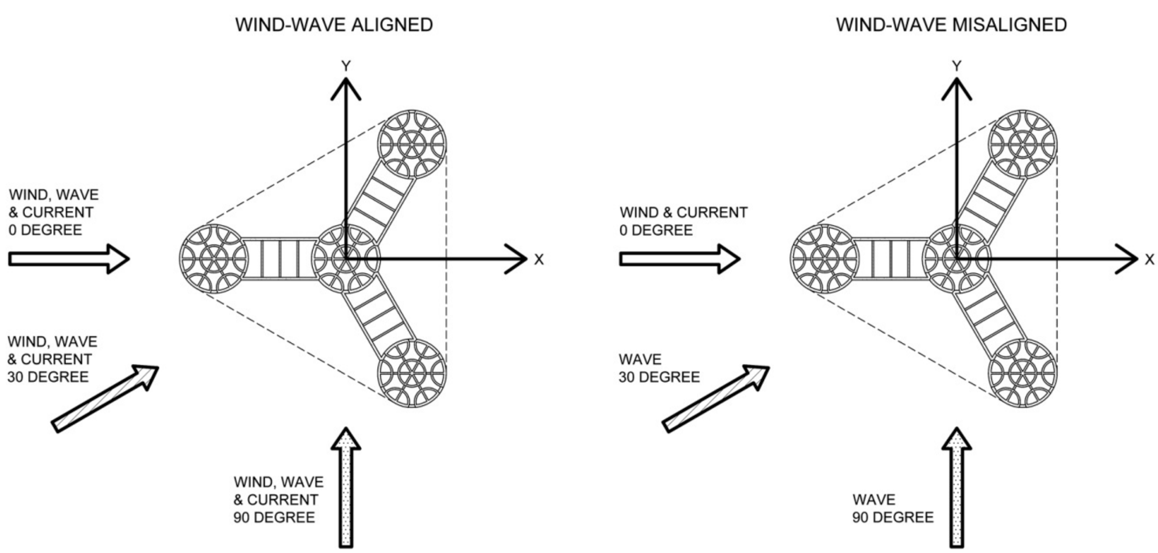


Figure 3.1. Definitions of aligned and misaligned 100 year storm wind and wave load cases.

3.4 Model Load Application

Permanent gravity loads, combined with a spectrum of environmental load cases, were applied to the analysis model in order to determine the most unfavorable effects on the platform. The resultant forces were then used to verify the sufficiency of the proposed platform layout and to determine the reinforcement required to resist the worst-case forces. Roddier et al. (2010) found the loading on the underwater portions of the structure, such as the base pontoon and floater columns, is dominated by wave loading. Aerodynamic loading due to the wind turbine has a more significant impact on platform elements, such as the top of the brace float walls and the upper section of the central column that serves as an interface between the tower and the platform (Roddier et al., 2010).

Loading from wind and waves oriented in the global positive x-direction of the model were applied as lateral forces. The environmental loads accounted for in the analysis model included static loading due to waves, wind pressure, currents, buoyancy forces, and thrust forces from the wind turbine. The load cases were applied in all directions with the different load combinations as per ABS (2013) criteria. As discussed previously, the misalignments of wind and wave forces were also accounted for in the load cases.

SAP2000 represents the application of static wave load patterns by a significant wave height, a specified wave return period, and a direction. The automatic wave loads are based on requirements presented by the American Petroleum Institute 2000 reference for designing fixed offshore platforms (Computers and Structures, Inc., 2011). Wave load patterns also account for current and buoyancy loads on immersed structural members and wind loads on members above the water surface. Wave velocity and acceleration fields are generated using Airy (linear) wave theory and the wave force on the member is calculated using Morison's equation (Computers and

Structures, Inc., 2011). The wave loading results from water pressure integrated over the wetted surface of the platform (Butterfield et al., 2007). SAP2000 automatically defines load cases for the defined wave load patterns. The magnitude of wave loads are assigned based on the surface area exposed to the wave loading and volume of a member. Wave loads are automatically applied to frame members in the model. Once a wave load case is defined, there is no need to assign the wave loads to a member, only to provide frame members with appropriately representative surface areas and volumes. Figure 3.2 shows the wave load pattern definition for the 100-year return storm condition with aligned wave, current, and wind. Figure 3.3 shows the plot of the load pattern displaying the resultant velocity of the wave.

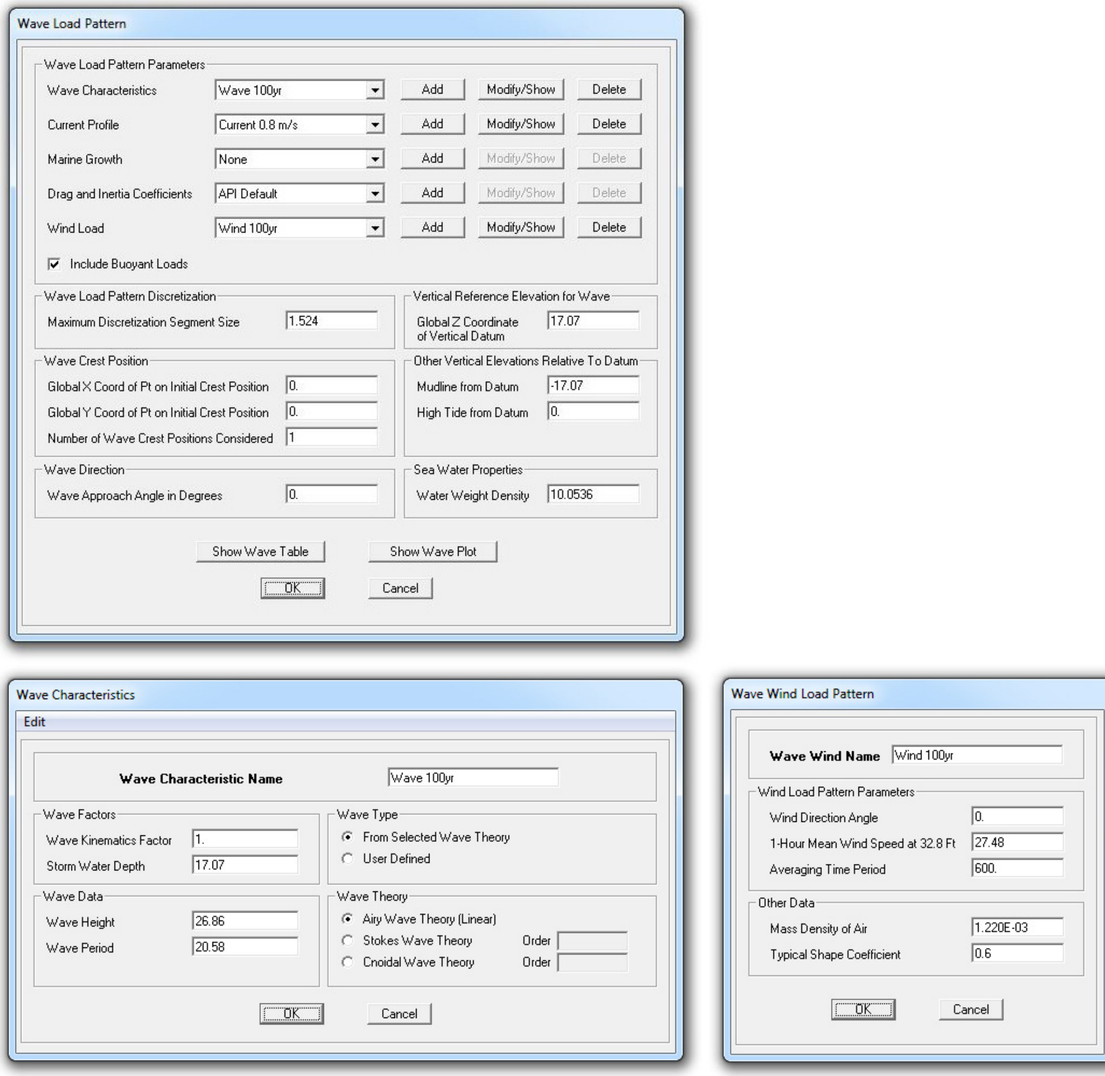


Figure 3.2. Wave load pattern definition in SAP2000 for aligned heading case of 100-year return storm condition (units in meters).

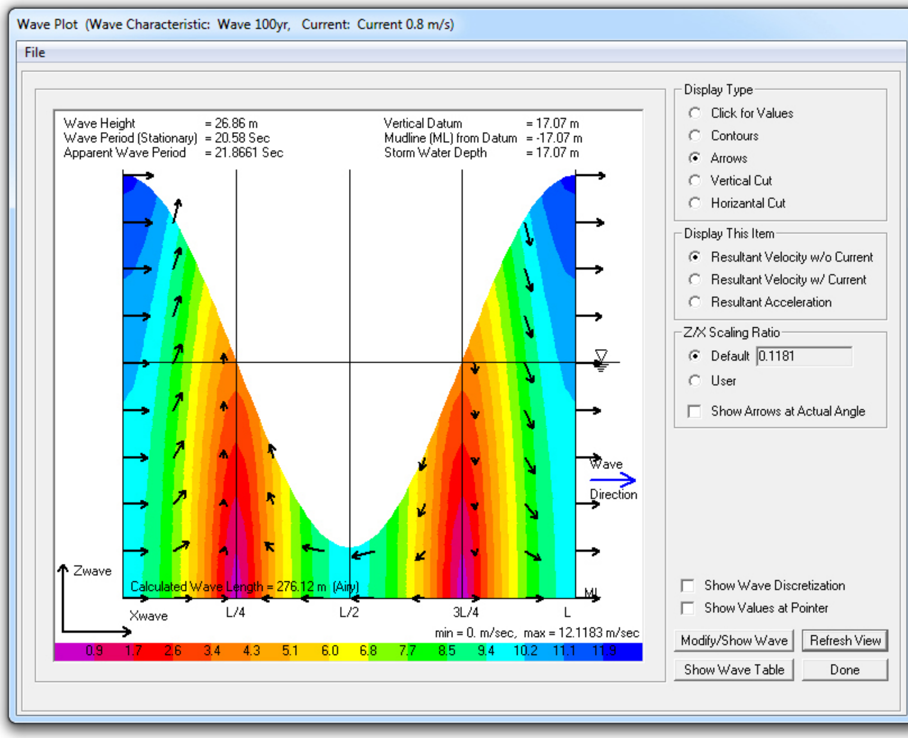


Figure 3.3. 100-year wave load pattern plot displaying the resultant velocity of the wave.

A physical representation of the wind turbine and tower were not included in the analysis model. The weight from the tower and RNA components were applied on the core column top deck as a distributed load around the circumference of the tower base. The total applied load due to the RNA and mast was 8376 kN (1883 kips). This load was factored as permanent load in the load combinations. Thrust forces due to wind turbine loading were broken into tower base shear and overturning moment components. These loads were based on the ABS (2012) case study results from the integrated dynamic analysis of the global system. The base shear forces and overturning moments had to be converted into distributed loads that could be applied to the analysis model. The overturning moments were decoupled into resultant forces in both the global positive and negative gravity directions. Figure 3.4 shows the application of a typical tower base shear load in

SAP2000. Figure 3.5 shows the application of a typical overturning moment load at the base of the tower in SAP2000.

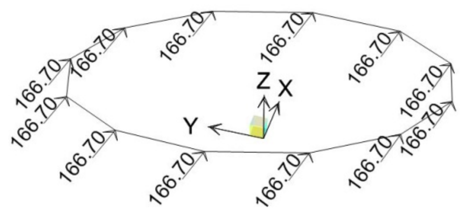


Figure 3.4. Base shear load applied at top of center honeycomb column due to load case considering the wind turbine operating at the rated wind speed with extreme turbulence.

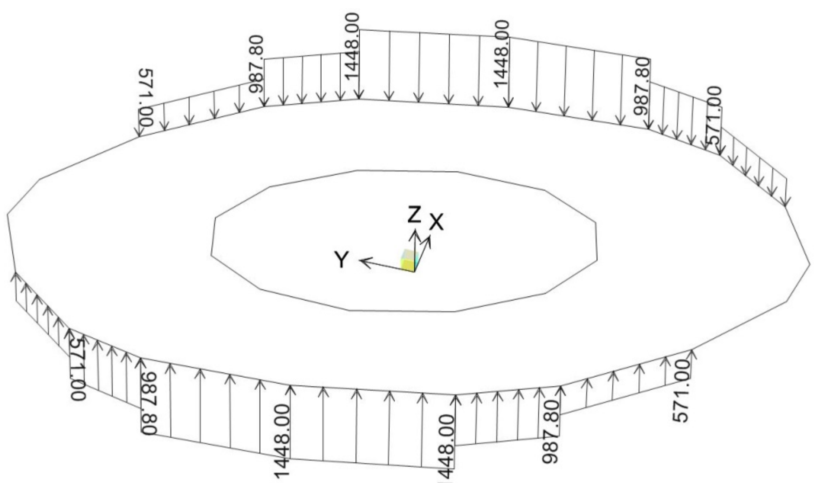


Figure 3.5. Overturning moment load applied at base of tower due to 100 year storm load case.

3.5 Model Characteristics

The initial scantling of the FOWT platform was determined with the aid of Autodesk Revit 2013. Various iterations of the FOWT platform were modeled at full scale giving concrete volume quantities which then were used to determine the draft of the global system, ensuring the platform would float and have sufficient air gap as specified per ABS (2013) 6/1.7 (minimum air gap requirement of 1.5 m during 50 year storm event). A maximum total wave height of 26 meters for a 100 year storm event in the Pacific Ocean was derived from FOWT cases studies found in the ABS (2012) report on floating wind turbines. The wave height determined that a minimum freeboard of 14.5 meters (half of the total wave height plus the minimum air gap requirement) was required for the concrete platform design. Horizontal dimensions of the platform were selected to be similar in scale to other FOWT platforms, such as WindFloat. Column center to center distance was minimized to obtain a more rigid structure, anchored by the pontoon base, although a large water plane area was desired for stability and buoyancy.

The platform was modeled so that the central column was braced against thrust loading from the turbine by all of the surrounding components. The surrounding components, consisting of the brace floats and floater columns, also provide floatation for the system. Each member must be substantially robust to resist wave loading in extreme weather events. The columns were modeled with an internal honeycomb structure consisting of cylindrical sectors and brace walls. The proportions of the honeycomb were selected to minimize the unbraced length of each wall component. The brace floats were modeled with thicker external walls braced perpendicularly by thinner internal walls. Please see Figure 3.6 below that provides main dimensions for the platform model. The platform was modeled to be is 32 m (105 ft) in height.

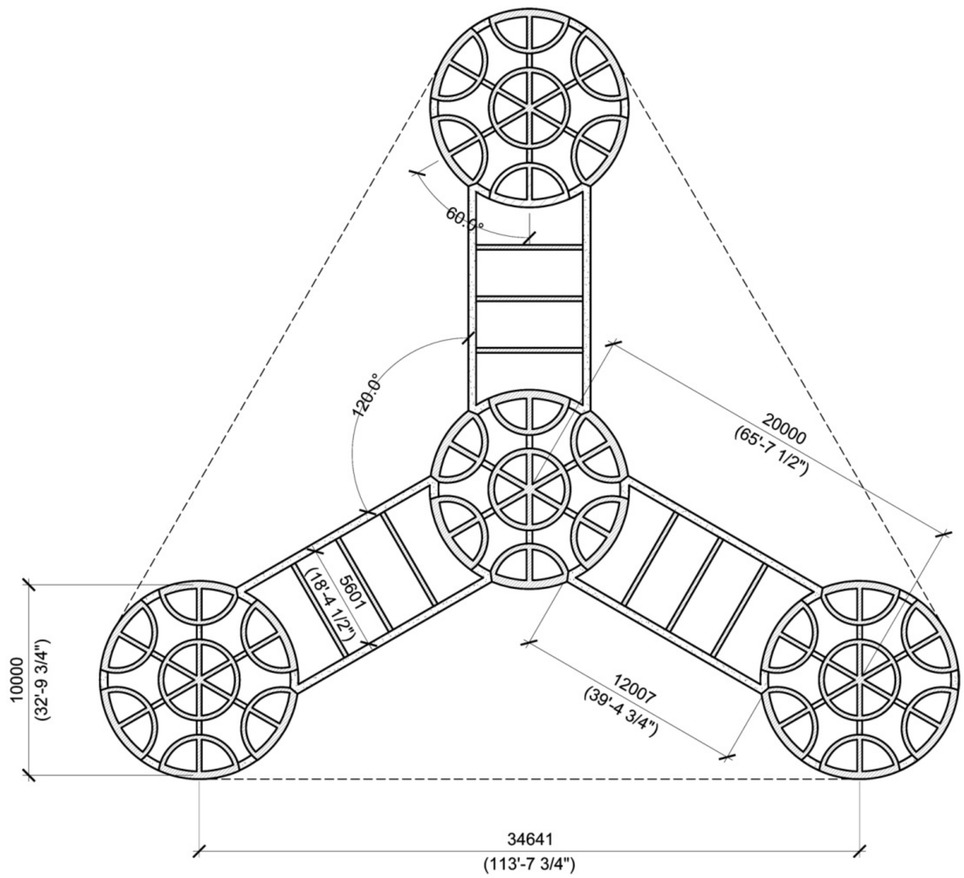


Figure 3.6. Plan view showing main platform dimensions (units in mm and degrees).

A course mesh model with thick-plate area shell finite elements was generated in SAP2000. Due to the shape of the platform, it was necessary to include some triangular area shell elements in certain locations, although four-node finite elements are preferred for more accurate results. In future work, finer meshes should be implemented in areas with high stress concentrations for a more accurate examination of stressed areas. The complete model was comprised of 17,454 area shell elements. Please see Table 3.2 for a summary of the analysis model element properties. 60 MPa (8700 psi) concrete was defined as the material for all shell elements. The modulus of elasticity of the concrete was defined as 36406 MPa (5317 ksi) as consistent with ACI 318-11 8.5.1. Element local axes were oriented to be aligned with the global axes in the x and y-directions for most elements. Only the brace floater deck slabs had their local axes rotated for a more convenient interpretation of the force results. No stiffness modification factors were applied in the section definitions, as permitted by ACI 318-11 10.10.4.1, because it was assumed that the sections were to remain uncracked.

Table 3.2

Analysis Model Components

Component	Section Name	Description	Element Type	Element Thickness	Number of Elements
Core Column	CoreOuter	Cylindrical outer core wall	Area thick shell	457 mm (18 in)	960
	CoreCenter	Cylindrical inner core wall	Area thick shell	305 mm (12 in)	384
	CoreBrace (1-6)	Honeycomb brace walls	Area thick shell	305 mm (12 in)	160 each (960 total)
	CoreSector (1-6)	Honeycomb half-cylinder walls	Area thick shell	305 mm (12 in)	192 each (1152 total)
	Deck1067	Deck supporting wind turbine tower	Area thick shell	1067 mm (42 in)	144
Floater Column (3 total)	FloatOuter	Cylindrical outer floater column wall	Area thick shell	406 mm (16 in)	960 (2880 total)
	FloatCenter	Cylindrical inner floater column wall	Area thick shell	254 mm (10 in)	384 (1152 total)
	FloatBrace (1-6)	Honeycomb brace walls	Area thick shell	254 mm (10 in)	160 each (960 per column)
	FloatSector (1-6)	Honeycomb half-cylinder walls	Area thick shell	254 mm (10 in)	192 each (1152 per column)
	Deck305	Floater column top deck	Area thick shell	305 mm (12 in)	144 (432 total)
Floater Brace (3 total)	BraceWall406	Outer walls spanning between columns	Area thick shell	406 mm (16 in)	768 each brace (2304 total)
	BraceWall254	Brace walls perpendicular to outer walls	Area thick shell	254 mm (10 in)	432 each brace (1296 total)
	Deck457	Floater brace top deck	Area thick shell	457 mm (18 in)	72 each brace (216 total)
Pontoon	PontoonSlab	Top, bottom, and perimeter pontoon slabs	Area thick shell	457 mm (18 in)	1782
					Total = 17454

Images of the SAP2000 analysis model are given below in Figures 3.7 and 3.8.

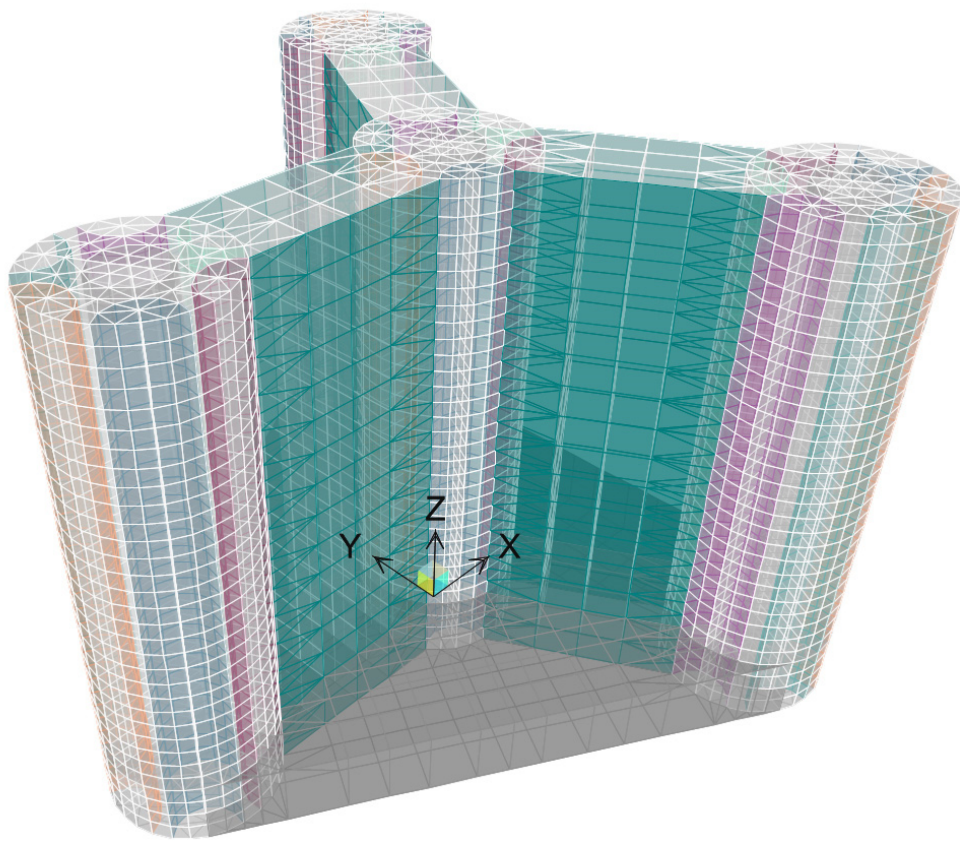


Figure 3.7. SAP2000 analysis model of concrete FOWT platform.

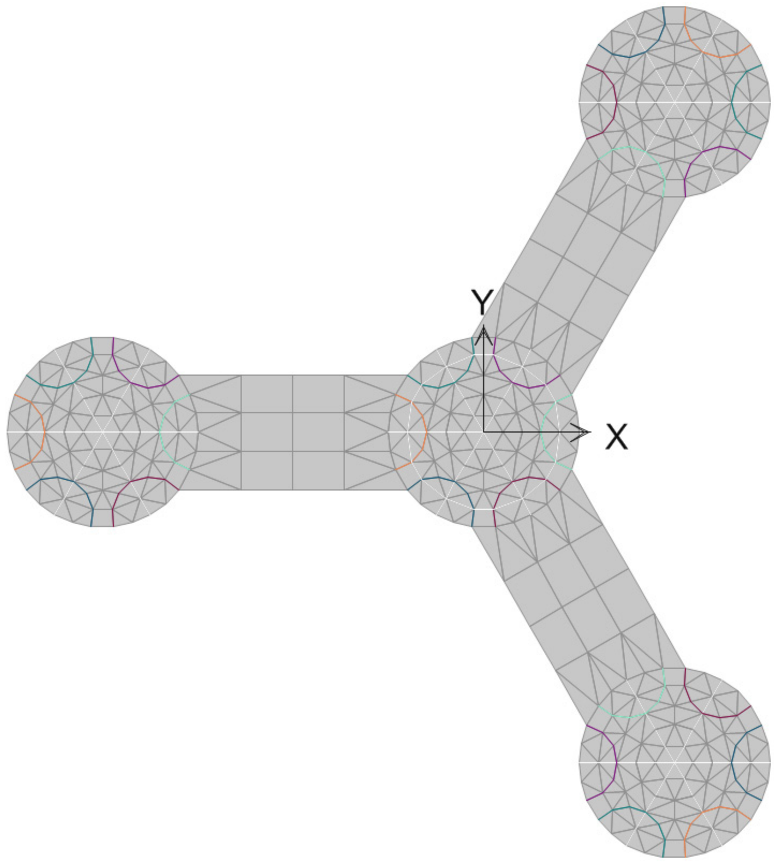


Figure 3.8. View of top deck of SAP2000 analysis model.

Vertical frame elements with equivalent surface area to the platform members were assigned to account for wave loading. The stiffness and mass parameters were modified by a factor of 0.00001 so that only the element surface areas resulting in wave loading would contribute to the model. Wave loads are applied with the wave crest positioned at the center of the core column (x = 0 and y = 0). SAP2000 offers the option to evaluate multiple wave crests, but as the period of the waves being evaluated are long, only looking at one wavelength is sufficient for static loading. Please see Figure 3.9 for an extruded view of the representative frame elements for wave loading and a view of wave loads applied to the core column representative frame element. Hydrostatic pressures were neglected in the model as they are insignificant in comparison to other loading sources. The buoyancy forces on the structure were automatically accounted for in SAP2000 when wave loading was applied.

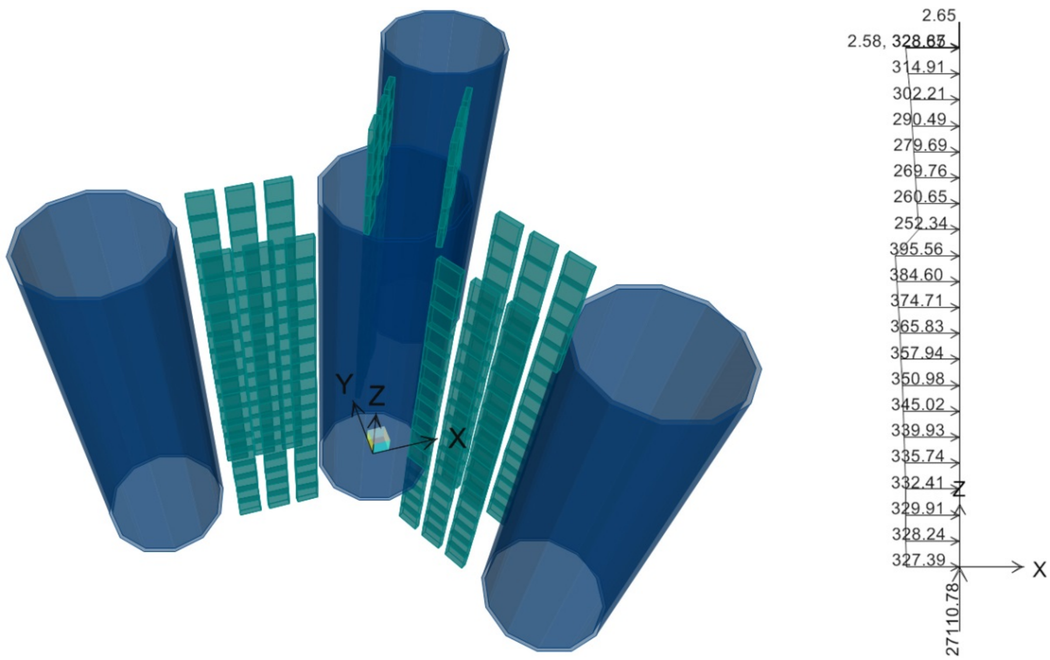


Figure 3.9. Right: Extruded view of frame elements representing columns and floater brace walls for wave loading; Left: 10 year storm wave load case applied to core column representative frame element (units are kN).

Establishing model boundary conditions was a significant challenge. The typical boundary conditions available in SAP2000, i.e. pinned or fixed conditions, did not properly reflect the conditions of a structure that was floating, as undue stress concentrations occurred at the base when lateral loads were applied. To avoid stress concentrations at the base of the structure, water surrounding the platform was modeled to simulate that the platform was floating. However, this approximation was not completely accurate because the base of the pontoon was “fixed” to the modeled water which still induced stress concentrations. It was difficult to determine the best location to approximate the water fixity as the pontoon will be fully submersed in the water. It was therefore determined that the platform would be modeled with a pinned base and that stress concentrations at the base would be assumed as potentially larger than might occur with more advanced modeling techniques. Accounting for the active water ballast was neglected in the analysis model due to the pinned base fixity. In a dynamic analysis, the ballast system should be modeled to resist the overturning moment on the global system.

3.6 Analysis Results

Maximum shell element forces for each structural member were obtained and evaluated from the finite element analysis of the FOWT platform. As previously stated, a general goal of the concrete FOWT analysis was for the platform model to experience no concrete cracking due to all applied combinations of loads. It was assumed that concrete will not crack unless the stress in a section is greater than the tensile strength of the concrete. Steel reinforcement bars will only be engaged once the concrete cracks. The cracking moment and tensile forces that would initiate cracking were evaluated for each section depth as per ACI 318-11 9.5.2.3. Please see Appendix C for calculations giving the cracking moment and tensile strength for each section depth. The maximum resultant forces found in the model were mostly due to the 100 year storm load cases applied at various headings and sometimes due to the rated wind speed load case with the largest overturning moment at the base of the tower. This was contrary to the finding of the ABS case studies where the maximum global responses tended to be from power production conditions. The forces found in the model never exceeded the critical cracking limits over a large area except for at one location, near the base of the structure on the float columns up to the intersection with the pontoon. These high stress concentrations were assumed to be due to modeling effects related to the pinned base fixity. The reinforcement at this location was designed for the maximum forces; however, it was assumed that the resultant high tensile forces will not be reached at this location. There were many instances where the cracking forces were exceeded over a small area, but due to the monolithic design of the structure and the nature of concrete structures, these forces may be averaged over a larger area as forces will redistribute within the section. Despite no other sections besides at the base of the structure were found to be cracked, reinforcement will be provided nonetheless.

The analysis showed that the platform layout worked as intended. Overturning moment loads and tower base shear loads due to thrust from the wind turbine primarily resulted in compression rather than tension in the bracing members, although tension in the members did occur for some load cases with large wave forces. In vertical members, the local axes have been assigned so that local axis 1 is horizontal and local axis 2 is vertical. F22 tension forces were generally greater then F11 tension in vertically oriented members (columns and brace walls) except for at a few locations that were evaluated case by case. Therefore, for most members the typical reinforcement determined by the evaluation of F22 forces was used as the reinforcement in the local axis 1 direction as well. The resultant compressive forces for all load cases were not close to the capacity of the concrete without reinforcement. Table 3.3 gives force envelopes for each structural member type. The definitions of the area shell element forces are as follows: F11 and F22 are direct tensile and compressive forces along local axes 1 and 2, respectively, with positive values for tension and negative values for compression; M11 and M22 are local bending forces along the respective local axes; V13 and V23 are out-of-plane transverse shear forces that run perpendicular to the local axes. Each force is given per one meter section width. The pontoon forces are not included as they are not considered to be realistic. This is discussed further in the following section of the report. Images of typical F11 and F22 force envelopes including all 60 load combinations and load combinations that resulted in maximum F11 and F22 forces are shown below the table.

Table 3.3

Structural Member Force Envelopes

Model Component	Section Name (Thickness)	F11 (kN/m)	F22 (kN/m)	M11 (kN-m/m)	M22 (kN-m/m)	V13 (kN/m)	V23 (kN/m)
Core	Outer Wall	2772	2149	96	40	363	130
Column	(457 mm)	-2830	-2774	-95	-52	-353	-139
	Honeycomb Walls	819	1437	44	24	127	81
	(305 mm)	-859	-1714	-45	-25	-127	-79
	Deck	7445	9960	912	521	1625	922
	(1067 mm)	-7670	-10141	-760	-464	-1613	-922
Floater	Outer Walls	1036	3666	58	98	87	208
Column	(406 mm)	-1033	-4111	-56	-88	-86	-195
	Honeycomb Walls	743	1857	22	10	116	63
	(254 mm)	-744	-2111	-23	-11	-120	-64
	Deck	912	1130	6	5	22	25
	(305 mm)	-906	-1122	-6	-5	-26	-25
Floater	Outer Walls	2808	2607	49	26	78	79
Brace	(406 mm)	-2916	-3240	-49	-29	-78	-79
	Brace Walls	254	1010	30	7	30	19
	(254 mm)	-251	-1400	-30	-7	-30	-19
	Deck	2396	2920	18	20	69	65
	(457 mm)	-2496	-3019	-18	-19	-70	-63

Note: Shaded values indicate forces and moments that govern design (subsequent chapters).

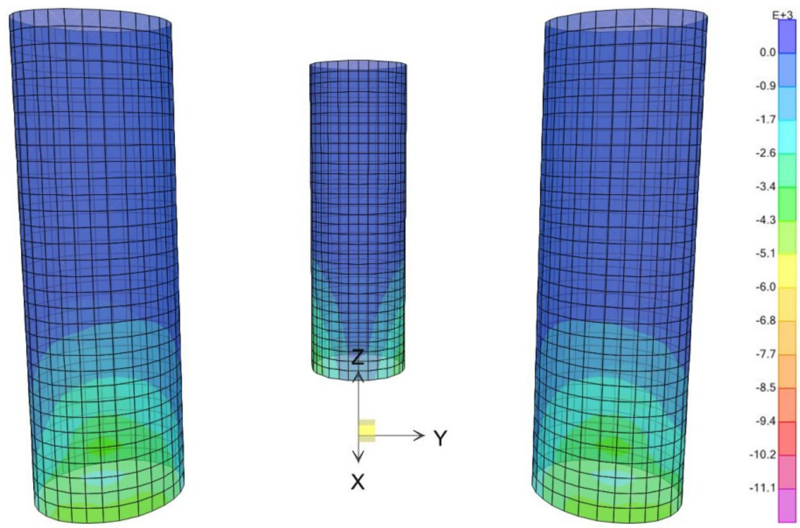


Figure 3.10. View of float columns; F22 minimum force envelope (kN/m).

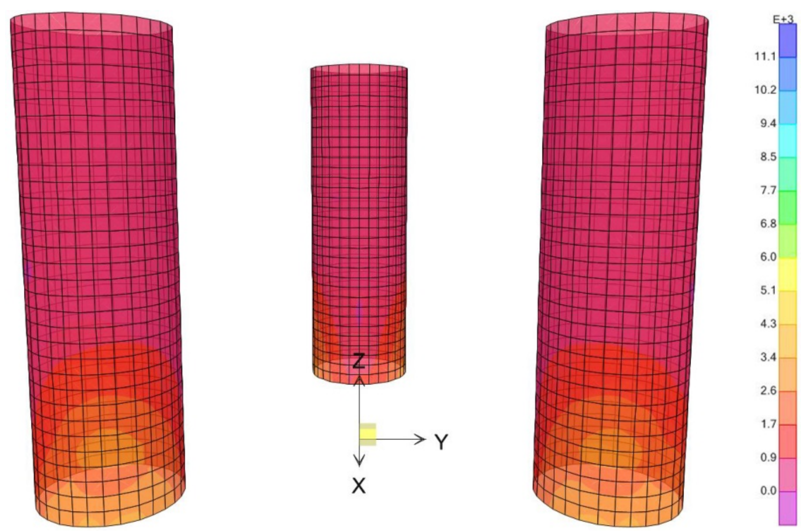


Figure 3.11. View of float columns; F22 maximum force envelope (kN/m).

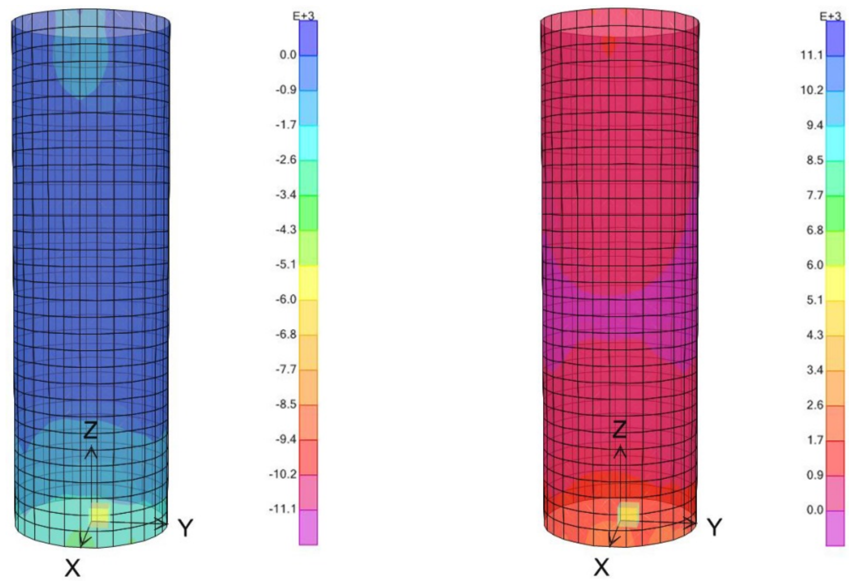


Figure 3.12. View of core column F22 minimum and maximum forces envelopes (kN/m).

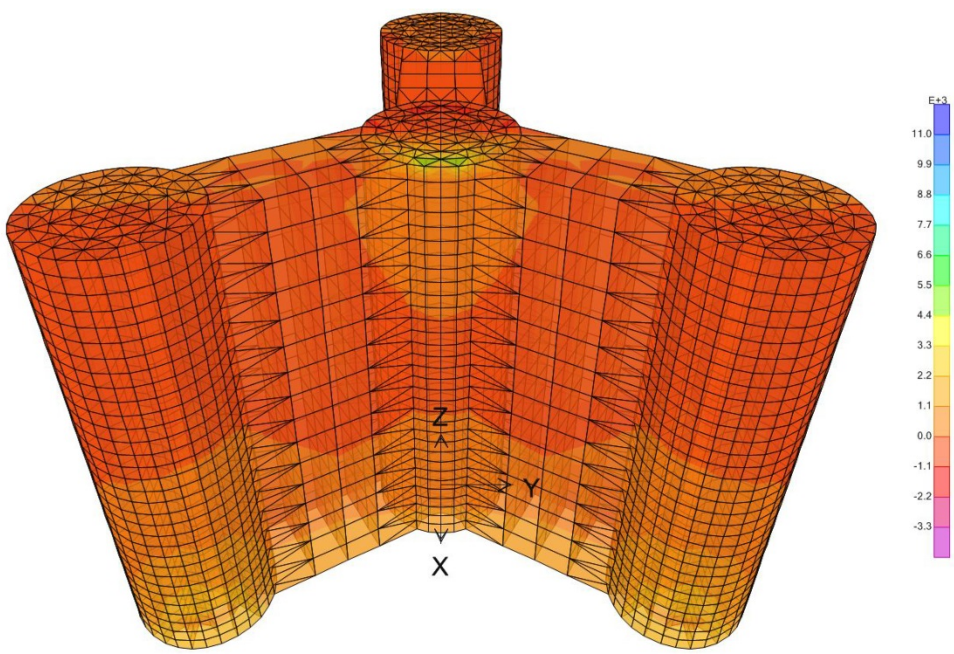


Figure 3.13. View of platform with pontoon removed; F22 forces (kN/m) due to load combination applying 100-year wave load case in global negative x-direction (11D).

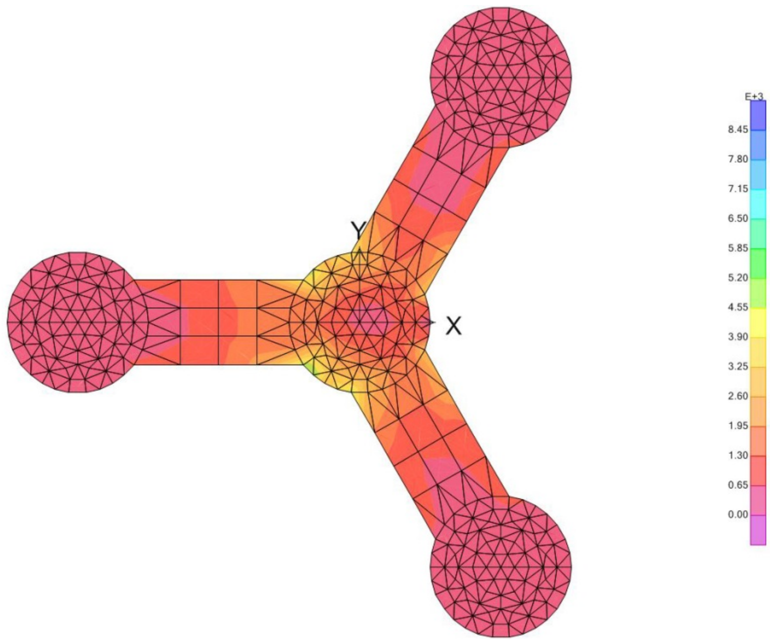


Figure 3.14. Top deck view; F11 maximum force envelope (kN/m).

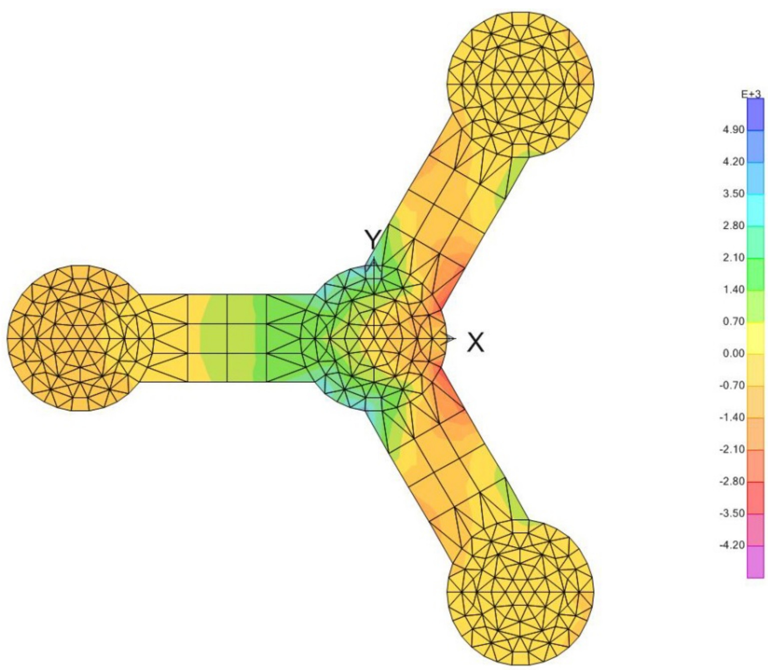


Figure 3.15. Top deck view; F11 forces (kN/m) due to load combination applying 100-year wave load case in global positive x-direction and 0.9 factor assigned to permanent loads (11C).

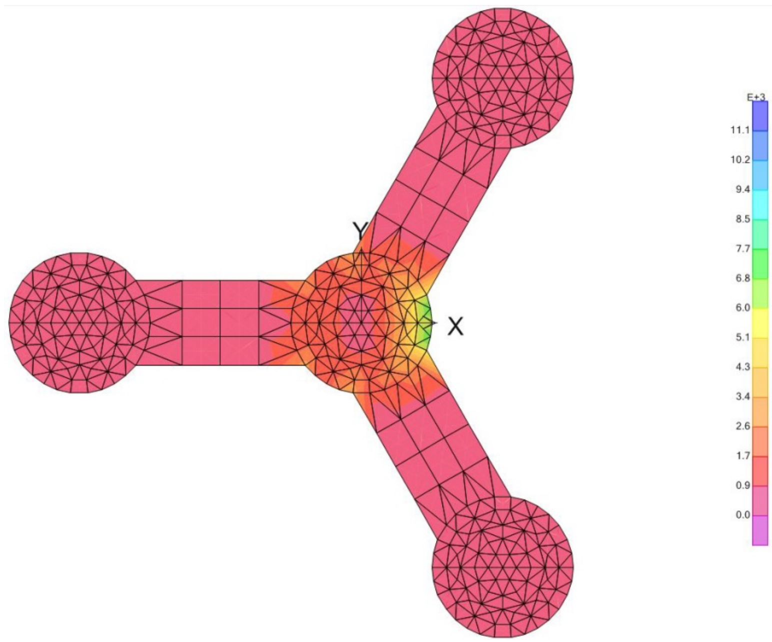


Figure 3.16. Top deck view; F22 maximum force envelope (kN/m).

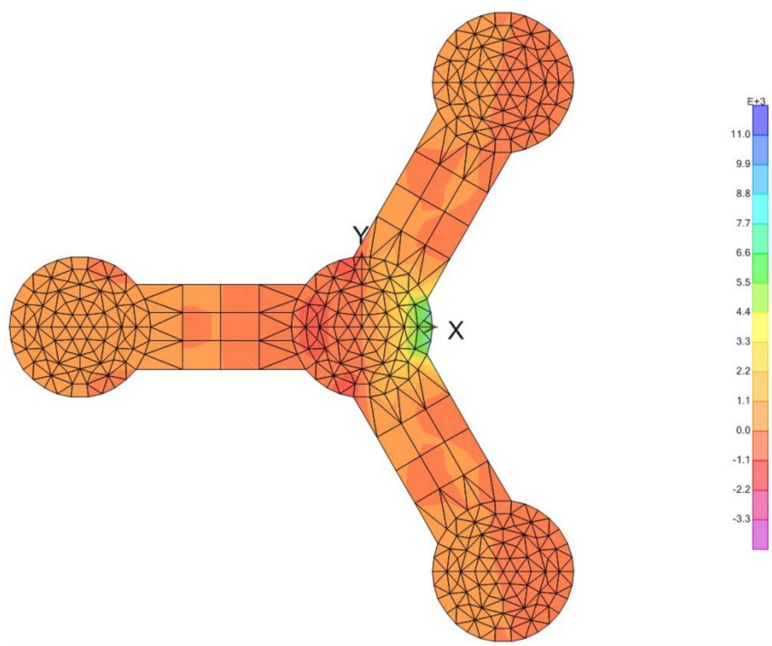


Figure 3.17. Top deck view; F22 forces (kN/m) due to load combination applying 100-year wave load case in global negative x-direction and 0.9 factor assigned to permanent loads (11D).

3.7 Limitations of Modeling and Analysis Methods

A dynamic analysis with coupled loading due to wind and wave forces on the turbine and floating support platform will be required to fully achieve accurate loading on a FOWT support structure analysis model. As stated previously, this analysis was not an option with SAP2000 finite element analysis software. The goal of this project was to use the dynamic analysis results from similar projects to realistically load the concrete FOWT platform finite element model that would result in forces that could be used to determine a preliminary design. While this was achieved for the main platform components, there remain uncertainties related to the results from some of the components due to adverse modeling effects.

The base pontoon will be submersed under the ocean surface, but due to the relatively shallow draft, hydrodynamic wave excitation forces should not be neglected. The pontoon will be constantly subjected to the complex hydrodynamic loading of radiation (linear drag) forces, diffraction (wave scattering) forces and nonlinear viscous shedding forces that will require sophisticated modeling to consider properly (Aubault et al., 2006). Also something to be investigated with hydrodynamic analysis, the maximum structural response of a platform may not be due to the largest sea state (100 year wave load case), but instead the dominant parameter for some structural modes may be due to the excitation from a specific wave period (Aubault et al., 2006).

The lifetime fatigue analysis of the structure is a major design issue not addressed by this project, but is required by all relevant code standards. A fatigue assessment requires a combined aerodynamic and hydrodynamic analysis with simulations typically performed in a time domain (Butterfield et al., 2007). The process involves choosing appropriate representative load cases ran for the proper time duration and then extrapolating the results to lifecycle load spectra

(Butterfield et al., 2007). Such analyses are complicated and only recently has research begun on the extrapolation of loading resulting from extreme environmental states (Butterfield et al., 2007). For example, how is the fatigue resultant from a 50 year storm event incorporated into a 20 year simulation of otherwise typical loading scenarios?

The maximum and minimum mooring line tension forces were included in the initial analysis model and load combinations, but were subsequently removed from consideration. The influence of the mooring system on the platform should be left to a dynamic analysis where the behavior of the platform relative to the mooring system will be properly accounted.

Chapter 4

CONCRETE FOWT PLATFORM DESIGN

4.1 Introduction

The design of a robust reinforced concrete support structure for a floating offshore wind turbine is presented here. A description of the structural design of the major platform components and the connection between the wind turbine tower and floating platform is provided. Construction recommendations are given regarding precasting protocol and post-tensioning of the structure. A concrete mix that will provide sufficient strength and durability is included. Lastly, the optimization of the platform design is discussed.

4.2 Structural Design and Construction Recommendations

The acceptance criteria adopted for this project was not typical of structures designed to protect life safety and to meet certain serviceability requirements related to the comfort of the occupants. For example, deflection was not evaluated as a parameter for design acceptance. Both lateral deflection and deflection due to gravity are irrelevant to a floating structure that will be pushed around by waves. The acceptance criteria of a FOWT platform design are based on parameters including stability and fatigue resistance which are evaluated with dynamic analyses, as previously discussed. The prevention of concrete cracking in order to avoid reinforcement corrosion was a major consideration for this project. The time of crack initiation in concrete cover can be considered as the most important criteria for the service life evaluation and prediction of reinforced concrete exposed to a marine environment (Nabavi, Nejadi, & Samali, 2012). Neglecting the contribution of prestressing, concrete cracking will not occur until the stresses within a section exceed the tensile stress of the concrete. Therefore, it was a goal for the tensile forces experienced by the analysis model to remain less than the tensile strength of concrete through the endurance of all design load cases.

The typical reinforcement for each structural member was determined using strength design (LRFD) with the maximum resultant forces found in the element. The structure was designed to resist the most critical effects from the various combinations of factored loads prescribed by ABS (2013). Structural members were proportioned such that the computed forces given by analysis do not exceed the forces at which section cracking is initiated. Regardless of whether a section was expected to crack, reinforcement shall be provided. Resultant shell element forces from the SAP2000 analysis were used to calculate typical reinforcement for 1-m design strips. Reinforcement to resist tension forces, bending moments, and shear forces were calculated in accordance to ACI 318-11 (2011). Local tension forces were the predominant factor governing reinforcement requirements. Local bending was considered along with tension forces for the evaluation of the required reinforcement, but did not ever govern design requirements. By observation, all sections were adequate to resist compressive forces. The design strength for each section was based on the nominal strength and strength reduction factors as defined by ACI 318-11. Please see Appendix D for reinforcement determination calculations. Please see Table 4.1 below summarizing typical reinforcement and spacing. The concrete compressive strength shall be specified to be no less than 60 MPa (8700 psi). The rebar shall be specified to have a yield strength of 420 MPa (60 ksi). All design calculations were performed assuming normal-weight concrete.

Table 4.1

Summary of Typical Reinforcement

Component	Section	Thickness	Reinforcement and Spacing
Core Column	Outer Wall	457 mm (18 in)	Horizontal: Provide No. 29 bars spaced at 152 mm (6 in) on center at each face for the top 4 m of the column, then increase spacing to 203 mm (8 in) throughout the remainder of the column height. Vertical: Provide No. 29 bars spaced at 152 mm (6 in) on center at each face throughout entire column height. Shear: Provide No. 16 shear links spaced at 152 mm (6 in) each way for the top 4 m of the column.
	Honeycomb	305 mm (12 in)	Horizontal: Provide No. 25 bars spaced at 203 mm (8 in) on center at each face within all core column honeycomb components. Vertical: Provide No. 25 bars spaced at 203 mm (8 in) on center at each face within all core column honeycomb components.
	Deck	1067 mm (42 in)	Provide No. 32 bars spaced at 102 mm (4 in) on center at each face in two directions. Shear: Provide No. 19 shear links spaced at 102 mm (4 in) each way.
Floater Columns	Outer Wall	406 mm (16 in)	Horizontal: Provide No. 25 bars spaced at 203 mm (8 in) on center at each face throughout entire column height. Vertical: Provide No. 32 bars spaced at 152 mm (6 in) on center at each face for bottom 8 m of column; provide No. 25 bars spaced at 203 mm (8 in) on center at each face starting from 8 m up the column for the remainder of the column height.
	Honeycomb	254 mm (10 in)	Provide No. 25 bars spaced at 203 mm (8 in) on center at each face in both directions within all floater column honeycomb components.

Table 4.1 (continued)

Component	Section	Thickness	Reinforcement and Spacing
	Deck	305 mm (12 in)	Provide No. 25 bars spaced at 203 mm (8 in) on center at each face in both directions.
Floater Braces	Outer Wall	406 mm (16 in)	Horizontal: Provide No. 25 bars spaced at 152 mm (6 in) on center at each face for the top 4 m of the brace wall, then increase spacing to 203 mm (8 in) throughout the remainder of the wall height. Vertical: Provide No. 32 bars spaced at 152 mm (6 in) on center at each face for bottom 8 m of column. Provide No. 25 bars spaced at 203 mm (8 in) on center at each face starting from 8 m up the column for the remainder of the column height.
	Brace Wall	254 mm (10 in)	Provide No. 19 bars spaced at 203 mm (8 in) on center at each face in both directions.
	Deck	457 mm (18 in)	Running longitudinal between columns: Provide No. 29 bars spaced at 152 mm (6 in) at each face for 1067 mm (42 in) along edges, then increase spacing to 203 mm (8 in) on center. Running transverse between columns: Provide No. 29 bars spaced at 203 mm (8 in) on center at each face.
Pontoon	Pontoon Slab	457 mm (18 in)	Provide No. 32 bars spaced at 203 mm (8 in) on center at each face in three directions.

Note: Transverse shear reinforcement is not required within a section unless indicated.

Despite post-tensioning being recommended for this project, the structure was evaluated according to ACI 318 guidelines considering no prestressing, which resulted in a more conservative design. The post-tensioning was treated purely as a means to close cracks if they are formed. In addition to increasing the tension capacity of a member, compressive stresses resulting from post-tensioning increase shear force resistance. This increased shear capacity was not considered in the design of the structure. All structural members were considered as solid slabs even though components shall be precast in sections. Therefore, minimum shear reinforcement was not required as per ACI 318-11 11.4.6.1 specifications. These provisions exclude solid slabs from the minimum shear reinforcement requirements because there is a possibility of load sharing between weak and strong areas (ACI 318-11 R11.4.6.1). The column honeycombs will not only provide a brace against shear forces, but also act to redistribute loads from weak to strong areas based on their relative stiffness. Compressive forces not due to prestressing were also excluded from consideration when evaluating shear reinforcement requirements. However, tensile forces that reduce shear resistance were considered. Transverse shear reinforcement was not required except for the top of the exterior walls and deck of the core column. Calculations giving the shear capacity of each section depth are included in Appendix C.

It is suggested that construction be performed in 4 m vertical segments. Sector and brace wall components of the core and floater columns shall be precast as shown below in Figure 4.1. The horizontal joints between the precast pieces shall be cast-in-place, tying all components together monolithically. Vertical longitudinal rebar shall be included in the joints as specified by the required spacing. Type 2 mechanical connectors or adequate tension lap splices must be provided in the joints to tie together the precast elements as according to ACI 318. The vertical segments shall be connected with mechanical connectors cast into the precast components with grouted joints between the segments. The brace float walls (exterior and interior) and pontoon slabs shall be cast-in-place. The decks shall be comprised of a precast plank and cast-in-place topping. The topping shall tie all rebar together from the precast components and joints at the top of the structure. The topping shall be the full specified thickness of the column decks with both the top and bottom layers of rebar as well as the required shear reinforcement located in the topping. The plank components shall have minimum thickness and reinforcement required to support the weight from the wet concrete topping. Please see Figure 4.2 below showing a section view of the core column and Figure 4.3 showing a plan view of the core column with a rebar layout typical of all columns.

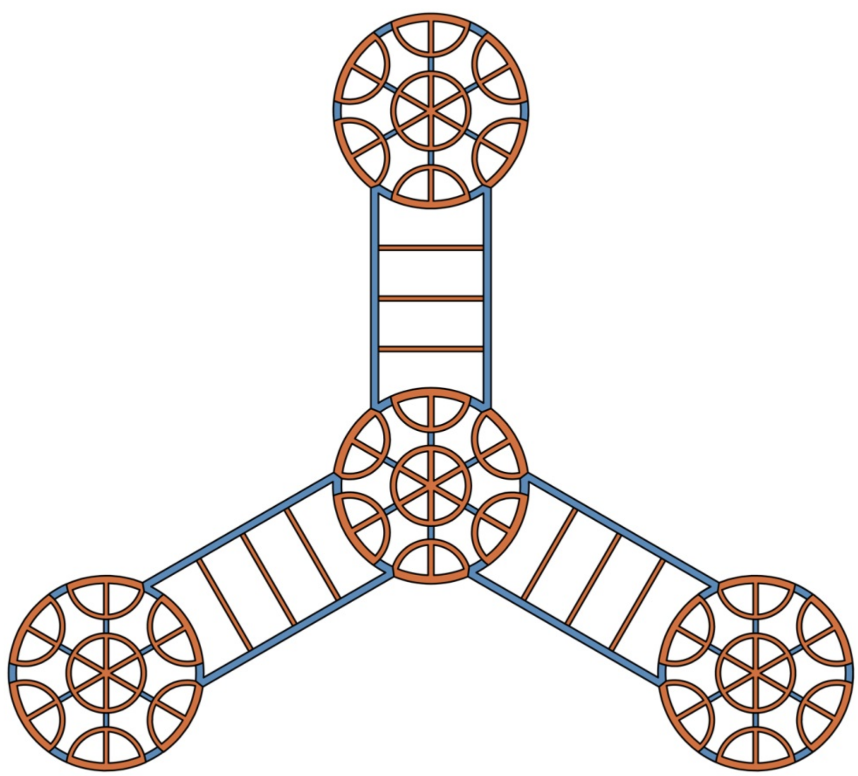


Figure 4.1. Precasting plan for concrete FOWT platform; components shaded in orange shall be precast and components shaded in blue shall be cast-in-place.

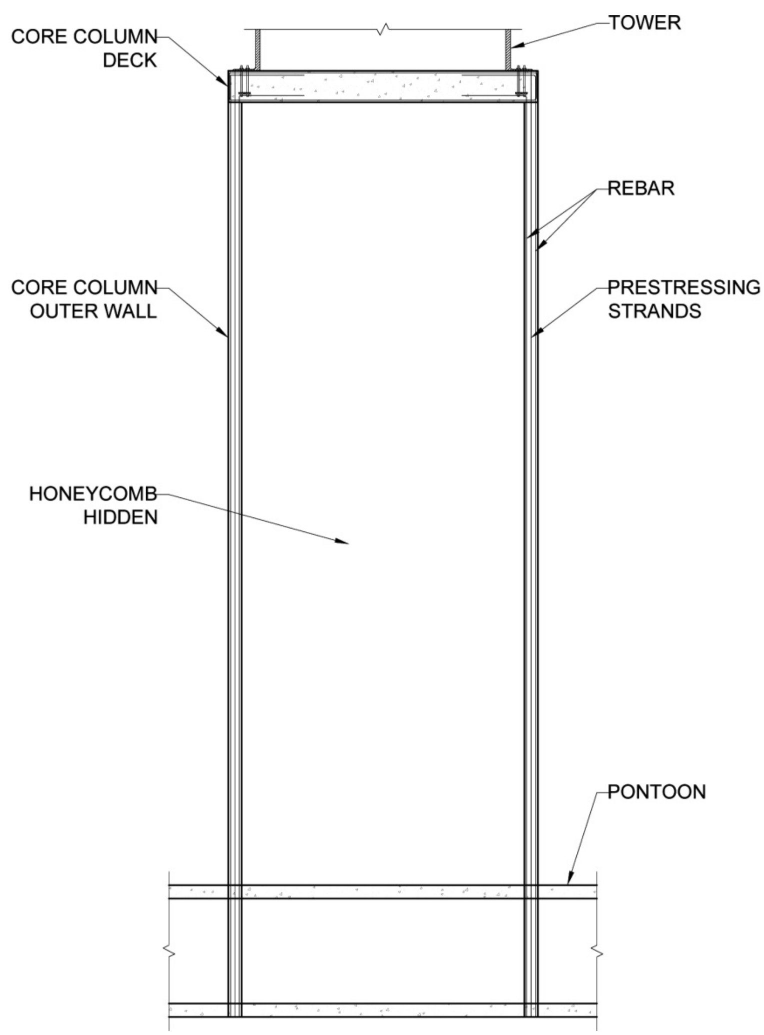


Figure 4.2. Section view of core column showing outer wall and deck rebar.

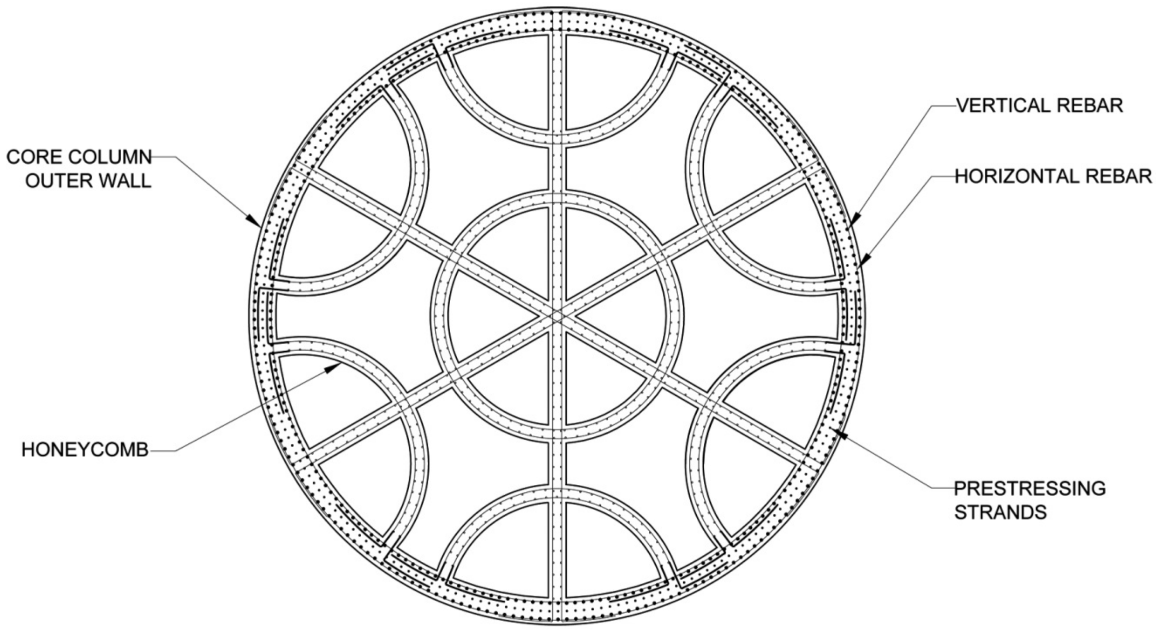


Figure 4.3. Plan view of core column showing rebar layout typical of all columns.

The design of the reinforcement required in the pontoon slabs was not performed for this project. The pinned base fixity in the analysis model limits the forces imposed on the pontoon due to environmental loading while exaggerating other forces due to the restrained base. The analysis model does not represent the conditions the pontoon must resist when being towed to the wind farm site. The forces resulting from the current analysis were therefore limited in their reflection of the realistic behavior of the pontoon. The complex hydrodynamic loading on the pontoon will need to be evaluated in order to properly capture appropriate design forces. Recommendations for the pontoon reinforcement and post-tensioning are provided in Table 4.1 and Table 4.2, respectively, but must be reevaluated with further analysis.

4.3 Post-Tensioning Design

Prestressing the concrete by means of bonded post-tensioning was recommended by Dr. Alfred A. Yee based on his experience designing floating concrete structures for over forty years (Dr. A Yee, personal communication, March 12, 2014). The prestressing shall be provided to prevent rebar corrosion by closing concrete cracks, not to resist any design forces experienced by a FOWT. Dr. Yee suggests that a prestressing of 200 psi (1.4 MPa) is applied longitudinally to all sections. Although 200 psi is not a large stress, Dr. Yee has found it sufficient to resist the formation of cracks for numerous marine projects. The tendons shall be spaced in the center of each section so that no eccentric load will be generated by the prestressing. Grouting the tendon ducts following the tensioning will be essential to create a bond between the tendons and the concrete superstructure and to protect the tendons from corrosion. The post-tensioning of the columns shall span from the top of the deck to the base of the pontoon slab. Similarly, the post-tensioning of the brace floaters shall be applied from the top deck to the base of the pontoon slab. Please see Table 4.2 below for a summary of the post-tensioning design.

Table 4.2

Summary of Post-Tensioning Design

Component	Thickness	Reinforcement and Spacing
Core Column Outer Wall	457 mm (18 in)	Provide 0.5" Ø 7 wire prestressing strands spaced at 6" around circumference of core column outer wall at section
Floater Columns Outer Wall	406 mm (16 in)	Provide 0.5" Ø 7 wire prestressing strands spaced at 6" around circumference of floater column outer walls at section
Floater Braces Outer Wall	406 mm (16 in)	Provide 0.5" Ø 7 wire prestressing strands spaced at 6" along floater brace outer walls at section centerline.
Pontoon	457 mm (18 in)	In top and bottom pontoon slabs provide 0.5" Ø 7 wire prestressing strands spaced at 6" in three directions placed symmetrically about section centerlines. Provide same strands and spacing oriented vertically along perimeter of pontoon at section centerline.

4.4 Design of Connection at Base of Tower

A bolted connection between the base of the tower and the core column deck was designed per ACI 318-11 Appendix D criteria. The bolted connection shall wrap around the circumference of the tower base attaching the steel tower base plate to the concrete deck. Groups of 2 in (50.8 mm) diameter ASTM A449 headed anchor bolts spaced at 12 in (305 mm) around the base of the tower were required to resist the base shear and overturning moment from the wind turbine. Each group has two bolts embedded 24 in (610 mm) into the concrete spaced 8 in (203 mm) from each other as per minimum spacing requirements set by ACI 318-11 D.8.1. The bolts shall be attached to a 1 in (25.4 mm) thick steel anchor plate that shall be also embedded in the concrete. The anchor plate may be a continuous piece of steel or plate segments welded together to form a large circle. The design calculations checked the connection for: (1) combined tension and shear force on the anchor bolts; (2) tension breakout of the anchor bolts from the concrete; (3) tension pullout of the anchor bolts from the concrete; (4) concrete breakout of the anchor bolts subjected to shear forces; (5) shear pryout of the anchor bolts subjected to shear forces; and (6) the bearing strength of the anchor plate. Tension failure mechanisms governed over shear failure mechanisms as the tension forces due to the overturning moment were far greater than the tower base shear. The governing load condition was tension breakout of the bolts from the concrete. Please see Appendix F for design calculations. Figures 4.4, 4.5, and 4.6 below show the layout of the bolted connection, dimensions of the connection, and a cross-section of the connection, respectively.

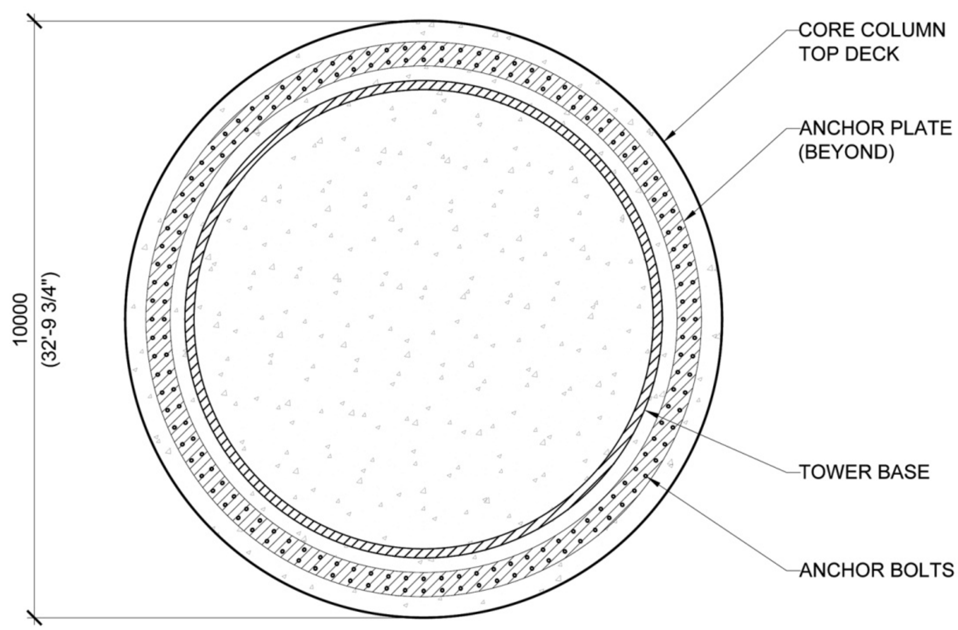


Figure 4.4. Overview of core column top deck showing bolted connection to tower.

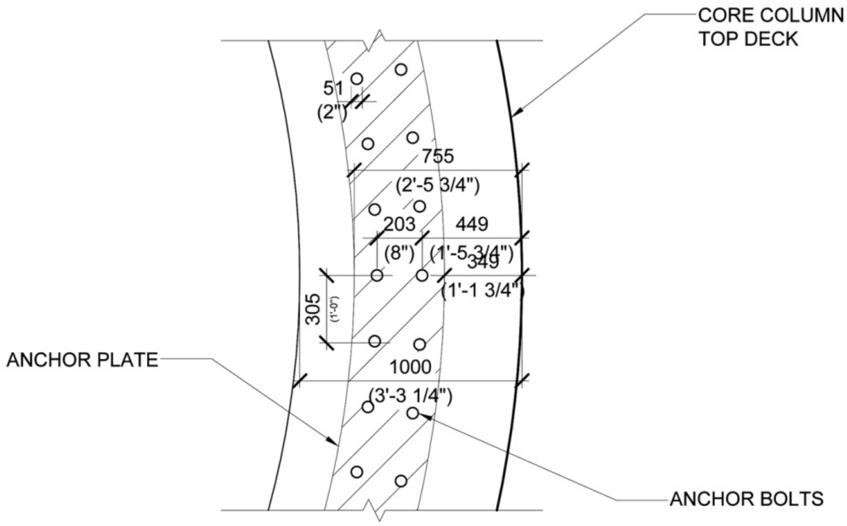


Figure 4.5. Tower to core column deck bolted connection dimensions.

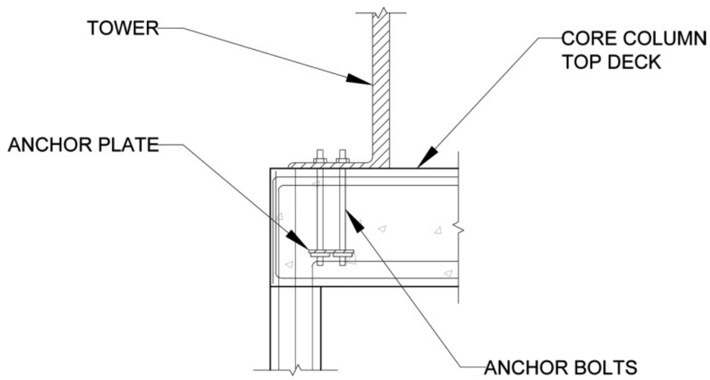


Figure 4.6. Section view of core column deck at bolted connection to tower.

4.5 Concrete Mix Design

The concrete mix design provided in Table 4.3 is recommended to achieve high-strength, light-weight concrete with a compressive strength of at least 60 MPa (8700 psi) and a target density of 2000 kg per cubic meter (125 pcf). The presented mix design was developed based on recommendations by concrete expert Dr. Sabet Bahador to attain a durable concrete with low permeability and a long service life (Dr. S. Bahador, personal communication, January 30, 2014).

Table 4.3

High-Strength, Light-Weight Concrete Mix

Material	Quantity per Cubic Meter
Total cementitious material	550 kg
Water	154 kg
Coarse aggregate	648 kg
Fine aggregate	648 kg
Cement − type I	330 kg
Undensified silica fume	55 kg
Granulated blast furnace slag	165 kg
Superplasticizer	5-9 liter
Target compressive strength =	60 MPa
Target density =	2000 kg/m^3

The final mix design shall be based on the availability and prices of materials. Anything between 550 to 580 kg per cubic meter of total cementitious material shall be sufficient. A water-cementitious material ratio of 0.28 is given here with 0.3 or less required. The fine aggregate shall be natural sand with the ratio of coarse to fine aggregate based on the density of the light-weight aggregate. The total aggregate content shall be determined by the limiting density of 2000 kg per cubic meter. Mineral admixtures silica fume and granulated blast furnace slag are recommended to enhance the concrete durability. The slump of the wet concrete shall be maximized based on the price of available superplasticizers.

4.6 Design Optimization

A sufficiently conservative design has been presented here. It was estimated that the platform may be overdesigned by up to 20 percent. Structural optimizations that will reduce the overall weight of the structure should be performed. Many areas of the platform may be reduced in depth, such as the center portions of the columns, where all resultant forces were found to be minimal. The reduction in the depth of certain column sections is readily accommodated with precasting. The center sections of the columns, perhaps from 12 m to 24 m, may be tapered into thinner sections, potentially resulting in significant material savings. While an overdesign makes the platforms seem less economical, it is preferred that a more robust structure is evaluated with the dynamic coupled analysis to ensure the design does not fail. Based on the results from the coupled analysis, sections can then be reduced accordingly, producing a more efficient system.

If the proposed concrete FOWT platform design is found to have inadequate dynamic performance, there are adjustments that can be made to the design that may lead to a more preferred behavior. The initial design was intended to maximize the economy of the platform by minimizing the dimensions and, therefore, material requirements. A simple way to potentially improve the dynamic characteristics of the structure is to increase the horizontal spread of the pontoon base resulting in a larger water plane area. Increasing the size of the pontoon, i.e. increasing the center to center dimension between the columns, will require the pontoon slabs to be made thicker and more braces added internally to keep the pontoon rigid. Another action that may improve the dynamic performance of the platform would be to increase the height of the columns, creating a deeper overall draft. This action would lower the platform center of gravity, providing greater stability for the structure. Both of the suggested revisions would impact the size and weight of the structure, perhaps decreasing the economic viability of the design concept.

If more concrete is added to the design, the structure will be heavier resulting in a reduced freeboard. The draft of the structure may be adjusted by simply increasing the height of the pontoon. Even if the dynamic performance and stability of the structure are found to be adequate, further work should be carried out to optimize the structural dimensions.

Chapter 5

FINDINGS AND INTERPRETATIONS

5.1 Introduction

This chapter presents a discussion of the findings and interpretations of this project. The feasibility of reinforced concrete FOWT support structures for use in offshore wind farms are evaluated based on economic parameters. Materials that should be considered in future evaluation of concrete FOWT platforms are discussed. Suggestions for analysis methods in future work are given. Finally, conclusions are made based on the objectives of the project.

5.2 Feasibility and Recommendations for Future Action

There are many limiting factors related to the construction of concrete FOWT platforms. The size and weight of the structure may be a disincentive for the adoption of concrete FOWT platforms, as sufficient dry dock facilities and giant cranes will be required for construction. A reliable source for large quantities of concrete that meets the specifications may be a restrictive factor as light-weight course aggregate supply that is adequate for use in high-strength concrete may be difficult to obtain. This may lead to the initial conclusion that a concrete floating support structure is uneconomical, but these assumptions require further investigation.

The cost of constructing a specialized marine vessel as proposed in this work is difficult to determine. Based on his experience, Dr. Yee estimates the cost of construction and installation of a floating concrete marine structure at $2500 per cubic metric of concrete (Dr. A. Yee, personal communication, April 22, 2014). His estimate includes all materials such as concrete, reinforcing steel, prestressing strands, mechanical connectors, and formwork for both precast and cast-in-place components, as well as the costs of labor, dry dock rental, launching costs, etc. The concrete FOWT platform designed for this project uses 6750 cubic meters of concrete, so based on Dr. Yee's cost estimate, the concrete platform would cost $16,868,860 (plus 20% margin equals $20,242,630). University of Hawaii Ocean Engineering graduate students estimated that the cost for one WindFloat platform to be constructed and installed offshore of the island of Oahu, Hawaii, would be $22,036,950 (Barnes, Casilio, Frederick, Nolte, & Schwartz, 2012). The price estimate includes the cost of the platform, mooring and installation costs, and dry dock fees. Based on this estimate comparison, there may be real potential for cost savings if a concrete platform is selected over a steel platform. There is also the potential for savings over the service life of the FOWTs due to lower down time and a reduction in maintenance requirements for concrete versus steel platforms.

This project addressed the feasibility of using traditional steel reinforced concrete as the building material for a floating platform to support a wind turbine, but alternative materials should also be explored. Research has found that chloride-induced corrosion was the primary cause of premature deterioration of reinforced concrete structures exposed to a marine environment (Nabavi et al., 2012). Chloride diffusion into concrete occurs due to concrete permeability and surface cracks from various sources of loading (Nabavi et al., 2012). Post-tensioning was suggested for this project as an added measure to increase the ability of the structure to resist concrete cracking, but there are other options to prevent cracking that should be investigated. Polymer-concrete composites such as polymer fiber reinforced concrete and polymer modified concrete should be explored as material options for concrete with and without steel reinforcement. Polymer-concrete composites have been found to provide higher durability and longer service life than conventional concrete, and corrosion will never be an issue if steel reinforcement is avoided altogether (Nabavi et al., 2012). Crystalline catalyst additives for concrete continuously exposed to water should also be considered for use in concrete FOWT platforms. Until recently, there have been questions on the impact that these crystalline additives will have on the performance of high-strength concrete. Studies are currently being conducted on the effect of these additives on the mechanical properties of concrete and recent research has shown that crystalline catalyst additives were effective in improving the durability of high performance concrete stressed by continuous mechanical loading (Takagi, Lima, & Helene). Steel fiber reinforcement that will increase the tensile strength of concrete should also be considered for use in concrete FOWT platforms, although much care must be given to ensure there is proper concrete cover over the steel fibers. Increasing the tensile strength and ductility of the concrete may potentially result in reduced platform section depths.

Dynamic analyses of this preliminary design should be pursued in future work. SAP2000 is limited in its capability to account for coupled wind and wave loading due to the necessity of imposing boundary conditions that do not properly account for the behavior of a floating structure. Attempts to approximate floating boundary conditions resulted in unrealistic forces in the structure. Global performance analyses must be performed with site-specific data on a truly representative model of a concrete FOWT platform to provide accurate loading for a finite element analysis model. The dynamic behavior of a concrete FOWT platform must be accounted for in a proper analysis. The stability of the floating platform is a primary concern as the overall dimensions are less than those of other platform concepts.

5.3 Conclusion

This thesis introduced a robust and durable reinforced concrete support structure design for a floating offshore wind turbine based on the results from a three-dimensional equivalent static analysis of a finite element model. The resultant design was determined to be potentially economically viable, although more work must be done to determine with certainty that reinforced concrete FOWT platforms are a practical alternative to steel platforms. The simplified method of analysis implemented in this project, which involved using the results from the global performance analysis of a case study by ABS to infer the equivalent static loading applied to the finite element model, was suitable for determining a preliminary design. The proposed design must be evaluated with a coupled aerodynamic and hydrodynamic analysis to assess the stability of the structural, loading on the pontoon base, and the interaction between the mooring system and the floating platform. It is also necessary to perform a fatigue assessment to ensure that the platform meets serviceability criteria established by adopted design guidelines.

The floating reinforced concrete platform concept proposed in this thesis has merit due to its economic viability and robust behavior when subjected to extreme environmental loads. While the pontoon base has yet to be verified in its performance capabilities, the remainder of the structure was found to function as intended.

Appendix A. Background and Context for Deep Water Offshore Wind Farms

The developed world has arrived at a critical turning point in how we produce and consume energy. No longer an item of debate, it is now widely accepted that we have past the peak production of fossil fuels and must now shift towards new sources of energy; energy that is sustainable and environmentally benign. With eminent climate change, nuclear disasters, and high energy prices, a major paradigm shift has occurred in our collective culture and developing the ability to generate renewable energy is now a prevailing focus of both societal aspirations and the majority of the industrial powers that currently provide our finite supply of fossil fuels. The financial costs of these renewable energy supplies are significant, but the oil industry is in good position to fund the infrastructure necessary to produce new sustainable sources of energy. The future financial, social, and environmental benefits are immeasurable for society and the oil industry alike. Our sustainable future is at a very exciting beginning.

According to the U.S. Energy Information Administration (2014), renewable energy sources accounted for 12.8 percent of the total energy that was domestically produced in the United States in 2013. Hydroelectric power is the largest producer of renewable energy, producing 50.8 percent of the renewable energy total and 6.5 percent of total energy production in the country (U.S. Energy Information Administration, 2014). Wind and solar energy production capacity have both been experiencing huge growths recently and have many new installations in the works. The American Wind Energy Association (2014) reported that 1,084 MW of wind capacity was installed in 2013 adding to the 61,108 MW total wind power installation capacity in the United States, enough to power over 15.3 million homes. There are currently over 12,000 MW in wind energy projects under construction (American Wind Energy Association, 2014). Solar energy accounted for 29 percent of new electricity generation capacity added in 2013 with 4,751 MW of new photovoltaic capacity installed across the country. This contributes to the 13,000 MW of cumulative solar electric capacity operating in the United States, enough to power more than 2.2 million homes (Solar Energy Industries Association, 2014). There are also many new projects that allow the capacity to store the power produced by these sustainable energy sources, enabling energy to be released to the grid as needed during peak consumption hours. This will decrease the need for peak power plants that burn coal, polluting our air and adding carbon to the atmosphere. It will take time to figure out the optimal solutions to our problems, but there are many great minds and significant funds involved with cultivating the integration of renewable energy sources into our established systems.

Solar energy systems have been widely accepted due to the accessibility of the infrastructure for individual homeowners and businesses. Almost anyone can put solar panels on their roof due to a plethora of financing options. Major utility projects are constructed in the middle of the desert, away from the eyes of anyone who might complain of their intrusive unsightliness. However, wind power projects are a different issue. The phrase "not in my backyard" has become synonymous with wind projects, whose massive towers can be seen for miles surrounding the wind farms, and whose enormous blades have been accused of causing harm to birds and other wildlife. While so far, all of the wind farm projects in the United States have been on land, Cape Wind, the first offshore wind project in Massachusetts, has taken ten years to transverse the permitting process and litigation hurdles at a huge expense (Milford, 2014). Opponents of the project protest the same complaints incurred by onshore wind projects, but also add concerns as to the effects on marine life, fishing practices and ship navigation.

Europe is leading the world in wind power and renewable energy production in general. In 2013, 49.1 percent of Spain's power capacity was provided by renewable sources with wind power accounting for 21.1 percent of the total supply (PennEnergy Editorial Staff, 2013). The United Kingdom, Germany, and Norway each have robust wind power programs that grow every year. These wind farms are located both on and offshore. Norway is fortunate to have an abundance of shallow waters where bottom-mounted wind turbine towers are easily constructed. The problem for the U.K. and for many other countries around the world, including much of the coastline on the west coast of the United States, is that the waters offshore are deep, and construction of bottom-mounted wind turbine platforms is not practical.

Floating offshore wind farms offer many solutions to the issues associated with wind projects. Floating offshore wind turbine (FOWT) platforms may be stationed at water depths starting at around 30 meters up to 1000 meters (Sclavounos, 2009). This enables wind farms to be located far enough from the coastline that they are not seen from the shore, immediately removing the "not in my back yard" dilemma. Offshore wind resources in the open ocean are more consistent and stronger than those found close to shore. Further from shore there are less seabirds and other flying wildlife to contend with and, as shown with offshore oil platforms, manmade objects in marine environments can serve as artificial reefs, providing habitat and protection for marine life. Opponents to floating offshore wind farms are concerned that they will be navigational hazards and affect the migrations of marine species, but proponents say that these issues will be minimal. A large cost associated with floating offshore wind farms is constructing the undersea cables that transmit the generated power from the wind farms to the onshore power grid, a process which can also be disruptive to marine life.

Offshore of the island of Oahu is the target site for the design of a wind farm with floating concrete platforms. The state of Hawaii has a unique set of issues regarding its energy supply. Located approximately 2500 miles from the nearest continent, the state currently imports fossil fuels to meet the vast majority of its energy demand. The state has set goals to produce 40 percent of its electrical energy from renewable resources by 2030 and reduce total electricity consumption by 30 percent (Hawaii State Energy Office, 2013). In 2012, distributed renewable energy system installations more than doubled, contributing to a total capacity of 138 MW that provided 13 percent of the state's energy needs (Hawaii State Energy Office, 2013). However, the primary energy source for the state remains fossil fuels, relying on burning oil for 74 percent of its electricity while the rest of the nation uses oil for less than one percent of electrify generation (Hawaii State Energy Office, 2013). On the island of Oahu, eight percent of energy needs are met by burning garbage, including plastics, while a small percentage of the energy is produced by onshore wind turbines and solar panels placed on the roofs of domestic housing and businesses (Prevedouros, 2013). On the Big Island, approximately two percent of the state's power is provided by thermal vents due to the active volcano that is still forming the island (Hawaii State Energy Office, 2013). Residents of Hawaii are subjected to the highest energy costs in the country, over three times higher than the national average (Hawaii State Energy Office, 2013). Indeed, the costs of shipping fuel supplies to the islands are the burden of the consumer.

Hawaii is abundant with natural resources that could be used to produce renewable energy. If the required infrastructure was constructed, the sun, wind, thermal, and tidal energies could easily provide all of the energy consumed by the 1.3 million residents of the state, the majority of whom live on the island of Oahu. The Hawaii State Legislature recently approved the financing structure for constructing an undersea cable linking the electricity grid systems of the islands.

This is a promising first step towards enabling the renewable energy generated on each island to be transported where it is needed. This move may also set a precedent for the construction of the transmission cable necessary to transport electricity from an offshore wind farm to the shore.

The same protests against wind farms are present in Hawaii as in the rest of the country. In addition to residents complaining that wind turbines pollute their scenic views, they also fear that wind farms will adversely affect tourism by making the islands less aesthetically appealing. However, building wind farms far from the shore in the deep waters surrounding the islands removes these issues. Potential wind farm sites have already been located in the waters around Oahu by ocean engineering graduate students from the University of Hawaii at Manoa. Floating offshore wind turbine platforms offer an opportunity to harness the abundant offshore wind resources of Hawaii to provide renewable energy for the state. The isolation of Hawaii, located in the center of the Pacific Ocean, and lack of construction resources requires that the materials to build an offshore wind farm be shipped from the mainland. To avoid the complications of shipping enormous FOWT platforms across the ocean, it would be necessary to construct the wind turbine platforms in the ship yards of Honolulu, and then tow the platforms into place at the wind farm site.

Although the cost of structural steel has gone down recently, there will be additional costs associated with the production of steel FOWT support structures in regards to an offshore wind farm for the island of Oahu. The steel platform components would be manufactured somewhere in mainland North America or China and then shipped thousands of miles to Hawaii. The platforms would be assembled once the components reached the island, requiring construction teams only consisting of highly skilled welders and crane operators. By contrast, every aspect of concrete FOWT platform construction could be performed on Oahu, bringing many new jobs of all trades to the island. Raw materials for the concrete that are not sourced locally, as well as the rebar, would need to be shipped to the island, but this would cost less than shipping the steel platform components from overseas. Concrete formwork construction, concrete mixing, pouring, curing, and assembly could all be performed at a shipyard with local contractors. This is an opportunity to stimulate the economy and to provide clean, renewable energy, potentially for many generations to come.

Appendix B. Load Combination Definitions

Count	Combo Name	Combo Type	Case Name	Load Factor	Case Type
1	1A	Linear Add	Wave Vin	1.35	Linear Static
			BS1	1.35	Linear Static
			OTM1	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
2	2A	Linear Add	Wave Vr	1.35	Linear Static
			BS2	1.35	Linear Static
			OTM2	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
3	3A	Linear Add	Wave Vout	1.35	Linear Static
			BS3	1.35	Linear Static
			OTM3	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
4	4A	Linear Add	Wave Vin	1.35	Linear Static
			BS4	1.35	Linear Static
			OTM4	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
5	5A	Linear Add	Wave Vr	1.35	Linear Static
			BS5	1.35	Linear Static
			OTM5	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
6	6A	Linear Add	Wave Vout	1.35	Linear Static
			BS6	1.35	Linear Static
			OTM6	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
7	7A	Linear Add	Wave Vr	1.35	Linear Static
			BS7	1.35	Linear Static
			OTM7	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
8	8A	Linear Add	Wave Vout	1.35	Linear Static
			BS8	1.35	Linear Static
			OTM8	1.35	Linear Static
			DEAD	1	Linear Static

			Rotor/Nacelle/Mast	1	Linear Static
9	9A	Linear Add	Wave 10yr	1.35	Linear Static
			BS 10yr	1.35	Linear Static
			OTM 10yr	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
10	10A	Linear Add	Wave 50yr	1.35	Linear Static
			BS 50yr	1.35	Linear Static
			OTM 50yr	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
11	11A	Linear Add	Wave 100yr (11)	1.35	Linear Static
			BS 100yr	1.35	Linear Static
			OTM 100yr	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
12	12A	Linear Add	Wave 100yr (12)	1.35	Linear Static
			BS 100yr 30deg	1.35	Linear Static
			OTM 100yr 30deg	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
13	13A	Linear Add	Wave 100yr (13)	1.35	Linear Static
			BS 100yr 90deg	1.35	Linear Static
			OTM 100yr 90deg	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
14	14A	Linear Add	Wave 100yr (14)	1.35	Linear Static
			BS 100yr	1.35	Linear Static
			OTM 100yr	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
15	15A	Linear Add	Wave 100yr (15)	1.35	Linear Static
			BS 100yr	1.35	Linear Static
			OTM 100yr	1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
16	1B	Linear Add	Wave Vin	-1.35	Linear Static
			BS1	-1.35	Linear Static
			OTM1	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
17	2B	Linear Add	Wave Vr	-1.35	Linear Static
			BS2	-1.35	Linear Static
			OTM2	-1.35	Linear Static

			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
18	3B	Linear Add	Wave Vout	-1.35	Linear Static
			BS3	-1.35	Linear Static
			OTM3	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
19	4B	Linear Add	Wave Vin	-1.35	Linear Static
			BS4	-1.35	Linear Static
			OTM4	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
20	5B	Linear Add	Wave Vr	-1.35	Linear Static
			BS5	-1.35	Linear Static
			OTM5	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
21	6B	Linear Add	Wave Vout	-1.35	Linear Static
			BS6	-1.35	Linear Static
			OTM6	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
22	7B	Linear Add	Wave Vr	-1.35	Linear Static
			BS7	-1.35	Linear Static
			OTM7	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
23	8B	Linear Add	Wave Vout	-1.35	Linear Static
			BS8	-1.35	Linear Static
			OTM8	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
24	9B	Linear Add	Wave 10yr	-1.35	Linear Static
			BS 10yr	-1.35	Linear Static
			OTM 10yr	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
25	10B	Linear Add	Wave 50yr	-1.35	Linear Static
			BS 50yr	-1.35	Linear Static
			OTM 50yr	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
26	11B	Linear Add	Wave 100yr (11)	-1.35	Linear Static
			BS 100yr	-1.35	Linear Static

			OTM 100yr	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
27	12B	Linear Add	Wave 100yr (12)	-1.35	Linear Static
			BS 100yr 30deg	-1.35	Linear Static
			OTM 100yr 30deg	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
28	13B	Linear Add	Wave 100yr (13)	-1.35	Linear Static
			BS 100yr 90deg	-1.35	Linear Static
			OTM 100yr 90deg	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
29	14B	Linear Add	Wave 100yr (14)	-1.35	Linear Static
			BS 100yr	-1.35	Linear Static
			OTM 100yr	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
30	15B	Linear Add	Wave 100yr (15)	-1.35	Linear Static
			BS 100yr	-1.35	Linear Static
			OTM 100yr	-1.35	Linear Static
			DEAD	1	Linear Static
			Rotor/Nacelle/Mast	1	Linear Static
31	1C	Linear Add	Wave Vin	1.35	Linear Static
			BS1	1.35	Linear Static
			OTM1	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
32	2C	Linear Add	Wave Vr	1.35	Linear Static
			BS2	1.35	Linear Static
			OTM2	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
33	3C	Linear Add	Wave Vout	1.35	Linear Static
			BS3	1.35	Linear Static
			OTM3	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
34	4C	Linear Add	Wave Vin	1.35	Linear Static
			BS4	1.35	Linear Static
			OTM4	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
35	5C	Linear Add	Wave Vr	1.35	Linear Static

			BS5	1.35	Linear Static
			OTM5	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
36	6C	Linear Add	Wave Vout	1.35	Linear Static
			BS6	1.35	Linear Static
			OTM6	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
37	7C	Linear Add	Wave Vr	1.35	Linear Static
			BS7	1.35	Linear Static
			OTM7	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
38	8C	Linear Add	Wave Vout	1.35	Linear Static
			BS8	1.35	Linear Static
			OTM8	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
39	9C	Linear Add	Wave 10yr	1.35	Linear Static
			BS 10yr	1.35	Linear Static
			OTM 10yr	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
40	10C	Linear Add	Wave 50yr	1.35	Linear Static
			BS 50yr	1.35	Linear Static
			OTM 50yr	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
41	11C	Linear Add	Wave 100yr (11)	1.35	Linear Static
			BS 100yr	1.35	Linear Static
			OTM 100yr	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
42	12C	Linear Add	Wave 100yr (12)	1.35	Linear Static
			BS 100yr 30deg	1.35	Linear Static
			OTM 100yr 30deg	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
43	13C	Linear Add	Wave 100yr (13)	1.35	Linear Static
			BS 100yr 90deg	1.35	Linear Static
			OTM 100yr 90deg	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static

44	14C	Linear Add	Wave 100yr (14)	1.35	Linear Static
			BS 100yr	1.35	Linear Static
			OTM 100yr	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
45	15C	Linear Add	Wave 100yr (15)	1.35	Linear Static
			BS 100yr	1.35	Linear Static
			OTM 100yr	1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
46	1D	Linear Add	Wave Vin	-1.35	Linear Static
			BS1	-1.35	Linear Static
			OTM1	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
47	2D	Linear Add	Wave Vr	-1.35	Linear Static
			BS2	-1.35	Linear Static
			OTM2	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
48	3D	Linear Add	Wave Vout	-1.35	Linear Static
			BS3	-1.35	Linear Static
			OTM3	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
49	4D	Linear Add	Wave Vin	-1.35	Linear Static
			BS4	-1.35	Linear Static
			OTM4	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
50	5D	Linear Add	Wave Vr	-1.35	Linear Static
			BS5	-1.35	Linear Static
			OTM5	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
51	6D	Linear Add	Wave Vout	-1.35	Linear Static
			BS6	-1.35	Linear Static
			OTM6	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
52	7D	Linear Add	Wave Vr	-1.35	Linear Static
			BS7	-1.35	Linear Static
			OTM7	-1.35	Linear Static
			DEAD	0.9	Linear Static

			Rotor/Nacelle/Mast	0.9	Linear Static
53	8D	Linear Add	Wave Vout	-1.35	Linear Static
			BS8	-1.35	Linear Static
			OTM8	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
54	9D	Linear Add	Wave 10yr	-1.35	Linear Static
			BS 10yr	-1.35	Linear Static
			OTM 10yr	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
55	10D	Linear Add	Wave 50yr	-1.35	Linear Static
			BS 50yr	-1.35	Linear Static
			OTM 50yr	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
56	11D	Linear Add	Wave 100yr (11)	-1.35	Linear Static
			BS 100yr	-1.35	Linear Static
			OTM 100yr	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
57	12D	Linear Add	Wave 100yr (12)	-1.35	Linear Static
			BS 100yr 30deg	-1.35	Linear Static
			OTM 100yr 30deg	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
58	13D	Linear Add	Wave 100yr (13)	-1.35	Linear Static
			BS 100yr 90deg	-1.35	Linear Static
			OTM 100yr 90deg	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
59	14D	Linear Add	Wave 100yr (14)	-1.35	Linear Static
			BS 100yr	-1.35	Linear Static
			OTM 100yr	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static
60	15D	Linear Add	Wave 100yr (15)	-1.35	Linear Static
			BS 100yr	-1.35	Linear Static
			OTM 100yr	-1.35	Linear Static
			DEAD	0.9	Linear Static
			Rotor/Nacelle/Mast	0.9	Linear Static

Appendix C. Section Capacities

C.1 Cracking Moment and Tensile Strength of Sections

$M_{cr} = f_r I_g / y_t = f_r S_c$ (ACI 318-11 Eqn. 9-9)

$f_r = 0.62\sqrt{f'_c} = (0.62)(60\ \text{MPa})^{1/2} =$ 4.802 N/mm^2 (ACI 318-11 Eqn. 9-10)

1067 mm Section:

Cracking moment:

$S_c = bh^2/6 = (1000\ \text{mm})(1066.8\ \text{mm})^2/6 =$ 189677040 mm^3

$M_{cr} = (4.802\ \text{N/mm}^2)(189677040\ \text{mm}^3) =$ 910.9 kN-m

Concrete tensile strength:

Section area = (1000 mm)(1066.8 mm) = 1066800 mm^2

$T_{cr} = (4.802\ \text{N/mm}^2)(1066800\ \text{mm}^2) =$ 5123.3 kN

457 mm Section:

Cracking moment:

$S_c = bh^2/6 = (1000\ \text{mm})(457.2\ \text{mm})^2/6 =$ 34838640 mm^3

$M_{cr} = (4.802\ \text{N/mm}^2)(34838640\ \text{mm}^3) =$ 167.3 kN-m

Concrete tensile strength:

Section area = (1000 mm)(457.2 mm) = 457200 mm^2

$T_{cr} = (4.802\ \text{N/mm}^2)(457200\ \text{mm}^2) =$ 2195.7 kN

406 mm Section:

Cracking moment:

$S_c = bh^2/6 = (1000\text{ mm})(406.4\text{ mm})^2/6 =$ 27526827 mm^3

$M_{cr} = (4.802\text{ N/mm}^2)(27526827\text{ mm}^3) =$ 132.2 kN-m

Concrete tensile strength:

Section area = (1000 mm)(406.4 mm) = 406400 mm^2

$T_{cr} = (4.802\text{ N/mm}^2)(406400\text{ mm}^2) =$ 1951.7 kN

305 mm Section:

Cracking moment:

$S_c = bh^2/6 = (1000\text{ mm})(304.8\text{ mm})^2/6 =$ 15483840 mm^3

$M_{cr} = (4.802\text{ N/mm}^2)(15483840\text{ mm}^3) =$ 74.36 kN-m

Concrete tensile strength:

Section area = (1000 mm)(304.8 mm) = 304800 mm^2

$T_{cr} = (4.802\text{ N/mm}^2)(304800\text{ mm}^2) =$ 1463.8 kN

254 mm Section:

Cracking moment:

$S_c = bh^2/6 = (1000\text{ mm})(254\text{ mm})^2/6 =$ 10752667 mm^3

$M_{cr} = (4.802\text{ N/mm}^2)(10752667\text{ mm}^3) =$ 51.64 kN-m

Concrete tensile strength:

Section area = (1000 mm)(254 mm) = 254000 mm^2

T_{cr} = (4.802 N/mm^2)(254000 mm^2) = 1219.8 kN

C.2 Section Shear Capacities

$V_c = 0.17\sqrt{f'_c} b_w d$ (ACI 318-11 Eqn. 11-3)

$\Phi =$ 0.75 (ACI 318-11 9.3.2.3)

1067 mm Section:

$b_w =$ 1000 mm $d =$ 972.7 mm

$V_c = (0.17)\sqrt{(60\text{ MPa})}(1000\text{ mm})(972.7\text{ mm}) =$ 1280.9 kN

$\Phi V_c =$ 960.6 kN

457 mm Section:

$b_w =$ 1000 mm $d =$ 366.7 mm

$V_c = (0.17)\sqrt{(60\text{ MPa})}(1000\text{ mm})(366.7\text{ mm}) =$ 482.9 kN

$\Phi V_c =$ 362.2 kN

406 mm Section:

$b_w =$ 1000 mm $d =$ 314.1 mm

$V_c = (0.17)\sqrt{(60\text{ MPa})}(1000\text{ mm})(314.4\text{ mm}) =$ 413.6 kN

$\Phi V_c =$ 310.2 kN

305 mm Section:

$b_w =$ 1000 mm $d =$ 214.3 mm

$V_c = (0.17)\sqrt{(60\text{ MPa})}(1000\text{ mm})(314.4\text{ mm}) =$ 282.2 kN

$\Phi V_c =$ 211.6 kN

254 mm Section:

$b_w =$ 1000 mm $d =$ 165.1 mm

$V_c = (0.17)\sqrt{(60\ \text{MPa})}(1000\ \text{mm})(314.4\ \text{mm}) =$ 217.4 kN

$\Phi V_c =$ 163.1 kN

Appendix D. Reinforcement Determination Calculations

D.1 Core Column Outer Shell Reinforcement

Reinforcement Required to Resist Horizontal Forces

Maximum horizontal tension force found at top of core column due to load combination 11D. Resultant shell element forces from SAP2000 analysis for 1-m design strip. Maximum force found concentrated within depth of 1067 mm deck, so evaluating forces for adjacent 1 m strip.

Required axial reinforcement to resist tension for 1 m column strip:

$T_u = F_{11} =$ 1964.7 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (1964.7\ \text{kN})/(0.9)(420\ \text{MPa}) =$ 5197.5 mm^2

Required bars = $A_s/645\ \text{mm}^2 =$ 8.058 No. 29 bars

Tension forces will be resisted equally by exterior and interior face reinforcement steel.

Required reinforcement to resist moment for 1 m column strip:

$M_u = M_{22} =$ 48.18 kN-m

Wall thickness = 457.2 mm
Compression face width, b = 1000 mm
Concrete cover = 76.20 mm
Rebar diameter = 28.65 mm (No. 29 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

d = 457.2 mm - 76.20 mm - (28.62 mm/2) = 366.7 mm

$$A_s = \frac{(0.85)(60\text{ MPa})(1000\text{ mm})}{(420\text{ MPa})} \times \left[366.7\text{ mm} - \sqrt{(366.7\text{ mm})^2 - \frac{(2)(48.18\text{ kN}-\text{m})}{(0.85)(0.9)(60\text{ MPa})(1000\text{ mm})}}\right]$$

$A_s =$ 349.0 mm^2

Required A_s/645 mm^2 = 0.541 No. 29 bars

Reinforcement to resist positive moment is located at the interior face of the column.

Required horizontal reinforcement for 1 m column strip:

Reinforcement required to resist tension = 5197.5 mm^2

Reinforcement required to resist moment = 349.0 mm^2

Total required exterior face reinforcement = 2598.8 mm^2 = 4.029 No. 29 bars

Total required interior face reinforcement = 2947.8 mm^2 = 4.570 No. 29 bars

Required bar spacing = 1000 mm/4.570 = 218.8 mm

Provide No. 29 bars spaced at 152 mm (6 in) on center at each face for the top 4 m of the column, then increase spacing to 203 mm (8 in) throughout the remainder of the column height.

Reinforcement Required to Resist Vertical Forces

Maximum vertical tension forces found at base of core columns due to load combination 11D. Resultant shell element forces from SAP2000 analysis for 1-m design strip.

Required axial reinforcement to resist tension for 1 m column strip:

$T_u = F_{22} =$ 2149.2 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (2149.2 \text{ kN})/(0.9)(420 \text{ MPa}) =$ 5685.8 mm^2

Required bars = $A_s/645 \text{ mm}^2 =$ 8.815 No. 29 bars

Tension forces will be resisted equally by exterior and interior face reinforcement steel.

Required reinforcement to resist moment for 1 m column strip:

$M_u = M_{22} =$ -49.83 kN-m

Wall thickness = 457.2 mm
Compression face width, b = 1000 mm
Concrete cover = 76.20 mm
Rebar diameter = 28.65 mm (No. 29 bars)

$$A_s = \frac{0.85 f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85 \Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

d = 457.2 mm - 76.20 mm - (28.62 mm/2) = 366.7 mm

$$A_s = \frac{(0.85)(60 \text{ MPa})(1000 \text{ mm})}{(420 \text{ MPa})} \times \left[366.7 \text{ mm} - \sqrt{(366.7 \text{ mm})^2 - \frac{(2)(49.83 \text{ kN} - \text{m})}{(0.85)(0.9)(60 \text{ MPa})(1000 \text{ mm})}} \right]$$

$A_s =$ 361.0 mm^2

Required $A_s/645 \text{ mm}^2 =$ 0.560 No. 29 bars

Reinforcement to resist negative moment is located at the exterior face of the column.

Required vertical reinforcement for 1 m column strip:

Reinforcement required to resist moment = 361.0 mm^2

Reinforcement required to resist tension = 5685.8 mm^2

Total required exterior face reinforcement = 3203.9 mm^2 = 4.967 No. 29 bars

Total required interior face reinforcement = 2842.9 mm^2 = 4.408 No. 29 bars

Required bar spacing = 1000 mm/4.967 = 201.3 mm

Provide No. 29 bars spaced at 152 mm (6 in) on center at each face throughout entire column height.

Reinforcement Required to Resist Out-of-Plane Shear

Resultant shell element forces from SAP2000 analysis for 1-m design strip. Transverse shear force found at top of column due to load combination 11A.

$V_u = V_{13} =$ 363.0 kN

$\Phi V_n \geq V_u$ (ACI 318-11 Eqn. 11-1)

$V_n = V_c + V_s$ (ACI 318-11 Eqn. 11-2)

$V_c = 0.17\sqrt{f'_c} b_w d$ (ACI 318-11 Eqn. 11-3)

$b_w =$ 1000 mm $d =$ 366.7 mm

$V_c = (0.17)\sqrt{(60\ \text{MPa})}(1000\ \text{mm})(366.7\ \text{mm}) =$ 473.4 kN

$\Phi =$ 0.75 (ACI 318-11 9.3.2.3)

$V_u/\Phi =$ 484.0 kN > 473.4 kN

Therefore, shear reinforcement is required.

$V_s = V_u/\Phi - V_c$ = 484 kN - 473.4 kN = 10.61 kN

$V_s = A_v f_{yt} d/s$ (ACI 318-11 Eqn. 11-15)

Try No. 16 shear links spaced at 152 mm:

V_s = (200 mm^2)(420 MPa)(366.7 mm)/152 mm = 202.1 kN > 10.61 kN

Provide No. 16 shear links spaced at 152 mm (6 in) each way for the top 4 m of the column.

<u>Reinforcement Required to Resist Out-of-Plane Shear</u>

Resultant shell element forces from SAP2000 analysis for 1-m design strip. Transverse shear force found at base of column due to load combination 11B.

$V_u = V_{23}$ = 138.9 kN

$\Phi V_n \geq V_u$ (ACI 318-11 Eqn. 11-1)

$V_n = V_c + V_s$ (ACI 318-11 Eqn. 11-2)

$V_c = 0.17\sqrt{f'_c}b_w d$ (ACI 318-11 Eqn. 11-3)

b_w = 1000 mm d = 366.7 mm

$V_c = (0.17)\sqrt{(60\text{ MPa})}$(1000 mm)(366.7 mm) = 473.4 kN

Φ = 0.75 (ACI 318-11 9.3.2.3)

V_u/Φ = 185.2 kN < 473.4 kN

Therefore, transverse shear reinforcement is not required. Minimum shear reinforcement is not required in solid slabs per ACI 318-11 Section 11.4.6.1 (a).

D.2 Core Column Honeycomb Reinforcement

Reinforcement Required to Resist Horizontal Forces

Maximum horizontal tension force found in honeycomb sector wall due to load combination 11D. Resultant shell element forces from SAP2000 analysis for 1-m design strip.

Required axial reinforcement to resist tension for 1 m wall strip:

$T_u = F_{11} =$ 819.4 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (819.4\ \text{kN})/(0.9)(420\ \text{MPa}) =$ 2167.6 mm^2

Required bars $= A_s/509\ \text{mm}^2 =$ 4.259 No. 25 bars

Tension forces will be resisted equally by reinforcement steel at both wall faces.

Required reinforcement to resist moment for 1 m wall strip:

$M_u = M_{22} =$ -4.227 kN-m

Wall thickness =	304.8	mm	
Compression face width, b =	1000	mm	
Concrete cover =	76.20	mm	
Rebar diameter =	25.40	mm	(No. 25 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$d = 304.8\ \text{mm} - 76.20\ \text{mm} - (25.40\ \text{mm}/2) =$ 215.9 mm

$$A_s = \frac{(0.85)(60\ \text{MPa})(1000\ \text{mm})}{(420\ \text{MPa})}$$

$$\times\left[215.9\text{ mm} - \sqrt{(215.9\text{ mm})^2 - \frac{(2)(4.227\text{ kN}-\text{m})}{(0.85)(0.9)(60\text{ MPa})(1000\text{ mm})}}\right]$$

$A_s =$ 51.84 mm^2

Required $A_s/509\ mm^2 =$ 0.102 No. 25 bars

Required horizontal reinforcement for 1 m wall strip:

Reinforcement required to resist tension = 2167.6 mm^2

Reinforcement required to resist moment = 51.84 mm^2

Total required exterior face reinforcement = 1083.8 mm^2 = 2.129 No. 25 bars

Total required interior face reinforcement = 1135.7 mm^2 = 2.231 No. 25 bars

Required bar spacing = 1000 mm/2.231 = 448.2 mm

Provide No. 25 bars spaced at 203 mm (8 in) on center at each face within all core column honeycomb components.

__Reinforcement Required to Resist Vertical Forces__

Maximum vertical tension forces found in honeycomb brace wall due to load combination 2C. Resultant shell element forces from SAP2000 analysis for 1-m design strip.

Required axial reinforcement to resist tension for 1 m wall strip:

$T_u = F_{22} =$ 1436.5 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (1436.5\text{ kN})/(0.9)(420\text{ MPa}) =$ 3800.3 mm^2

Required bars = A_s/509 mm^2 = 7.466 No. 25 bars

Tension forces will be resisted equally by reinforcement steel at both wall faces.

Required reinforcement to resist moment for 1 m wall strip:

$M_u = M_{22}$ = 0.000234 kN-m

Moment force occuring with maximum tension is negligible; required reinforcement will not be evaluated.

Required vertical reinforcement for 1 m wall strip:

Reinforcement required to resist tension = 3800.3 mm^2

Total required exterior face reinforcement = 1900.2 mm^2 = 3.733 No. 25 bars

Total required interior face reinforcement = 1900.2 mm^2 = 3.733 No. 25 bars

Required bar spacing = 1000 mm/3.733 267.9 mm

Provide No. 25 bars spaced at 203 mm (8 in) on center at each face within all core column honeycomb components.

<u>Reinforcement Required to Resist Out-of-Plane Shear</u>

Resultant shell element forces from SAP2000 analysis for 1-m design strip. Transverse shear force found in honeycomb sector wall due to load combination 11A.

$V_u = V_{13}$ = 127.0 kN

$\Phi V_n \geq V_u$ (ACI 318-11 Eqn. 11-1)

$V_n = V_c + V_s$ (ACI 318-11 Eqn. 11-2)

$V_c = 0.17\sqrt{f'_c} b_w d$ (ACI 318-11 Eqn. 11-3)

b_w = 1000 mm d = 215.9 mm

$V_c = (0.17)\sqrt{(60\ MPa)}(1000\ mm)(215.9\ mm) =$ 278.7 kN

$\Phi =$ 0.75 (ACI 318-11 9.3.2.3)

$V_u/\Phi =$ 169.3 kN $<$ 278.7 kN

Therefore, transverse shear reinforcement is not required. Minimum shear reinforcement is not required in solid slabs per ACI 318-11 Section 11.4.6.1 (a).

D.3 Core Column Deck Reinforcement

<u>Reinforcement Required to Resist Forces Along Local Axis 1</u>

Maximum tension forces found at intersection with brace deck due to load combination 15C. Resultant shell element forces from SAP2000 analysis averaged from 2 m deck strip for 1 m design strip.

Required axial reinforcement to resist tension for 1 m deck strip:

$T_u = F_{11} =$ 4809.5 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (4809.5\text{ kN})/(0.9)(420\text{ MPa}) =$ 12723.5 mm^2

Required bars = $A_s/819\text{ mm}^2 =$ 15.54 No. 32 bars

Tension forces will be resisted equally by exterior and interior face reinforcement steel.

Required reinforcement to resist moment for 1 m deck strip:

$M_u = M_{11} =$ -194.9 kN-m

Deck thickness =	1066.8 mm	
Compression face width, b =	1000 mm	
Concrete cover =	76.20 mm	
Rebar diameter =	35.81 mm	(No. 32 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$d = 1066.8\text{ mm} - 76.20\text{ mm} - (35.81\text{ mm}/2) =$ 972.7 mm

$$A_s = \frac{(0.85)(60\ \text{MPa})(1000\ \text{mm})}{(420\ \text{MPa})} \times \left[972.7\ \text{mm} - \sqrt{(972.7\ \text{mm})^2 - \frac{(2)(194.9\ \text{kN} - \text{m})}{(0.85)(0.9)(60\ \text{MPa})(1000\ \text{mm})}}\right]$$

$A_s =$ 531.2 mm^2

Required A_s/819 mm^2 = 0.649 No. 32 bars

Reinforcement to resist negative moment is located at the exterior face of the deck.

Required reinforcement in local axis 1 direction for 1 m deck strip:

Reinforcement required to resist moment = 531.2 mm^2

Reinforcement required to resist tension = 12723.5 mm^2

Total required exterior face reinforcement = 6893.0 mm^2 = 8.416 No. 32 bars

Total required interior face reinforcement = 6361.8 mm^2 = 7.768 No. 32 bars

Required bar spacing = 1000 mm/8.416 = 118.8 mm

Provide No. 32 bars spaced at 102 mm (4 in) on center at each face.

Reinforcement Required to Resist Maximum Moment Along Local Axis 1

Maximum moment due to load combination 2B. Maximum moment occurs with compressive forces that will not require reinforcement. Resultant shell element forces from SAP2000 analysis for 1-m design strip.

Required reinforcement to resist moment for 1 m deck strip:

$M_u = M_{11} =$ 911.5 kN-m

Deck thickness = 1066.8 mm
Compression face width, b = 1000 mm

Concrete cover = 76.20 mm

Rebar diameter = 35.81 mm (No. 32 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

d = 1066.8 mm - 76.20 mm - (35.81 mm/2) = 972.7 mm

$$A_s = \frac{(0.85)(60\ \text{MPa})(1000\ \text{mm})}{(420\ \text{MPa})} \times \left[972.7\ \text{mm} - \sqrt{(972.7\ \text{mm})^2 - \frac{(2)(911.5\ \text{kN}-\text{m})}{(0.85)(0.9)(60\ \text{MPa})(1000\ \text{mm})}} \right]$$

$A_s =$ 2505.7 mm^2

Required $A_s/819\ mm^2 =$ 3.059 No. 32 bars

Reinforcement to resist positive moment is located at the interior face of the deck.

Reinforcement provided at interior face = 9.804 bars > 3.059 bars (OK)

__Reinforcement Required to Resist Forces Along Local Axis 2__

Maximum tension forces found at edge of deck due to load combination 11D. Resultant shell element forces from SAP2000 analysis averaged from 3 m deck strip for 1 m design strip.

Required axial reinforcement to resist tension for 1 m deck strip:

$T_u = F_{22} =$ 5086.2 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (5086.2\ \text{kN})/(0.9)(420\ \text{MPa}) =$ 13455.5 mm^2

Required bars = A_s/819 mm^2 = 16.43 No. 32 bars

Tension forces will be resisted equally by exterior and interior face reinforcement steel.

Required reinforcement to resist moment for 1 m deck strip:

$M_u = M_{22}$ = -147.9 kN-m

Deck thickness = 1066.8 mm
Compression face width, b = 1000 mm
Concrete cover = 76.20 mm
Rebar diameter = 35.81 mm (No. 32 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}}\right]$$

Φ = 0.90 (ACI 318-11 9.3.2.1)

d = 1066.8 mm - 76.20 mm - (35.81 mm/2) = 972.7 mm

$$A_s = \frac{(0.85)(60\text{ MPa})(1000\text{ mm})}{(420\text{ MPa})} \times \left[972.7\text{ mm} - \sqrt{(972.7\text{ mm})^2 - \frac{(2)(147.9\text{ kN}-\text{m})}{(0.85)(0.9)(60\text{ MPa})(1000\text{ mm})}}\right]$$

A_s = 403.0 mm^2

Required A_s/819 mm^2 = 0.492 No. 32 bars

Reinforcement to resist negative moment is located at the exterior face of the deck.

Required reinforcement in local axis 2 direction for 1 m deck strip:

Reinforcement required to resist moment = 403.0 mm^2

Reinforcement required to resist tension = 13455.5 mm^2

Total required exterior face reinforcement = 7130.8 mm^2 = 8.707 No. 32 bars

Total required interior face reinforcement = 6727.8 mm^2 = 8.215 No. 32 bars

Required bar spacing = 1000 mm/8.707 = 114.9 mm

Provide No. 32 bars spaced at 102 mm (4 in) on center at each face.

<u>Reinforcement Required to Resist Out-of-Plane Shear</u>

Resultant shell element forces from SAP2000 analysis for 1-m design strip. Transverse shear forces due to load combination 2B.

$V_u = V_{13} =$ 1625.3 kN

$\Phi V_n \geq V_u$ (ACI 318-11 Eqn. 11-1)

$V_n = V_c + V_s$ (ACI 318-11 Eqn. 11-2)

$V_c = 0.17\sqrt{f'_c}b_w d$ (ACI 318-11 Eqn. 11-3)

$b_w =$ 1000 mm d = 972.7 mm

$V_c = (0.17)\sqrt{(60\text{ MPa})}(1000\text{ mm})(972.7\text{ mm}) =$ 1255.7 kN

$\Phi =$ 0.75 (ACI 318-11 9.3.2.3)

$V_u/\Phi =$ 2167.0 kN > 1255.7 kN

Therefore, shear reinforcement is required.

Required $V_s = V_u/\Phi - V_c =$ 2166.9 kN - 1255.7 kN = 911.3 kN

$V_s = A_v f_{yt} d/s$ (ACI 318-11 Eqn. 11-15)

Try No. 19 shear links spaced at 102 mm:

$V_s = (284\text{ mm}^2)(420\text{ MPa})(972.7\text{ mm})/102\text{ mm} =$ 1137.5 kN > 911.3 kN

Provide No. 19 shear links spaced at 102 mm (4 in) each way.

Check shear reinforcement when deck is in tension:

Forces due to load combination 14C for 1 m deck strip.

$V_u = V_{13} =$ 519.1 kN $N_u = F_{11} =$ 1647.1 kN (tension)

For walls subject to axial tension:

$$V_c = 0.17\left(1 + \frac{0.29N_u}{A_g}\right)\sqrt{f'_c}b_w d$$ (ACI 318-11 Eqn. 11-8)

A_g = (1000 mm)(1066.8 mm) = 1066800 mm^2

V_c = 0.17 x [1 + (0.29)(-1647.1 kN)/1066800 mm^2]

x (√60 MPa)(1000 mm)(972.7 mm) = 707.3 kN

$V_n = V_c + V_s$ = 707.3 kN + 1137.5 kN = 1844.8 kN

ΦV_n = (0.75)(1844.8 kN) = 1383.6 kN > 519.1 kN

D.4 Floater Column Outer Shell Reinforcement

<u>Reinforcement Required to Resist Horizontal Forces</u>

Maximum horizontal tension force found at top of floater column due to load combination 11A. Resultant shell element forces from SAP2000 analysis for 1-m design strip.

Required axial reinforcement to resist tension for 1 m wall strip:

$T_u = F_{11} =$ 1035.9 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (1035.9\text{ kN})/(0.9)(420\text{ MPa}) =$ 2740.4 mm^2

Required bars = $A_s/509\text{ mm}^2 =$ 5.384 No. 25 bars

Tension forces will be resisted equally by reinforcement steel at both wall faces.

Required reinforcement to resist moment for 1 m wall strip:

$M_u = M_{22} =$ -16.03 kN-m

Wall thickness =	406.4	mm	
Compression face width, b =	1000	mm	
Concrete cover =	76.20	mm	
Rebar diameter =	25.40	mm	(No. 25 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$d = 406.4\text{ mm} - 76.20\text{ mm} - (25.40\text{ mm}/2) =$ 317.5 mm

$$A_s = \frac{(0.85)(60\text{ MPa})(1000\text{ mm})}{(420\text{ MPa})}$$

$$\times \left[317.5 \text{ mm} - \sqrt{(317.5 \text{ mm})^2 - \frac{(2)(16.03 \text{ kN} - \text{m})}{(0.85)(0.9)(60 \text{ MPa})(1000 \text{ mm})}} \right]$$

$A_s =$ 133.8 mm^2

Required A_s/509 mm^2 = 0.263 No. 25 bars

Reinforcement to resist negative moment is located at the exterior face of the column.

Required horizontal reinforcement for 1 m wall strip:

Reinforcement required to resist tension = 2740.4 mm^2

Reinforcement required to resist moment = 133.8 mm^2

Total required exterior face reinforcement = 1370.2 mm^2 = 2.692 No. 25 bars

Total required interior face reinforcement = 1504.0 mm^2 = 2.955 No. 25 bars

Required bar spacing = 1000 mm/2.955 = 338.4 mm

Provide No. 25 bars spaced at 203 mm (8 in) on center at each face throughout entire column height.

<u>**Reinforcement Required to Resist Vertical Forces**</u>

Maximum tension forces found at base of floater columns due to load combination 11D. Resultant shell element forces from SAP2000 analysis averaged from 6 m horizontal column strip for 1 m design strip. Modeling effects likely resulted in amplified forces at column base and therefore it is assumed that the forces exceeding concrete tensile strength at this location will not occur in operation.

Required axial reinforcement to resist tension for 1 m column strip:

$T_u = F_{22} =$ 3407.5 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y$ = (3407.5 kN)/(0.9)(420 MPa) = 9014.5 mm^2

Required bars = A_s/819 mm^2 = 11.01 No. 32 bars

Tension forces will be resisted equally by exterior and interior face reinforcement steel.

Required reinforcement to resist moment for 1 m column strip:

$M_u = M_{22}$ = -38.63 kN-m

Wall thickness = 406.4 mm
Compression face width, b = 1000 mm
Concrete cover = 76.20 mm
Rebar diameter = 32.26 mm (No. 32 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

d = 406.4 mm - 76.20 mm - (32.26 mm/2) = 314.1 mm

$$A_s = \frac{(0.85)(60\text{ MPa})(1000\text{ mm})}{(420\text{ MPa})} \times \left[314.1\text{ mm} - \sqrt{(314.1\text{ mm})^2 - \frac{(2)(38.63\text{ kN}-\text{m})}{(0.85)(0.9)(60\text{ MPa})(1000\text{ mm})}} \right]$$

A_s = 326.8 mm^2

Required A_s/819 mm^2 = 0.399 No. 32 bars

Reinforcement to resist negative moment is located at the exterior face of the column.

Required vertical reinforcement for 1 m column strip:

Reinforcement required to resist moment = 326.8 mm^2

Reinforcement required to resist tension = 9014.5 mm^2

Total required exterior face reinforcement = 4834.0 mm^2 = 5.902 No. 32 bars

Total required interior face reinforcement = 4507.2 mm^2 = 5.503 No. 32 bars

Required bar spacing = 1000 mm/5.902 = 169.4 mm

Provide No. 32 bars spaced at 152 mm (6 in) on center at each face for bottom 8 m of column.

Above 8 m height on floater column tension forces reduce significantly and provided rebar may be reduced. Maximum tension forces above 8 m due to load combination 11D. Resultant shell element forces from SAP2000 analysis for 1 m design strip.

Required axial reinforcement to resist tension for 1 m column strip:

$T_u = F_{22} =$ 1715.4 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y$ = (1715.4 kN)/(0.9)(420 MPa) = 4538.0 mm^2

Required bars = A_s/509 mm^2 = 8.916 No. 25 bars

Tension forces will be resisted equally by exterior and interior face reinforcement steel.

Required reinforcement to resist moment for 1 m column strip:

$M_u = M_{22} =$ -4.983 kN-m

Wall thickness = 406.4 mm
Compression face width, b = 1000 mm
Concrete cover = 76.20 mm
Rebar diameter = 25.40 mm (No. 25 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

d = 406.4 mm - 76.20 mm - (25.40 mm/2) = 317.5 mm

$$A_s = \frac{(0.85)(60\ \text{MPa})(1000\ \text{mm})}{(420\ \text{MPa})} \times \left[317.5\ \text{mm} - \sqrt{(317.5\ \text{mm})^2 - \frac{(2)(4.983\ \text{kN} - \text{m})}{(0.85)(0.9)(60\ \text{MPa})(1000\ \text{mm})}} \right]$$

$A_s =$ 41.54 mm^2

Required A_s/509 mm^2 = 0.0816 No. 25 bars

Reinforcement to resist negative moment is located at the exterior face of the column.

Required vertical reinforcement for 1 m column strip:

Reinforcement required to resist moment = 41.54 mm^2

Reinforcement required to resist tension = 4538.0 mm^2

Total required exterior face reinforcement = 2310.6 mm^2 = 4.539 No. 25 bars

Total required interior face reinforcement = 2269.0 mm^2 = 4.458 No. 25 bars

Required bar spacing = 1000 mm/4.539 = 220.3 mm

Provide No. 25 bars spaced at 203 mm (8 in) on center at each face starting from 8 m up the column for the remainder of the column height.

Reinforcement Required to Resist Out-of-Plane Shear

Resultant shell element forces from SAP2000 analysis for 1-m design strip. Transverse shear forces found at intersection with base pontoon due to load combination 12B.

$V_u = V_{23} =$ 207.8 kN

$\Phi V_n \geq V_u$ (ACI 318-11 Eqn. 11-1)

$V_n = V_c + V_s$ (ACI 318-11 Eqn. 11-2)

$V_c = 0.17\sqrt{f'_c} b_w d$ (ACI 318-11 Eqn. 11-3)

$b_w =$ 1000 mm $d =$ 314.1 mm

$V_c = (0.17)\sqrt{(60 \text{ MPa})}(1000 \text{ mm})(314.1 \text{ mm}) =$ 405.5 kN

$\Phi =$ 0.75 (ACI 318-11 9.3.2.3)

$V_u/\Phi =$ 277.1 kN $<$ 405.5 kN

Therefore, transverse shear reinforcement is not required. Minimum shear reinforcement is not required in solid slabs per ACI 318-11 Section 11.4.6.1 (a).

D.5 Floater Column Honeycomb Reinforcement

Reinforcement Required to Resist Maximum Tension Forces

Maximum tension forces found spanning vertically at base of honeycomb sector wall at intersection with outer floater column wall due to load combination 11D. Resultant shell element forces from SAP2000 analysis for 1-m design strip.

Vertical tension forces govern over horizontal forces. The rebar determined for the vertical direction will be used in the horizontal direction as well.

Required axial reinforcement to resist tension for 1 m wall strip:

$T_u = F_{22} =$ 1856.8 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (1856.8\ \text{kN})/(0.9)(420\ \text{MPa}) =$ 4912.1 mm^2

Required bars = $A_s/509\ \text{mm}^2 =$ 9.651 No. 25 bars

Tension forces will be resisted equally by reinforcement steel at both wall faces.

Required reinforcement to resist moment for 1 m wall strip:

$M_u = M_{22} =$ -4.811 kN-m

Wall thickness =	254	mm	
Compression face width, b =	1000	mm	
Concrete cover =	76.20	mm	
Rebar diameter =	25.40	mm	(No. 25 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

d = 254 mm - 76.20 mm - (25.40 mm/2) = 165.1 mm

$$A_s = \frac{(0.85)(60\text{ MPa})(1000\text{ mm})}{(420\text{ MPa})} \times \left[165.1\text{ mm} - \sqrt{(165.1\text{ mm})^2 - \frac{(2)(4.811\text{ kN}-\text{m})}{(0.85)(0.9)(60\text{ MPa})(1000\text{ mm})}}\right]$$

A_s = 77.23 mm^2

Required A_s/509 mm^2 = 0.152 No. 25 bars

Required vertical reinforcement for 1 m wall strip:

Reinforcement required to resist moment = 77.23 mm^2

Reinforcement required to resist tension = 4912.1 mm^2

Total required exterior face reinforcement = 2533.3 mm^2 = 4.977 No. 25 bars

Total required interior face reinforcement = 2456.1 mm^2 = 4.825 No. 25 bars

Required bar spacing = 1000 mm/4.977 = 200.9 mm

Provide No. 25 bars spaced at 203 mm (8 in) on center at each face in both directions within all floater column honeycomb components.

<u>Reinforcement Required to Resist Out-of-Plane Shear</u>

Resultant shell element forces from SAP2000 analysis for 1-m design strip. Transverse shear forces found mid-height of brace wall at intersection with sector wall due to load combination 12B.

$V_u = V_{13}$ = 119.6 kN

$\Phi V_n \geq V_u$ (ACI 318-11 Eqn. 11-1)

$V_n = V_c + V_s$ (ACI 318-11 Eqn. 11-2)

$V_c = 0.17\sqrt{f'_c}b_w d$ (ACI 318-11 Eqn. 11-3)

$b_w =$ 1000 mm $d =$ 165.1 mm

$V_c = (0.17)\sqrt{(60\text{ MPa})}(1000\text{ mm})(165.1\text{ mm}) =$ 213.1 kN

$\Phi =$ 0.75 (ACI 318-11 9.3.2.3)

$V_u/\Phi =$ 159.4 kN < 213.1 kN

Therefore, transverse shear reinforcement is not required. Minimum shear reinforcement is not required in solid slabs per ACI 318-11 Section 11.4.6.1 (a).

D.6 Floater Column Deck Reinforcement

Reinforcement Required to Resist Maximum Tension Forces

Maximum tension forces found at intersection with brace deck due to load combination 14B. Resultant shell element forces from SAP2000 analysis for 1-m design strip.

Tension forces along local axis 2 govern over local axis 1 forces. The rebar determined for the local axis 2 direction will be used in the local axis 1 direction as well.

Required axial reinforcement to resist tension for 1 m deck strip:

$T_u = F_{22} =$ 1130.4 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (1130.4\text{ kN})/(0.9)(420\text{ MPa}) =$ 2990.5 mm^2

Required bars = $A_s/509\text{ mm}^2 =$ 5.875 No. 25 bars

Tension forces will be resisted equally by exterior and interior face reinforcement steel.

Required reinforcement to resist moment for 1 m deck strip:

$M_u = M_{22} =$ -4.690 kN-m

Deck thickness =	304.8 mm	
Compression face width, b =	1000 mm	
Concrete cover =	76.20 mm	
Rebar diameter =	25.40 mm	(No. 25 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$d = 304.8\text{ mm} - 76.20\text{ mm} - (25.40\text{ mm}/2) =$ 215.9 mm

$$A_s = \frac{(0.85)(60 \text{ MPa})(1000 \text{ mm})}{(420 \text{ MPa})} \times \left[972.7 \text{ mm} - \sqrt{(972.7 \text{ mm})^2 - \frac{(2)(4.690 \text{ kN} - \text{m})}{(0.85)(0.9)(60 \text{ MPa})(1000 \text{ mm})}}\right]$$

$A_s =$ 57.54 mm^2

Required A_s/509 mm^2 = 0.113 No. 25 bars

Reinforcement to resist negative moment is located at the exterior face of the deck.

Required reinforcement in local axis 2 direction for 1 m deck strip:

Reinforcement required to resist moment = 57.54 mm^2

Reinforcement required to resist tension = 2990.5 mm^2

Total required exterior face reinforcement = 1552.8 mm^2 = 3.051 No. 25 bars

Total required interior face reinforcement = 1495.3 mm^2 = 2.938 No. 25 bars

Required bar spacing = 1000 mm/3.051 = 327.8 mm

Provide No. 25 bars spaced at 203 mm (8 in) on center at each face in both directions.

D.7 Floater Brace Outer Wall Reinforcement

<u>Reinforcement Required to Resist Horizontal Forces</u>

Maximum horizontal tension force found at intersection with core column deck due to load combination 11D. Resultant shell element forces from SAP2000 analysis averaged from 2 m wall strip for 1 m design strip.

Required axial reinforcement to resist tension for 1 m wall strip:

$T_u = F_{11} =$ 2105.8 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (2105.8\text{ kN})/(0.9)(420\text{ MPa}) =$ 5570.9 mm^2

Required bars = $A_s/509\text{ mm}^2 =$ 10.94 No. 25 bars

Tension forces will be resisted equally by reinforcement steel at both wall faces.

Required reinforcement to resist moment for 1 m wall strip:

$M_u = M_{22} =$ -28.88 kN-m

Wall thickness =	406.4 mm	
Compression face width, b =	1000 mm	
Concrete cover =	76.20 mm	
Rebar diameter =	25.40 mm	(No. 25 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$d = 406.4\text{ mm} - 76.20\text{ mm} - (25.40\text{ mm}/2) =$ 317.5 mm

$$A_s = \frac{(0.85)(60\text{ MPa})(1000\text{ mm})}{(420\text{ MPa})} \times \left[317.5\text{ mm} - \sqrt{(317.5\text{ mm})^2 - \frac{(2)(28.88\text{ kN}-\text{m})}{(0.85)(0.9)(60\text{ MPa})(1000\text{ mm})}}\right]$$

$A_s =$ 241.4 mm^2

Required $A_s/509\ mm^2 =$ 0.474 No. 25 bars

Reinforcement to resist negative moment is located at the exterior face of the wall

Required horizontal reinforcement for 1 m wall strip:

Reinforcement required to resist tension = 5570.9 mm^2

Reinforcement required to resist moment = 241.4 mm^2

Total required exterior face reinforcement = 2785.4 mm^2 = 5.472 No. 25 bars

Total required interior face reinforcement = 3026.9 mm^2 = 5.947 No. 25 bars

Required bar spacing = 1000 mm/5.947 = 168.2 mm

Provide No. 25 bars spaced at 152 mm (6 in) on center at each face for the top 4 m of the brace wall, then increase spacing to 203 mm (8 in) throughout the remainder of the wall height.

<u>Reinforcement Required to Resist Vertical Forces</u>

Maximum tension forces found at base of floater brace wall due to load combination 11D. Resultant shell element forces from SAP2000 analysis averaged from 12 m horizontal wall strip for 1 m design strip. High tensile forces are concentrated at the base of the wall. Modeling effects likely resulted in amplified forces at wall base and therefore it is assumed that the forces exceeding concrete tensile strength at this location will not occur in operation.

Required axial reinforcement to resist tension for 1 m wall strip:

$T_u = F_{22} =$ 2163.0 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (2163\ \text{kN})/(0.9)(420\ \text{MPa}) =$ 5722.2 mm^2

Required bars = $A_s/819\ \text{mm}^2 =$ 6.987 No. 32 bars

Tension forces will be resisted equally by exterior and interior face reinforcement steel.

Required reinforcement to resist moment for 1 m wall strip:

$M_u = M_{22} =$ -22.89 kN-m

Wall thickness = 406.4 mm
Compression face width, b = 1000 mm
Concrete cover = 76.20 mm
Rebar diameter = 32.26 mm (No. 32 bars)

$$A_s = \frac{0.85 f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85 \Phi f'_c b}} \right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$d = 406.4\ \text{mm} - 76.20\ \text{mm} - (32.26\ \text{mm}/2) =$ 314.1 mm

$$A_s = \frac{(0.85)(60\ \text{MPa})(1000\ \text{mm})}{(420\ \text{MPa})} \times \left[314.1\ \text{mm} - \sqrt{(314.1\ \text{mm})^2 - \frac{(2)(22.89\ \text{kN}-\text{m})}{(0.85)(0.9)(60\ \text{MPa})(1000\ \text{mm})}} \right]$$

$A_s =$ 193.3 mm^2

Required $A_s/819\ \text{mm}^2 =$ 0.236 No. 32 bars

Reinforcement to resist negative moment is located at the exterior face of the wall.

Required vertical reinforcement for 1 m wall strip:

Reinforcement required to resist moment =	193.3 mm^2		
Reinforcement required to resist tension =	5722.2 mm^2		
Total required exterior face reinforcement =	3054.4 mm^2	=	6.001 No. 32 bars
Total required interior face reinforcement =	2861.1 mm^2	=	5.621 No. 32 bars
Required bar spacing = 1000 mm/6.001 =	166.6 mm		

Provide No. 32 bars spaced at 152 mm (6 in) on center at each face for bottom 8 m of column. Provide No. 25 bars spaced at 203 mm (8 in) on center at each face starting from 8 m up the column for the remainder of the column height.

D.8 Floater Brace Inner Wall Reinforcement

__Reinforcement Required to Resist Maximum Tension Forces__

Maximum tension forces found spanning vertically at base of brace wall at perpendicular intersection with outer brace floater wall due to load combination 12C. Resultant shell element forces from SAP2000 analysis for 1-m design strip.

Vertical tension forces govern over horizontal forces. The rebar determined for the vertical direction will be used in the horizontal direction as well.

Required axial reinforcement to resist tension for 1 m wall strip:

$T_u = F_{22} =$ 1009.7 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (1009.7\ \text{kN})/(0.9)(420\ \text{MPa}) =$ 2671.1 mm^2

Required bars = $A_s/284\ \text{mm}^2 =$ 9.405 No. 19 bars

Tension forces will be resisted equally by reinforcement steel at both wall faces.

Required reinforcement to resist moment for 1 m wall strip:

$M_u = M_{22} =$ 0.282 kN-m

Moment force occuring with maximum tension is negligible; required reinforcement will not be evaluated.

Required vertical reinforcement for 1 m wall strip:

Reinforcement required to resist tension = 2671.1 mm^2

Total required exterior face reinforcement = 1335.6 mm^2 = 4.703 No. 19 bars

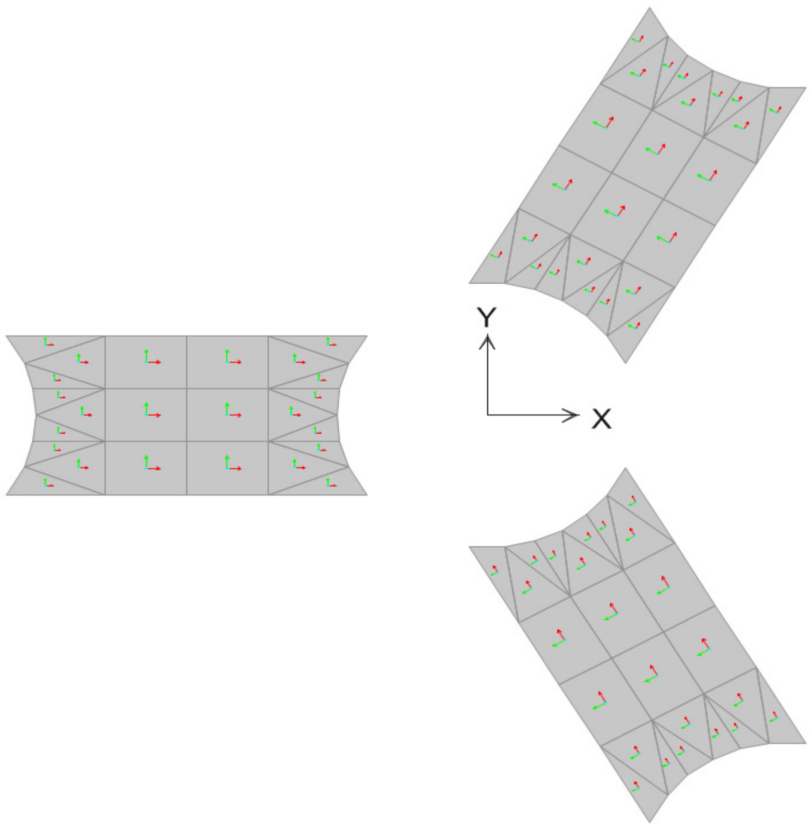
Total required interior face reinforcement = 1335.6 mm^2 = 4.703 No. 19 bars

Required bar spacing = 1000 mm/4.703 = 212.6 mm

Provide No. 19 bars spaced at 203 mm (8 in) on center at each face in both directions within all brace floater brace walls.

D.9 Floater Brace Deck Reinforcement

The brace float decks have local axes that are rotated from the default orientation for more clear interpretation of the analysis results. Local axis 1 (red) runs longitudinal between the floater columns and local axis 2 (green) runs transverse to the columns.



Reinforcement Required to Resist Forces Along Local Axis 1

Maximum tension forces found at intersection with core columns due to load combination 11D. Resultant shell element forces from SAP2000 analysis averaged from 2 m deck strip for 1 m design strip.

Required axial reinforcement to resist tension for 1 m deck strip:

$T_u = F_{11} =$ 2174.0 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y$ = (2174 kN)/(0.9)(420 MPa) = 5751.2 mm^2

Required bars = A_s/645 mm^2 = 8.917 No. 29 bars

Tension forces will be resisted equally by exterior and interior face reinforcement steel.

Required reinforcement to resist moment for 1 m deck strip:

$M_u = M_{11}$ = -10.57 kN-m

Deck thickness = 457.2 mm
Compression face width, b = 1000 mm
Concrete cover = 76.20 mm
Rebar diameter = 28.65 mm (No. 29 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}}\right]$$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

d = 457.2 mm - 76.20 mm - (28.65 mm/2) = 366.7 mm

$$A_s = \frac{(0.85)(60\ \text{MPa})(1000\ \text{mm})}{(420\ \text{MPa})} \times \left[366.7\ \text{mm} - \sqrt{(366.7\ \text{mm})^2 - \frac{(2)(10.57\ \text{kN}-\text{m})}{(0.85)(0.9)(60\ \text{MPa})(1000\ \text{mm})}}\right]$$

A_s = 76.30 mm^2

Required A_s/645 mm^2 = 0.118 No. 29 bars

Reinforcement to resist negative moment is located at the exterior face of the deck.

Required reinforcement in local axis 1 direction for 1 m deck strip:

Reinforcement required to resist moment = 76.30 mm^2

Reinforcement required to resist tension = 5751.2 mm^2

Total required exterior face reinforcement = 2951.9 mm^2 = 4.577 No. 29 bars

Total required interior face reinforcement = 2875.6 mm^2 = 4.458 No. 29 bars

Required bar spacing = 1000 mm/4.577 = 218.5 mm

Provide No. 29 bars spaced at 152 mm (6 in) at each face for 1067 mm (42 in) along edges, then increase spacing to 203 mm (8 in) on center.

Reinforcement Required to Resist Forces Along Local Axis 2

Maximum tension forces found at intersection with core columns. Maximum tension forces due to load combination 11D. Resultant shell element forces from SAP2000 analysis for 1-m design strip.

Required axial reinforcement to resist tension for 1 m deck strip:

$T_u = F_{22} =$ 1323.5 kN

$T_u \leq \Phi A_s f_y$

$\Phi =$ 0.90 (ACI 318-11 9.3.2.1)

$A_s = T_u/\Phi f_y = (1323.5 \text{ kN})/(0.9)(420 \text{ MPa}) =$ 3501.4 mm^2

Required bars = $A_s/645\ mm^2$ = 5.428 No. 29 bars

Tension forces will be resisted equally by exterior and interior face reinforcement steel.

Required reinforcement to resist moment for 1 m deck strip:

$M_u = M_{22} =$ 7.783 kN-m

Deck thickness = 457.2 mm
Compression face width, b = 1000 mm
Concrete cover = 76.20 mm

Rebar diameter = 28.65 mm (No. 29 bars)

$$A_s = \frac{0.85f'_c b}{f_y} \times \left[d - \sqrt{d^2 - \frac{2M_u}{0.85\Phi f'_c b}} \right]$$

Φ = 0.90 (ACI 318-11 9.3.2.1)

d = 457.2 mm - 76.20 mm - (28.65 mm/2) = 366.7 mm

$$A_s = \frac{(0.85)(60\ \text{MPa})(1000\ \text{mm})}{(420\ \text{MPa})} \times \left[366.7\ \text{mm} - \sqrt{(366.7\ \text{mm})^2 - \frac{(2)(7.783\ \text{kN}-\text{m})}{(0.85)(0.9)(60\ \text{MPa})(1000\ \text{mm})}} \right]$$

A_s = 56.19 mm^2

Required A_s/645 mm^2 = 0.087 No. 29 bars

Reinforcement to resist positive moment is located at the interior face of the deck.

Required reinforcement in local axis 2 direction for 1 m deck strip:

Reinforcement required to resist moment = 56.19 mm^2

Reinforcement required to resist tension = 3501.4 mm^2

Total required exterior face reinforcement = 1806.9 mm^2 = 2.801 No. 29 bars

Total required interior face reinforcement = 1750.7 mm^2 = 2.714 No. 29 bars

Required bar spacing = 1000 mm/2.801 = 357.0 mm

Provide No. 29 bars spaced at 203 mm (8 in) on center at each face.

Appendix E. Post-Tensioning Design Calculations

Prestressing strands shall be ASTM A416 0.5" Ø 7 wire strands. Initial jacking is 70% of ultimate strength. 20% loss of prestress is assumed at service stage. The tendons shall be spaced in the center of each section so that no eccentric load will be generated by the prestressing.

f_{pu} =	270 ksi	
$f_{se} = (0.7)(0.8)f_{pu}$ =	151.2 ksi	
f'_c =	8700 psi	
f_s =	200 psi	(target stress in section due to prestressing)

Limit to compressive stresses in concrete at service level due to prestress plus total load:

$0.60f'_c$ = 5220 psi (ACI 318-11 18.4.2)

Required Axial Compression

Section	A (mm^2)	A (in^2)	P (kip)
Core column	13706669	21245.3	4249.1
Floater column	12248564	18985.3	3797.1
Brace floater	4879022	7562.5	1512.5

Core Column Outer Wall

$A_{ps} = P_s/f_{se}$ = 28.10 in^2

Area of strands = 0.153 in^2

No. of strands = 183.7

Circumference = 1180.3 in

Required spacing = 6.426 in

Provide 0.5" Ø 7 wire prestressing strands spaced at 6" around circumference of core column outer wall at section centerline.

Check maximum allowable compressive stress:

Use no. strands =	198		Spacing =	5.961 in	
$P = (198)(A_s)(f_{se}) =$	4580.5 kips				
P/A =	215.6 psi				
Max stress =	880.1 psi		(compressive stress due to load combination 11A)		
Total stress =	1095.7	<	5220 psi		(OK)

Floater Column Outer Wall

$A_{ps} = P_s/f_{se} =$	25.11 in^2
Area of strands =	0.153 in^2
No. of strands =	164.1
Circumference =	1186.6 in
Spacing =	7.229 in

Provide 0.5" Ø 7 wire prestressing strands spaced at 6" around circumference of floater column outer walls at section centerline.

Check maximum allowable compressive stress:

Use no. strands =	198		Spacing =	5.993 in	
$P = (198)(A_s)(f_{se}) =$	4580.5 kips				
P/A =	241.3 psi				
Max stress =	1467.1 psi		(compressive stress due to load combination 11A)		
Total stress =	1708.3	<	5220 psi		(OK)

Floater Brace Outer Wall

$A_{ps} = P_s/f_{se}$ =	10.00 in^2			
Area of strands =	0.153 in^2			
No. of strands =	65.38			
Wall length =	472.7 in			
Spacing =	7.229 in			

Provide 0.5" Ø 7 wire prestressing strands spaced at 6" along floater brace outer walls at section centerline.

Check maximum allowable compressive stress:

Use no. strands =	80		Spacing =	5.908 in
$P = (198)(A_s)(f_{se})$ =	1850.7 kips			
P/A =	244.7 psi			
Max stress =	1156.4 psi		(compressive stress due to load combination 12B)	
Total stress =	1401.2	<	5220 psi	(OK)

Appendix F. Connection Design at Base of Tower

Design of bolted connection attaching wind turbine tower to deck slab of core column.

Proposed connection:

- 2" Ø ASTM A449 headed anchor bolts, 58 ksi yield strength, 90 ksi tensile strength.
- Cast-in-place group of 2 - 2" anchor bolts spaced at 12" around base of tower with 30" embedment into deck.

Check for:

- Bolts subjected to combined tension and shear.
- Concrete failure modes.
- Plate bearing capacity.

Assumptions:

- Bolts take shear loading equally.
- Considering worst-case loading due to OTM; approximately 1/4 of bolts take decoupled OTM as tension force.
- Simple connection (no eccentricity considered) subject to tension from OTM and shear forces from tower base shear.

Minimum spacing between 2 bolts in group = $4d_a$ = (4)(2 in) = 8 in (ACI 318-11 D.8.1)

Base of tower circumference = 25132.7 mm 989.5 in 82.46 ft

Loading:

Case (1) due to load case 2 with turbine rated wind speed and extreme turbulence.

Tower base shear =	2000 kN	449.6 kips
Tower base OTM =	130500 kN-m	96251.7 kip-ft
Decoupled OTM =	3149.4 kN/m	215.8 klf

Shear:	4.887 kips per 2 bolts	x 1.35 =	6.598 kips
Tension:	215.8 kips per 2 bolts	x 1.35 =	291.3 kips

<u>Case (2)</u> due to load case 5 with turbine rated wind speed and extreme operating gust.

Tower base shear =	2175	kN	489.0	kips
Tower base OTM =	127000	kN-m	93670.2	kip-ft
Decoupled OTM =	3064.9	kN/m	210.0	klf

Shear:	5.315	kips per 2 bolts	x 1.35 =	7.175	kips
Tension:	210.0	kips per 2 bolts	x 1.35 =	283.5	kips

F.1 Anchor Bolt Tensile and Shear Capacities

Anchor bolts are 2" Ø ASTM A449 headed anchor bolts which are embedded 30" into the core column deck. Anchor bolts embedded in the deck will be checked for tension combined with shear force loading.

Yield strength, f_y = 58 ksi
Tensile strength, f_{uta} = 90 ksi

2" Ø anchor bolt tensile strength:

Per ACI 318-11 Appendix D.5.1.2 , the nominal strength of a single anchor in tension is

$N_{sa} = A_{se,N}f_{uta}$ (ACI 318-11 Eqn. D-2)

$A_{se,N}$ = 2.5 in^2 (Tbl. 7-18 AISC Steel Manual 13th Ed.)

Φ = 0.75 (ACI 318-11 D.4.3 (a), ductile steel element subjected to tensile loads)

$\Phi N_{sa} = (0.75)(2.5 \text{ in}^2)(90 \text{ ksi}) =$ 168.8 kips per bolt

ΦN_{sa} 2 bolts = 337.5 kips

2" Ø anchor bolt shear strength:

Per ACI 318-11 Appendix D.6.1.2 (b), the nominal strength of a single post-installed anchor in shear is

$V_{sa} = 0.6A_{se,V}f_{uta}$ (ACI 318-11 Eqn. D-29)

$A_{se,V} = (\pi/4)(d_a - 0.9743/n_t)^2 = (\pi/4)(2 \text{ in} - 0.9743/4.5)^2 =$ 2.498 in^2

(ACI 318-11 RD.6.1.2)

n_t is the number of threads per inch, which is taken as 4.5 per AISC Steel Manual Tbl. 7-18.

$\Phi =$ 0.65 (ACI 318-11 D.4.3, ductile steel element subjected to shear loads)

$\Phi V_{sa} = (0.65)(0.6)(2.498\ in^2)(90\ ksi) =$ 87.69 kips

ΦV_{sa} 2 bolts = 175.4 kips

Check combined tension and shear:

Derived from AISC Eqn. J3-3a:

$$\frac{\text{Required tensile strength}}{\text{Available tensile strength}} + \frac{\text{Required shear strength}}{\text{Available shear strength}} \leq 1.3$$

Case (1):

$$\frac{291.3}{337.5} + \frac{6.598}{175.4} = 0.901 \quad < 1.3 \qquad \text{(OK)}$$

Case (2):

$$\frac{283.5}{337.5} + \frac{7.175}{175.4} = 0.881 \quad < 1.3 \qquad \text{(OK)}$$

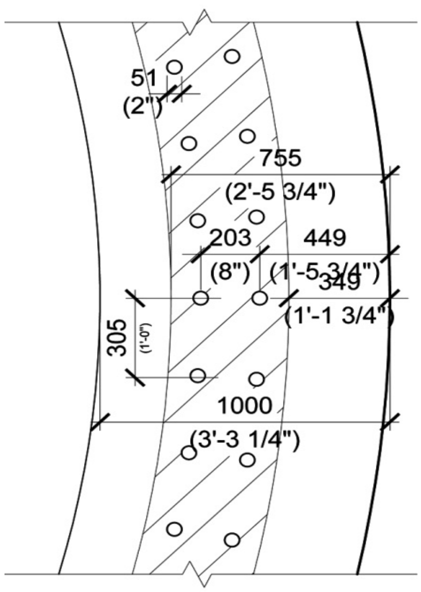
F.2 Concrete Breakout Strength − Anchors in Tension

According to ACI 318-11 Appendix D.5.2.1, the nominal concrete breakout strength of a group of anchors in tension shall not exceed:

$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (ACI 318-11 Eqn. D-4)

$A_{Nc} = (1.5h_{ef} + s_1 + c_{a1})$ x s (ACI 318-11 Fig. RD.5.2.1(b))

$A_{Nco} = 9h_{ef}'^2$ (ACI 318-11 Eqn. D-5 and RD.5.2.3)



$h_{ef} =$ 30 in (anchor effective embedment depth)

$c_{a1} =$ 17.75 in (distance from edge to center of closest anchor)

$h_{ef}' = c_{a1}/1.5 = 17.75$ in/1.5 = 11.83 in (limiting value of anchor effective embedment depth)

$s_1 =$ 8 in (center-to-center spacing of anchors in group)

s = 12 in (center-to-center spacing between anchor groups)

$A_{Nc} = [(1.5)(30 \text{ in}) + 8 \text{ in} + 17.75] \times 12 \text{ in} =$ 849.0 in^2

$A_{Nco} = (9)(11.83\ in)^2 =$ 1260.3 in^2

$A_{Nc}/A_{Nco} =$ 0.674

$\psi_{ec,N} =$ 1.0 (ACI 318-11 D.5.2.4, assuming anchor group not loaded eccentrically in tension)

$\psi_{ed,N} = 0.7 + (0.3)(c_{a1}/1.5h_{ef}) =$ 0.818 (ACI 318-11 Eqn. D-10, $c_{a,min} < 1.5h_{ef}$)

$\psi_{c,N} =$ 1.25 (ACI 318-11 D.5.2.6, cast-in anchors)

$\psi_{cp,N} =$ 1.0 (ACI 318-11 D.5.2.7, cast-in anchors)

$N_b = 16\lambda\sqrt{f'_c}h_{ef}^{5/3}$ (ACI 318-11 Eqn. D-7)

$\lambda =$ 1.0 (ACI 318-11 8.6.1, normal weight concrete)

$f'_c =$ 8700 psi

$N_b = (16)(1.0)(\sqrt{8700\ psi})(24\ in)^{5/3} =$ 432.3 kips

$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$

$= (674)(1.0)(0.818)(1.25)(1.0)(17.75\ kips) =$ 297.9 kips

Adhering to the conservative requirements set forth in ACI 318-08 D.3.3.3, the anchor design strength associated with concrete failure modes shall be taken as $0.75\Phi N_n$ where Φ is given in ACI 318-11 D.4.3(c) as 0.75 for cast-in anchors subjected to tension loads where supplementary reinforcement is present.

$0.75\Phi N_{cbg} = (0.75)(0.75)(297.9\ kips) =$ 167.6 kips

Loading on anchor group = 291.3 kips > 167.6 kips (NOT OK)

1" thick continous plate shall be embedded with the headed anchor bolts to expand the project concrete failure area of the bolt group and a larger bolt group shall be considered. See next page for calculations checking this scheme.

F.3 Concrete Breakout Strength − Anchors in Tension

A larger bolt group is considered than with previous calculations. Bolts shall be attached to a 1 in (25.4 mm) thick steel anchor plate that is also embedded in the concrete.

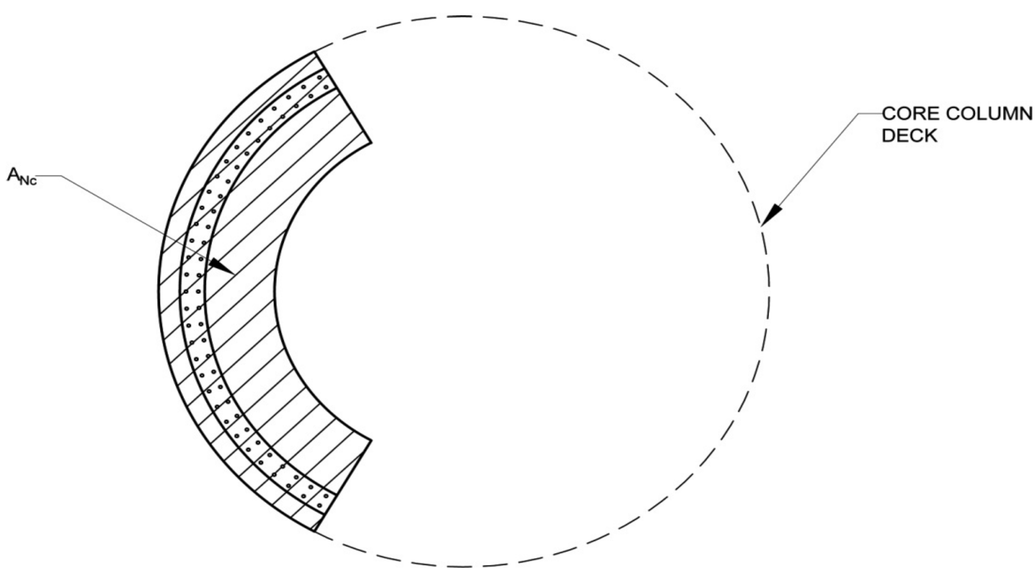
Tension loading on anchor group: 16312.5 kN = 3667.2 kips

x 1.35 = 4950.7 kips

According to ACI 318-11 Appendix D.5.2.1, the nominal concrete breakout strength of a group of anchors in tension shall not exceed:

$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (ACI 318-11 Eqn. D-4)

$A_{Nc} =$ 25238.2 in^2 (measured in AutoCAD)



$A_{Nco} = 9h_{ef}'^2$ (ACI 318-11 Eqn. D-5 and RD.5.2.3)

$h_{ef} =$ 30 in (anchor effective embedment depth)

$c_{a1} =$ 17.75 in (distance from edge to center of closest anchor)

$h_{ef}' = c_{a1}/1.5 = 17.75$ in/1.5 = 11.83 in (limiting value of anchor effective embedment depth)

$A_{Nco} = (9)(11.83\ in)^2 =$ 1260.3 in^2

$A_{Nc}/A_{Nco} =$ 20.03

$\psi_{ec,N} =$ 1.0 (ACI 318-11 D.5.2.4, assuming anchor group not loaded eccentrically in tension)

$\psi_{ed,N} = 0.7 + (0.3)(c_{a1}/1.5h_{ef}) =$ 0.818 (ACI 318-11 Eqn. D-10, $c_{a,min} < 1.5h_{ef}$)

$\psi_{c,N} =$ 1.25 (ACI 318-11 D.5.2.6, cast-in anchors)

$\psi_{cp,N} =$ 1.0 (ACI 318-11 D.5.2.7, cast-in anchors)

$N_b = 16\lambda\sqrt{f'_c}h_{ef}^{5/3}$ (ACI 318-11 Eqn. D-7)

$\lambda =$ 1.0 (ACI 318-11 8.6.1, normal weight concrete)

$f'_c =$ 8700 psi

$N_b = (16)(1.0)(\sqrt{8700\ psi})(30\ in)^{5/3} =$ 432.3 kips

$N_{cbg} = (A_{Nc}/A_{Nco})\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$

$= (20.03)(1.0)(0.818)(1.25)(1.0)(432.3\ kips) =$ 8855.0 kips

Adhering to the conservative requirements set forth in ACI 318-08 D.3.3.3, the anchor design strength associated with concrete failure modes shall be taken as $0.75\Phi N_n$ where Φ is given in ACI 318-11 D.4.3(c) as 0.75 for cast-in anchors subjected to tension loads where supplementary reinforcement is present.

$0.75\Phi N_{cbg} = (0.75)(0.75)(5719.6\ kips) =$ 4981.0 kips

Loading on anchor group = 4950.7 kips < 4981.0 kips (OK)

F.4 Pullout Strength − Anchor in Tension

According to ACI 318-11 Appendix D.5.3.1, the nominal pullout strength of a single anchor in tension shall not exceed

$N_{pn} = \psi_{c,P}N_p$ (ACI 318-11 Eqn. D-13)

$\psi_{c,P} =$ 1.0 (ACI 318-11 D.5.3.6, cracking at service level)

$N_p = 8A_{brg}f'_c$ (ACI 318-11 Eqn. D-14)

$A_{brg} =$ 9.766 in^2 (3.125"x3.125" head of stud)

$f'_c =$ 8700 psi

$N_p = (8)(9.766\text{ in}^2)(8700\text{ psi}) =$ 679.7 kips

$N_{pn} = (1.0)(679.7\text{ kips}) =$ 679.7 kips

Adhering to the conservative requirements set forth in ACI 318-08 D.3.3.3, the anchor design strength associated with concrete failure modes shall be taken as $0.75\Phi N_n$ where Φ is given in ACI 318-11 D.4.3(c) as 0.70 for cast-in anchors subjected to tension loads.

$0.75\Phi N_{pn} = (0.75)(0.70)(679.9\text{ kips}) =$ 356.8 kips

Loading on single anchor = 145.7 kips < 356.8 kips (OK)

F.5 Concrete Breakout Strength − Anchor Subjected to Shear

According to ACI 318-11 Appendix D.6.2.1, the nominal concrete breakout strength of a group of anchors in shear shall not exceed the following.

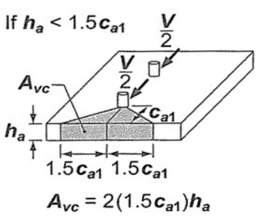
$V_{cbg} = (A_{Vc}/A_{Vco})\psi_{ec,V}\psi_{ed,V}\psi_{c,V}\psi_{h,V}V_b$ (ACI 318-11 Eqn. D-31)

c_{a1} = 17.75 in (distance to edge in direction of applied shear)

s_1 = 8 in (center-to-center spacing of anchors)

h_a = 42 in (diaphragm thickness)

ACI 318-11 Figure RD.6.2.1(b):



$A_{Vc} = 2(1.5c_{a1})h_a = (2)(1.5)(17.75 \text{ in})(42 \text{ in}) =$ 2236.5 in^2 (ACI 318-11 Fig. RD.6.2.1(b) Case 1)

$A_{Vco} = 4.5c_{a1}^2 = (4.5)(17.75 \text{ in})^2 =$ 1417.8 in^2 (ACI 318-11 Eqn. D-32)

$A_{Vc}/A_{Vco} =$ 1.577

$\psi_{ec,V} = 1/(1 + 2e'_V/3c_{a1}) =$ 0.906 (ACI 318-11 Eqn. D-36, assuming anchor group loaded eccentrically in shear where e'_V = 2.75 in)

$\psi_{ed,V} =$ 1.0 (ACI 318-11 Eqn. D-37, $c_{a2} \geq 1.5c_{a1}$)

$\psi_{c,V} =$ 1.2 (ACI 318-11 D.6.2.7, cracked concrete with reinforcement between anchor and edge)

$\psi_{h,V} =$ 1.0 (ACI 318-11 D.6.2.8, $h_a > 1.5c_{a1}$)

$V_b = (7(l_e/d_a)^{0.2}\sqrt{d_a})\lambda\sqrt{f'_c}(c_{a1})^{1.5}$ (ACI 318-11 Eqn. D-33)

$d_a =$ 2 in

$l_e = 8d_a =$ 16 in (ACI 318-11 D.6.2.2)

$\lambda =$ 1.0 (ACI 318-11 8.6.1, normal weight concrete)

$f'_c =$ 8700 psi

$V_b = [(7)(16\text{ in}/2\text{ in})^{0.2}(\sqrt{2\text{ in}})](\sqrt{8700\text{ psi}})(17.75\text{ in})^{1.5} =$ 104.7 kips

$V_{cbg} = (A_{Vc}/A_{Vco})\psi_{ec,V}\psi_{ed,V}\psi_{c,V}\psi_{h,V}V_b$

$= (1.577)(0.906)(1.0)(1.2)(1.0)(104.7\text{ kips}) =$ 179.6 kips

Adhering to the conservative requirements set forth in ACI 318-08 D.3.3.3, the anchor design strength associated with concrete failure modes shall be taken as $0.75\Phi V_n$ where Φ is given in ACI 318-11 D.4.3(c) as 0.70 for post-installed anchors subjected to shear loads.

$0.75\Phi V_{cbg} = (0.75)(0.70)(179.6\text{ kips}) =$ 94.28 kips

Loading on anchor group = 7.175 kips < 94.28 kips (OK)

F.6 Concrete Pryout Strength − Anchor Subjected to Shear

According to ACI 318-11 Appendix D.6.3.1(b), the nominal pryout strength for a group of anchors in shear shall not exceed

$V_{cpg} = k_{cp}N_{cbg}$ (ACI 318-11 Eqn. D-41)

$k_{cp} =$ 2.0 (ACI 318-11 D.6.3.1, $h_{ef} \geq 2.5$ in)

$N_{cbg} =$ 297.9 kips (ACI 318-11 Eqn. D-4, calculated for tension breakout)

$V_{cpg} = (2)(297.9 \text{ kips}) =$ 595.8 kips

Adhering to the conservative requirements set forth in ACI 318-08 D.3.3.3, the anchor design strength associated with concrete failure modes shall be taken as $0.75\Phi V_n$ where Φ is given in ACI 318-11 D.4.3(c) as 0.70 for post-installed anchors subjected to shear loads.

$0.75\Phi V_{cpg} = (0.75)(0.70)(595.8 \text{ kips}) =$ 312.8 kips

Loading on anchor group = 7.175 kips < 312.8 kips (OK)

F.7 Concrete Side-Face Blowout Strength − Anchor in Tension

According to ACI 318-11 Appendix D.5.4.2, h_{ef} must be greater than $2.5c_{a1}$ to necessitate side-face blowout strength calculation.

$c_{a1} =$ 17.75 in

$h_{ef} =$ 30 in

$2.5c_{a1} =$ 44.38 in > 30 in

Therefore, concrete side-face blowout will not govern.

F.8 Plate Bearing Strength at Bolt Holes

f_y = 36 ksi
f_u = 58 ksi
Plate thickness = 1 in

Hole diameter = d + 1/16 in = 2 in + 1/16 in = 33/16 in

As both holes are near edge of plate, use

$L_c = L_e$ - h/2 = 4 in - (33/16 in)/2 = 2.969 in

tf_u = (1 in)(58 ksi) = 58 kips/in

$R_n = 1.2L_ctf_u$ = (1.2)(2.696 in)(58 kips/in) = 206.6 kips (AISC Eqn. J3-6a)

Upper limit:

$2.4dtf_u$ = (2.4)(2 in)(58 kips/in) = 278.4 kips > 206.6 kips

Plate bearing strength = Σ bearing values = (2)(206.6 kips) = 413.3 kips

→ Compare this bearing capacity value to bolt shear strength.

The anchor shear capacities are less than the bearing capacity of the plate, therefore, the anchor capacities govern the strength of the steel components in the connection.

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