Final Project Report

Preliminary Design and Cost Assessment of the Use of Composite Reinforcements in Place of Traditional Steel Rebar and Post-tensioning Tendons for Concrete Floating Wind Turbine Foundations to Lower Cost, Increase Durability, and Reduce the Carbon Footprint

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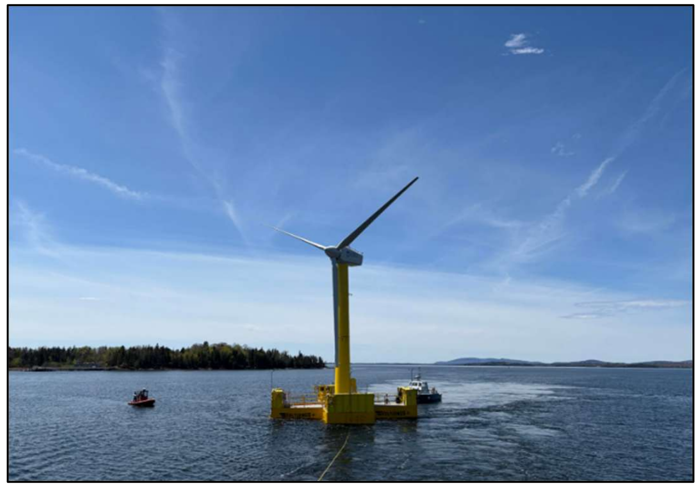

Figure 1: UMaine VolturnUS+ Floating Concrete Foundation Supporting a Wind Turbine Deployed in 2025

UMaine completed a preliminary design and cost assessment to explore replacing traditional steel rebar and post-tensioning tendons with composite reinforcements in floating concrete wind turbine foundations. The aim was to reduce overall costs, improve durability, and lower the carbon footprint.

The study evaluated the use of three alternative reinforcing materials for a UMaine developed VolturnUS+ 22MW floating concrete foundation (**see Figure 1**) with a total concrete mass of 26,250MT, maximum width of 84m, and draft of 12m. The floating foundation is a steel reinforced concrete barge designed to resist wave and wind loading, buoyancy, live loads, and self-weight. A commercial scale 1GW farm would require about 45 of these units. Each foundation uses roughly 1800MT US tons of steel rebar, totaling 81,000MT tons for the full 1GW farm. Traditional steel reinforcement offers strength and ductility but is prone to corrosion in chloride-rich marine environments, which necessitates thick concrete cover and adds weight.

Three alternative materials were evaluated for their feasibility in this structure:

1. Glass Reinforced Fiber Polymer (GFRP) bars to replace steel bars and reduce concrete cover and reduce labor costs
2. Polyethylene Synthetic Macro Structural Fibers to reduce steel rebar reinforcement
3. Carbon fiber Post-tensioning (PT) Tendons to replace Steel PT Tendons

A preliminary design for a 22 MW concrete floating wind foundation was developed using both steel and composite (GFRP) reinforcement which showed several advantages. The design followed offshore concrete standards from the American Bureau of Shipping and included buoyancy and stability analyses based on naval architecture principles. Cost estimates were generated using UMaine's internal cost and production tools, incorporating input from project partners.

Key Findings

1. **Use of GFRP Reinforcement**
    - **Structural Application**: GFRP bars are advantageous in non-watertight elements like the top slab and interior bracing walls (**see Figure 2**), where reduced stiffness is acceptable. In watertight walls, however, GFRP's low stiffness makes crack control challenging. In the non-watertight areas, the high strength of the GFRP bars is actually more efficient than steel.

- **Weight Reduction**: A 1-inch reduction in concrete cover (vs. steel) is permissible per international design codes for the top slab and interior walls, resulting in a 7.3% decrease in total structural weight.

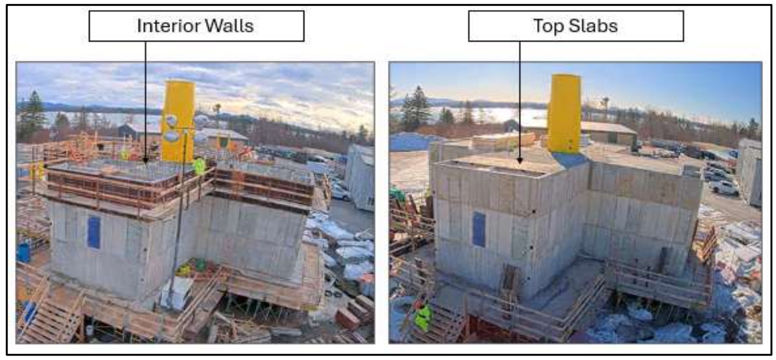

Figure 2: VolturnUS+ Interior Walls and Top Slabs

- **Cost Savings**: The hybrid GFRP and steel design showed a 7-11% cost savings for the foundation compared to the all-steel alternative due to material and labor savings. These costs are based on material and labor input costs from contractors and GFRP suppliers.
- **Reinforcement Quantities**: Each unit used ~196 metric tons of GFRP. A 1 GW farm (45 units) would require ~<u>8,820 metric tons of GFRP</u>. U.S. lease areas (California and Maine) include ~12 GW of floating wind potential, indicating a strong market opportunity for concrete-GFRP foundations. The global market is larger.
- **Structural Performance**: The hybrid system met all strength and serviceability criteria. Durability and long-term maintenance benefits of FRP were not included in this analysis.

2. **Synthetic Macrofibers**
    - Added to reduce steel reinforcement and enhance ductility, crack control, and fire resistance
    - Provided 10–20% contribution to flexural and shear strength in some sections.
    - Minimal cost increase; net cost/performance benefit seen.
    - Not suitable as primary reinforcement for this application.
3. **Carbon Fiber Tendons**
    - Considered for post-tensioning applications.
    - Offer strong mechanical performance but require major redesigns (e.g., external PT systems).
    - Not able to fully evaluate current scope due to high cost and complexity.

Based on the findings of this study, incorporating GFRP bars and synthetic macrofibers in floating concrete wind turbine foundations has strong potential to improve structural efficiency and durability while achieving measurable cost savings. However, to fully implement these solutions, a more detailed structural design study is required, along with an evaluation of the supply chain's ability to deliver the large volumes of GFRP needed for serial production. Additionally, labor savings associated with the reduced weight of FRP reinforcement—an important factor in overall cost reduction—should be further validated with contractors. Carbon fiber tendons also warrant additional investigation, as their high cost and the substantial design modifications they require must be carefully weighed against potential performance benefits.

1. INTRODUCTION

The project aims to assess the feasibility and economic benefits of replacing traditional steel reinforcement with composite materials, specifically Glass Fiber Reinforced Polymer (GFRP) reinforcement, Carbon Fiber Reinforced Polymer (CFRP) strands, and macro synthetic structural fibers, in concrete foundations for floating offshore wind turbines. In the highly corrosive marine environment, where weight is also a critical factor, using non-metallic reinforcements has the potential to improve durability, reduce material demands, and lower overall costs.

The primary goals of this initiative are to reduce construction and lifecycle costs by eliminating the need for heavy, corrosion-prone steel; to increase the long-term durability of floating concrete structures; to reduce embodied carbon by lowering concrete volume; and to assess the commercial opportunity presented by a growing offshore wind sector.

The principal challenge to adoption remains the high upfront capital cost. A technoeconomic analysis is required to demonstrate that alternative composite material systems can be cost-competitive with traditional steel-reinforced foundations. While design codes have begun to recognize composite reinforcements, further engineering validation and design adaptation are required to support widespread deployment.

To address these challenges, the University of Maine has leveraged its DOE-funded design, cost and production models and extensive research history in offshore wind to evaluate the performance and cost implications of the alternative materials in floating foundation designs for wind turbines.

UMaine has developed, patented, and deployed two floating concrete foundations called the VolturnUS. This project involved modifications to a 22MW VolturnUS+ concrete barge foundation baseline all-steel rebar design to include integration of GFRP reinforcement, CFRP strands, and synthetic macrofibers. These changes were assessed through structural analysis, preliminary design calculations, and material property data provided by Mateenbar (GFRP), Tokyo Rope (CFRP), and Sika (synthetic fibers). Applicable standards such as the following will guide design decisions:

- American Bureau of Shipping (ABS) Guide for Building and Classing Floating Offshore Wind Turbines (January 2024) [1]
- Det Norske Veritas (DNV) Offshore Concrete structures (DNV-ST-C502) [2]
- American Concrete Institute (ACI) Building Code Requirements for Structural Concrete (ACI-318-19) [3]
- ACI Building Code Requirements for Structural Concrete Reinforced with Glass Fiber-Reinforced Polymer (GFRP) Bars (ACI 440.11-22) [4]
- ACI Guide to Design with Fiber-Reinforced Concrete (ACI 544.45-18) [5]
- Florida Department of Transportation (FDOT) Fiber Reinforced Polymer Guidelines (FRPG) FDOT Structures Manual (Vol. 4) [6]

Cost estimates were generated using UMaine's internal cost and production tools, incorporating input from project partners. A summary of the next steps to further evaluate the implementation of these materials in these applications is also provided.

2. STEEL AND GFRP REINFORCEMENT STRUCTURAL DESIGN FOR VOLTURNUS + CONCRETE FOUNDATION

2.1 OVERVIEW

This section presents the results of a structural assessment at key design locations on a 22MW floating wind turbine concreate foundation(see Figure 3). The goal of this work is to get a reasonable estimate for the quantities of reinforcement which were then used as input in a cost estimate.

For this preliminary study, wall and slab sections of the hollow concrete foundation were designed only for local forces due to self-weight, live loads, and hydrostatic pressures which is a reasonable assumption based on Maine's past experience. The walls are also subjected to global bending loads due to the wind turbine and motions of the hull. However, these loads are largely carried by post-tensioning tendons.

2.2 REINFORCEMENTS

Two reinforcement materials were evaluated: traditional grade 60 uncoated steel reinforcement and Mateenbar 60 GFRP reinforcement for use in the walls and slabs which make up the hollow concrete floating structure (see Figure 4). Appendix A presents the technical data for Mateenbar 60.

2.3 CONCRETE COVER REQUIREMENTS

The use of GFRP reinforcement provides the opportunity to reduce concrete cover requirements which can lead to a decrease in concrete weight. Concrete cover values are presented in Table 1 and are based on applicable design standards. A 1in reduction was selected for the GFRP designs based on FDOT and DNV guides which are more advanced and specifically address the use of GFRP for marine structures.

2.4 STRUCTURAL DESIGN CRITERIA

The concrete walls and slabs were evaluated for serviceability and strength conditions following ABS Guide for Building and Classing Floating Wind Turbine Foundations.[1]

Strength conditions follow standard concrete design guides found in ACI 318[3] which allow the concrete to crack under unfactored loads to meet the required moment capacity.

Serviceability requirements are dictated by the ABS Guide for Building and Classing Floating Wind Turbine Foundations. For watertight liquid containing concrete walls resisting hydrostatic pressures, the structure must meet the following criteria outlined in 7-3-5.9.2:

5.9.2 Liquid-Containing Structures

The following criteria are to be satisfied for liquid-containing structures to verify adequate design against leakage.

i) The reinforcing steel stresses are to be in accordance with 7-3/5.9.1.

ii) The compression zone is to extend over 25% of the wall thickness or 205 mm (8 in), whichever is less.

iii) There is no membrane tensile stress unless other construction arrangements are made, such as the use of special barriers to prevent leakage.

[1] https://ww2.eagle.org/content/dam/eagle/rules-and-guides/current/offshore/195_fowti/fowt-guide-july20.pdf

Allowable Tensile Stresses for Prestress and Reinforcing Steel to Control Cracking

Stage	*Loading*	*Allowable Stress, MPa (ksi)*	
		Reinforcing Steel, f_s	***Prestressing Tendons, D_{ps}***
Construction: where cracking during construction would be detrimental to the completed structure	All loads on the structure during construction	160 (23.0)	130 (18.5)
Construction: where cracking during construction is not detrimental to the completed structure	All loads on the structure during construction	210 (30.0) or 0.6 f_y, whichever is less	130 (18.5)
Transportation and installation	All loads on the structure during transportation and installation	160 (23.0)	130 (18.5)
At offshore site	Permanent and variable loads plus environmental loads	0.8 f_y	
f_y = yield stress of the reinforcing steel f_s = allowable stress in the reinforcing steel D_{ps} = increase in tensile stress in prestressing steel with reference to the stress at zero strain in the concrete.			

2.5 Structural Philosophy for Preliminary Design

Two methods were implemented to size the concrete wall and slab sections.

- Method 1 (uncracked under service loads): 1) the wall or slab is designed to remain uncracked which meets ABS watertightness requirements under unfactored loads and resistances. 2) The reinforcement is sized for a cracked section to meet ultimate strength requirements under factored loads and resistances. The GFRP design used the conservative nominal moment capacity estimate outlined in ACI-440.11-22 Chapter R22.3.1.1 [4].
- Method 2 (cracked under service loads): 1) the wall or slab is designed for a cracked section to meet watertightness requirements under unfactored loads 2) the ultimate strength requirements are checked under unfactored loads. The watertightness requirements are applicable for all exterior walls and slabs below the water line and some internal walls containing ballast. Watertightness criteria from ABS require that a compression zone be maintained within the concrete section under bending and axial loads.

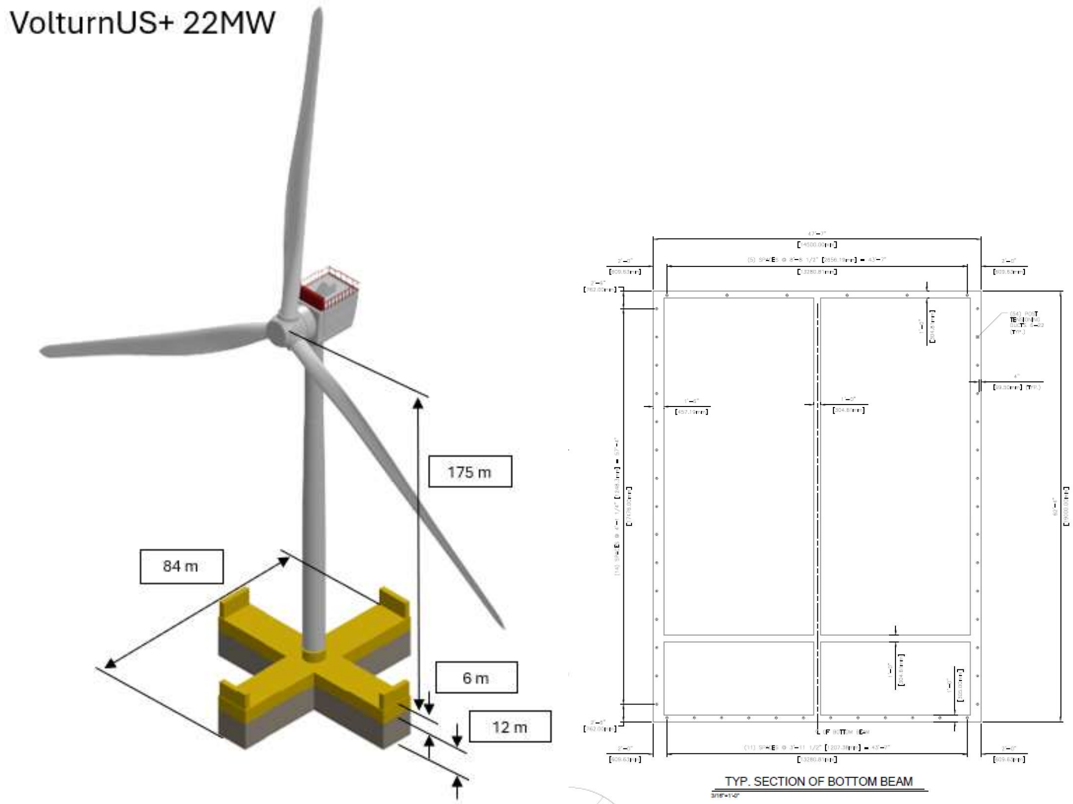

Figure 3: (Left) UMaine patented VolturnUS+ 22MW (right) Section cut through one leg of the foundation showing hollow void and walls and slabs analyzed in this study.

Figure 4: Steel Reinforcement (left) GFRP Reinforcement (right)

Table 1: Concrete Cover as Dictated by Different Design Guides

Comparison of Concrete Cover Requirements: 50 yr Design Life, Exposure: Splash Zone, XS3						
	Bar Size	Bar Diameter	ABS	DNV	ACI	FDOT
	-	in	in	in	in	in
Steel	8	1	2.5	2.4	2	3
GFRP	10	1.378	N/A	1.4	2	2
Reduction			N/A	1.0	0	1
Comparison of Concrete Cover Requirements: 50 yr Design Life, Exposure: Fully Submerged Zone, XS2						
	Bar Size	Bar Diameter	ABS	DNV	ACI	FDOT
	-	in	in	in	in	in
Steel	8	1	2	2.0	2	3
GFRP	10	1.378	N/A	1.4	2	2
Reduction			N/A	0.6	0	1

2.6 Structural Design Results

For this preliminary study, UMaine analyzed the bottom and top slabs to bound the reinforcement quantities required for the foundation. A full structural design of every wall and slab in the foundation is typically a 6-month effort including Finite Element Analys which was beyond the scope of this small study.

Each slab was subjected to a specific pressure:

- the bottom slab experienced both hydrostatic and hydrodynamic pressures
- the top slab was subjected to its dead load and an industrial live load.

See Figure 5 for a sketch depicting the slab loading. Shell theory for a rectangular shell subjected to uniform pressure was then applied to determine the bending load in each slab section. Using a one-foot-wide strip of the slab, both concrete design methods were applied to determine required steel and GFRP reinforcement.

Table 2 presents the bottom slab results using concrete design method one. Steel reinforcement results are shown in the second-to-last column and GFRP results are shown in the last column. Each row represents a design property used in determining the reinforcement ratio area per foot, presented in the last row.

Table 3 presents the bottom slab results using concrete design method two, with the same design properties as in Table 2. However, it separates the steel and GFRP reinforcement results into multiple columns to account for multiple concrete thicknesses options.

Table 4 presents the top slab results using concrete design method one. The layout follows the same format as Table 2. Appendix B-G presents Mathcad example calculations for steel and GFRP using both methods.

Key Observations

3. The top slab and interior walls do not require ABS watertightness conditions to be met. The design is driven by strength requirements. This is an ideal place for GFRP reinforcement. GFRP reinforcement has a rupture strength approximately 2.4 times greater than the yield strength of

steel reinforcement. As shown in Table 2 and Table 4, **this higher strength translates to requiring less GFRP reinforcement for the same section under Concrete Design Method 1 (see final rows)**. The top slab design is driven by ACI's minimum required reinforcement because the load is quite low, and the span is selected for the hydrodynamic and hydrostatic loads on the bottom slab. **The top slab also shaved 2" of concrete from the top slab.**

4. The bottom slab and exterior walls are designed to resist extreme water pressures. Concrete Design Method 2 yields the lightest structure. **GFRP has difficulty providing a competitive design with Method 2 due to the decreased bar stiffness which results in difficulty maintaining a compression zone required by the ABS code.** Therefore, GFRP does not appear to make economic sense to include in the watertight bottom slabs or vertical exterior walls. This is because steel reinforcement has a modulus of elasticity approximately 3.3 times greater than that of GFRP reinforcement. As shown in Table 3, this results in steel requiring less reinforcement than GFRP under Concrete Design Method 2 (see final row).
5. Both steel and GFRP reinforcement requirements decrease when concrete is designed to remain uncracked under ABS watertightness conditions. However, this design approach necessitates a thicker concrete section with no savings in weight and is not ideal from a buoyancy perspective.

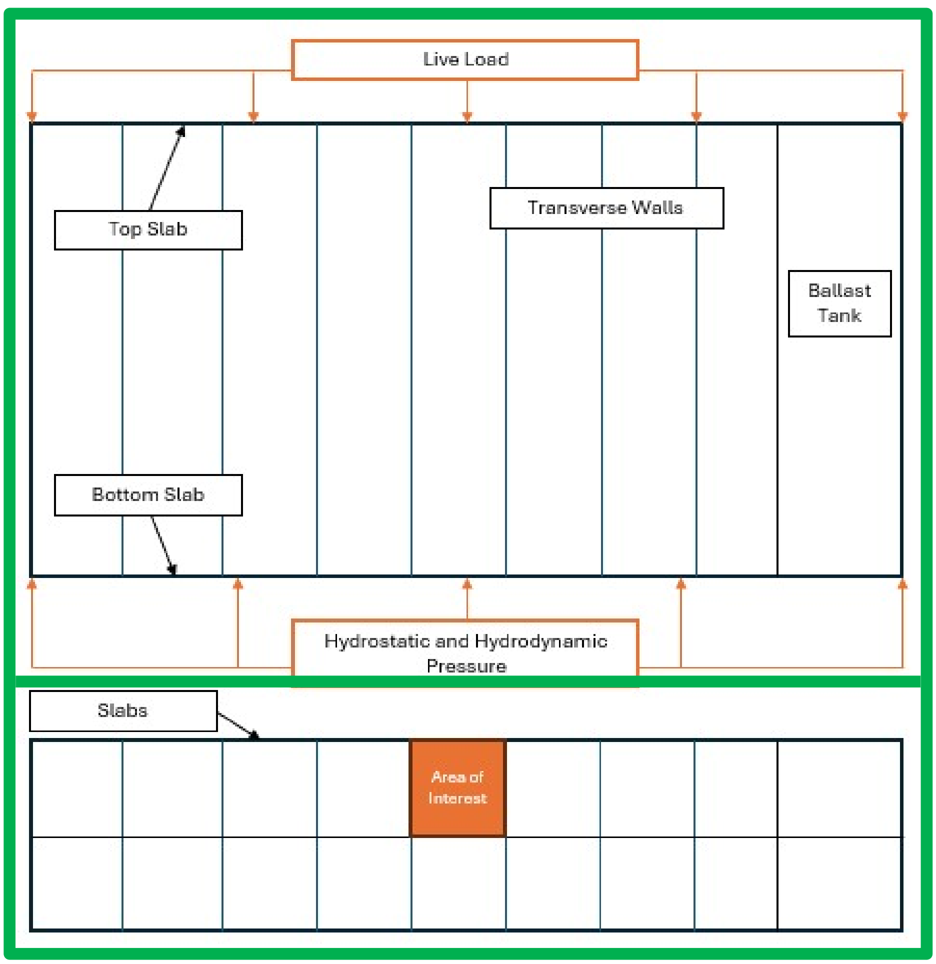

Figure **5**: (top) Elevation view of single leg of the concrete foundation and (bottom) plan view of the internal and external walls

Table 2: Bottom Slab Concrete Design Method 1 (uncracked under service loads)

Bottom Slab Method 1: Uncrack Concrete for Watertightness Requirements. Reinforcement Sized for Ultimate Strength Conditions.			
Reinforcement Material	Unit	Steel	GFRP
Clear Cover	in	2.5	1.5
Concrete Compressive Strength	psi	6000	6000
Concrete Rupture Strength	psi	581	581
Applied Moment	kip*ft	40	40
Cracking Moment	kip*ft	42.7	42.7
Applied / Cracking Moment	-	0.937	0.937
		Uncracked	Uncracked
Thickness	in	21	21
Reinforce Size	#	8	7
Reinforcement Diameter	in	1	0.949
Reinforcement Spacing	in	9	10
Reinforcement Area	in^2	0.785	0.600
Reinforcement Area Per Foot	in^2/ft	1.047	0.720

Table 3: Bottom Slab Concrete Design Method 2 (cracked under service loads)

Bottom Slab Method 2: Allow Concrete to Crack for Watertightness Requirements.							
Reinforcement Material	-	Steel			GFRP		
Clear Cover	in	2.5			1.5		
Concrete Compressive Strength	psi	6000			6000		
Concrete Rupture Strength	psi	581			581		
Applied Moment	kip*ft	40			40		
Cracking Moment	kip*ft	42.7	31.4	21.8	42.7	31.4	21.8
Applied / Cracking Moment	-	0.937	1.274	1.835	0.937	1.274	1.835
		Uncracked	Cracked	Cracked	Uncracked	Cracked	Cracked
Thickness	in	21	18	15	21	18	15
Reinforce Size	#	10	10	10	10	10	10
Reinforcement Diameter	in	1.27	1.27	1.27	1.378	1.378	Not enough depth in section for all reinforcement layers required.
Reinforcement Spacing	in	6.5	7.25	8	3.25	3.5	
Reinforcement Area	in^2	1.27	1.27	1.27	1.27	1.27	
Reinforcement Area Per Foot	in^2/ft	2.339	2.097	1.900	9.378	8.709	
					GFRP is a Double Stacked Mat		

Table 4: Top Slab Concrete Design Method 1 (uncracked under service loads)

Top Slab			
Reinforcement Material	Unit	Steel	GFRP
Clear Cover	in	2.5	1.5
Concrete Compressive Strength	psi	6000	6000
Concrete Rupture Strength	psi	581	581
Applied Moment	kip*ft	5.7	5.4
Cracking Moment	kip*ft	13.9	9.7
Applied / Cracking Moment	-	0.410	0.557
		Uncracked	Uncracked
Thickness	in	12	10
Reinforce Size	#	5	4
Reinforcement Diameter	in	0.625	0.551
Reinforcement Spacing	in	9	8.25
Reinforcement Area	in^2	0.307	0.200
Reinforcement Area Per Foot	in^2/ft	0.409	0.291

2.7 Summary

A preliminary structural analysis was conducted for the VolturnUS+ 22MW floating concrete foundation, focusing on the slabs and walls that form the primary load-bearing structure of the hollow floating concrete foundation. The purpose of this assessment was to establish initial quantities of concrete and reinforcement to inform the cost modeling and comparative analysis between conventional steel and alternative composite reinforcements.

The analysis concentrated on localized loading conditions that typically govern reinforcement design. These include hydrostatic pressures on the bottom slab and external walls, self-weight, and superimposed service loads. The approach applied standard offshore concrete design methodologies, including provisions from relevant ABS codes and naval architecture principles. The key outcome was to develop conservative but reasonable reinforcement schemes that reflect expected structural demands under both operational and extreme loading scenarios.

A key focus of the study was evaluating the structural implications of using Glass Fiber Reinforced Polymer (GFRP) bars as a partial replacement for traditional steel reinforcement. The performance of GFRP bars was assessed in various structural zones based on their exposure to water pressure, crack control requirements, and stiffness demands.

GFRP bars were found to be advantageous in non-watertight areas such as the top slab and interior bracing walls as shown in green below in Figure 6. In these components, the reduced stiffness of GFRP compared to steel does not significantly impact serviceability or structural performance. Moreover, the high tensile strength of GFRP can be fully utilized, often making it more efficient than steel in tension-controlled sections. International concrete design codes permit a 1-inch reduction in concrete cover when using GFRP due to its corrosion resistance. This leads to a measurable reduction in total concrete volume and structural weight.

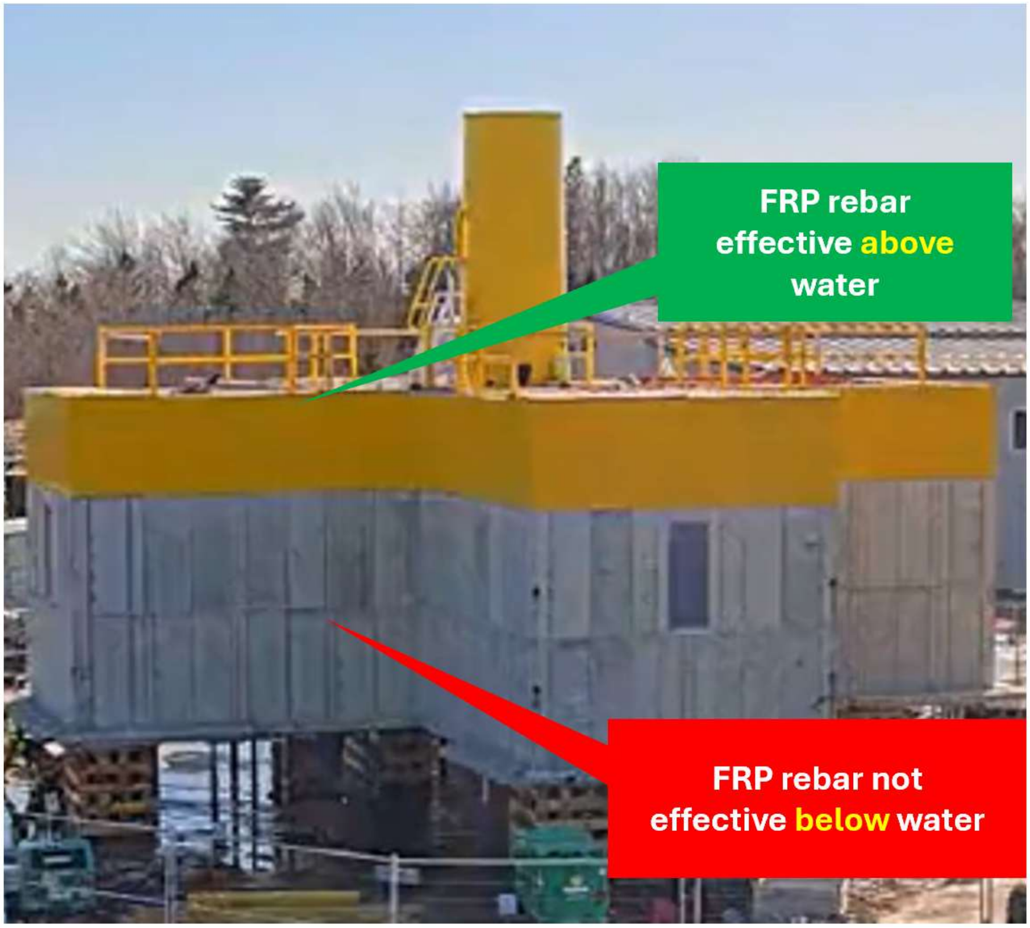

Figure 6 Preferred Reinforcing Types for Locations within the VolturnUS Concrete Foundation

Conversely, in watertight sections such as the bottom slab and external vertical walls (shown in red above), GFRP's lower modulus of elasticity raises concerns related to crack width control under hydrostatic loads. Maintaining watertightness in these components is critical for the long-term durability and performance of the floating foundation. As a result, conventional steel reinforcement appears more favorable for these elements, particularly where post-tensioning is not applied.

The structural configuration that emerged as most promising involves a hybrid reinforcement approach. Steel is used in elements that resist hydrostatic pressure and require high stiffness and crack control, such as the bottom slab and exterior walls. GFRP bars are used in the top slab and interior partitions, where watertightness is not essential, and stiffness demands are lower. This hybrid system optimizes both structural performance and cost, as reflected in the preliminary cost analysis presented in Section 5.

In conclusion, the structural assessment supports the feasibility of a mixed-material reinforcement strategy in the VolturnUS+ floating foundation. While further detailed analysis and validation are required for final design development, this early-stage evaluation provides a reasonable basis for estimating quantities and understanding how different reinforcement materials can be most effectively deployed across the structural system.

3. Synthetic Macrofibres Structural Design

Synthetic macrofibers in concrete offer several advantages, particularly in enhancing durability and crack control. They act as supplementary reinforcement, improving ductility and crack control, and can enhance fire resistance by mitigating explosive spalling. However, they have limitations. Synthetic macrofibers contribute little to structural strength and cannot replace primary steel reinforcement.

A structural analysis was conducted to compare steel reinforced concrete and hybrid fiber-steel reinforced concrete at various thicknesses**. Table 5** presents the results. Steel reinforcement results are shown in the third column and macro fiber with steel reinforcement results are shown in the last column. Each row represents a design property used in determining the percent capacity increase and percent reinforcement area decrease, presented in the last two rows. With the inclusion of synthetic macrofibers the steel area dropped 16% while keeping the same moment capacity. The fibers help to bridge the crack induced during bending.

To achieve the designed fiber reinforced residual concrete strength approximately 3 lbs of synthetic macrofibers is required, costing approximately $12 per cubic yard of concrete. This is an 8.5% premium for each cubic yard of concrete but reduces steel reinforcement costs by 16%. The synthetic marcofibers also increases shear capacity by 35%, which could have significant benefit for global shear loading which was not investigated in this preliminary study. Appendix H presents Mathcad example calculations for hybrid fiber-steel.

Table 5: Steel and Hybrid Fiber-Steel Results

Synthetic Fiber Results			
Design	-	Steel Bars Only	Steel Bars w/ Macro Structural Fibers
Slab	-	Top	Top
Concrete Thickness	in	12	12
Reinforcement Area/ft	in^2/ft	0.409	0.342
Reinforcement Size	in	5	5
Reinforcement Spacing	in	9	10.75
Fiber Reinforced Concrete Residual Strength	psi	120	120
Factored Moment Capacity	kip*ft	11.1	11.2
Percent Capacity Increase	%	1	
Percent Reinforcement Area Decrease	%	16	
Shear Stress	psi	155	208
Shear Strength	Kip/ft	14.9	20.0
Percent Capacity Increase	%	35	

4. System Design for Steel Rebar and GFRP Concrete Floating Foundation

Using the results from the structural assessment in Chapter 2, a modified version of the 22MW VolturnUS+ floating concrete foundation system was developed to evaluate the structural and economic impacts of reducing concrete wall/slab thickness in top slabs and interior walls with the use of GFRP reinforcement. The results are presented in Table 6.

The modified design has GFRP in lieu of steel in the top slab and all interior walls where watertightness is not required. The new design maintains the same hull displacement, draft, freeboard, and external geometry, ensuring buoyancy and stability characteristics are preserved while optimizing material efficiency.

The most significant outcome is a 7.3% reduction in total hull structural mass, achieved by decreasing the volume of reinforced concrete. This translates to 1,852 metric ton savings in concrete, which decreases material and labor costs, lowers embodied carbon, and simplifies transportation and assembly logistics.

To offset the decreased structural mass and maintain the desired draft and system stability, ballast mass was increased by 24.3%, rising from 6,605 to 8,433 metric tons. Hydrostatic stability requirements were also met and are presented in the table.

Mass reductions were especially evident in structural components:

- Internal wall sections (across 28 walls in four legs) experienced a 17.9% mass reduction.
- Top slabs (four in total) experienced a 19.0% mass reduction.

Table 6: 22MW VolturnUS+ Steel and GFRP System Comparison

22MW VolturnUS+				
Property	Units	Steel System	Steel & GFRP System	% Diff.
Hull Displacement	m3	34,227	34,227	0.0%
Hull Structural Mass	mt	26,250	24,398	7.3%
Hull Concrete Density	kg/m3	2,478	2,478	0.0%
Ballast Mass	mt	6,605	8,433	24.3%
Draft	m	12	12	0.0%
Freeboard	m	6	6	0.0%
Leg Radius/Radial Spacing of Columns	m	42	42	0.0%
Hull Width/Radial Column Diameter	m	19	19	0.0%
Leg/Bottom Beam Height	m	18	18	0.0%
Vertical Venter of Gravity (KG)	m	15.24	14.93	2.1%
Vertical Venter of Buoyancy (KB)	m	6.05	6.05	0.0%
Metacentric Radius (BM)	m	23.85	23.43	1.8%
Metacentric Height (GM)	m	14.66	14.55	0.7%
Concrete Weight	mt	26,250	24,398	7.3%
Heel Angle at Rated Thrust	deg	6.0	6.0	0.8%
Mass of Internal Dia (7 walls/leg x 4 legs)	mt	6,571	5,492	17.9%
Mass of Top Slab (1 slab/leg x 4 legs)	mt	1,708	1,412	19.0%

5. Summary of Cost Analysis

The cost estimate was developed using a proprietary detailed cost model created by UMaine under a prior Department of Energy-funded project. This model incorporates data receive in quotes from industry, RS Means data, and input from experienced collaborators and partners. The analysis assumes a mature production scenario for a 1 GW offshore wind farm, with fabrication occurring at a dedicated facility

Production is based on an output rate of about 25 floating foundations per year after a 6–12-month ramp-up, with construction taking place 24 hours per day, 5 days per week, throughout the year. The material cost estimate includes items such as cement, aggregate, reinforcing steel (rebar), post-tensioning strands, coatings, piping, electrical components, and instrumentation. Labor costs account for construction personnel including equipment operators, skilled trades, quality control staff, engineers, and management. The model is built from the bottom up based on the materials and procedures.

Material quantities for both the conventional all-steel rebar design and a hybrid design using both GFRP and steel rebar are summarized in **Table 7**.

Table 7. Quantities used for Cost Estimation (per platform)

Item	Unit	Baseline Design Steel	Hybrid Design Steel/GFRP	% dif	Notes
Concrete	m^3	10,593	9,846	7.1%	Reduced for reduction in cover
Steel Rebar	MT	1,867	955	48.9%	For Hybrid: Only steel in bottom slab and exterior walls.
FRP Rebar	MT	-	196	-	For Hybrid: GFRP bars only in top slabs and interior non-watertight walls
PT	MT	209	209	0%	

The cost analysis effort showed that the hybrid steel-GFRP floating turbine foundation is approximately 6.9% to 10.7% less expensive than the all-steel version. This is a significant reduction. These cost savings stem from reductions in both material quantities and labor required to place concrete and rebar. In particular, the hybrid design requires less concrete volume and rebar weight, resulting in lower overall costs.

6. Conclusions

This study conducted by UMaine explored the feasibility of using composite reinforcement systems—namely GFRP bars, synthetic macrofibers, and carbon fiber tendons—as alternatives to traditional steel reinforcement and post-tensioning systems in large-scale floating concrete wind turbine foundations. The goal was to assess potential benefits in cost, durability, and weight savings, while evaluating structural performance at commercial production scale. UMaine has been developing concrete floating foundations for about 15 years and has deployed two prototypes with steel reinforcing systems which is the industry norm.

A preliminary design was completed for the 22 MW VolturnUS+ floating foundation—a steel-reinforced concrete barge with a concrete mass of 26,250 metric tons, 84 meters in beam, and 12 meters in draft. UMaine recently deployed a demo unit offshore in May 2025 for system testing. This structure is intended to resist environmental and operational loads and would be serially produced for a 1 GW wind farm at a rate of 25 units per year. Each unit incorporates around 1,800 metric tons of steel rebar, or 81,000 metric tons total across a commercial-scale deployment. Given the marine exposure, the corrosive risks of steel reinforcement were a key factor in evaluating alternative materials.

Three types of composite reinforcements were reviewed. GFRP bars, used in hybrid designs alongside steel, demonstrated notable structural and economic advantages. They allowed reduced concrete cover in select structural elements, cutting total weight by approximately 7.3%, and resulted in estimated cost savings of 6.9% to 10.7% per foundation. These savings were primarily due to reduced material volumes and associated labor costs. However, their use must be tailored to non-watertight components due to stiffness limitations affecting crack control in watertight sections. **For a single full 1 GW farm, the hybrid design would require approximately 8,820 metric tons of GFRP—highlighting the need to assess supply chain readiness for scaled delivery. 7GW of floating wind lease areas have been leased in the US alone.**

Two other alternative materials were investigated. Synthetic macrofibers were also integrated to supplement reinforcement, offering 10–20% contribution to strength and enhancing crack control and ductility. While they cannot replace primary reinforcement, they provide modest performance gain with cost benefits despite a premium on the concrete mix. Carbon fiber tendons were also considered as a possible substitute but were not evaluated in detail in this limited study.

In summary, integrating GFRP bars and synthetic macrofibers presents a viable path to enhancing the cost-effectiveness and durability of floating wind foundations. The technology readiness level of these composite materials proposed is mature with many demonstrations and commercial applications completed and design standards adopting their use. To advance the use of these materials in a floating wind project, further engineering validation and market readiness analysis are required—particularly regarding labor pricing assumptions and the ability to source and install large quantities of FRP materials at scale.

7. Works Cited

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A. Mateenbar 60 GFRP Technical Data

Proudly Made in the USA
from local and imported materials

Technical Data

Mateenbar™ 60 GFRP Rebar

Mateenbar™ 60 (ASTM D8505, CSA-S807 Grade III)

	Units	#2 (6mm)	#3 (10mm)	#4 (13mm)	#5 (15/16mm)	#6 (19/20mm)	#7 (22mm)	#8 (25mm)	#9 (30mm)	#10 (32mm)
Guaranteed tensile force	kN	27	71	129	199	284	387	510	600	735
	kip	7.2	16.0	29.0	44.0	64.0	87.0	115.0	134.9	165.2
Elastic Modulus	GPa	60								
	ksi	8700								
Guaranteed transverse shear capacity	MPa	180								
	ksi	26.1								
Weight	g/m	97	185	315	476	702	960	1252	1575	2050
	lb/ft	0.065	0.124	0.212	0.320	0.472	0.645	0.841	1.058	1.378
Nominal cross-sectional area	mm²	32	71	129	199	284	387	510	645	819
	in²	0.049	0.110	0.200	0.310	0.440	0.600	0.790	1.000	1.270
Outer diameter (including ribs)	mm	8.2	10.8	14.0	17.2	20.6	24.1	27.4	30.8	35.0
	in	0.315	0.425	0.551	0.677	0.807	0.949	1.087	1.213	1.378
Primary Materials	Epoxy Backboned Vinylester and Corrosion Resistant E-CR Glass									

The data herein applies to straight bars only. For data on Mateenbar™ rebar bends, please refer to the Mateenbar™ rebar bends data sheet.

Code-Approved and Proven Performance

MATERIAL STANDARDS
Mateenbar™ 60 complies with ASTM D8505 and CSA-S807 Grade III material standards.

RESIDENTIAL CONCRETE
Mateenbar™ 60 can be used in residential concrete, including footings and foundation walls, as prescribed in ICC-EER 5548, or as designed using ACI 332 and ACI 440 design methodology.

COMMERCIAL CONCRETE
Mateenbar™ 60 can be used in commercial concrete design using concrete code ACI 440.11-22, ICC-ESR 5548 and AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete.

MASONRY
Mateenbar™ 60 can be used with TMS 402/602-22 Appendix D as reinforcing for masonry walls.

Leading the world in durable GFRP rebar
mateenbar.com

mateenbar™

B. Bottom Slab Concrete Design Method 1, Steel Example

Concrete Material Properties

Concrete Density	$\rho_c := 145 \dfrac{lbm}{ft^3}$	
Lightweight Concrete Factor	$\lambda := min\left(1, 0.0075 \cdot \rho_c \cdot \dfrac{ft^3}{lbm}\right) = 1$	(ACI 318-19 Ch. 19.2.4)
Concrete Strength	$f'_c := 6000\ psi$	S.W. COLE Concrete 28 Day Test Data
Concrete Rupture Strength	$f_r := \lambda \cdot 7.5 \cdot \sqrt{f'_c \cdot psi} = 580.948\ psi$	(ACI 318-19 Ch. 19.2.3)
Concrete Young's Modulus	$E_c := 33 \cdot \left(\rho_c \cdot \dfrac{ft^3}{lbm}\right)^{1.5} \cdot \sqrt{f'_c \cdot psi} = 4463.151\ ksi$	(ACI 318-19 Ch. 19.2.2)
Strain at Ultimate in the Concrete Extreme Compression Fiber	$\varepsilon_{cu} := 0.003$	
Equivalent Stress Block Depth Factor	$\beta_1 := \max\left(0.65, 0.85 - \dfrac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) = 0.75$	(ACI 318-19 Ch. 22.3.3.4.3)

Reinforcement Steel Material Properties

Reinforcement Yield Strength	$f_y := 60\ ksi$	
Steel Young's Modulus	$E_s := 29000\ ksi$	(ABS Floating 2020 7-3/7.3.7)
Allowable Reinforcement Stress	$f_{s\_allow} := 0.8 \cdot f_y = 48\ ksi$	(ABS Floating 2020 7-3/5.9.1 Table 1)

Geometry

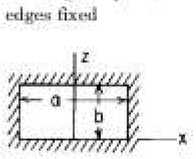
Thickness	$t := 21\ in$	
Analyzed Section Width	$b_w := 1\ ft$	
Gross Moment of Inertia	$I_g := \dfrac{b_w \cdot t^3}{12} = 9261\ in^4$	

Hydrostatic Loading

Seawater Density	$\rho_{SW} := 1025 \dfrac{kg}{m^3}$	
Factor of Safety	$FOS := 1$	
Hull Width	$W_H := 19\ m$	
Hull Height	$H_{Hull} := 18.16\ m$	
Hull Draft	$H_{Draft} := 12.09\ m$	
Hull Radius	$R_{Hull} := 42.25\ m$	
Bottom Beam End Wall Thickness	$t_{BBEW} := 0.4573\ m$	
Vertical Ballast Tank Width	$W_{VBT} := 3.5\ m$	
Bottom Beam Interior End Wall Thickness	$t_{BBIEW} := 0.4573\ m$	
Bottom Beam Interior Transverse Wall Width	$W_{BBITW} := 18.08\ m$	
Bottom Beam Interior Longitudinal Wall Thickness	$t_{BBILW} := 0.25\ m$	
Hydrostatic Pressure	$P_{HS} := \rho_{SW} \cdot g \cdot H_{Draft} = 17.626\ psi$	
Number of Transverse Walls	$n_{Walls} := 7$	
Distance Between Bottom Beam Transverse Walls	$d1 := \dfrac{R_{Hull} - W_H \div 2 - t_{BBEW} - W_{VBT} - t_{BBIEW} - n_{Walls} \cdot t_{BBILW}}{n_{Walls} + 1} = 3.323\ m$	$d1 = 10.903\ ft$
Distance Between Bottom Beam Side Wall and Interior Longitudingal Wall	$d2 := \dfrac{W_{BBITW} - t_{BBILW}}{2} = 8.915\ m$	

Roark's Tabl 11.4 Case 8a

8. Rectangular plate, all edges fixed



8a. Uniform over entire plate

(At center of long edge) $\sigma_{max} = \frac{-\beta_1 q b^2}{t^2}$

(At center) $\sigma = \frac{\beta_2 q b^2}{t^2}$ and $y_{max} = \frac{\alpha q b^4}{E t^3}$

a/b	1.0	1.2	1.4	1.6	1.8	2.0	∞
β_1	0.3078	0.3834	0.4356	0.4680	0.4872	0.4974	0.5000
β_2	0.1386	0.1794	0.2094	0.2286	0.2406	0.2472	0.2500
α	0.0138	0.0188	0.0226	0.0251	0.0267	0.0277	0.0284

(Refs. 7 and 25 and Ref. 21 for $\nu = 0.3$)

[CHAP. 11

Roark's Constants $q := P_{HS} = 17.626\ psi$

$a := \max(d1, d2) = 8.915\ m$

$b := min(d1, d2) = 3.323\ m$

$\frac{a}{b} = 2.683$

$\beta 1 := 0.5$

$\beta 2 := 0.25$

$\sigma_{Edge} := \frac{\beta 1 \cdot q \cdot b^2}{t^2} = 342\ psi$

$\sigma_{Center} := \frac{\beta 2 \cdot q \cdot b^2}{t^2} = 171\ psi$

Hydrostatic Stress $\sigma_{HSL} := \max(\sigma_{Edge}, \sigma_{Edge}) = 342.076\ psi$

Hydrodynamic Load

Wave Period $T_p := 14.2\ s$

Significant Wave Height $H_s := 9.8\ m$

Maximum Wave Height $H_{max} := 1.86 \cdot H_s = 18.228\ m$

Wave Amplitude $A := \frac{H_{max}}{2} = 9.114\ m$

Water Depth $d := 60\ m$

Wave Length $\lambda := \frac{g \cdot T_p^2}{2 \cdot \pi} = 314.715\ m$ (DNVGL-RP-C205 3.2.2)

Wave Number $k := \frac{2 \cdot \pi}{\lambda} = 0.02\ \frac{1}{m}$

$z := 0\ m$

Point of Interest Depth $z := -H_{Draft} = -12.09\ m$

Hydrodynamic Presure $P_{HD} := \frac{1}{2} \cdot \rho_{SW} \cdot g \cdot H_{max} \cdot e^{k \cdot z} = 10.438\ psi$

(DNVGL-RP-C205 3.2.2)

Roark's Constants $q := P_{HD} = 10.438\ psi$

$\sigma_{Edge} := \frac{\beta 1 \cdot q \cdot b^2}{t^2} = 203\ psi$

$\sigma_{Center} := \frac{\beta 2 \cdot q \cdot b^2}{t^2} = 101\ psi$

Hydrodynamic Stress $\sigma_{HDL} := \max(\sigma_{Edge}, \sigma_{Edge}) = 202.571\ psi$

Combined Load

Applied Stress $\sigma_{Applied} := \sigma_{HSL} + \sigma_{HDL} = 544.647\ psi$

Applied Moment $M := \frac{\sigma_{Applied} \cdot I_g}{\frac{t}{2}} = 40.032\ kip \cdot ft$

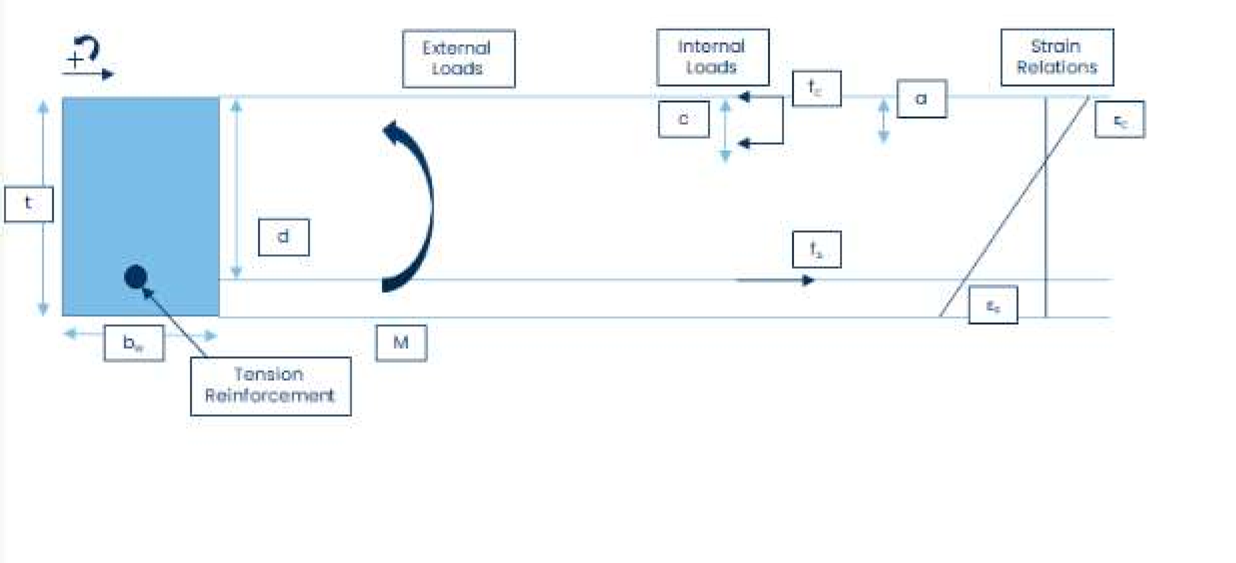
Design Moment $M_D := FOS \cdot M = 40.032\ kip \cdot ft$

Concrete Design Method 1 (for Steel)

Cracking Moment

Strength Reduction Factor	$\phi_{Serivce} := 1.0$	(ABS Floating 2020 7-3/5.9.1) (Serviceability)	
Cracking Moment	$M_{cr} := \dfrac{f_r \cdot I_g}{t \div 2} = 42.7\ \mathbf{kip \cdot ft}$		
Factored Cracking Moment	$\phi_{Serivce} \cdot M_{cr} = 43\ \mathbf{kip \cdot ft}$	$> M_D = 40\ \mathbf{kip \cdot ft}$	GOOD
Utilization Ratio	$\dfrac{M_D}{M_{cr}} = 0.938$	< 1	GOOD

Strength: Ultimate Section Analysis: Tension - Reinforcement



Variable	Definition
t	Thickness
b_w	Unit Width
d	Depth to Reinforcement
M	Applied Moment
P	Post Tensioning Load
A	Applied Axial Load
c	Distance to Neutral Axis
f_c	Compressive Stress
a	Compression Block Depth
f_{fiber}	Fiber Stress
f_s	Tensile Reinforcement Stress
f'_s	Compression Reinforcement Stress
ε_s	Tensile Reinforcement Strain
ε'_s	Compression Reinforcement Strain
ε_c	Concrete Strain

Concrete Cover	$cover := 2.5\ \mathbf{in}$		
Reinforcement Diameter	$d_{bar} := \dfrac{8}{8}\ \mathbf{in}$		
Reinforcement Area	$A_{bar} := \dfrac{\pi}{4} \cdot d_{bar}^{\ 2} = 0.785\ \mathbf{in}^2$		
Depth to Reinforcement Centriod	$d := t - cover - d_{bar} - \dfrac{d_{bar}}{2} = 17\ \mathbf{in}$		
Minimum Flexural Reinforcement in Nonprestressed Beams	$A_{f\_min\_a} := \dfrac{3 \cdot \sqrt{f'_c \cdot \mathbf{psi}}}{f_y} \cdot b_w \cdot d = 0.79\ \mathbf{in}^2$	(ACI 318-19 Ch. 9.6.1.2)	
	$A_{f\_min\_b} := \dfrac{200}{f_y \div \mathbf{psi}} \cdot b_w \cdot d = 0.68\ \mathbf{in}^2$		
	$A_{f\_min} := \max\left(A_{f\_min\_a}, A_{f\_min\_b}\right) = 0.79\ \mathbf{in}^2$		
Reinforcement Spacing	$s := 9\ \mathbf{in}$		
Reinforcement Count Per Unit Width	$n_{bar} := \dfrac{b_w}{s} = 1.333$		
Reinforcement Area	$A_s := A_{bar} \cdot n_{bar} = 1.047\ \mathbf{in}^2$	$>= A_{f\_min} = 0.7901\ \mathbf{in}^2$	GOOD

Sum of Forces

$$0 = -0.85 \cdot f'_c \cdot a \cdot b_w + A_s \cdot f_y$$

Compression Block Depth $\quad a := \dfrac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b_w} = 1.027\ \mathbf{in}$

Neutral Axis Depth $\quad c := \dfrac{a}{\beta_1} = 1.369\ \mathbf{in}$

Sum of Moments about Top of Section

$$M = -0.85 \cdot f'_c \cdot a \cdot b_w \cdot \left(\frac{1}{2} \cdot a\right) + A_s \cdot f_y \cdot d$$

Unfactor Moment Capacity	$M_{n\_RC} := -0.85 \cdot f'_c \cdot a \cdot b_w \cdot \left(\dfrac{1}{2} \cdot a\right) + A_s \cdot f_y \cdot d = 86\ \mathbf{kip \cdot ft}$		
Partial Safety Factor	$c_k := 1.35$	(ABS Floating 2020 7-3/5.3)	
Strength Reduction Factor	$\phi_{Strength} := 0.65$	(ABS Floating 2020 7-3/5.5) (Strength)	
Moment Capacity	$\phi M_n := \phi_{Strength} \cdot M_{n\_RC} = 56\ \mathbf{kip \cdot ft}$	$> c_k \cdot M_D = 54\ \mathbf{kip \cdot ft}$	GOOD
Utilization Ratio	$\dfrac{c_k \cdot M_D}{\phi M_n} = 0.963$	< 1	GOOD

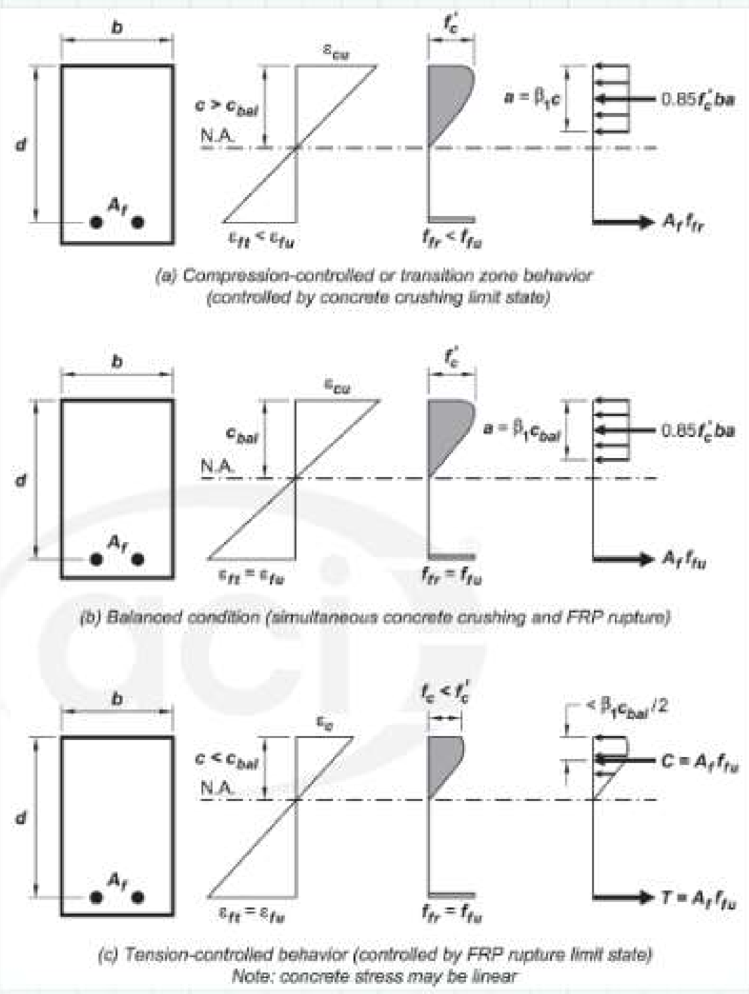
C. Bottom Slab Concrete Design Method 1, GFRP Example

Concrete Design Method 1 (for GFRP)

Cracking Moment

Strength Reduction Factor	$\phi_{Service} := 1.0$	(ABS Floating 2020 7-3/5.9.1) (Serviceability)	
Cracking Moment	$M_{cr} := \dfrac{f_r \cdot I_g}{t \div 2} = 42.7\ \boldsymbol{kip \cdot ft}$		
Factored Cracking Moment	$\phi_{Service} \cdot M_{cr} = 43\ \boldsymbol{kip \cdot ft}$	$> M_D = 40\ \boldsymbol{kip \cdot ft}$	GOOD
Utilization Ratio	$\dfrac{M_D}{M_{cr}} = 0.938$	< 1	GOOD

Strength: Ultimate Section Analysis: Tension - Reinforcement (Balanced)



(a) Compression-controlled or transition zone behavior (controlled by concrete crushing limit state)

(b) Balanced condition (simultaneous concrete crushing and FRP rupture)

(c) Tension-controlled behavior (controlled by FRP rupture limit state) Note: concrete stress may be linear

Concrete Cover	$cover := 1.5\ \boldsymbol{in}$		
Reinforcement Diameter	$d_{bar} := 0.949\ \boldsymbol{in}$	(Mateenbar 60 GFRP Rebar #7)	
Reinforcement Area	$A_{bar} := 0.60\ \boldsymbol{in}^2$	(Mateenbar 60 GFRP Rebar #7)	
Depth to Reinforcement Centriod	$d := t - cover - d_{bar} - \dfrac{d_{bar}}{2} = 18.077\ \boldsymbol{in}$		
Minimum Flexural Reinforcement in Nonprestressed Beams	$A_{f\_min\_a} := \dfrac{4.9 \cdot \sqrt{f'_c \cdot \boldsymbol{psi}}}{f_{fu}} \cdot b_w \cdot d = 0.665\ \boldsymbol{in}^2$	(ACI 440.11-22 Ch. 9.6.1.2)	
	$A_{f\_min\_b} := \dfrac{330}{f_{fu} \div \boldsymbol{psi}} \cdot b_w \cdot d = 0.579\ \boldsymbol{in}^2$		
	$A_{f\_min} := \max\left(A_{f\_min\_a}, A_{f\_min\_b}\right) = 0.665\ \boldsymbol{in}^2$		
Reinforcement Spacing	$s := 10\ \boldsymbol{in}$		
Reinforcement Count Per Unit Width	$n_{bar} := \dfrac{b_w}{s} = 1.2$		
Reinforcement Area	$A_f := A_{bar} \cdot n_{bar} = 0.72\ \boldsymbol{in}^2$	$>= A_{f\_min} = 0.6654\ \boldsymbol{in}^2$	GOOD
Reinforcement to Concrete Ratio	$\rho_f := \dfrac{A_f}{t \cdot b_w} = 0.00286$	$< \rho_{fb} = 0.00538$	TENSION-CONTROLLED
Neutral Axis Depth	$c_{bal} := \dfrac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} \cdot d = 3.149\ \boldsymbol{in}$	(ACI 440.11-22 Ch. 22.3.1.1)	
Unfactored Moment Capacity	$M_{n\_RC} := A_f \cdot f_{fu} \cdot \left(d - \dfrac{\beta_1 \cdot c_{bal}}{2}\right) = 125.435\ \boldsymbol{kip \cdot ft}$		
Partial Safety Factor	$c_k := 1.35$	(ABS Floating 2020 7-3/5.3)	
Strength Reduction Factor	$\phi_{Strength} := 0.65$	(ABS Floating 2020 7-3/5.5) (Strength)	
Moment Capacity	$\phi M_{n\_RC} := \phi_{Strength} \cdot M_{n\_RC} = 81.532\ \boldsymbol{kip \cdot ft}$	$> c_k \cdot M_D = 54.043\ \boldsymbol{kip \cdot ft}$	GOOD
Utilization Ratio	$\dfrac{c_k \cdot M_D}{\phi M_{n\_RC}} = 0.663$	< 1	GOOD

D. Bottom Slab Concrete Design Method 2, Steel Example

Concrete Design Method 2 (for Steel)

Cracking Moment

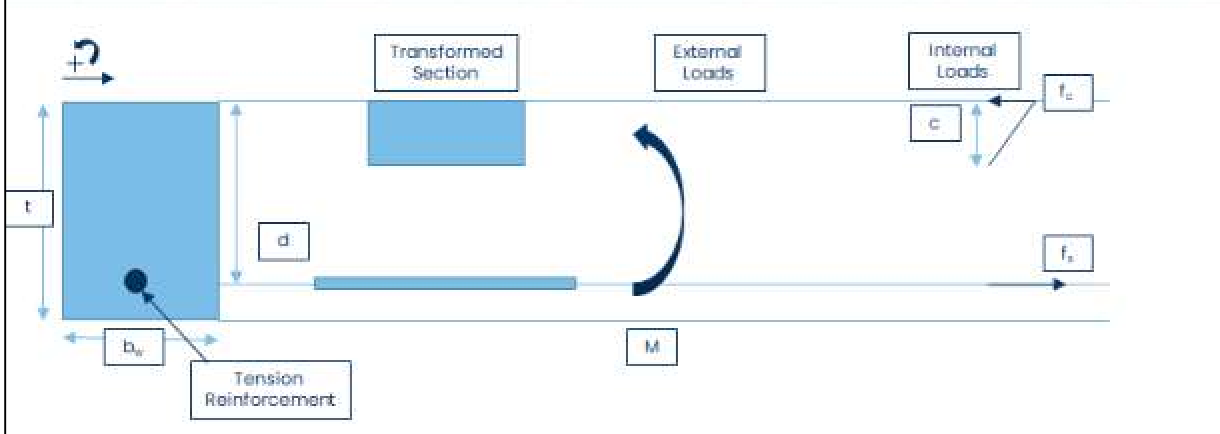
Strength Reduction Factor $\phi_{Serivce} := 1.0$ (ABS Floating 2020 7-3/5.9.1) (Serviceability)

Cracking Moment $M_{cr} := \dfrac{f_r \cdot I_g}{t \div 2} = 31.4\ \textbf{kip}\cdot\textbf{ft}$

Factored Cracking Moment $\phi_{Serivce} \cdot M_{cr} = 31\ \textbf{kip}\cdot\textbf{ft}$ $> M_D = 40\ \textbf{kip}\cdot\textbf{ft}$ DESIGN FOR CRACKED SECTION

Utilization Ratio $\dfrac{M_D}{M_{cr}} = 1.276$ < 1 DESIGN FOR CRACKED SECTION

Service: Cracked Transfrom Section Analysis: Tension - Reinforcement



Variable	Definition
t	Thickness
b_w	Unit Width
d	Depth to Reinforcement
M	Applied Moment
P	Post Tensioning Load
A	Applied Axial Load
c	Distance to Neutral Axis
f_c	Compressive Stress
a	Compression Block Depth
f_{fiber}	Fiber Stress
f_s	Tensile Reinforcement Stress
f'_s	Compression Reinforcement Stress
ε_s	Tensile Reinforcement Strain
ε'_s	Compression Reinforcement Strain
ε_c	Concrete Strain

Concrete Cover $cover := 2.5\ \textbf{in}$

Reinforcement Diameter $d_{bar} := 1.270\ \textbf{in}$

Reinforcement Area $A_{bar} := \dfrac{\pi}{4} \cdot d_{bar}^{\ 2} = 1.267\ \textbf{in}^2$

Depth to Reinforcement Centriod $d := t - cover - d_{bar} - \dfrac{d_{bar}}{2} = 13.595\ \textbf{in}$

Reinforcement Spacing $s := 7.25\ \textbf{in}$

Reinforcement Count Per Unit Width $n_{bar} := \dfrac{b_w}{s} = 1.655$

Reinforcement Area $A_s := A_{bar} \cdot n_{bar} = 2.097\ \textbf{in}^2$

Minimum Flexural Reinforcement in Nonprestressed Beams $A_{s\_min\_a} := \dfrac{3 \cdot \sqrt{f'_c \cdot \textbf{psi}}}{f_y} \cdot b_w \cdot d = 0.632\ \textbf{in}^2$ (ACI 318-19 Ch. 9.6.1.2)

$A_{s\_min\_b} := \dfrac{200}{f_y \div \textbf{psi}} \cdot b_w \cdot d = 0.544\ \textbf{in}^2$

$A_{s\_min} := \max\left(A_{s\_min\_a}, A_{s\_min\_b}\right) = 0.632\ \textbf{in}^2$

Reinforcement Ratio $\rho := \dfrac{A_s}{2\ b_w \cdot d} = 0.00643$ $>= \rho_{shrink\_\&\_temp} := 0.0018$ GOOD (ACI 318-19 Ch. 24.4.3.2)

Steel Modular Ratio $n_s := \dfrac{E_s}{E_c} = 6.498$

Steel Transformed Area $A\_T_s := n_s \cdot A_s = 13.624\ \textbf{in}^2$

Sum of Moments About Neutral Axis

$$b_w \cdot c \cdot \left(\frac{c}{2}\right) = A\_T_s \cdot (d - c)$$

$$b_w \cdot c \cdot \left(\frac{c}{2}\right) = A\_T_s \cdot (d) - A\_T_s \cdot (c)$$

$$\frac{b_w}{2} \cdot c^2 + A\_T_s \cdot c - A\_T_s \cdot d = 0$$

Neutral Axis Depth $c := \dfrac{-A\_T_s + \sqrt{A\_T_s^{\ 2} - 4 \cdot \dfrac{b_w}{2} \cdot \left(-A\_T_s \cdot d\right)}}{2 \cdot \dfrac{b_w}{2}} = 4.535\ \textbf{in}$

Utilization Ratio $\dfrac{\dfrac{t}{4}}{c} = 0.992$ < 1 GOOD

New Moment of Inertia From Top of Section

Concrete Moment of Inertia $I_{o\_c} := \dfrac{b_w \cdot c^3}{12} = 93.299\ \textbf{in}^4$ $d + cover = 16.095\ \textbf{in}$

Concrete Area $A_c := b_w \cdot c = 54.426\ \textbf{in}^2$ $t = 18\ \textbf{in}$

Concrete Distance to Centroid $d_c := \frac{c}{2} = 2.268\ in$

Steel Moment of Inertia $I_{o\_s} := 0\ in^4$

Steel Transformed Area $A_s := A\_T_s$

Concrete Distance to Centroid $d_s := d = 13.595\ in$

Moment of Inertia $I := I_{o\_c} + A_c \cdot d_c^{\,2} + I_{o\_s} + A_s \cdot (d_s - c)^2 = 1491.359\ in^4$

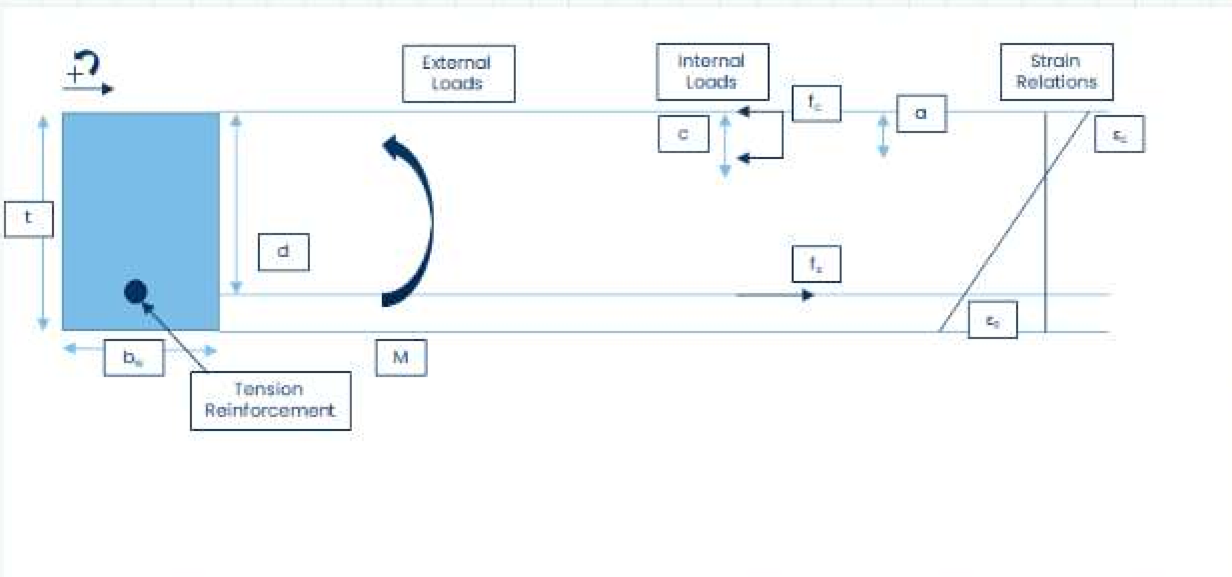
Concrete Stress $f_c := \frac{M_D \cdot c}{I} = 1461\ psi$ $< 0.5 \cdot f'_c = 3000\ psi$ GOOD

Utilization Ratio $\frac{f_c}{0.5 \cdot f'_c} = 0.487$ < 1 GOOD

Steel Stress $f_s := \frac{M_D \cdot (d_s - c)}{I} \cdot n_s = 18961\ psi$ $< f_{s\_allow} = 48000\ psi$ GOOD

Utilization Ratio $\frac{f_s}{f_{s\_allow}} = 0.395$ < 1 GOOD

Strength: Ultimate Section Analysis: Tension - Reinforcement



Variable	Definition
t	Thickness
b_w	Unit Width
d	Depth to Reinforcement
M	Applied Moment
P	Post Tensioning Load
A	Applied Axial Load
c	Distance to Neutral Axis
f_c	Compressive Stress
a	Compression Block Depth
f_{fiber}	Fiber Stress
f_s	Tensile Reinforcement Stress
f'_s	Compression Reinforcement Stress
ε_s	Tensile Reinforcement Strain
ε'_s	Compression Reinforcement Strain
ε_c	Concrete Strain

Concrete Cover $cover := 2.5\ in$

Reinforcement Diameter $d_{bar} := 1.270\ in$

Reinforcement Area $A_{bar} := \frac{\pi}{4} \cdot d_{bar}^{\,2} = 1.267\ in^2$

Depth to Reinforcement Centroid $d := t - cover - d_{bar} - \frac{d_{bar}}{2} = 13.595\ in$

Minimum Flexural Reinforcement in Nonprestressed Beams $A_{f\_min\_a} := \frac{3 \cdot \sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot d = 0.632\ in^2$ (ACI 318-19 Ch. 9.6.1.2)

$A_{f\_min\_b} := \frac{200}{f_y \div psi} \cdot b_w \cdot d = 0.544\ in^2$

$A_{f\_min} := \max\left(A_{f\_min\_a}, A_{f\_min\_b}\right) = 0.632\ in^2$

Reinforcement Spacing $s := 7.25\ in$

Reinforcement Count Per Unit Width $n_{bar} := \frac{b_w}{s} = 1.655$

Reinforcement Area $A_s := A_{bar} \cdot n_{bar} = 2.097\ in^2$ $>= A_{f\_min} = 0.6318\ in^2$ GOOD

Sum of Forces

$0 = -0.85 \cdot f'_c \cdot a \cdot b_w + A_s \cdot f_y$

Compression Block Depth $a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b_w} = 2.056\ in$

Neutral Axis Depth $c := \frac{a}{\beta_1} = 2.741\ in$

Concrete Design Method 2 (for GFRP)

Cracking Moment

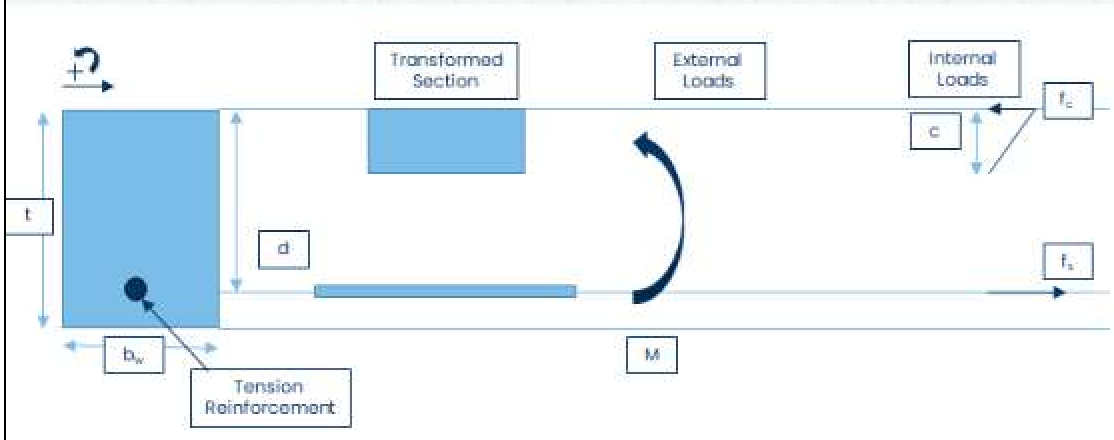
Strength Reduction Factor $\phi_{Service} := 1.0$ (ABS Floating 2020 7-3/5.9.1) (Serviceability)

Cracking Moment $M_{cr} := \dfrac{f_r \cdot I_g}{t \div 2} = 31.4\ kip \cdot ft$

Factored Cracking Moment $\phi_{Service} \cdot M_{cr} = 31\ kip \cdot ft$ $> M_D = 40\ kip \cdot ft$ GOOD

Utilization Ratio $\dfrac{M_D}{M_{cr}} = 1.276$ < 1 GOOD

Service: Cracked Transfrom Section Analysis: Tension - Reinforcement



Variable	Definition
t	Thickness
b_w	Unit Width
d	Depth to Reinforcement
M	Applied Moment
P	Post Tensioning Load
A	Applied Axial Load
c	Distance to Neutral Axis
f_c	Compressive Stress
a	Compression Block Depth
f_{fiber}	Fiber Stress
f_s	Tensile Reinforcement Stress
f'_s	Compression Reinforcement Stress
ε_s	Tensile Reinforcement Strain
ε'_s	Compression Reinforcement Strain
ε_c	Concrete Strain

Concrete Cover $cover := 1.5\ in$

Reinforcement Diameter $d_{bar} := 1.378\ in$ (Mateenbar 60 GFRP Rebar #10)

Reinforcement Area $A_{bar} := 1.270\ in^2 \cdot 2$ *2 for double Mat (Mateenbar 60 GFRP Rebar #10)

Depth to Reinforcement Centriod $d := t - cover - 3 \cdot d_{bar} - \dfrac{d_{bar}}{2} = 11.677\ in$

Minimum Flexural Reinforcement in Nonprestressed Beams $A_{f\_min\_a} := \dfrac{4.9 \cdot \sqrt{f'_c \cdot psi}}{f_{fu}} \cdot b_w \cdot d = 0.43\ in^2$ (ACI 440.11-22 Ch. 9.6.1.2)

$A_{f\_min\_b} := \dfrac{330}{f_{fu} \div psi} \cdot b_w \cdot d = 0.374\ in^2$

$A_{f\_min} := \max\left(A_{f\_min\_a}, A_{f\_min\_b}\right) = 0.43\ in^2$

Reinforcement Spacing $s := 3.5\ in$

Reinforcement Count Per Unit Width $n_{bar} := \dfrac{b_w}{s} = 3.429$

Reinforcement Area $A_f := A_{bar} \cdot n_{bar} = 8.709\ in^2$ $>= A_{f\_min} = 0.4298\ in^2$ GOOD

GFRP Modular Ratio $n_f := \dfrac{E_f}{E_c} = 1.949$

GFRP Transformed Area $A\_T_f := n_f \cdot A_f = 16.976\ in^2$

Sum of Moments About Neutral Axis

$$b_w \cdot c \cdot \left(\frac{c}{2}\right) = A\_T_f \cdot (d - c)$$

$$b_w \cdot c \cdot \left(\frac{c}{2}\right) = A\_T_f \cdot (d) - A\_T_f \cdot (c)$$

$$\frac{b_w}{2} \cdot c^2 + A\_T_f \cdot c - A\_T_f \cdot d = 0$$

Neutral Axis Depth $c := \dfrac{-A\_T_f + \sqrt{A\_T_f^{\,2} - 4 \cdot \dfrac{b_w}{2} \cdot (-A\_T_f \cdot d)}}{2 \cdot \dfrac{b_w}{2}} = 4.505\ in$

Utilization Ratio $\dfrac{\frac{t}{4}}{c} = 0.999$ < 1 GOOD

New Moment of Inertia From Top of Section

Concrete Moment of Inertia	$I_{o\_c} := \frac{b_w \cdot c^3}{12} = 91.411\ in^4$	
Concrete Area	$A_c := b_w \cdot c = 54.056\ in^2$	
Concrete Distance to Centroid	$d_c := \frac{c}{2} = 2.252\ in$	
GFRP Moment of Inertia	$I_{o\_f} := 0\ in^4$	
GFRP Transformed Area	$A_f := A\_T_f$	
Concrete Distance to Centroid	$d_f := d = 11.677\ in$	
Moment of Inertia	$I := I_{o\_c} + A_c \cdot d_c^2 + I_{o\_f} + A_f \cdot \left(d_f - c\right)^2 = 1238.899\ in^4$	
Concrete Stress	$f_c := \frac{M_D \cdot c}{I} = 1747\ psi$	$< 0.5 \cdot f'_c = 3000\ psi$ GOOD
Utilization Ratio	$\frac{f_c}{0.5 \cdot f'_c} = 0.582$	< 1 GOOD
GFRP Stress	$f_f := \frac{M_D \cdot \left(d_f - c\right)}{I} \cdot n_f = 5421\ psi$	$< 0.8 \cdot f_{fu} = 98987\ psi$ GOOD ACI 440.11-22 21.2.2 Limit
Utilization Ratio	$\frac{f_f}{0.8 \cdot f_{fu}} = 0.055$	< 1 GOOD
Ratio of the Distance from the Elastic Cracked Section Neutral Acis to the extreme tension fiber to the distance from the Elastic Crack section Neutral Axis to the centroid of the longitudinal tensile reinforcement	$\beta_{cr} := \frac{t - c}{d - c} = 1.882$	(ACI 440.11-22 Ch. 24.3.2.2)
GFRP Bond Factor	$k_b := 1.2$	(ACI 440.11-22 Ch. 24.3.2.3)
Thickness of concrete cover measured from extreme tension fiber to cneter of bar	$d_c := cover + d_{bar} + \frac{d_{bar}}{2} = 3.567\ in$	
Allowable Tensile Stress	$f_{fs\_allow} := \frac{0.014 \cdot in \cdot E_f}{d_c \cdot \beta_{cr} \cdot k_b} = 15123.031\ psi$	(ACI 440.11-22 24.3.2.2)
GFRP Stress	$f_f := \frac{M_D \cdot \left(d_f - c\right)}{I} \cdot n_f = 5421\ psi$	$< f_{fs\_allow} = 15123\ psi$ GOOD
Utilization Ratio	$\frac{f_f}{0.8 \cdot f_{fu}} = 0.055$	< 1 GOOD

F. Top Slab Concrete Design Method 1, Steel Example

Concrete Material Properties

Concrete Density	$\rho_c := 145 \frac{lbm}{ft^3}$	
Lightweight Concrete Factor	$\lambda := min\left(1, 0.0075 \cdot \rho_c \cdot \frac{ft^3}{lbm}\right) = 1$	(ACI 318-19 Ch. 19.2.4)
Concrete Strength	$f'_c := 6000\ psi$	S.W. COLE Concrete 28 Day Test Data
Concrete Rupture Strength	$f_r := \lambda \cdot 7.5 \cdot \sqrt{f'_c \cdot psi} = 580.948\ psi$	(ACI 318-19 Ch. 19.2.3)
Concrete Young's Modulus	$E_c := 33 \cdot \left(\rho_c \cdot \frac{ft^3}{lbm}\right)^{1.5} \cdot \sqrt{f'_c \cdot psi} = 4463.151\ ksi$	(ACI 318-19 Ch. 19.2.2)
Strain at Ultimate in the Concrete Extreme Compression Fiber	$\varepsilon_{cu} := 0.003$	
Equivalent Stress Block Depth Factor	$\beta_1 := \max\left(0.65, 0.85 - \frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) = 0.75$	(ACI 318-19 Ch. 22.3.3.4.3)

Reinforcement Steel Material Properties

Reinforcement Yield Strength	$f_y := 60\ ksi$	
Steel Young's Modulus	$E_s := 29000\ ksi$	(ABS Floating 2020 7-3/7.3.7)
Allowable Reinforcement Stress	$f_{s\_allow} := 0.8 \cdot f_y = 48\ ksi$	(ABS Floating 2020 7-3/5.9.1 Table 1)

Geometry

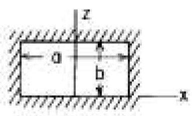
Thickness	$t := 12\ in$
Analyzed Section Width	$b_w := 1\ ft$
Gross Moment of Inertia	$I_g := \frac{b_w \cdot t^3}{12} = 1728\ in^4$

Slab Loading

Factor of Safety	$FOS := 1$	
Hull Width	$W_H := 19\ m$	
Hull Radius	$R_{Hull} := 42.25\ m$	
Bottom Beam End Wall Thickness	$t_{BBEW} := 0.4572\ m$	
Vertical Ballast Tank Width	$W_{VBT} := 3.5\ m$	
Bottom Beam Interior End Wall Thickness	$t_{BBIEW} := 0.4573\ m$	
Bottom Beam Interior Transverse Wall Width	$W_{BBITW} := 18.08\ m$	
Bottom Beam Interior Longitudinal Wall Thickness	$t_{BBILW} := 0.25\ m$	
Dead Load	$P_{DL} := \rho_c \cdot t \cdot g = 1.007\ psi$	
Live Load	$P_{LL} := 250\ psf = 1.736\ psi$	
Number of Transverse Walls	$n_{Walls} := 7$	
Distance Between Bottom Beam Transverse Walls	$d1 := \frac{R_{Hull} - W_H \div 2 - t_{BBEW} - W_{VBT} - t_{BBIEW} - n_{Walls} \cdot t_{BBILW}}{n_{Walls} + 1} = 3.323\ m$	$d1 = 10.903\ ft$
Distance Between Bottom Beam Side Wall and Interior Longitudingal Wall	$d2 := \frac{W_{BBITW} - 2 \cdot t_{BBILW}}{2} = 8.79\ m$	

Roark's Tabl 11.4 Case 8a

8. Rectangular plate, all edges fixed



8a. Uniform over entire plate

(At center of long edge) $\sigma_{max} = \frac{-\beta_1 q b^2}{t^2}$

(At center) $\sigma = \frac{\beta_2 q b^2}{t^2}$ and $y_{max} = \frac{\alpha q b^4}{E t^3}$

a/b	1.0	1.2	1.4	1.6	1.8	2.0	∞
β_1	0.3078	0.3834	0.4356	0.4680	0.4872	0.4974	0.5000
β_2	0.1386	0.1794	0.2094	0.2286	0.2406	0.2472	0.2500
α	0.0138	0.0188	0.0226	0.0251	0.0267	0.0277	0.0284

(Refs. 7 and 25 and Ref. 21 for $\nu = 0.3$)

[CHAP. 11

Roark's Constants $q1 := 1.4\ P_{DL} = 1.41\ psi$ (ACI 318-19 Ch. 5.3.1)

$q2 := 1.2\ P_{DL} + 1.6\ P_{LL} = 3.986\ psi$ (ACI 318-19 Ch. 5.3.1)

$q := \max(q1, q2) = 3.986\ psi$

$a := \max(d1, d2) = 8.79\ m$

$b := min(d1, d2) = 3.323\ m$

$\frac{a}{b} = 2.645$

$\beta 1 := 0.5$

$\beta 2 := 0.25$

$\sigma_{Edge} := \frac{\beta 1 \cdot q \cdot b^2}{t^2} = 237\ psi$

$\sigma_{Center} := \frac{\beta 2 \cdot q \cdot b^2}{t^2} = 118\ psi$

Slab Stress $\sigma := \max(\sigma_{Edge}, \sigma_{Edge}) = 236.919\ psi$

Applied Load

Applied Stress $\sigma_{Applied} := \sigma = 236.919\ psi$

Applied Moment $M := \frac{\sigma_{Applied} \cdot I_g}{\frac{t}{2}} = 5.686\ kip \cdot ft$

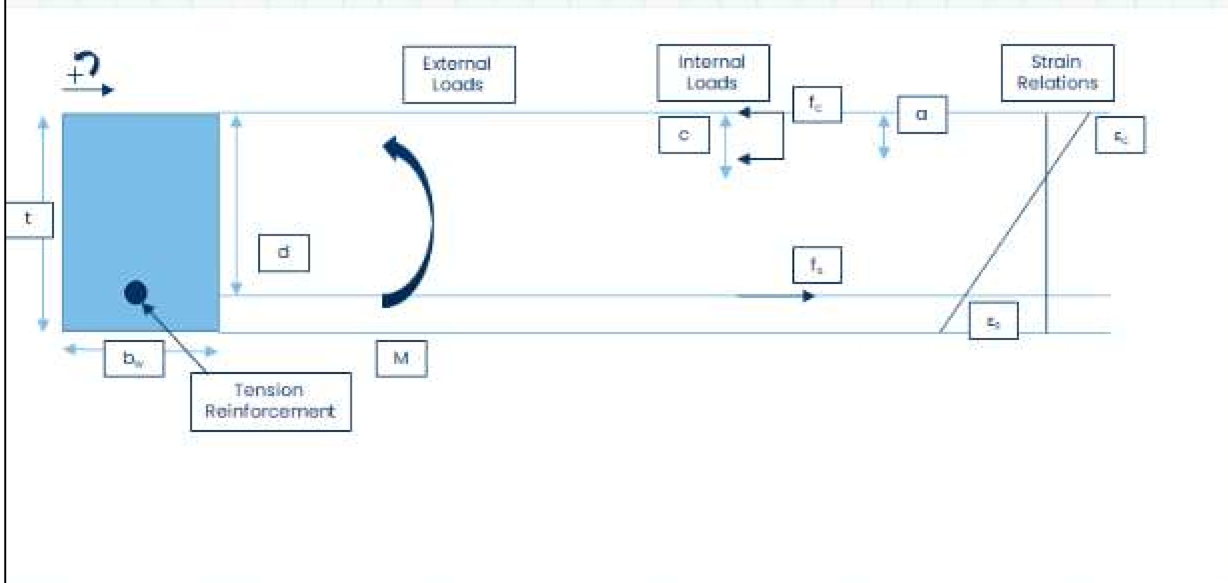
Design Moment $M_D := FOS \cdot M = 5.686\ kip \cdot ft$

Concrete Design Method 1 (for Steel)

Cracking Moment

Strength Reduction Factor	$\phi_{Serivce} := 1.0$	(ABS Floating 2020 7-3/5.9.1) (Serviceability)	
Cracking Moment	$M_{cr} := \dfrac{f_r \cdot I_g}{t \div 2} = 13.9\ \boldsymbol{kip \cdot ft}$		
Factored Cracking Moment	$\phi_{Serivce} \cdot M_{cr} = 14\ \boldsymbol{kip \cdot ft}$	$> M_D = 6\ \boldsymbol{kip \cdot ft}$	GOOD
Utilization Ratio	$\dfrac{M_D}{M_{cr}} = 0.408$	< 1	GOOD

Strength: Ultimate Section Analysis: Tension - Reinforcement



Variable	Definition
t	Thickness
b_w	Unit Width
d	Depth to Reinforcement
M	Applied Moment
P	Post Tensioning Load
A	Applied Axial Load
c	Distance to Neutral Axis
f_c	Compressive Stress
a	Compression Block Depth
f_{fiber}	Fiber Stress
f_s	Tensile Reinforcement Stress
f'_s	Compression Reinforcement Stress
ε_s	Tensile Reinforcement Strain
ε'_s	Compression Reinforcement Strain
ε_c	Concrete Strain

Concrete Cover $\quad cover := 2.5\ \boldsymbol{in}$

Reinforcement Diameter $\quad d_{bar} := \dfrac{5}{8}\ \boldsymbol{in}$

Reinforcement Area $\quad A_{bar} := \dfrac{\pi}{4} \cdot d_{bar}^{\ 2} = 0.307\ \boldsymbol{in}^2$

Depth to Reinforcement Centriod $\quad d := t - cover - d_{bar} - \dfrac{d_{bar}}{2} = 8.563\ \boldsymbol{in}$

Minimum Flexural Reinforcement in Nonprestressed Beams $\quad A_{f\_min\_a} := \dfrac{3 \cdot \sqrt{f'_c \cdot \boldsymbol{psi}}}{f_y} \cdot b_w \cdot d = 0.398\ \boldsymbol{in}^2$ $\quad$ (ACI 318-19 Ch. 9.6.1.2)

$$A_{f\_min\_b} := \frac{200}{f_y \div \boldsymbol{psi}} \cdot b_w \cdot d = 0.343\ \boldsymbol{in}^2$$

$$A_{f\_min} := \max\left(A_{f\_min\_a}, A_{f\_min\_b}\right) = 0.398\ \boldsymbol{in}^2$$

Reinforcement Spacing $\quad s := 9\ \boldsymbol{in}$

Reinforcement Count Per Unit Width $\quad n_{bar} := \dfrac{b_w}{s} = 1.333$

Reinforcement Area $\quad A_s := A_{bar} \cdot n_{bar} = 0.409\ \boldsymbol{in}^2$ $\quad > A_{f\_min} = 0.398\ \boldsymbol{in}^2$ GOOD

Sum of Forces

$$0 = -0.85 \cdot f'_c \cdot a \cdot b_w + A_s \cdot f_y$$

Compression Block Depth $\quad a := \dfrac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b_w} = 0.401\ \boldsymbol{in}$

Neutral Axis Depth $\quad c := \dfrac{a}{\beta_1} = 0.535\ \boldsymbol{in}$

Sum of Moments about Top of Section

$$M = -0.85 \cdot f'_c \cdot a \cdot b_w \cdot \left(\frac{1}{2} \cdot a\right) + A_s \cdot f_y \cdot d$$

Unfactor Moment Capacity $\quad M_{n\_RC} := -0.85 \cdot f'_c \cdot a \cdot b_w \cdot \left(\dfrac{1}{2} \cdot a\right) + A_s \cdot f_y \cdot d = 17\ \boldsymbol{kip \cdot ft}$

Partial Safety Factor $\quad c_k := 1.35$ $\quad$ (ABS Floating 2020 7-3/5.3)

Strength Reduction Factor $\quad \phi_{Strength} := 0.65$ $\quad$ (ABS Floating 2020 7-3/5.5) (Strength)

Moment Capacity $\quad \phi M_n := \phi_{Strength} \cdot M_{n\_RC} = 11\ \boldsymbol{kip \cdot ft}$ $\quad > c_k \cdot M_D = 8\ \boldsymbol{kip \cdot ft}$ GOOD

Utilization Ratio $\quad \dfrac{c_k \cdot M_D}{\phi M_n} = 0.69$ $\quad < 1$ GOOD

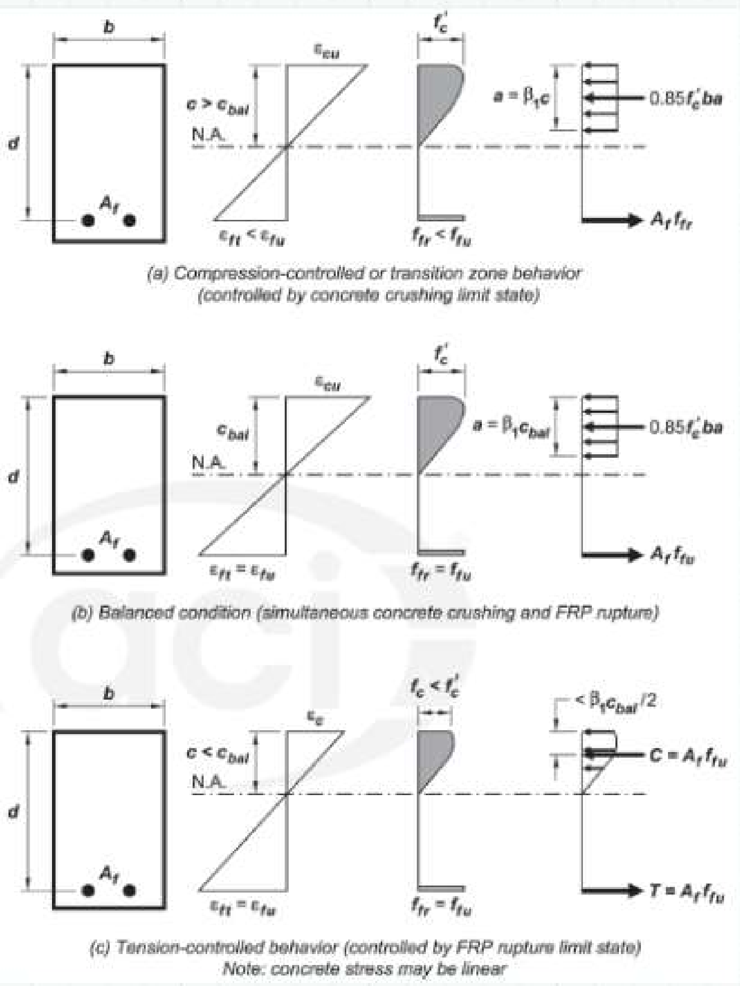
G. Top Slab Concrete Design Method 1, GFRP Example

Concrete Design Method 1 (for GFRP)

Cracking Moment

Strength Reduction Factor	$\phi_{Serivce} := 1.0$	(ABS Floating 2020 7-3/5.9.1) (Serviceability)
Cracking Moment	$M_{cr} := \dfrac{f_r \cdot I_g}{t \div 2} = 9.7\ \mathbf{kip \cdot ft}$	
Factored Cracking Moment	$\phi_{Serivce} \cdot M_{cr} = 10\ \mathbf{kip \cdot ft}$	$> M_D = 5\ \mathbf{kip \cdot ft}$ GOOD
Utilization Ratio	$\dfrac{M_D}{M_{cr}} = 0.558$	< 1 GOOD

Strength: Ultimate Section Analysis: Tension - Reinforcement (Balanced)



Concrete Cover	$cover := 1.5\ \mathbf{in}$	
Reinforcement Diameter	$d_{bar} := 0.551\ \mathbf{in}$	(Mateenbar 60 GFRP Rebar #4)
Reinforcement Area	$A_{bar} := 0.2\ \mathbf{in}^2$	(Mateenbar 60 GFRP Rebar #4)
Depth to Reinforcement Centriod	$d := t - cover - d_{bar} - \dfrac{d_{bar}}{2} = 7.674\ \mathbf{in}$	
Minimum Flexural Reinforcement in Nonprestressed Beams	$A_{f\_min\_a} := \dfrac{4.9 \cdot \sqrt{f'_c \cdot \mathbf{psi}}}{f_{fu}} \cdot b_w \cdot d = 0.282\ \mathbf{in}^2$	(ACI 440.11-22 Ch. 9.6.1.2)
	$A_{f\_min\_b} := \dfrac{330}{f_{fu} \div \mathbf{psi}} \cdot b_w \cdot d = 0.246\ \mathbf{in}^2$	
	$A_{f\_min} := \max\left(A_{f\_min\_a}, A_{f\_min\_b}\right) = 0.282\ \mathbf{in}^2$	
Reinforcement Spacing	$s := 8.25\ \mathbf{in}$	
Reinforcement Count Per Unit Width	$n_{bar} := \dfrac{b_w}{s} = 1.455$	
Reinforcement Area	$A_f := A_{bar} \cdot n_{bar} = 0.291\ \mathbf{in}^2$	$>= A_{f\_min} = 0.2825\ \mathbf{in}^2$ GOOD
Reinforcement to Concrete Ratio	$\rho_f := \dfrac{A_f}{t \cdot b_w} = 0.00242$	$< \rho_{fb} = 0.00538$ TENSION-CONTROLLED
Neutral Axis Depth	$c_{bal} := \dfrac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} \cdot d = 1.337\ \mathbf{in}$	(ACI 440.11-22 Ch. 22.3.1.1)
Unfactored Moment Capacity	$M_{n\_RC} := A_f \cdot f_{fu} \cdot \left(d - \dfrac{\beta_1 \cdot c_{bal}}{2}\right) = 21.514\ \mathbf{kip \cdot ft}$	
Partial Safety Factor	$c_k := 1.35$	(ABS Floating 2020 7-3/5.3)
Strength Reduction Factor	$\phi_{Strength} := 0.65$	(ABS Floating 2020 7-3/5.5) (Strength)
Moment Capacity	$\phi M_{n\_RC} := \phi_{Strength} \cdot M_{n\_RC} = 13.984\ \mathbf{kip \cdot ft}$	$> c_k \cdot M_D = 7.288\ \mathbf{kip \cdot ft}$ GOOD
Utilization Ratio	$\dfrac{c_k \cdot M_D}{\phi M_{n\_RC}} = 0.521$	< 1 GOOD

H. Top Slab Hybrid Steel-Fiber Example

Concrete Material Properties

Concrete Density	$\rho_c := 145 \frac{lbm}{ft^3}$	
Lightweight Concrete Factor	$\lambda := min\left(1, 0.0075 \cdot \rho_c \cdot \frac{ft^3}{lbm}\right) = 1$	(ACI 318-19 Ch. 19.2.4)
Concrete Strength	$f'_c := 6000\ psi$	S.W. COLE Concrete 28 Day Test Data
Concrete Rupture Strength	$f_r := \lambda \cdot 7.5 \cdot \sqrt{f'_c \cdot psi} = 580.948\ psi$	(ACI 318-19 Ch. 19.2.3)
Concrete Young's Modulus	$E_c := 33 \cdot \left(\rho_c \cdot \frac{ft^3}{lbm}\right)^{1.5} \cdot \sqrt{f'_c \cdot psi} = 4463.151\ ksi$	(ACI 318-19 Ch. 19.2.2)
Strain at Ultimate in the Concrete Extreme Compression Fiber	$\varepsilon_{cu} := 0.003$	
Equivalent Stress Block Depth Factor	$\beta_1 := \max\left(0.65, 0.85 - \frac{0.05 \cdot (f'_c - 4000\ psi)}{1000\ psi}\right) = 0.75$	(ACI 318-19 Ch. 22.3.3.4.3)

Reinforcement Steel Material Properties

Reinforcement Yield Strength	$f_y := 60\ ksi$	
Steel Young's Modulus	$E_s := 29000\ ksi$	(ABS Floating 2020 7-3/7.3.7)
Allowable Reinforcement Stress	$f_{s\_allow} := 0.8 \cdot f_y = 48\ ksi$	(ABS Floating 2020 7-3/5.9.1 Table 1)

Geometry

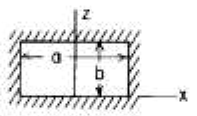
Thickness	$t := 12\ in$
Analyzed Section Width	$b_w := 1\ ft$
Gross Moment of Inertia	$I_g := \frac{b_w \cdot t^3}{12} = 1728\ in^4$

Slab Loading

Factor of Safety	$FOS := 1$	
Hull Width	$W_H := 19\ m$	
Hull Radius	$R_{Hull} := 42.25\ m$	
Bottom Beam End Wall Thickness	$t_{BBEW} := 0.4572\ m$	
Vertical Ballast Tank Width	$W_{VBT} := 3.5\ m$	
Bottom Beam Interior End Wall Thickness	$t_{BBIEW} := 0.4573\ m$	
Bottom Beam Interior Transverse Wall Width	$W_{BBITW} := 18.08\ m$	
Bottom Beam Interior Longitudinal Wall Thickness	$t_{BBILW} := 0.25\ m$	
Dead Load	$P_{DL} := \rho_c \cdot t \cdot g = 1.007\ psi$	
Live Load	$P_{LL} := 250\ psf = 1.736\ psi$	
Number of Transverse Walls	$n_{Walls} := 7$	
Distance Between Bottom Beam Transverse Walls	$d1 := \frac{R_{Hull} - W_H \div 2 - t_{BBEW} - W_{VBT} - t_{BBIEW} - n_{Walls} \cdot t_{BBILW}}{n_{Walls} + 1} = 3.323\ m$	$d1 = 10.903\ ft$
Distance Between Bottom Beam Side Wall and Interior Longitudinal Wall	$d2 := \frac{W_{BBITW} - 2 \cdot t_{BBILW}}{2} = 8.79\ m$	

Roark's Tabl 11.4 Case 8a

8. Rectangular plate, all edges fixed



8a. Uniform over entire plate

(At center of long edge) $\sigma_{max} = \frac{-\beta_1 q b^2}{t^2}$

(At center) $\sigma = \frac{\beta_2 q b^2}{t^2}$ and $y_{max} = \frac{\alpha q b^4}{E t^3}$

a/b	1.0	1.2	1.4	1.6	1.8	2.0	∞
β_1	0.3078	0.3834	0.4356	0.4680	0.4872	0.4974	0.5000
β_2	0.1386	0.1794	0.2094	0.2286	0.2406	0.2472	0.2500
α	0.0138	0.0188	0.0226	0.0251	0.0267	0.0277	0.0284

(Refs. 7 and 25 and Ref. 21 for $\nu = 0.3$)

[CHAP. 11

Roark's Constants

$q1 := 1.4\ P_{DL} = 1.41\ psi$ (ACI 318-19 Ch. 5.3.1)

$q2 := 1.2\ P_{DL} + 1.6\ P_{LL} = 3.986\ psi$ (ACI 318-19 Ch. 5.3.1)

$q := \max(q1, q2) = 3.986\ psi$

$a := \max(d1, d2) = 8.79\ m$

$b := min(d1, d2) = 3.323\ m$

$\frac{a}{b} = 2.645$

$\beta1 := 0.5$

$\beta2 := 0.25$

$\sigma_{Edge} := \frac{\beta1 \cdot q \cdot b^2}{t^2} = 237\ psi$

$\sigma_{Center} := \frac{\beta2 \cdot q \cdot b^2}{t^2} = 118\ psi$

Slab Stress $\sigma := \max(\sigma_{Edge}, \sigma_{Edge}) = 236.919\ psi$

Applied Load

Applied Stress $\sigma_{Applied} := \sigma = 236.919\ psi$

Applied Moment $M := \frac{\sigma_{Applied} \cdot I_g}{\frac{t}{2}} = 5.686\ kip \cdot ft$

Design Moment $M_D := FOS \cdot M = 5.686\ kip \cdot ft$

Concrete Design Method 1 (for Steel)

Cracking Moment

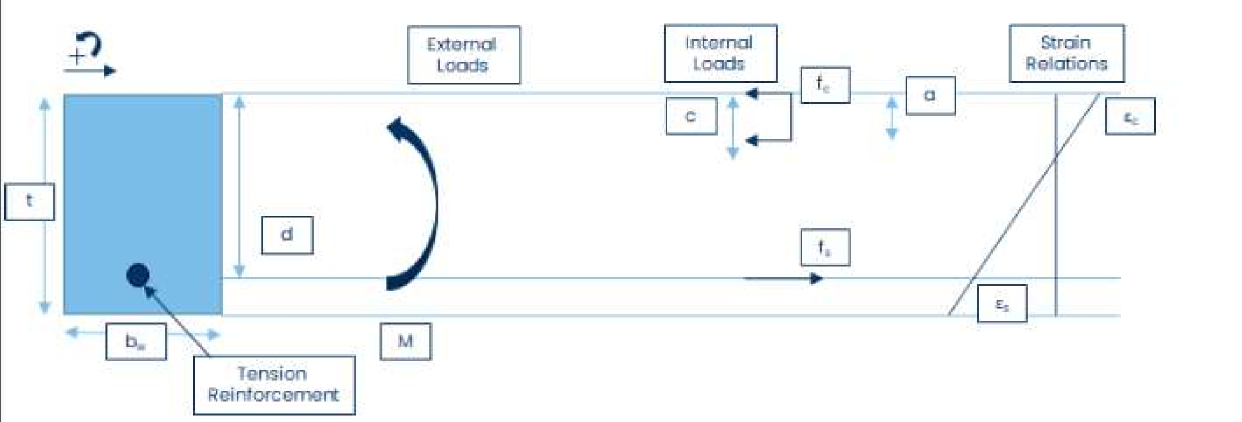
Strength Reduction Factor $\phi_{Serivce} := 1.0$ (ABS Floating 2020 7-3/5.9.1) (Serviceability)

Cracking Moment $M_{cr} := \dfrac{f_r \cdot I_g}{t \div 2} = 13.9\ kip \cdot ft$

Factored Cracking Moment $\phi_{Serivce} \cdot M_{cr} = 14\ kip \cdot ft$ $> M_D = 6\ kip \cdot ft$ GOOD

Utilization Ratio $\dfrac{M_D}{M_{cr}} = 0.408$ < 1 GOOD

Strength: Ultimate Section Analysis: Tension - Reinforcement



Variable	Definition
t	Thickness
b_w	Unit Width
d	Depth to Reinforcement
M	Applied Moment
P	Post Tensioning Load
A	Applied Axial Load
c	Distance to Neutral Axis
f_c	Compressive Stress
a	Compression Block Depth
f_{fiber}	Fiber Stress
f_s	Tensile Reinforcement Stress
f'_s	Compression Reinforcement Stress
ε_s	Tensile Reinforcement Strain
ε'_s	Compression Reinforcement Strain
ε_c	Concrete Strain

Concrete Cover $cover := 2.5\ in$

Reinforcement Diameter $d_{bar} := \dfrac{5}{8}\ in$

Reinforcement Area $A_{bar} := \dfrac{\pi}{4} \cdot d_{bar}^2 = 0.307\ in^2$

Depth to Reinforcement Centriod $d := t - cover - d_{bar} - \dfrac{d_{bar}}{2} = 8.563\ in$

Minimum Flexural Reinforcement in Nonprestressed Beams $A_{f\_min\_a} := \dfrac{3 \cdot \sqrt{f'_c \cdot psi}}{f_y} \cdot b_w \cdot d = 0.398\ in^2$ (ACI 318-19 Ch. 9.6.1.2)

$A_{f\_min\_b} := \dfrac{200}{f_y \div psi} \cdot b_w \cdot d = 0.343\ in^2$

$A_{f\_min} := \max\left(A_{f\_min\_a}, A_{f\_min\_b}\right) = 0.398\ in^2$

Reinforcement Spacing $s := 10.75\ in$

Reinforcement Count Per Unit Width $n_{bar} := \dfrac{b_w}{s} = 1.333$

Reinforcement Area $A_s := A_{bar} \cdot n_{bar} = 0.409\ in^2$ $> A_{f\_min} = 0.398\ in^2$ GOOD

Sum of Forces

$0 = -0.85 \cdot f'_c \cdot a \cdot b_w + A_s \cdot f_y$

Compression Block Depth $a := \dfrac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b_w} = 0.401\ in$

Neutral Axis Depth $c := \dfrac{a}{\beta_1} = 0.535\ in$

Sum of Moments about Top of Section

$M = -0.85 \cdot f'_c \cdot a \cdot b_w \cdot \left(\dfrac{1}{2} \cdot a\right) + A_s \cdot f_y \cdot d$

Unfactor Moment Capacity $M_{n\_RC} := -0.85 \cdot f'_c \cdot a \cdot b_w \cdot \left(\dfrac{1}{2} \cdot a\right) + A_s \cdot f_y \cdot d = 17\ kip \cdot ft$

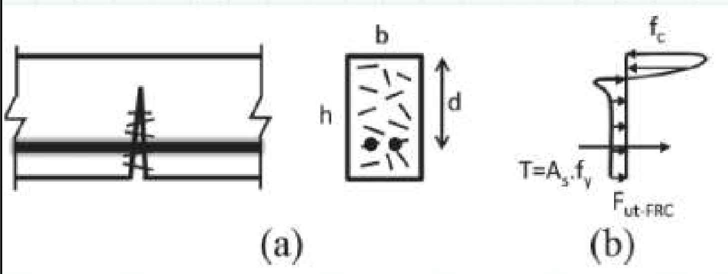
Partial Safety Factor $c_k := 1.35$ (ABS Floating 2020 7-3/5.3)

Strength Reduction Factor $\phi_{Strength} := 0.65$ (ABS Floating 2020 7-3/5.5) (Strength)

Moment Capacity $\phi M_n := \phi_{Strength} \cdot M_{n\_RC} = 11.117\ kip \cdot ft$ $> c_k \cdot M_D = 8\ kip \cdot ft$ GOOD

Utilization Ratio $\dfrac{c_k \cdot M_D}{\phi M_n} = 0.69$ < 1 GOOD

Strength: Ultimate Section Analysis: Tension - Reinforcement, Fiber



Synthetic Fiber Ultimate Analysis

Fiber Reinforced Concrete Flexural Residual Strength — $f_{e3} := 120\ psi$

Nominal Moment Capacity of FRC — $M_{n\_FRC} := f_{e3} \cdot \frac{b_w \cdot t^2}{6} = 2.88\ kip \cdot ft$ — (ACI 444.4R-18 Ch. 4.5)

Combined Analysis

Nominal Moment Capacity — $M_{n\_HFRC} := M_{n\_RC} + M_{n\_FRC} = 17.255\ kip \cdot ft$ — (ACI 444.4R-18 Ch. 4.7)

Partial Safety Factor — $c_k := 1.35$ — (ABS Floating 2020 7-3/5.3)

Strength Reduction Factor — $\phi_{Strength} := 0.65$ — (ABS Floating 2020 7-3/5.5) (Strength)

Moment Capacity — $\phi M_{n\_HFRC} := \phi_{Strength} \cdot M_{n\_HFRC} = 11.215\ kip \cdot ft$ — $> c_k \cdot M_D = 7.676\ kip \cdot ft$ GOOD

Utilization Ratio — $\frac{c_k \cdot M_D}{\phi M_n} = 0.822$ — < 1 GOOD

Shear: Reinforcred Concrete

Unfactor Concrete Shear Capacity — $v_{c\_RC} := 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} = 154.919\ psi$ — (ACI 318 Ch. 22.5.5)

Shear: Synthetic Fiber

Partial Safety Factor for Plain Concrete — $\gamma_c := 0.7$

Size Effect Factor — $k_s := min\left(2, 1 + \left(\frac{8\ in}{d}\right)^{0.5}\right) = 1.967$ — (ACI 544.4R-18 Ch. 4.8)

Steel to Concrete Area Ratio — $\rho := \frac{A_s}{b_w \cdot t} = 0.002$

Average Normal Stress Acting on Concrete Cross Section — $\sigma_{cp} := 0\ psi$

Unfactor Concrete Shear Capacity with Synthetic Fiber — $v_{c\_FRC} := 26.8 \cdot \left(\frac{0.18}{\gamma_c} \cdot k_s \cdot \left(100 \cdot \rho \cdot \left(1 + 7.5 \cdot \frac{f_{e3}}{f_r}\right) \cdot \frac{f'_c}{psi}\right)^{\frac{1}{3}} + \frac{0.15 \cdot \sigma_{cp}}{psi}\right) \cdot psi = 208.431\ psi$ — (ACI 544.4R-18 Ch. 4.8)

Ratio of Concrete Shear FRC:RC — $\frac{v_{c\_FRC}}{v_{c\_RC}} = 1.345$

I. Reinforcement Weight Tables

Steel Design																				
Steel Weight Table																				
			0 = Thickness Direction																	
Component			(Along Leg)	(Across Leg)	(Up Leg)	Component	Reinforcement Properties													
		Thickness	Legnth	Width	Height	Count	Spacing	Count		Layers	Length	Size	Diameter	Area	Volume	Density	Mass			
		in	in	in	in	-	in	-	-	-	in	-	in	in^2	in^3	lb/in^3	lb	kg	mt	mt
Leg	Side Wall (+x)	21	1280	0	715	4	9	79.44	80.00	4	1637792	8	1	0.785	1286319	0.284	365315	165676	165.68	
	Side Wall (-x)	21	1280	0	715	4	9	79.44	80	4	1637792	8	1	0.785	1286319	0.284	365315	165676	165.68	
	Internal Longitudinal Wall	12	1280	0	715	4	9	79.44	80	4	1637792	8	1	0.785	1286319	0.284	365315	165676	165.68	
	End Wall	21	0	748	715	4	9	79.44	80	4	957478	8	1	0.785	752002	0.284	213569	96856	96.86	
	Internal End Wall	21	0	748	715	4	9	79.44	80	4	957478	8	1	0.785	752002	0.284	213569	96856	96.86	
	Internal Transverse Wall	12	0	748	715	28	9	79.44	80	4	6702349	8	1	0.785	5264012	0.284	1494980	677995	678.00	
	Bottom Slab	21	1280	748	0	4	9	83.11	84	4	1719682	8	1	0.785	1350635	0.284	383580	173959	173.96	
	Top Slab	21	1280	748	0	4	9	83.11	84	4	1719682	5	0.625	0.307	527592	0.284	149836	67953	67.95	
Keystone	Size Wall (+x)	23	748	0	715	1	9	79.44	80	4	239370	8	1	0.785	188000	0.284	53392	24214	24.21	1867.32
	Side Wall (-x)	23	748	0	715	1	9	79.44	80	4	239370	8	1	0.785	188000	0.284	53392	24214	24.21	
	Spoke (+x)	36	748	0	614	1	9	68.24	69	4	206456	8	1	0.785	162150	0.284	46051	20885	20.88	
	Spoke (-x)	36	748	0	614	1	9	68.24	69	4	206456	8	1	0.785	162150	0.284	46051	20885	20.88	
	Size Wall (+y)	23	0	748	715	1	9	79.44	80	4	239370	8	1	0.785	188000	0.284	53392	24214	24.21	
	Side Wall (-y)	23	0	748	715	1	9	79.44	80	4	239370	8	1	0.785	188000	0.284	53392	24214	24.21	
	Spoke (+y)	36	0	748	614	1	9	68.24	69	4	206456	8	1	0.785	162150	0.284	46051	20885	20.88	
	Spoke (-y)	36	0	748	614	1	9	68.24	69	4	206456	8	1	0.785	162150	0.284	46051	20885	20.88	
	Bottom Slab	23	748	748	0	1	9	83.11	84	4	251338	8	1	0.785	197400	0.284	56062	25425	25.42	
	Internal Slab	23	748	748	0	1	9	83.11	84	4	251338	8	1	0.785	197400	0.284	56062	25425	25.42	
	Top Slab	23	748	748	0	1	9	83.11	84	4	251338	8	1	0.785	197400	0.284	56062	25425	25.42	

Hybrid Design (Steel and GFRP Reinforcement)																					
Steel Weight Table																					
			0 = Thickness Direction																		
Component			(Along Leg)	(Across Leg)	(Up Leg)	Component	Reinforcement Properties														TOTAL MASS
		Thickness	Legnth	Width	Height	Count	Spacing	Count		Layers	Length	Size	Diameter	Area	Volume	Density	Mass				
		in	in	in	in	-	in	-	-	-	in	-	in	in^2	in^3	lb/in^3	lb	kg	mt	mt	mt
Leg	Side Wall (+x)	21	1280	0	715	4	9	79.44	80	4	1637792	8	1	0.785	1286319	0.284	365315	165676	165.68	955.69	1151.85
	Side Wall (-x)	21	1280	0	715	4	9	79.44	80	4	1637792	8	1	0.785	1286319	0.284	365315	165676	165.68		
	Internal Longitudinal Wall	12	1280	0	715	4															
	End Wall	21	0	748	715	4	9	79.44	80	4	957478	8	1	0.785	752002	0.284	213569	96856	96.86		
	Internal End Wall	21	0	748	715	4	9	79.44	80	4	957478	8	1	0.785	752002	0.284	213569	96856	96.86		
	Internal Transverse Wall	12	0	748	715	28															
	Bottom Slab	21	1280	748	0	4	9	83.11	84	4	1719682	8	1	0.785	1350635	0.284	383580	173959	173.96		
	Top Slab	12	1280	748	0	4															
Keystone	Size Wall (+x)	23	748	0	715	1	9	79.44	80	4	239370	8	1	0.785	188000	0.284	53392	24214	24.21		
	Side Wall (-x)	23	748	0	715	1	9	79.44	80	4	239370	8	1	0.785	188000	0.284	53392	24214	24.21		
	Spoke (+x)	36	748	0	614	1	9	68.24	69	4	206456	8	1	0.785	162150	0.284	46051	20885	20.88		
	Spoke (-x)	36	748	0	614	1	9	68.24	69	4	206456	8	1	0.785	162150	0.284	46051	20885	20.88		
	Size Wall (+y)	23	0	748	715	1	9	79.44	80	4	239370	8	1	0.785	188000	0.284	53392	24214	24.21		
	Side Wall (-y)	23	0	748	715	1	9	79.44	80	4	239370	8	1	0.785	188000	0.284	53392	24214	24.21		
	Spoke (+y)	36	0	748	614	1	9	68.24	69	4	206456	8	1	0.785	162150	0.284	46051	20885	20.88		
	Spoke (-y)	36	0	748	614	1	9	68.24	69	4	206456	8	1	0.785	162150	0.284	46051	20885	20.88		
	Bottom Slab	23	748	748	0	1	9	83.11	84	4	251338	8	1	0.785	197400	0.284	56062	25425	25.42		
	Internal Slab	23	748	748	0	1	9	83.11	84	4	251338	8	1	0.785	197400	0.284	56062	25425	25.42		
	Top Slab	23	748	748	0	1	9	83.11	84	4	251338	8	1	0.785	197400	0.284	56062	25425	25.42		
GFRP Weight Table																					
			0 = Thickness Direction																		
Component			(Along Leg)	(Across Leg)	(Up Leg)	Component	Reinforcement Properties														
		Thickness	Legnth	Width	Height	Count	Spacing	Count		Layers	Length	Size	Diameter	Area	Volume	Density	Mass				
		in	in	in	in	-	in	-	-	-	in	-	in	in^2	in^3	lb/in^3	lb	kg		mt	
Leg	Side Wall (+x)	21	1280	0	715	4														196.16	
	Side Wall (-x)	21	1280	0	715	4															
	Internal Longitudinal Wall	10	1280	0	715	4	10	71.50	72	4	1474013	7	1	0.6	884408	0.089	78712	35697	35.70		
	End Wall	21	0	748	715	4															
	Internal End Wall	21	0	748	715	4															
	Internal Transverse Wall	10	0	748	715	28	10	71.50	72	4	6032114	7	1	0.6	3619268	0.089	322115	146084	146.08		
	Bottom Slab	21	1280	748	0	4															
	Top Slab	10	1280	748	0	4	8.65	86.48	87	4	1781099	4	0.625	0.2	356220	0.089	31704	14378	14.38		
Keystone	Size Wall (+x)	23	748	0	715	1															
	Side Wall (-x)	23	748	0	715	1															
	Spoke (+x)	36	748	0	614	1															
	Spoke (-x)	36	748	0	614	1															
	Size Wall (+y)	23	0	748	715	1															
	Side Wall (-y)	23	0	748	715	1															
	Spoke (+y)	36	0	748	614	1															
	Spoke (-y)	36	0	748	614	1															
	Bottom Slab	23	748	748	0	1															
	Internal Slab	23	748	748	0	1															
	Top Slab	23	748	748	0	1															