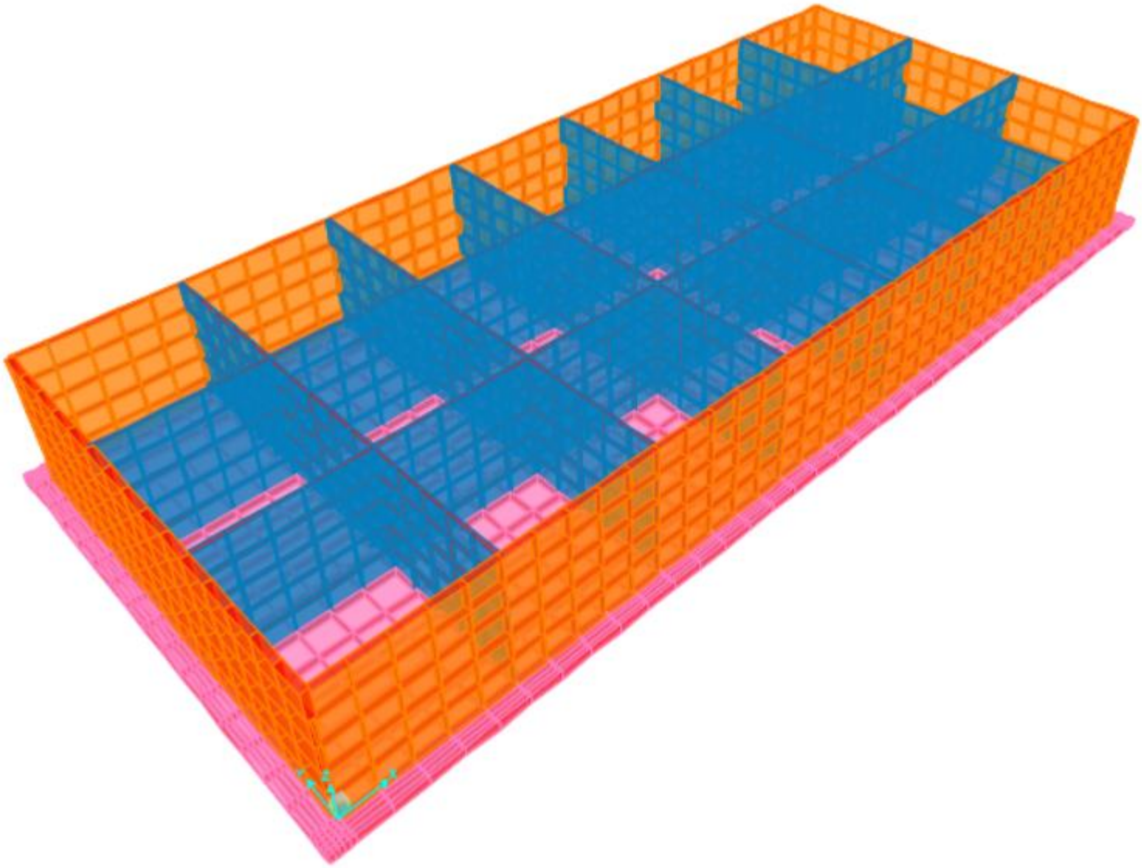
Offshore Water Purification and Power Generation Plant



CIE-4170 Construction Technology of Civil Engineering Projects

GROUP# 20

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Introduction

This report summarises all the alternatives considered and analysis done by Offshore Consultants to finally come up with an optimum solution for the construction and placement of a water purification and power generation unit.

This concept design report includes the consideration of all the boundary conditions defined by the functional requirements, location, construction technology and economy which lead to a decision for an optimum dimensional layout of the compartments and depth-width-length ratio of the floating body. The floating body is required to have a water storage capacity of 20000 m^3 in addition to enough storage space for ballast to ensure the 1.20 factor of safety at the final location against uplifting.

The floating body is to be constructed either in a floating dock or in a self-excavated graving dock, both the options have different cost, risk, draft and take different time so some decisions were required based on the cost and value-added aspect of the early completion. After construction of bottom slab and walls the floating body is taken to a dry dock where top slab is placed along with the superstructures on top including evaporators, generators and buildings. This structure is then towed to its final location by a tug boat, during this journey the structure is being acted upon by, among other loadings, the wave action with a wave height equal to half the draft of the structure with a minimum of 3m. Once at the final location the structure is immersed in the water using ballast. The weight of the ballast should be such that it gives the tank a factor of safety of 1.20 against uplifting even if all the compartments are empty, as mentioned earlier, and with all the tank full the pressure on the bearing soil should be more than 120 KN/m^2. The freeboard at the final location should be 4m. At the final location, another decision was required between keeping the structure offshore and connecting it to the land by a landing stage, strong enough to support vehicular loads, or making it an onshore structure after dredging operations.

This report includes the details for cost, risk and functional requirements which helped in making all these decisions and lead to the decision of an optimized solution for the design and construction of the floating body with water purification and power generation unit. After having a global layout of the structure, the design was optimised in terms of cost and efficiency.

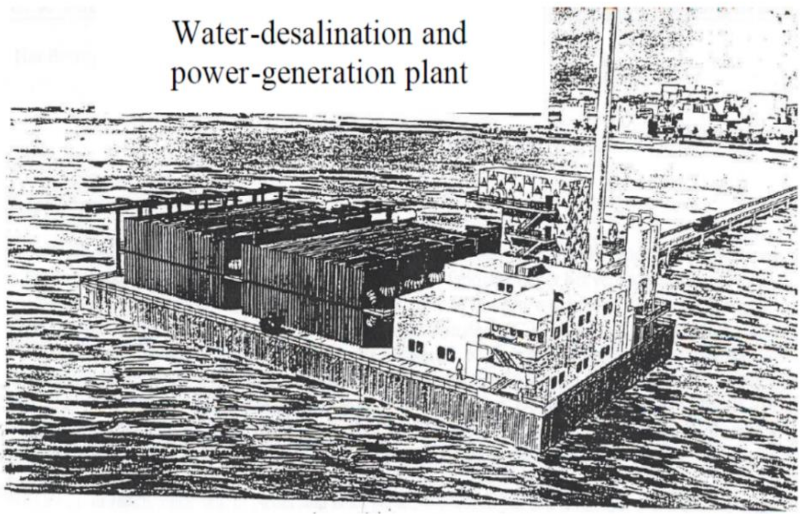

FIGURE 1 TYPICAL OFFSHORE WATER PURIFICATION PLANT

Chapter 1 Quality Control Plan

To complete the assignment in the most efficient way, we also need to have a detailed working plan which would help us achieve the goals within the specified boundary conditions and on time. For this purpose, we developed a quality control plan stating the responsibility of every person involved in the assignment along with specified deadlines. For ease of the management, this was divided in to 4 project phases.

1.1 Phase 1: Understanding the problem and recognizing all the boundary conditions

In phase one, we, as a team, tried to understand the problem while recognizing all the boundary conditions specified in the question which in the later phases would act as guiding rules. We noted down all the boundary conditions along with details on how these boundary conditions would affect certain aspects of our project. It was in this phase that we also realised the leading variables that would help us eliminate some of the alternatives considered.

1.2 Phase 2: Consideration and elimination of alternatives

During phase two, based on our understanding of the problem in phase 1, we came up with a number of alternatives based on the shape, layout, d-w-l ratio, ballast space distribution and placement of fuel tank. Six of such alternatives were considered and some were cancelled out based on the cost/risk analysis. As one of the boundary conditions restricts the maximum width of the floating body to 40m, any alternative can be constructed in both the graving and the dry dock. One of the objectives in this phase was to come up with a suitable comparison method using which we can cancel out the alternatives and choose one or two most economical alternatives for phase 3.

1.3 Phase 3: Detailed design, risk calculation and optimization of the chosen alternatives

In this phase, we work in detail on two of the most economically attractive alternatives including the risk assessment and optimization of the structural details assumed earlier for initial evaluation. Some of the side decisions were also made in this phase by comparing effect or different possible options to deal with a certain issue for example the distribution of ballast – done on the basis of cost/risk involved in each option. In this phase the most suitable alternative with the optimized structural dimensions was realised. Some other questions were also considered in this phase like the condition in case of one compartment failure and design of a body that can transfer 100 tonne mooring force.

1.4 Phase 4: Recommendation and Improvements

The difficulties we phased during the project time and the choices we had to make were discussed in this phase and some of the recommendations were provided to help improve these decisions.

1.5 Activity Plan

The activity plan was developed and modified in time as we proceeded with our project.

TABLE 1 ACTIVITY PLAN FOR THE PROJECT

Sr.	Activity	Duration (days)	Start (Dates)	Finish (Dates)	Dependency	Member
1	Understand the exercise and discuss all the boundary conditions for all the project phases	2	05-12-17	07-12-17		All Members
2	Discuss possible alternatives that we can consider for the floating body	6	08-12-17	14-12-17	1	All Members
3	Discuss and eliminate the obviously non-economical and non-feasible alternatives	1	15-12-17	15-12-2017	2	All Members
4	Make the activity plan	1	16-12-17	16-12-17	1,2,3	All Members
5	Make some side decisions which are important in calculating cost and risk of each alternative considered	2	17-12-17	18-12-17	2	All Members
6	**Consider Alternative 1 in General**	3	20-12-17	22-12-17	2,5,36	All Members
7	Hydraulic Design Safety	1	20-12-17	20-12-17	6	Ascha
8	Structural Design	1	21-12-17	21-12-17	6	Shozab
9	Cost Analysis	1	22-12-17	22-12-17	6,7,8	Tihitina
10	**Consider Alternative 2 in General**	3	23-12-17	25-12-17	2,5,36	All Members
11	Hydraulic Design Safety	1	23-12-17	23-12-17	10	Ascha
12	Structural Design	1	24-12-17	24-12-17	10	Shozab
13	Cost Analysis	1	25-12-17	25-12-17	10,11,12,36	Tihitina
14	**Consider Alternative 3 in General**	2	26-12-17	27-12-17	2,5	All Members
15	Hydraulic Design Safety	0.5	26-12-17	26-12-17	14	Ascha
16	Structural Design	0.5	26-12-17	26-12-17	14	Shozab
17	Cost Analysis	1	27-12-17	27-12-17	14,15,16	Tihitina
18	**Consider Alternative 4 in General**	2	28-12-17	29-12-17	2,5,6,10,14,36	Shozab and Tihitina
19	Hydraulic Design Safety	0.5	28-12-17	28-12-17	18	Shozab and Tihitina
20	Structural Design	0.5	28-12-17	28-12-17	18	Shozab
21	Cost Analysis	1	29-12-17	29-12-17	19,20	Tihitina
22	Realise the most suitable d-w-l ratio for layout option of other alternatives	1	29-12-17	29-12-17	6,10,14,18	Shozab and Tihitina
23	**Consider Alternative 5 in General**	2	02-01-18	03-01-18	6,10,14,18,22, 36	Shozab and Tihitina
24	Hydraulic Design Safety	0.5	02-01-18	02-01-18	23	Shozab and Tihitina
25	Structural Design	0.5	02-01-18	02-01-18	23	Shozab
26	Cost Analysis	0.5	02-01-18	03-01-18	24,25	Tihitina
27	**Consider Alternative 6 in General**	2	04-01-18	05-01-18	23,36	Shozab and Tihitina
28	Hydraulic Design Safety	0.5	04-01-18	04-01-18	27	Shozab and Tihitina
29	Structural Design	0.5	04-01-18	04-01-18	27	Shozab
30	Cost Analysis	0.5	04-01-18	05-01-18	28,29	Tihitina
31	Risk analysis and realising the most suitable alternative	3	06-01-18	08-01-18	28,29,30	Shozab and Tihitina
32	Dry Dock or Graving Dock	1	08-01-18	08-01-18	31	Shozab and

	decision					Tihitina
33	Ballast distribution decision	1	19-12-17	19-12-17	6,10,14,18,23, 27,31	Shozab and Tihitina
34	Structural Design and dimension of the planks of top slab	1	19-12-17	19-12-17	6,10,14,18,23, 27	Shozab
35	Cost and Time for Top Slab execution				34	Tihitina
36	Structural Design and dimensions of walls – inner and outer	1	19-12-17	19-12-17	5	Shozab
37	Cost and Time of Walls	1	09-01-18	09-01-18	36	Tihitina
38	Structural Design of Bottom Slab	1	10-01-18	10-01-18	31	Shozab
39	Cost and Time of Bottom Slab	1	11-01-18	11-01-18	38	Tihitina
40	Final Cost Estimation and Risk Analysis	4	12-01-18	15-01-18	38,39,37,36, 31	Tihitina
41	Optimization and Execution of the Structure	1	16-01-18	16-01-18	40	Shozab
42	Drawing details of connection and figures	4	17-01-18	20-01-18	41	Shozab
43	Recommendation and Comments	1	21-01-18	22-01-18	All	Shozab and Tihitina
44	Designing Mooring Structure	1	23-01-18	23-01-18	-	Shozab
45	Compiling Report	2	24-01-18	25-01-18	All	Shozab and Tihitina

1.6 Quality Assurance Plan

TABLE 2 QUALITY ASSURANCE PLAN

Task	Deadline Set	Task Completed
Understanding the problem and boundary conditions	18-12-2017	15-12-2017
Come up with at least 4 possible alternative and compare them	01-01-2018	05-01-2018
Detail the main alternative	10-01-2018	11-01-2018
Design the mooring force member	13-01-2018	15-01-2018
Compile the report	26-01-2018	26-01-2018

Chapter 2 Technical Details and Specifications

2.1 General Considerations

The aim of this project is to have a concept design for an offshore water purification unit and power generation plant. The dimensions (Length, Width and Height) of the floating body are optimised considering all the boundary conditions mentioned in the technical specifications. For ease of understanding and working on the project, it has been divided in four stages:

- Stage 1 – Construction of the floating body
- Stage 2 – Installation of Top Slab, Evaporators and Structures at the quay site
- Stage 3 – Towing journey of the structure from quay site to the final location
- Stage 4 – Immersion of the structure using ballast at the final location

The governing parameters and boundary conditions of each stage have been listed below.

2.2 Boundary Conditions and Technical Assumptions

2.2.1 Stage 1 – Construction of the floating body

Alternative 1 – Floating Dry Dock

- The width of the dry dock is 46m and length 260m – considering 3m of working space on both sides of the floating body we can have a maximum width of the floating body equal to 40m, also this is the maximum reach of the crane at the quay side.
- The crane available within the dry dock has a capacity of 140 tonnes and can reach any location within the dry dock – Limitation on the size of the prefabricated element, if any.
- The maximum draft available for the floating out of the floating body is 5.7m at high tide – governs the overall dimensioning and weight of the floating body to ensure that the submerged depth is less than 5.7m.

FIGURE 2 FLOATING DRY DOCK [1]

Alternative 2 – Graving Dock

- There are no specifications with respect to the dimensions of the graving dock, so we can choose them as per the governing boundary conditions in other stages of the construction. As the maximum reach of the crane at quayside is 40m, we limit our width to 40m which will be governing for the possible lengths of body.

- There is no data regarding the available crane at the graving dock, so we can decide the capacity and reach of the crane based on the maximum weight of the elements.
- The maximum draft available for the floating out of the floating body is 5.0m at high tide – governs the overall dimensioning and weight of the floating body to ensure that the submerged depth is less than 5.0m.

FIGURE 3 CAISSON IN GRAVING DOCK [2]

2.2.2 Stage 2 – Installation of the Evaporators and Structures at the Quay Site

- Crane - 100-tonne capacity with a reach of 40m and 1500 tonne-m moment capacity – Limits the size of prefabricated elements, if any (Max 37.5 tonnes at 40 m reach).
- The available draft at all time is 6.7m – governs the size of the floating body as the submerged depth cannot be more than 6.7m plus the requirement for minimum clearance from the sea bed during towing.

FIGURE 4 CONSTRUCTION AT QUAY SIDE [3]

2.2.3 Stage 3 – Towing Journey to the Final Location

- Wave height is half the draught – governs the unsubmerged depth of the structure for stability and the reinforcement in the walls as now the maximum water height is draught + half the wave height.
- Centre of gravity of the structure – governs the minimum weight and draft of the structure while floating to the final location.
- One compartment failure of the body should also be checked – to ensure that we don't lose the entire floating body in case of failure of the outer wall, preparing for an unforeseen event.

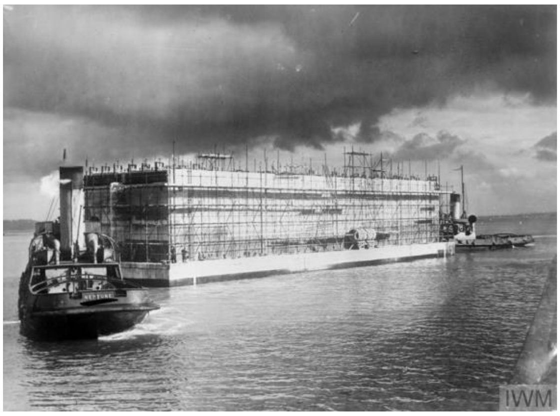

FIGURE 5 TOWING JOURNEY

2.2.4 Stage 4 – Immersion and Operation

- Free board of the structure at final location should be 4m – governs the total height and dimensioning of the structure.
- The structure should have a safety factor of 1.20 in case of uplifting considering the worst-case scenario i.e. all the compartments are empty – governs the total amount of the ballast required.
- In case that all the compartments are full i.e. there is 20000 m^3 of water in the structure, the bearing capacity of 120 KN/m^2 should not be violated – governs the overall area in relation to the weight of the structure.

2.3 Material Properties

- Compressive strength of concrete = 35 MPa
- Yield strength of reinforcement = 500 MPa
- Unit weight of reinforced concrete = 25 KN/m^3
- Unit weight of fresh water = 10 KN/m^3
- Unit weight of sea water = 10.2 KNm^3
- Unit weight of fuel = 8 KN/m^3
- Unit weight of steel = 80 KN/m^3
- Unit weight of the ballast = 30 KN/m^3 or 18 KN/m^3
- Bearing capacity of soil at final location = 120 KN/m^2

2.4 Loading on the structural elements

- The top slab is loaded by the super structure which consists of 2 evaporators weighing 12480 KN (20m x 30m) each, buildings/generator weighing 5000 KN (18m x 24m) and in some of the possible alternatives, the weight of the fuel tank.
- The outer walls of the floating body have to support the hydrostatic pressure from the seawater. The most critical condition for this would be once the body has been lowered down on to the sea bed but all the inner compartments are still empty. Along with this, the loading from the height of the ballast layer is also to be supported.
- The inner walls of the floating body have to resist the lateral pressure caused by the height of the ballast and the differences in the height of the water in adjacent compartment – which over the height of 1.375 m is equal to zero because of the provision of 1000mm diameter closable openings in the walls, the same opening will also be used by worker during construction and in case of maintenance requirement.

- The bottom slab of the floating body will be analysed for 3 conditions. First, while being towed to the final location when it is loaded by the uplift water pressure, second when it has been immersed in the water but still has zero fresh water and third while operational so all the loads are present.

2.5 Technical Considerations for various aspects of the project

2.5.1 Top Slab

We considered having both, prefabricated and cast in-situ top slab for the structure. There ease of execution and functional requirements of the structure were analysed to finally reach an optimum solution.

Cast in-situ top slab

- Easy wall to slab connection.
- No joints throughout the top slab.
- Ensures water tightness of the structure.
- Requires formwork and propping – which would be difficult to remove, so has an execution disadvantage.

Prefabricated top slab

- A lot of joints throughout the slab
- Complex wall to slab connection
- No formwork or maybe propping required – advantageous from execution point of view
- Special considerations for water tightness, if required

Considering the abovementioned pros and cons of both the systems, we decided to use a combined approach – Lattice Girder System with a part of top slab being prefabricated and the rest being cast in-situ. This would help us have simple and watertight wall to base connections without having to put formwork under the structure as the prefabricated part would act as formwork for the cast in-situ slab.

Another decision for top slab was to either construct it in the dry dock or at the quay side. The considerations of quay side include

- Easy transportation of the prefabricated planks to construction side as no transport over water required
- Lower rent of quay side compared to the rent of the dry dock
- Higher strength of walls to ensure structural safety
- Easy transport of the fuel tank for installation for alternatives with fuel tank inside one of the compartments – which is the case for most of the alternatives as it is much cheaper.

2.5.2 Positioning of the fuel tank

For positioning of the fuel tank, both the alternatives were considered, placing is on the top of top slab or placing it inside one of the compartment. Evaluating both the alternatives, following observations were made.

Fuel being in cylindrical steel tank would cause no lateral forces on the walls and they would be loaded only by water hydrostatic pressure on one side, again leading to thickening of the inner walls and execution difficulties. Although, the probability of occurrence is very low but in case of fire accident the fuel tank inside compartment may cause the structure to undergo severe damage unless concrete around is design as blast resistance. On the other hand, placing the fuel tank in one of the compartments help us reduce the size of the floating body considerably, saving time and cost.

Placing the fuel tank on the top slab causes very high uniformly distributed loads on the top slab which cost us extra reinforcement in the top slab below the fuel tank and also to have these loads within reasonable range without having to increase the thickness of the top slab locally, we need to have a tank with large diameter, requiring increase in the size of floating body. The cost and risk of both the scenarios was considered to reach an optimum solution.

2.5.3 Ballast type used for immersion of structure at final location

Two types of ballast were available for immersion of the floating body at the final location with different unit weights and costs. The cost of having both the types of ballast for a certain weight is considered to check which one costs us the least and a decision was reached on the type of ballast for lowering down the structure.

- Sand Type Ballast (Unit Weight = 18 KN/m^3)
- Gravel Type Ballast with Pore Water (Unit Weight = 30 KNm^3)

Using the ballast with lower unit weight will require higher volume for the same weight which may lead to the requirement of increasing the dimension of the floating body causing extra time and cost. This will also have a higher height and as the pressure on the walls is $P = \frac{1}{2}.\gamma_b.h^2$, having a higher height will cause more pressure on the walls.

TABLE 3 PRESSURE COMPARISON FOR DIFFERENT BALLASTS

Pressure on Walls			
Unit Weight (KN/m3)	Weight (KN)	Height (m/m^2)	**Force on Wall (KN)**
18	100	5.56	**277.78**
30	100	3.33	**166.67**

As seen from the table above, using the ballast with higher unit weight is more optimal than using the ballast with lower unit weight in terms of pressure on the walls. Comparison was also made based on the cost for a given weight for both type of ballast.

TABLE 4 COST COMPARISON FOR DIFFERENT BALLAST

Cost of Ballast				
Unit Weight (KN/m3)	Weight (KN)	Volume (m^3)	Unit Cost (€/m^3)	**Cost (€)**
18	100	5.56	25	**138**
30	100	3.33	40	**133**

As seen from the above figures, the ballast with unit weight of 30 KN/m^3 is beneficial in terms of both, the forces on the wall and cost – also it takes lower volumes and allows for reduction in volume of the overall floating body but it requires more time to ballast load the entire floating body while ballasting can be done in a day for 18 KN/m^3 ballast. The effect of this additional time was also analysed as an alternative – Alternative D which is solely considered to calculate the impact on height increase due to lower unit weight ballast and time increase due to higher unit weight ballast. The results will be presented in chapter 3.

2.5.4 Distribution of Ballast and Fresh Water

4 different alternatives were considered in terms of ballast space and fresh water distribution and the one with the least structural requirement – saving the cost of concrete and reinforcement, and the easiest execution with highest repetition factor was chosen keeping in mind the constraints caused by the functional requirements of the body.

The different ballast and fresh water distributions are listed below

- Ballast only in the corner compartments
- Ballast only in the edge compartments
- Ballast in all the compartments
- Ballast on top

Ballast only in the corner compartment

Having a higher height of ballast in some of the compartments rather than having it spread out uniformly throughout is not very appealing from a structural point of view. This would require thicker walls of the corner compartments because of the difference in the lateral loads caused by water and ballast, also the extreme case would be an empty compartment next to the ballast filled compartment, this would require very high percentages of reinforcement and thickening of walls.

From the execution point of view thicker walls of corner compartments will lead to a lower repetition factor in terms of formwork. However, it has a higher repetition factor in terms of ballast placing inside the compartment.

Considering the plant in operation, we can also say that if such a corner compartment fails, we would lose a significant amount of ballast – although the probability of this happening is very low but in such a scenario we would induce significant torsion in the structure and it might lead to complete failure, also the boundary condition for a 1.20 factor of safety against uplifting might be violated.

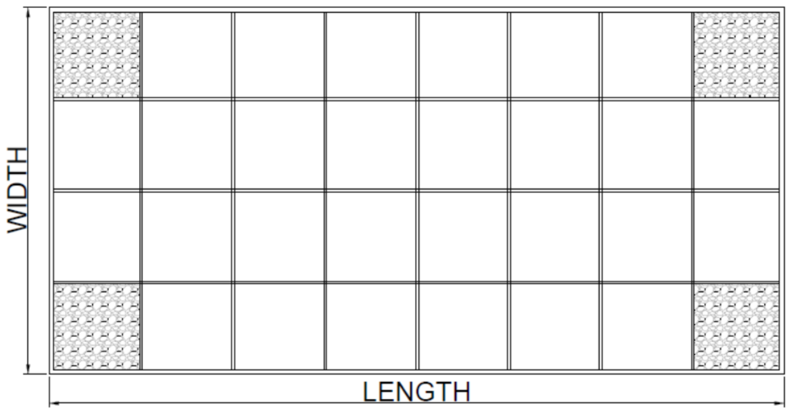

FIGURE 6 BALLAST DISTRIBUTION OPTION 1

Ballast only in the edge compartments

In our design we are providing 2 x 2 m manholes in the top slab to access the compartments in all the edge compartments which connect to the inner compartment through an opening in the wall. Simply speaking when the structure is complete we can only add the ballast in the edge compartments due to functional requirements of the floating body. Also, the height of the ballast required when all the edge compartments are used is such that the lateral forces on the inner walls are not very significant and the minimum reinforcing steel can take the loading – saving us cost of extra reinforced concrete required when higher heights of ballast are provided.

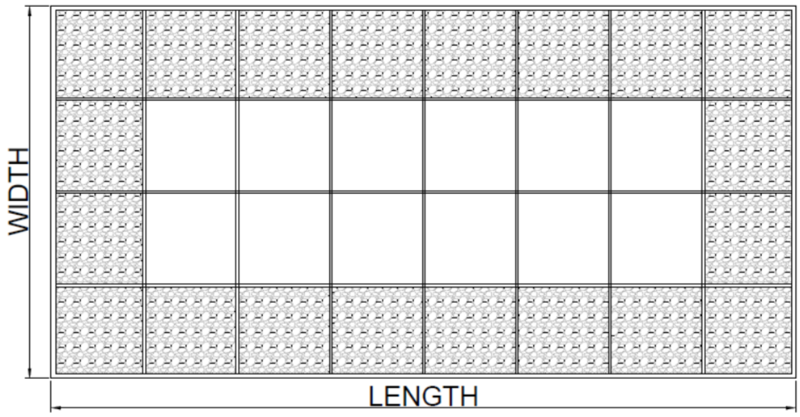

FIGURE 7 BALLAST DISTRIBUTION OPTION 2

Ballast in all the compartments

Structurally and in terms of load distribution this seems like the most reasonable alternative because in this case the height of ballast would be the least so very low loads on outer walls and no lateral loads on the inner walls because ballast it is in every compartment and the loading from one cancels the loading from others but as we have opening only in the edge compartment, if we choose to go by this alternative then we would need to add some ballast before casting the top slab while at quay side. This would cause very complex loading on the bottom slab while at the quay side and will also increase the draft of the floating body causing an increase in the lateral forces during the towing journey.

To consider this alternative another option would be to have openings in the centre compartments too but that is not possible given the layout of superstructure on the top slab. Considering all the pros and cons – we can say that this alternative gives equal advantage in terms of structural considerations but causes more time or requires some special arrangements which are not very feasible so we will cancel this one out.

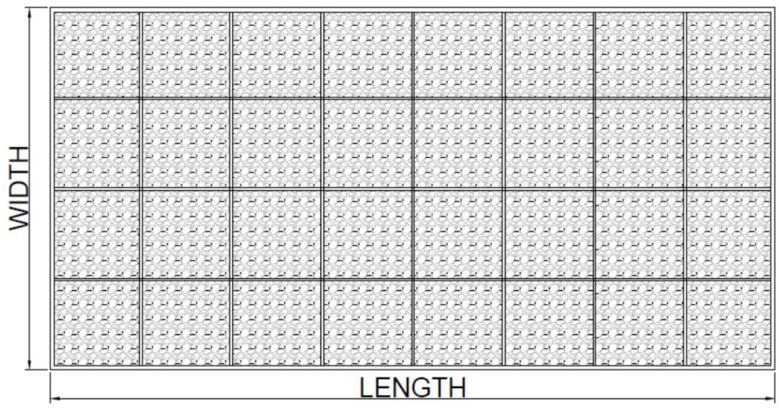

FIGURE 8 BALLAST DISTRIBUTION OPTION 3

Ballast on top slab

Placing the ballast on the top slab is also practiced, but in our case, this does not seem like a suitable choice to consider because of the excessive weight and volume of the ballast required.

2.5.5 Formwork

We can choose between plywood formwork or steel formwork. The steel formwork has higher initial cost but if the repetition factor of formwork is greater than 13 it is actually cheaper than plywood formwork because the plywood formwork requires repairs after certain cycles. In the considered alternatives, we have a maximum clear height if 9.4 m which gives us a repetition factor of 6 with casting of 1.5m of concrete each cycle. The height of 1.5m for one cycle is also attractive in terms of the maximum concrete height that can be poured without segregation, higher heights however can be achieved with some simple techniques. As the repetition factor is less than 13 and as evident from the figure 9 the plywood formwork is cheaper so we decided to use that.

The possibility of self-climbing formwork was also considered but the initial cost of having such a system is too high. If we had to construct a higher structure with a number of copies, simply with a high repetition factor, that might have been the best option for formwork but in our case, the plywood formwork is used in segmented crane climbing formwork form. As the plywood formwork would need to be lifted in segments, the workers can use the opening in the compartments (to ensure that the water level rise equally in each compartment) to access the inside if each compartment. As this is a rather difficult task, this might cause us some extra time but a lot of construction experience and knowledge is required to estimate the increase in time caused by such an execution method, so for now we stick to the cost values specified in the cost estimate table.

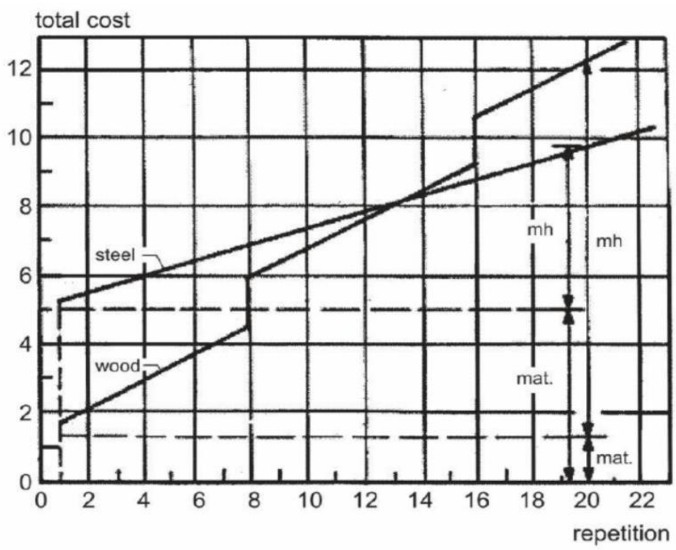

FIGURE 9 PLYWOOD AND STEEL FORMWORK COMPARISON

2.5.6 Dredging-Platform Cost Analysis

At the final location, there was another decision to be made between providing a long platform or dredging the soil for a longer length. To decide between these two options, we did the cost analysis between the two. As seen from the figure 10, once the dredging units are installed and mobilized, the rate of increment in cost per meter of dredging is very low compared to the increment in cost per meter of the platform provided. Also, the risks involved in platform are comparatively more than in dredging so we decided to provide the minimum length of the platform and dredge out most of the soil. For this purpose, at the final location the platform equal to the length for allowance of 1:5 slope was provided for platform and the rest of the volume of soil was dredged out.

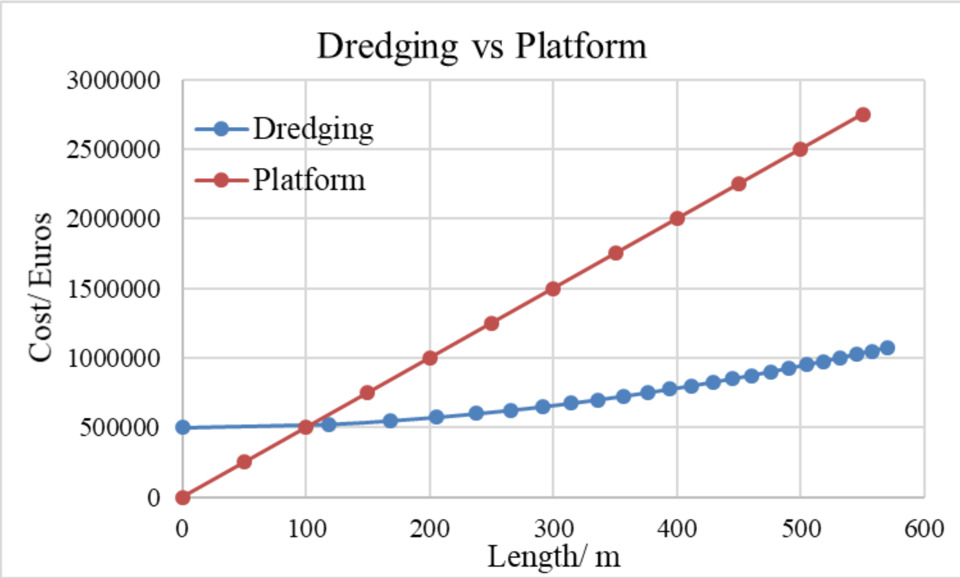

FIGURE 10 COST COMPARISON FOR DREDGING AND PLATFORM

Chapter 3 Alternatives

The decision for consideration of alternatives is also a cyclic process. To consider executable alternatives the following iterative method was used in which special attention was given to fulfilling of all the boundary conditions set in the project description.

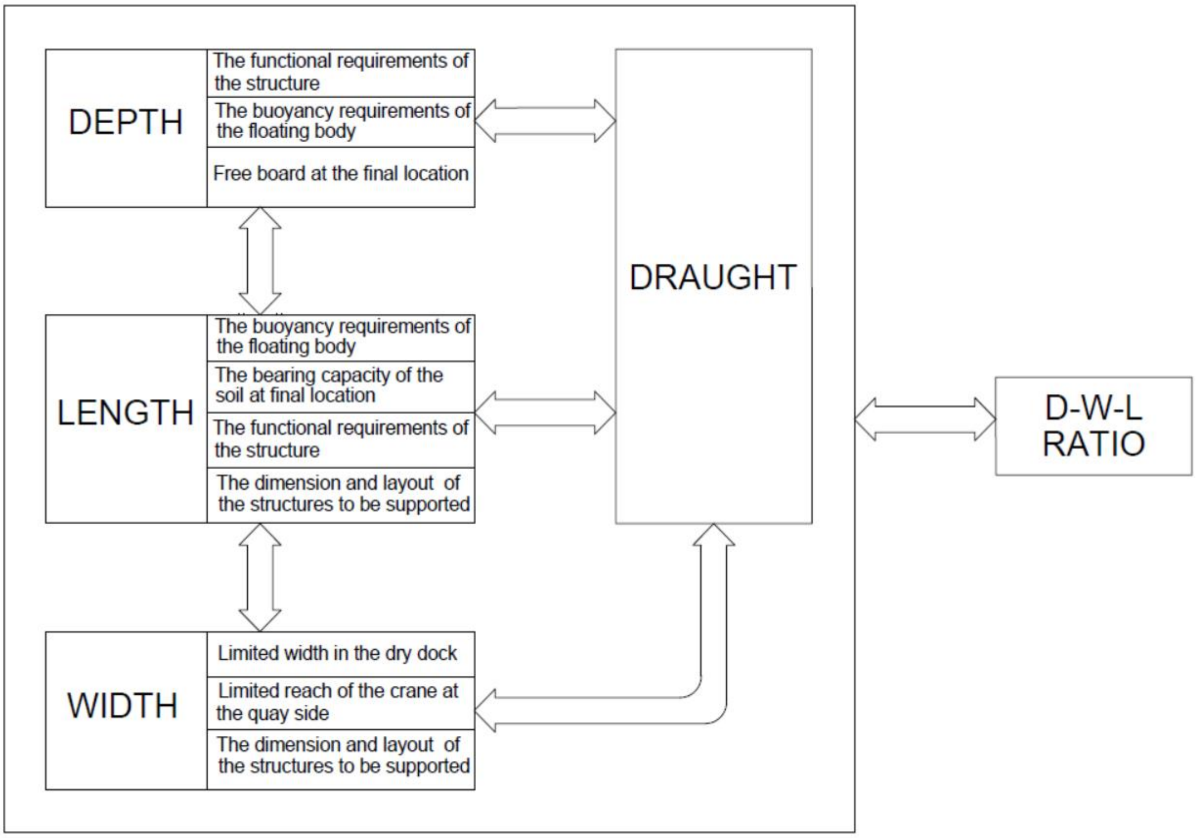

FIGURE 11 ITERATIONS FOR OPTIMUM D-W-L RATIO

To reach an optimal layout and d-w-l ratio of the floating body alternatives were considered based on the various depth-width-length ratio, layout of superstructure on the top slab, dimensions of internal compartment and ballast distribution. We considered also the variation in shape of the floating body, for example cylindrical and hexagonal but as the boundary conditions restrict us in one direction to 40m and to utilize the maximum in the second direction, rectangular floating body is optimal. Also, the repetition factor for formwork and training of the workers is optimal if all compartments have the same shape and dimension which is not possible for other structural shapes. Therefore, only the alternatives with rectangular floating body were considered. For dimensions and layout of the compartments, a 9.70m width was chosen based on the dimensions of the fuel tank which would fit in one of the compartments with least waste of volume. This was further changed to check the effect of varying dimension of cell on the cost of structure. The brief description of all the alternatives is given below – this is further explained in the section 3.1 to 3.6.

- A – Minimum dimensions of compartment size and body
- B – Effect of height reduction and increasing length
- C – Effect of placing fuel tank at top

- D – Effect of using the ballast with 18 KN/m^3 on the best from the above 3 alternatives
- E – Effect of reducing the compartment size of the best of the first 3 alternatives
- F – Effect of increasing the compartment size of the best of the first 3 alternatives

TABLE 5 DIMENSION AND COMPARTMENT SIZE OF ALTERNATIVES

Alternative	D-W-L	Fuel Tank	Compartment Size
A	10.0 – 40.7 – 70.7	Inside	9.70 x 9.70
B	9.5 – 40.7 – 80.7	Inside	9.70 x 9.70
C	9.25 – 40.7 – 90.7	Outside	9.70 x 9.70
D	10.5 – 40.7 – 70.7	Inside	9.70 x 9.70
E	10.5 – 37.7 – 82.1	Inside	7.10 x 7.10
F	10.5 – 33.7 – 77.7	Inside	10.70 x 10.70

The cost/risk analysis was done for all the alternatives considering both, the construction in graving dock and construction in dry dock. For initial cost estimate and comparison of the alternatives, the steel reinforcement is assumed to be equal to 140 kg/m^3 of concrete for alternative A, B, C and D, 120kg/m^3 for alternative E and 160 kg/m^3 for alternative F. Detailed cost and risk analysis will be done for the most economically attractive alternatives. To include the value-added time aspect in the alternative selection, the reference is the alternative which requires the maximum time for completion.

3.1 Alternative A

This gives us the least volume of concrete because it has the least dimension so the effect of reduction in volume of concrete can be compared with other factors in this case. However, this alternative does not allow the use of 18 KN/m^3 ballast due to its lower volume compared to other designs and as a consequence we would get some extra time to immerse it at the final location. The boundary condition check for alternative A and the costs of both, the construction in dry dock and the construction in graving dock is considered below.

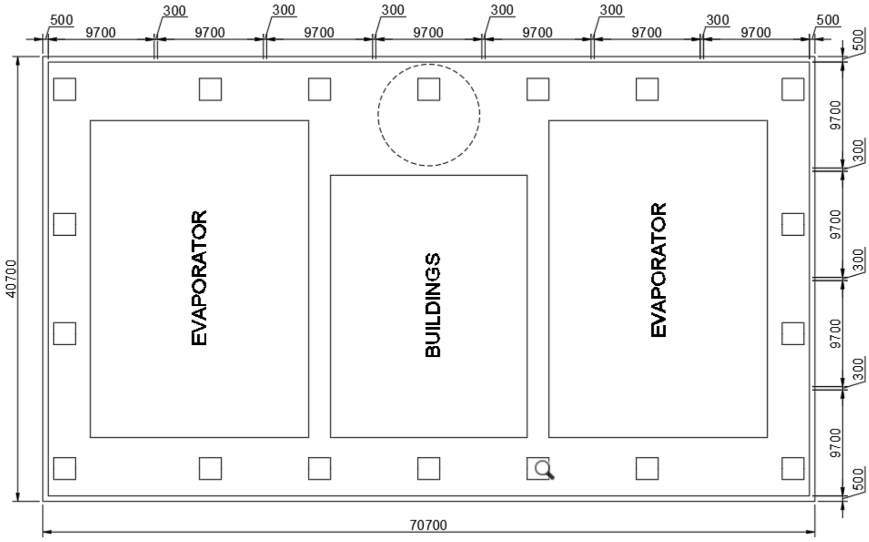

FIGURE 12 PLAN OF ALTERNATIVE A

The boundary conditions and dimensions are also tabulated.

TABLE 6 DIMENSIONS AND BOUNDARY CONDITIONS FOR ALTERNATIVE A

Length	70.7	m
Width	40.7	m
Height	10	m
Top Slab Thickness	0.4	m
Bottom Slab Thickness	0.7	m
Outer Wall Thickness	0.5	m
Inner Wall Thickness	0.3	m
Draft - Stage 1	3.49	m
Draft - Stage 2	5.64	m
Final Freeboard	4.36	m
Compartment Size	9.7	m
Unit Weight of Ballast	30	KN/m3
Height of Ballast	1.12	m
Soil Pressure	77.60	KN/m2
Total Free Volume	3.37	%

TABLE 7 COST ESTIMATE FOR ALTERNATIVE A - GRAVING DOCK

COST				
Graving Dock				
Activity	Quantity	Unit	Cost	Con Time
Graving Dock - Earthwork				
Dry Excavation (Including slope levelling and transport)	17013	m3	68052	2
Sheet Piling	353	Ton	617511	13
Dewatering (Installation plus per meter dewatering)	6439	m2	48939	3
Civil Work				
Concrete Bottom Slab	2014	m3	241709	3
Concrete walls	2162	m3	259492	6
Steel Reinforcement (including cutting and Installation)	585	Ton	584735	19
Formwork (Plywood Panels)	3354	m2	67081	
Formwork Preparation	14685	m2	734267	12
Repetition Factor=6	243	m2	m	
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	1703	m3	68106	4
Graving dock Site Overhead Cost	12	weeks	248000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation	1151	m3	138120	7
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	5	weeks	93695	27
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	114852	m3	574261	3
Platform	30	m	150000	
Sea bed protection	84	m2	3342	1
Ballasting				
Volume of Ballast	1480	m3	59212	3
Labour				
Concrete	750	mnh	30000	
Reinforcement	5847	mnh	233894	
Formwork	1677	mnh	67081	
Cost			4787496	
5% Indexing			239375	
Total Cost			5026871	
Time Value	0	week	0	
Net Cost			5026871	
Risk			1379394	
Total Time – week				19.1

The cost estimation for construction in dry dock is as follows.

TABLE 8 COST ESTIMATE FOR ALTERNATE A - DRY DOCK

COST				
Dry Dock				
Activity	Quantity	Unit	Cost	Con Time
Civil Work				
Concrete Bottom Slab	2014	m3	241709	3
Concrete Wall	2162	m3	259492	6
Steel Reinforcement (including cutting and Installation)	585	Ton	584735	19
Formwork (Plywood Panels)	3354	m2	67081	
Formwork Preparation	14685	m2	734267	12
Repetition Factor=6				
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	1703	m3	68106	4
Time for flooding dock and towing				1
Rent (including crew and energy supplies)	9	weeks	990000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation	1151	m3	138120	7
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	5	weeks	93695	27
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	114852	m3	574261	3
Sea bed protection	84	m2	3342	1
Platform	30	m	150000	
Ballasting				
Volume of Ballast	1480	m3	59212	3
Labour				
Concrete	750	mnh	30000	
Reinforcement	5847	mnh	233894	
Formwork	1677	mnh	67081	
Cost			4794994	
5% Indexing			239750	
Total Cost			5034744	
Time Value	3	week	850000	
Net Cost			4184744	
Risk			1441033	
Total Time – Week				15.7

3.2 Alternative B

The dimensions of the Alternative A have been varied in such a way that the use of both the ballast type is possible so a comparison can be made between the delay caused by using the heavier ballast type and the difference in application cost of them as mentioned in the section 2.5.3. The reduction in height also gives an advantage in terms of dredging at the final location. The combined effect of these factors will be visible in the cost of this alternative.

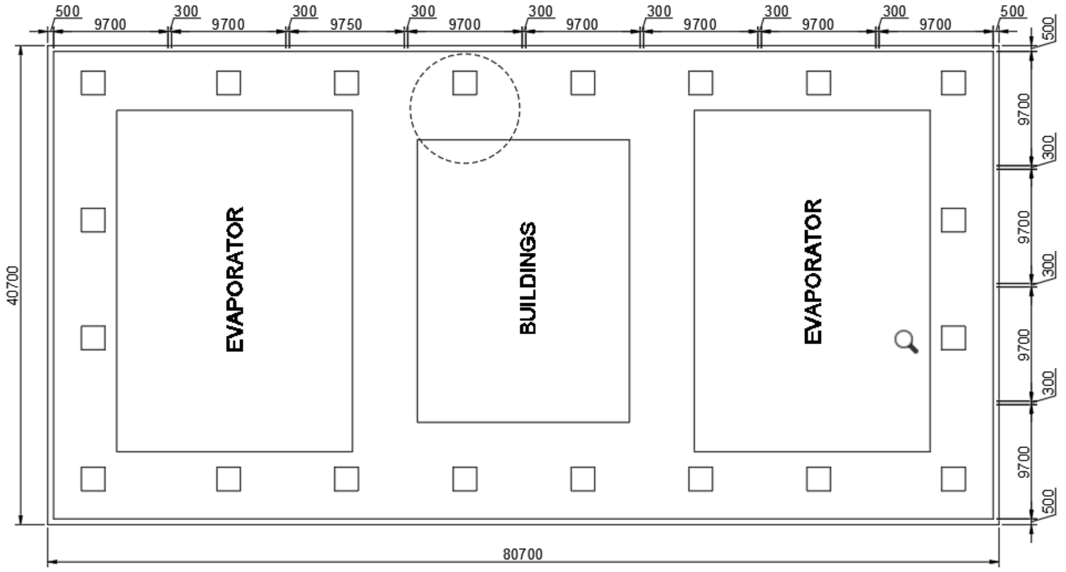

FIGURE 13 PLAN OF ALTERNATIVE B

TABLE 9 DIMENSIONS AND BOUNDARY CONDITIONS FOR ALTERNATIVE B

Length	80.7	m
Width	40.7	m
Height	9.5	m
Top Slab Thickness	0.4	m
Bottom Slab Thickness	0.7	m
Outer Wall Thickness	0.5	m
Inner Wall Thickness	0.3	m
Draft - Stage 1	3.63	m
Draft - Stage 2	5.38	m
Final Freeboard	4.12	m
Compartment Size	9.7	m
Unit Weight of Ballast	30	KN/m3
Height of Ballast	1.31	m
Soil Pressure	69.63	KN/m2
Total Free Volume	7.53	%

TABLE 10 COST ESTIMATE FOR ALTERNATIVE B - GRAVING DOCK

COST				
Graving Dock				
Activity	Quantity	Unit	Cost	Con Time
Graving Dock - Earthwork				
Dry Excavation (Including slope levelling and transport)	18714	m3	74856	2
Sheet Piling	378	Ton	661183	14
Dewatering (Installation plus per meter dewatering)	7066	m2	53704	4
Civil Work				
Concrete Bottom Slab	2299	m3	275897	3
Concrete walls	2298	m3	275799	6
Steel Reinforcement (including cutting and Installation)	644	Ton	643645	21
Formwork (Plywood Panels)	3799	m2	75982	
Formwork Preparation	15923	m2	796171	13
Repetition Factor = 6	274	m2	m	
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	1929	m3	77146	4
Graving dock Site Overhead Cost	13	weeks	268000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation	1314	m3	157656	7
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	5	weeks	99131	29
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	110158	m3	550790	3
Platform	28	m	137500	
Sea bed protection	91	m2	3642	1
Ballasting				
Volume of Ballast	1980	m3	79185	4
Labour				
Concrete	822	mnh	32880	
Reinforcement	6436	mnh	257458	
Formwork	1900	mnh	75982	
Cost			5096605	
5% Indexing			254830	
Total Cost			5351435	
Time Value	0	week	0	
Net Cost			5351435	
Risk			1458068	
Total Time – Weeks				20.5

The cost estimate for alternate B in Dry Dock.

TABLE 11 COST ESTIMATE FOR ALTERNATIVE B - DRY DOCK

COST				
Dry Dock				
Activity	Quantity	Unit	Cost	Con Time
Civil Work				
Concrete Bottom Slab	2299	m3	275897	3
Concrete Wall	2298	m3	275799	6
Steel Reinforcement (including cutting and Installation)	644	Ton	643645	21
Formwork (Plywood Panels)	3799	m2	75982	
Formwork Preparation	15923	m2	796171	13
Repetition Factor = 6				
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	1929	m3	77146	4
Time for flooding dock and towing				1
Rent (including crew and energy supplies)	10	weeks	1056000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation				7
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	5	weeks	99131	29
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	110158	m3	550790	3
Platform	28	m	137500	
Sea bed protection	91	m2	3642	1
Ballasting				
Volume of Ballast	1980	m3	79185	4
Labour				
Concrete	822	mnh	32880	
Reinforcement	6436	mnh	257458	
Formwork	1900	mnh	75982	
Cost			4937207	
5% Indexing			246860	
Total Cost			5184067	
Time Value	4	week	950000	
Net Cost			4234067	
Risk			1518764	
Total Time - Weeks				16.7

3.3 Alternative C

This alternative allows the placement of the fuel tank on the top slab instead of inside one of the compartments which a significant reduction in height but increased length. This compared to Alternative A will also give us an idea of the variation in cost with increasing length and reducing height of the structure so an optimum can be realised.

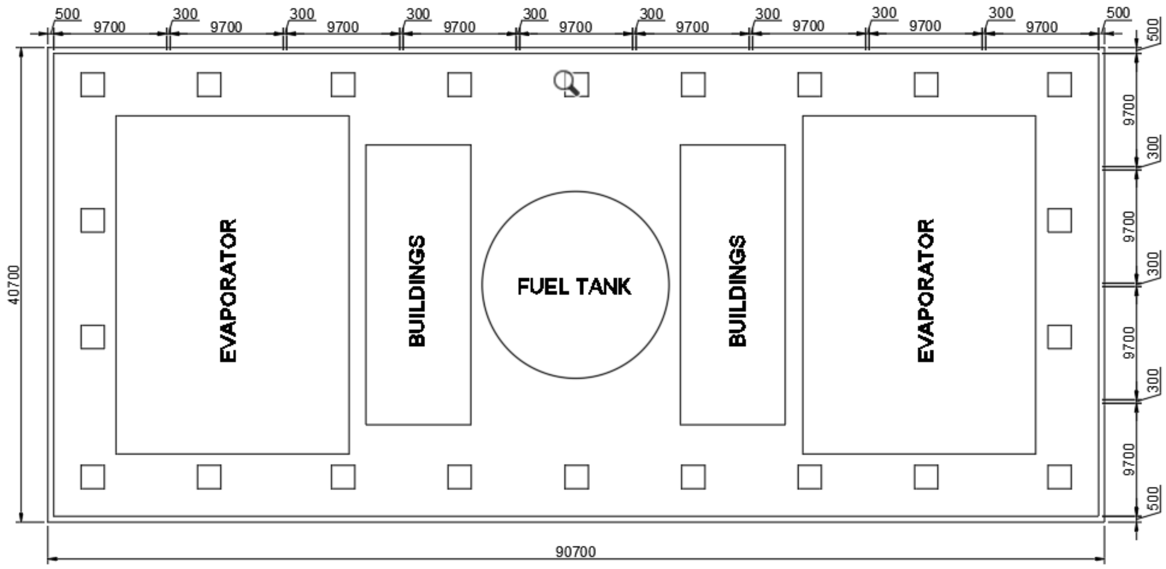

FIGURE 14 PLAN OF ALTERNATIVE C

TABLE 12 DIMENSIONS AND BOUNDARY CONDITIONS FOR ALTERNATIVE C

Length	90.7	m
Width	40.7	m
Height	9.25	m
Top Slab Thickness	0.4	m
Bottom Slab Thickness	0.7	m
Outer Wall Thickness	0.5	m
Inner Wall Thickness	0.3	m
Draft - Stage 1	3.56	m
Draft - Stage 2	5.20	m
Final Freeboard	4.05	m
Compartment Size	9.7	m
Unit Weight of Ballast	30	KN/m3
Height of Ballast	1.44	m
Soil Pressure	63.38	KN/m2
Total Free Volume	12.33	%

TABLE 13 COST ESTIMATE FOR ALTERNATIVE C - GRAVING DOCK

COST				
Graving Dock				
Activity	Quantity	Unit	Cost	Con Time
Graving Dock - Earthwork				
Dry Excavation (Including slope levelling and transport)	20415	m3	81660	3
Sheet Piling	403	Ton	704854	15
Dewatering (Installation plus per meter dewatering)	7693	m2	58469	4
Civil Work				
Concrete Bottom Slab	2584	m3	310085	4
Concrete walls	2666	m3	319968	6
Steel Reinforcement (including cutting and Installation)	735	Ton	735062	23
Formwork (Plywood Panels)	4244	m2	84888	
Formwork Preparation	17401	m2	870060	15
Repetition Factor = 6	327	m2	m	
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	2155	m3	86186	5
Graving dock Site Overhead Cost	15	weeks	300000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation	1477	m3	177192	8
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	5	weeks	104568	30
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	112922	m3	564608	3
Platform	26	m	131250	
Sea bed protection	99	m2	3942	1
Ballasting				
Volume of Ballast	2441	m3	97644	5
Labour				
Concrete	941	mnh	37640	
Reinforcement	7351	mnh	294025	
Formwork	2122	mnh	84888	
Cost			5546987	
5% Indexing			277349	
Total Cost			5824336	
Time Value	0	week	0	
Net Cost			5824336	
Risk			1558962	
Total Time - Weeks				22.6

The cost estimate of Alternative C for construction in dry floating dock.

TABLE 14 COST ESTIMATE FOR ALTERNATIVE C - DRY DOCK

COST				
Dry Dock				
Activity	Quantity	Unit	Cost	Con Time
Civil Work				
Concrete Bottom Slab	2584	m3	310085	4
Concrete Wall	2666	m3	319968	6
Steel Reinforcement (including cutting and Installation)	735	Ton	735062	23
Formwork (Plywood Panels)	4244	m2	84888	
Formwork Preparation	17401	m2	870060	15
Repetition Factor = 6				
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	2155	m3	86186	5
Time for flooding dock and towing				1
Rent (including crew and energy supplies)	11	weeks	1188000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation	1477	m3	177192	8
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	5	weeks	104568	30
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	112922	m3	564608	3
Platform	26	m	130000	
Sea bed protection	99	m2	3942	1
Ballasting				
Volume of Ballast	2441	m3	97644	5
Labour				
Concrete	941	mnh	37640	
Reinforcement	7351	mnh	294025	
Formwork	2122	mnh	84888	
Cost			5588754	
5% Indexing			279438	
Total Cost			5868191	
Time Value	3	week	750000	
Net Cost			5118191	
Risk			1618715	
Total Time - Weeks				18.4

3.4 Alternative D

The forth alternative was decide after comparing the cost of the first three. As seen from the tables above, alternative A is most economically attractive so it was further modified for further optimization.

This alternative is similar to alternative A except that in this we have increased the height of the structure to allow for the use of ballast with unit weight of 18 KN/m^3 – so essentially these are the same but when using the ballast with lower unit weight. The comparison between the cost of Alternative D and Alternative A will give us an idea of the effect on cost of using different ballast so we can choose the most economical one.

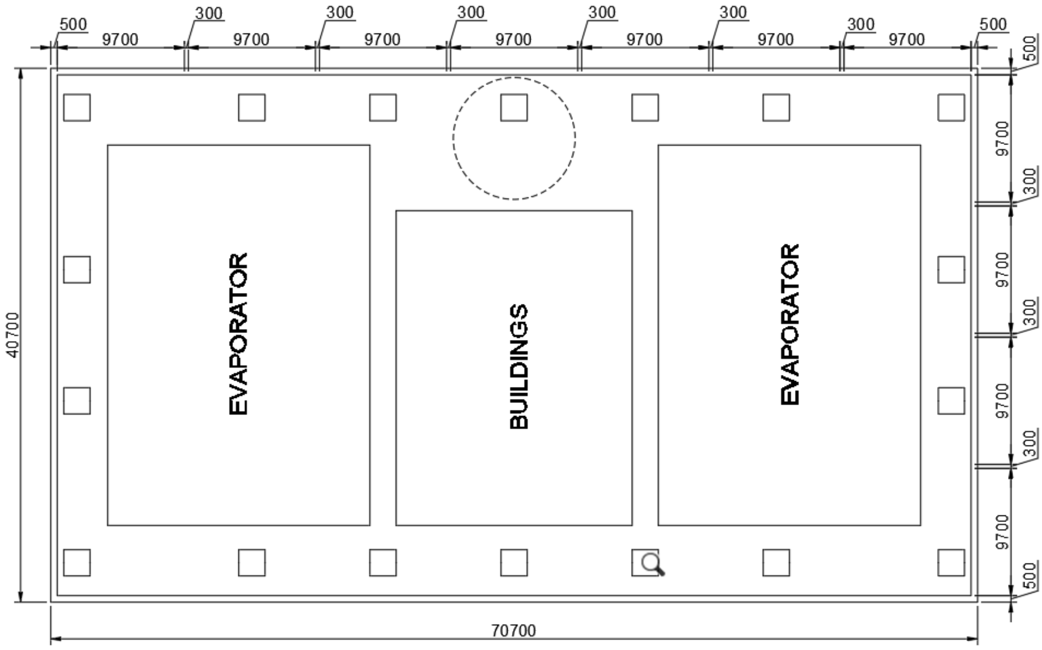

FIGURE 15 PLAN FOR ALTERNATIVE D

TABLE 15 DIMENSIONS AND BOUNDARY CONDITIONS FOR ALTERNATIVE D

Length	70.7	m
Width	40.7	m
Height	10.5	m
Top Slab Thickness	0.4	m
Bottom Slab Thickness	0.7	m
Outer Wall Thickness	0.5	m
Inner Wall Thickness	0.3	m
Draft - Stage 1	3.87	m
Draft - Stage 2	5.74	m
Final Freeboard	4.76	m
Compartment Size	9.7	m
Unit Weight of Ballast	18	KN/m3
Height of Ballast	2.49	m
Soil Pressure	78.55	KN/m2
Total Free Volume	1.47	%

TABLE 16 COST ESTIMATE FOR ALTERNATIVE D - GRAVING DOCK

COST				
Graving Dock				
Activity	Quantity	Unit	Cost	Con Time
Graving Dock - Earthwork				
Dry Excavation (Including slope levelling and transport)	17013	m3	68052	2
Sheet Piling	353	Ton	617511	13
Dewatering (Installation plus per meter dewatering)	6439	m2	48939	3
Civil Work				
Concrete Bottom Slab	2158	m3	258974	3
Concrete walls	2442	m3	293024	6
Steel Reinforcement (including cutting and Installation)	644	Ton	643998	21
Formwork (Plywood Panels)	3388	m2	67769	
Formwork Preparation	15286	m2	764281	12
Repetition Factor = 6	261	m2	m	
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	1703	m3	68106	4
Graving dock Site Overhead Cost	13	weeks	256000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation	1151	m3	138120	7
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	5	weeks	93695	27
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	134790	m3	673948	3
Platform	33	m	162500	
Sea bed protection	84	m2	3342	1
Ballasting				
Volume of Ballast	3079	m3	76976	1
Labour				
Concrete	820	mnh	32800	
Reinforcement	6440	mnh	257599	
Formwork	1694	mnh	67769	
Cost			5093402	
5% Indexing			254670	
Total Cost			5348072	
Time Value	0	week	0	
Net Cost			5348072	
Risk			1438117	
Total Time - Weeks				19.1

The cost estimate for the construction of Alternative D in dry dock is listed below.

TABLE 17 COST ESTIMATE FOR ALTERNATIVE D - DRY DOCK

COST				
Dry Dock				
Activity	Quantity	Unit	Cost	Con Time
Civil Work				
Concrete Bottom Slab	2158	m3	258974	3
Concrete Wall	2442	m3	293024	6
Steel Reinforcement (including cutting and Installation)	644	Ton	643998	21
Formwork (Plywood Panels)	3388	m2	67769	
Formwork Preparation	15286	m2	764281	12
Repetition Factor = 6				
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	1703	m3	68106	4
Time for flooding dock and towing				1
Rent (including crew and energy supplies)	9	weeks	1034000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation				7
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	5	weeks	93695	27
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	134790	m3	673948	3
Sea bed protection	84	m2	3342	1
Platform	33	m	162500	
Ballasting				
Volume of Ballast	3079	m3	123162	1
Labour				
Concrete	820	mnh	32800	
Reinforcement	6440	mnh	257599	
Formwork	1694	mnh	67769	
Cost			5044966	
5% Indexing			252248	
Total Cost			5297214	
Time Value	3	week	850000	
Net Cost			4447214	
Risk			1499756	
Total Time - Weeks				15.7

3.5 Alternative E

As per the results of cost analysis of the previous alternatives, the general d-w-l ratio which proved to be most attractive was considered for other internal layouts of walls. Alternative E is beneficial in terms of availability of formwork as 7.20m is a standard size used in almost every building construction. This added benefit might be used for reduction in the total cost of formwork but it was not done here as it requires a lot of experience. As the dimensions of the unsupported concrete are lower in this alternative, we used a steel ratio of 120 Kg/m^3 for this alternative.

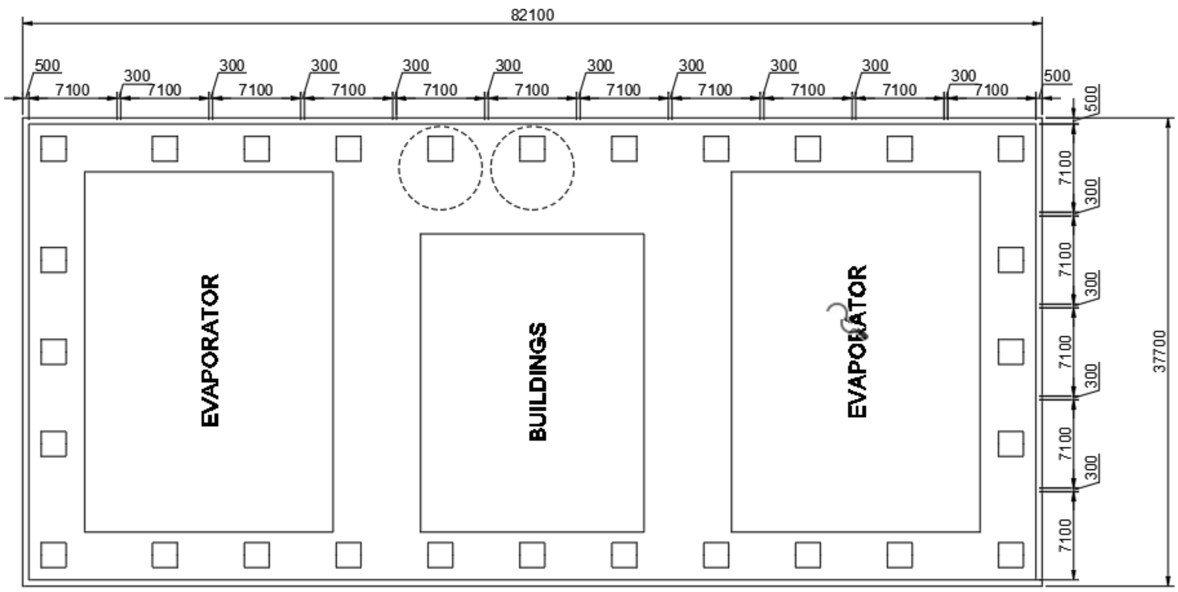

FIGURE 16PLAN FOR ALTERNATIVE E

The dimension and cost estimates are as follows.

TABLE 18 DIMENSIONS AND BOUNDARY CONDITIONS FOR ALTERNATIVE E

Length	82.1	m
Width	37.7	m
Height	10.5	m
Top Slab Thickness	0.4	m
Bottom Slab Thickness	0.7	m
Outer Wall Thickness	0.5	m
Inner Wall Thickness	0.3	m
Draft - Stage 1	4.34	m
Draft - Stage 2	6.08	m
Final Freeboard	4.42	m
Compartment Size	7.1	m
Unit Weight of Ballast	30	KN/m3
Height of Ballast	0.89	m
Soil Pressure	72.95	KN/m2
Total Free Volume	11.89	%

TABLE 19 COST ESTIMATE FOR ALTERNATIVE E - GRAVING DOCK

COST				
Graving Dock				
Activity	Quantity	Unit	Cost	Con Time
Graving Dock - Earthwork				
Dry Excavation (Including slope levelling and transport)	17952	m3	71809	2
Sheet Piling	374	Ton	654195	14
Dewatering (Installation plus per meter dewatering)	6812	m2	51769	3
Civil Work				
Concrete Bottom Slab	2167	m3	259994	3
Concrete walls	3033	m3	363915	6
Steel Reinforcement (including cutting and Installation)	624	Ton	623910	20
Formwork (Plywood Panels)	3721	m2	74417	
Formwork Preparation	20198	m2	1009897	15
Repetition Factor=6	323	m2	m	
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	1830	m3	73215	4
Graving dock Site Overhead Cost	13	weeks	268000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation	1238	m3	148568	13
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	6	weeks	121280	34
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	156483	m3	782413	4
Platform	33	m	162500	
Sea bed protection	90	m2	3594	1
Ballasting				
Volume of Ballast	1079	m3	43168	3
Labour				
Concrete	948	mnh	37920	
Reinforcement	6239	mnh	249564	
Formwork	1860	mnh	74417	
Cost			5574546	
5% Indexing			278727	
Total Cost			5853273	
Time Value	0	week	0	
Net Cost			5853273	
Risk			1517269	
Total Time - Weeks				21.7

The cost of alternative E in Dry Dock.

TABLE 20 COST ESTIMATE FOR ALTERNATIVE E - DRY DOCK

COST				
Dry Dock				
Activity	Quantity	Unit	Cost	Con Time
Civil Work				
Concrete Bottom Slab	2167	m3	259994	3
Concrete Wall	3033	m3	363915	6
Steel Reinforcement (including cutting and Installation)	624	Ton	623910	20
Formwork (Plywood Panels)	3721	m2	74417	
Formwork Preparation	20198	m2	1009897	15
Repetition Factor=6				
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	1830	m3	73215	4
Time for flooding dock and towing				1
Rent (including crew and energy supplies)	10	weeks	1078000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation	1238	m3	148568	13
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	6	weeks	121280	34
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	156483	m3	782413	4
Sea bed protection	90	m2	3594	1
Platform	33	m	162500	
Ballasting				
Volume of Ballast	1079	m3	43168	3
Labour				
Concrete	948	mnh	37920	
Reinforcement	6239	mnh	249564	
Formwork	1860	mnh	74417	
Cost			5606772	
5% Indexing			280339	
Total Cost			5887111	
Time Value	4	week	900000	
Net Cost			4987111	
Risk			1578302	
Total Time - Weeks				18.1

3.6 Alternative F

Alternative F help us realise the effect of increasing the dimension of walls and slabs compared to the most suitable alternative from the above-mentioned alternatives. For this case the highest steel ratio of 160Kg/m^3 was used to include the effect of increasing the dimension of the unsupported concrete. Also, due to relatively shorter dimension in width, we cannot access all the edge compartments but we choose a configuration for filling of ballast that is similar to the edge configuration in terms of area – so no changes in cost or lateral forces. The ballast distribution is explained in chapter 8.

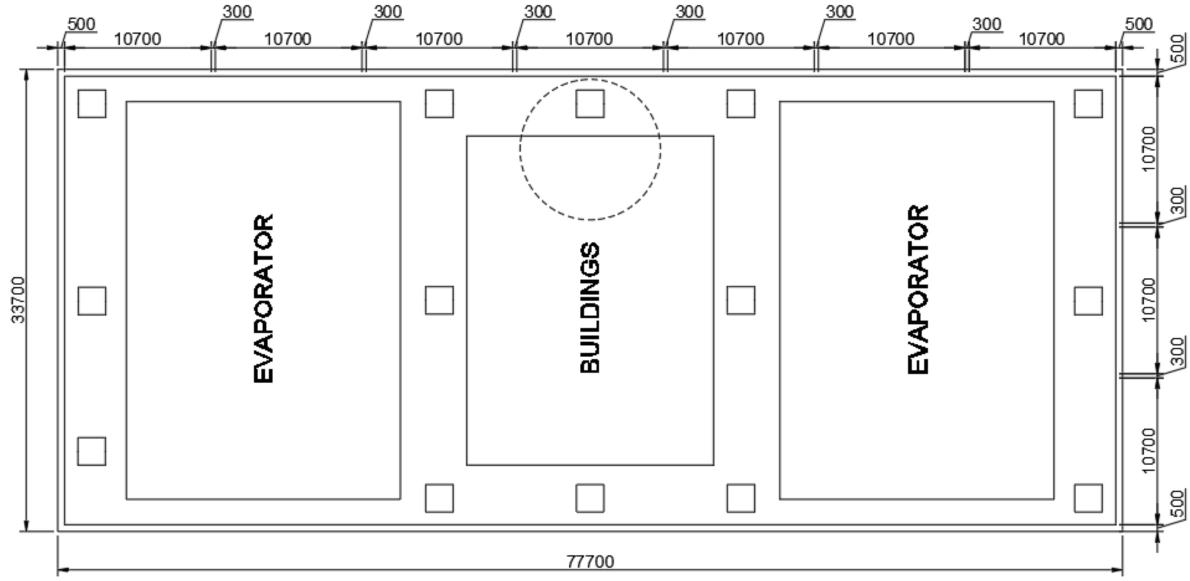

FIGURE 17 PLAN OF ALTERNATIVE F

TABLE 21 DIMENSIONS AND BOUNDARY CONDITIONS FOR ALTERNATIVE F

Length	77.7	m
Width	33.7	m
Height	10.5	m
Top Slab Thickness	0.4	m
Bottom Slab Thickness	0.7	m
Outer Wall Thickness	0.5	m
Inner Wall Thickness	0.3	m
Draft - Stage 1	3.82	m
Draft - Stage 2	5.77	m
Final Freeboard	4.73	m
Compartment Size	10.7	m
Unit Weight of Ballast	30	KN/m3
Height of Ballast	0.88	m
Soil Pressure	83.55	KN/m2
Total Free Volume	0.58	%

TABLE 22COST ESTIMATE FOR ALTERNATIVE F - GRAVING DOCK

COST				
Graving Dock				
Activity	Quantity	Unit	Cost	Con Time
Graving Dock - Earthwork				
Dry Excavation (Including slope levelling and transport)	15963	m3	63852	2
Sheet Piling	353	Ton	617511	13
Dewatering (Installation plus per meter dewatering)	6110	m2	46438	3
Civil Work				
Concrete Bottom Slab	1833	m3	219953	3
Concrete walls	2013	m3	241618	6
Steel Reinforcement (including cutting and Installation)	615	Ton	615428	20
Formwork (Plywood Panels)	3063	m2	61268	
Formwork Preparation	13317	m2	665875	10
Repetition Factor=6				
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	1573	m3	62926	4
Graving dock Site Overhead Cost	12	weeks	244000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation	1047	m3	125688	5
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	4	weeks	87400	26
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	148133	m3	740664	3
Platform	33	m	162500	
Sea bed protection	84	m2	3342	1
Ballasting				
Volume of Ballast	1207	m3	48288	3
Labour				
Concrete	692	mnh	27680	
Reinforcement	6154	mnh	246171	
Formwork	1532	mnh	61268	
Cost			4841869	
5% Indexing			242093	
Total Cost			5083962	
Time Value	0	week	0	
Net Cost			5083962	
Risk			1381224	
Total Time - Weeks				18.8

The cost of alternative F in Dry Dock.

TABLE 23 COST ESTIMATE FOR ALTERNATIVE F - DRY DOCK

COST				
Dry Dock				
Activity	Quantity	Unit	Cost	Con Time
Civil Work				
Concrete Bottom Slab	1833	m3	219953	3
Concrete Wall	2013	m3	241618	6
Steel Reinforcement (including cutting and Installation)	615	Ton	615428	20
Formwork (Plywood Panels)	3063	m2	61268	
Formwork Preparation	13317	m2	665875	10
Repetition Factor=6				
Gravel Layer to Build Up Water Pressure				
Volume of Gravel	1573	m3	62926	4
Time for flooding dock and towing				1
Rent (including crew and energy supplies)	9	weeks	968000	
Quay Side Super Structure Installation				
Top Slab Prefab slab installation	1047	m3	125688	5
Top Slab reinforcement				3
Top Slab concreting				14
Super Structure Installation				5
Rent	4	weeks	87400	26
Dredging, Shore Protection work and Landing Platform				
Installation and Operation	1	Fix Cost	500000	
Volume of Soil	148133	m3	740664	3
Platform	33	m	162500	
Sea bed protection	84	m2	3342	0
Ballasting				
Volume of Ballast	1207	m3	48288	3
Labour				
Concrete	692	mnh	27680	
Reinforcement	6154	mnh	246171	
Formwork	1532	mnh	61268	
Cost			4838067	
5% Indexing			241903	
Total Cost			5079971	
Time Value	4	week	900000	
Net Cost			4179971	
Risk			1391194	
Total Time - Weeks				15.2

3.7 Risk Analysis

All construction projects face uncertainties at one phase or another, therefore risk analysis and subsequent management schemes are necessary. For our project we have analysed the possible risks and their monetary values which we added to the total cost of our alternatives so that a proper comparison can be drawn. After discussion we decided to calculate risk for several cases including Monetary, Environmental, Technical, Project, Human, Social and Mechanical. For each category different possible events are listed with their respective cause and consequence. To compute the risk cost of an event, we graded the probability of that event occurring and its impact on cost and time as 'Very High', 'High', 'Moderate', 'Low', 'Very Low' as mentioned in the reference document [4].The grade was then related to a value in table 24, which was later multiplied by the cost if the event occurs. To compute the cost of an event occurs is also a very detailed and complex task, we tried to do it in a simplified manner by calculating the total global effect of the occurrence of event, like for a bad slab panel – damage equal to cost of one panel was assumed.

Although grading the likelihood of occurrence and impact a certain event will have on a project requires rigorous research, analysis and vast experience, we have made our best estimate considering the information at hand with the given time.

TABLE 24 PROBABILITY/IMPACT MATRIX [4]

Probability / Impact matrix						
VH	0.8	0.08	0.24	0.4	0.56	0.72
H	0.4	0.04	0.12	0.2	0.28	0.36
M	0.2	0.02	0.06	0.1	0.14	0.18
L	0.1	0.01	0.03	0.05	0.07	0.09
VL	0.05	0.005	0.015	0.025	0.035	0.045
Probability / Impact		0.1	0.3	0.5	0.7	0.9
		VL	L	M	H	VH

The results for risk of all alternatives in dry and graving dock are listed below.

TABLE 25 RISK VALUES FOR ALTERNATIVE

	Graving Dock Risk (€)	Dry Dock Risk (€)
Alternative A	1379394	1441033
Alternative B	1458068	1518764
Alternative C	1558962	1618715
Alternative D	1438117	1499756
Alternative E	1517269	1578302
Alternative F	1381224	1391194

3.8 Alternative Comparison

The data in the previous cost tables has been summarised to choose the most economical alternative so we can work on that in detail and apply all the design checks and calculate the actual cost of construction. As these cost values alone are not an appropriate measure of comparison because we also need to keep in mind the associated risks, the risk analysis was also done for construction in both, the dry dock and the graving dock for all alternatives mentioned in table 25.

To have a proper comparison of the costs of alternatives, the risk associated with each alternative was added to the total cost of the alternative and a comparison between the final costs was drawn.

First the cost of construction of an alternative in both the graving and dry dock is listed. The values of dry dock (as they requires less time) includes the time value aspect. From these two, the best alternative was considered and the total cost of the best alternatives were then compared between alternatives. The project with the least cost is the most optimal one.

TABLE 26 COST COMPARISON OF ALTERNATIVES

	Graving Dock		Dry Dock		Optimal Cost	Time	Time Value	Net Cost
	Cost	Time	Cost	Time				
Alternative A	6406265	19.1	5625777	15.7	6475777	15.7	2.7	5800777
Alternative B	6809503	20.5	5752831	16.7	6702831	16.7	1.7	6277831
Alternative C	7383298	22.6	6736906	18.4	7486906	18.4	0	7486906
Alternative D	6786189	19.1	5946970	15.7	6796970	15.7	2.7	6121970
Alternative E	7370542	21.7	6565413	18.1	7465413	18.1	0.3	7390413
Alternative F	6465186	18.8	5571165	15.2	6471165	15.2	3.2	5671165

As seen from table 26, the alternative F has the least cost with alternative A very close to it so for further analysis and design both the alternatives were considered so the optimal can be reached in a more detailed and reliable way.

Chapter 4 Structural Design – Alternative A

4.1 Draught Calculation

The draft of each alternative was calculated using the balance between the uplifting forces caused by water and the weight of the floating body. The following expression was used

- Stage 1: $Draught = \frac{(V_{bs} + V_{walls}) * \gamma_c}{Area_{bs} * \gamma_w} = \frac{108386}{3104*10} = 3.49m$
- Stage 2: $Draught = \frac{V_c * \gamma_c + W_{evap} + W_{str} + W_{ft}}{A_{bs} * \gamma_w} = \frac{175004}{3104*10} = 5.64m$

4.2 Water Tightness

As this is a hydraulic structure and the structural walls are required to separate fresh water from sea water, we need our structure to be water tight. Therefore, the checks of water tightness have been applied on outer walls and bottom slab. Many researchers have provided useful crack width calculations and approximation to ensure water tightness of the structures, some of these curves are shown in figure 18.

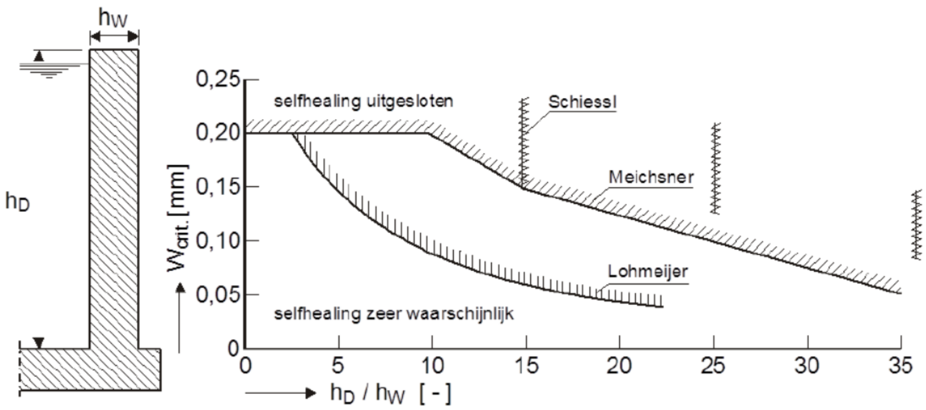

FIGURE 18 VARIOUS CURVES FOR WATER TIGHTNESS CALCULATION

For this project we decided to use the Lohmeijer [5] approximation for allowable crack width and subsequent reinforcement calculations as per the recommended method in Capita Selecta Concrete Structures course.

Input data for calculation

- Type of curve used: Lohmeijer
- Height of Liquid
- Slab Thickness
- W_{crit} – read off from the graph
- ϕ - Diameter of the reinforcing bar

$$w_{mo} = \frac{w_{crit}}{\gamma_s \cdot \gamma_\infty}; \gamma_s \text{ and } \gamma_\infty \text{ are taken equal to 1.3 as per Eurocode}$$

$$\sigma_{s,cr} = \frac{n.\sigma_{cr}}{2} + \sqrt{(\frac{n.\sigma_{cr}}{2})^2 + \frac{f_{ccm}.E_s.(0.5w_{mo})^{1.18}}{0.4\phi}}$$

σ_{cr} = Nominal Tensile stress caused by hydrostatic load:0.3N/mm^2

$$n = \frac{E_s}{E_c}$$

$$N = \frac{1}{2}.\gamma_w.(H_l)^2$$

$$A_s = \frac{N}{\sigma_{s,cr}}$$

The area of steel calculated from this calculation was compared with the area of steel required to resist the flexural moments acting on the structure and the bigger was provided so both the actions can be ensured safely. The tables for calculations are attached in the calculation section of each element for both alternatives.

4.3 Top Slab

The unit weights of the super structures were analysed and the maximum loading was considered for the design of the top slab for two of the selected alternatives. The same reinforcement was provided throughout the top slab in all directions for simplification of the execution at the site except for underneath the fuel tank, in the alternatives having fuel tank on the top slab but that alternative was not chosen so equal reinforcement in all directions. The details of reinforcement with sketches are mentioned in the following chapters.

TABLE 27 SUPERSTRUCTURE LOADS ON TOP SLAB

Structure	Area (m^2)	Loading (KN/m^2)
Evaporators	600	20.8
Building and Generators	432	11.57
Fuel Tank	Variable	Variable

As seen from the table, the maximum loading is caused by the evaporators so 20.8 KN/m^2 loading was used for reinforcement except of under the fuel tank – which was varied for optimal solution.

The first alternative basis was the various layouts possible for the distribution of structures on the top slab. To keep calculation simple and avoid varying heights of ballast in the compartments, all the layouts considered have zero net moments globally around the centre of gravity in either direction – this is also important for towing journey.

4.3.1 Moments

The moments on the top slab were calculated using SAP 2000 (FEM based design software). The figures of moments for general reinforcement distribution is shown in the figures 19-24 below.

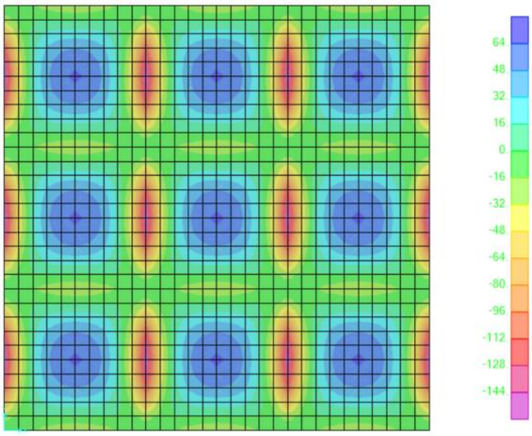

FIGURE 19 MXX TOP SLAB

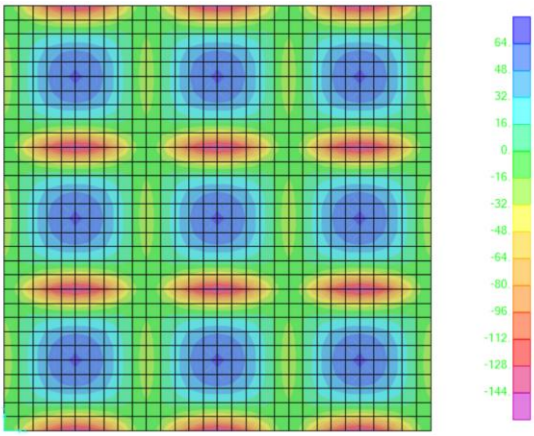

FIGURE 20 MYY TOP SLAB

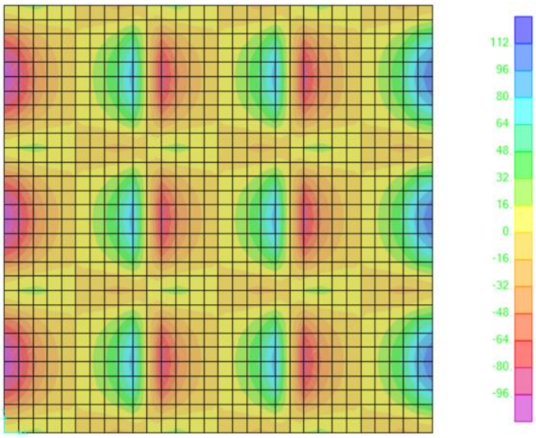

FIGURE 21 VX TOP SLAB

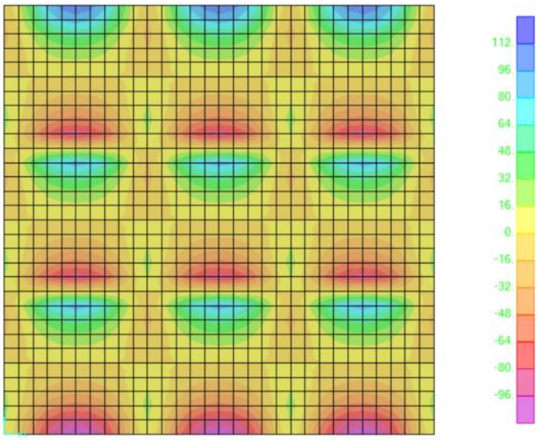

FIGURE 22 VY TOP SLAB

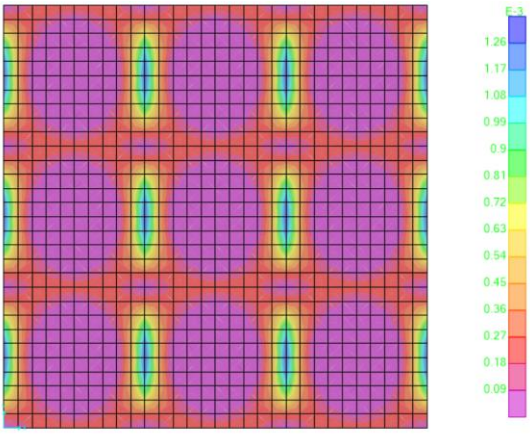

FIGURE 23 STEEL AREA TOP FACE TOP SLAB

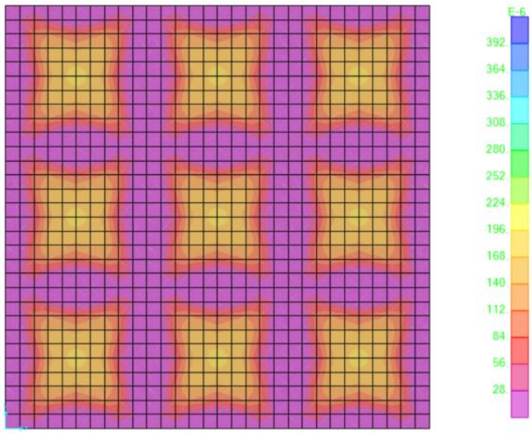

FIGURE 24 STEEL AREA BOTTOM FACE TOP SLAB

To calculate the amount of reinforcement required to resist these moments, following procedure was used. For top slab the clear cover is taken as 40 mm.

$$d = total\ height - cover = 400 - 40 - 0.5 * \phi(16) = 352mm$$

$$z = 0.87d = 0.87(352) = 306mm$$

As mentioned earlier, the global safety factor of 1.7 is used for calculation of required reinforcing area.

$$As = \frac{1.7 * M}{z * fy} = \frac{1.7(144 * 10^6\ (Nmm))}{306\ (\text{mm}) * 500\ (\text{N}/mm^2)} = 1600\ \text{mm2}$$

TABLE 28 MOMENTS AND STEEL AREA FOR TOP SLAB

	c (mm)	d (mm)	z (mm)	Moments (KNm)	Area (mm^2)
Mvx	50	352	306	144	1600
Msx	50	352	306	64	711
Mvy	50	352	306	144	1600
Msy	50	352	306	64	711

The spacing and bar provided are calculated as mentioned in the table below.

TABLE 29 REINFORCING BARS AND SPACING TOP SLAB

Area (mm^2)	Roh (%)	Roh Min (%)	Roh Provided (%)	Bars
1600	0.40	0.21	0.40	ϕ 16 @ 125
711	0.18	0.21	0.21	ϕ 16 @ 275
1600	0.40	0.21	0..40	ϕ 16 @ 125
711	0.18	0.21	0.21	ϕ 16 @ 275

To calculate the amount of steel per meter cube of concrete in each element the following table was used. The results of the total steel to concrete ratio for the floating body are mentioned in the summary as well. To calculate the length of the bar, the length was assumed 1.125 time as a thumb rule to cater for the lap splicing and all extra steel in connections.

TABLE 30 REINFORCEMENT RATIO OF TOP SLAB

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	16	16	16	16
Spacing	125	275	125	275
L1 (m)	40.7	40.7	70.7	70.7
L2 (m)	70.7	70.7	40.7	40.7
Number of Bars	326.0	148.0	566.0	257.0
Bar Length (m)	45.8	45.8	79.5	79.5
Mass (kg)	24159.6	10968.2	72864.2	33085.0
Total Steel (kg)	141077.0			
Ratio (kg/m3)	**122.6**			

4.3.2 Shear Check

The maximum shear force as calculated in SAP 2000 (figure 21-22) is checked against the shear resistance of the slab. The procedure for top slab is outlined below.

$$v_{rd} = C_{rd} * k * (100 * \rho * fck)^{\frac{1}{3}} = 0.12 * \left(1 + \sqrt{\frac{200}{352}}\right) * (100 * 0.0040 * 35)^{\frac{1}{3}} = 0.507\ \frac{N}{m}/mm2$$

$$v_{min} = 0.035 * k^{\frac{2}{3}} * \sqrt{fck} = 0.035 * \left(1 + \sqrt{\frac{200}{352}}\right)^{\frac{2}{3}} * \sqrt{35} = 0.301\ \frac{N}{m}/mm2$$

$$v_{Ed} = \frac{Vx}{b * d} = \frac{60000}{1000 * 352} = 0.170\ \frac{N}{m}/mm2$$

TABLE 31 SHEAR CHECK OF TOP SLAB

	V (KN/m)	VEd(N/m/mm2)	VRd(N/m/mm2)	Vmin(N/m/mm2)	Status
Vx	60	0.170	0.507	0.301	OK
Vy	60	0.170	0.507	0.301	OK

4.3.3 Deflection

The deflection of the top slab was calculated for two cases. First, when the lattice girder is loaded by the dead weight of fresh concrete and second when the combined thickness of floor is loaded by the live loads and dead loads of the top structure i.e. evaporators and buildings.

The following equation was used to calculate the deflection of the floor

$$w = \frac{5}{384 * EI} * w * l^4 < \frac{l}{250}$$

As there is no specific criterion for the deflection of the top slab of the caisson because generally such criterion is based on the comfort of inhabitants, still we assumed the same maximum allowable deflection.

- Case 1: Fresh concrete on 200mm thick lattice girder

$$w = \frac{5.q.l^4}{384.EI} = \frac{5 * 5 * 9700^4}{384 * (34000 * \frac{1}{12} * 1000 * 200^3)} = 25.42mm$$

So, propping is required while pouring the top slab.

- Case 2: Final deflection when loaded with super structure

$$w = \frac{5.q.l^4}{384.EI} = \frac{5 * 25.8 * 9700^4}{384 * (34000 * \frac{1}{12} * 1000 * 400^3)} = 16.4mm$$

The calculated value was checked against 38.8mm as mentioned earlier. This value is also a very conservative value for the deflection. The obtained value of deflection from SAP analysis is considerably lower.

4.4 Bottom Slab

4.4.1 Moments

The bottom slab of the floating body will have to sustain 3 different loading conditions. The loading on the bottom slab for these conditions was calculated and the one with the most extreme loading was used to calculate the moments and subsequent reinforcing steel.

- Loading Case 1: Floating

To calculate the loading on the bottom slab during floating phase we assumed that the height of the water that cause the uplift on the bottom slab is draught plus one fourth of the draught with a minimum of 1.5m.

$$UDL = -(Draught + 1.5)\gamma_w + Thichness * \gamma_c = -(5.64 + 1.5) * 10.2 + 0.70 * 25 = -55.32 KN / m^2$$

- Loading Case 2: Ballasted but Empty

This is when the floating body is immersed at its final location but the compartments are still empty with respect to the fresh water storage.

$$UDL = -BP + Thichness * \gamma_c + \frac{W_{Ballast}}{Area_{Ballast}} = -13.17 + 0.7 * 25 + \frac{44409}{1694} = 30.54 KN / m^2$$

- Loading Case 3: Operational (Water + Ballast)

During the operational phase the bottom slab will be exposed to the combined load of ballast and fresh water. The loading in this phase is

$$UDL = -BP + Thickness * \gamma_c + \frac{W_{ballast}}{Area_{ballast}} + \frac{W_{water}}{Area_{water}} = -77.60 + 0.7 * 25 + \frac{44409}{1694} + \frac{200000}{2634.52} = 42.03 KN / m^2$$

As seen from the above calculations that, for bottom slab, the most critical phase is while its floating so the reinforcement was calculated for this phase for both top and bottom steel as the values are not so different. The analysis results of SAP2000 are shown in the figures below.

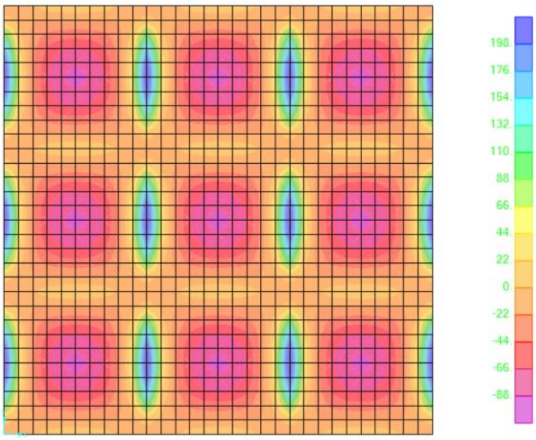

FIGURE 25BOTTOM SLAB MXX

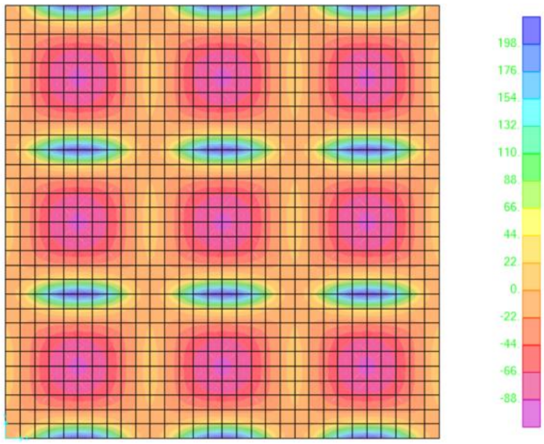

FIGURE 26BOTTOM SLAB MYY

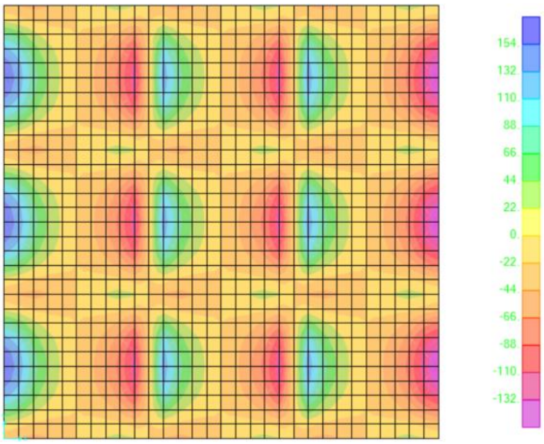

FIGURE 27 BOTTOM SLAB VX

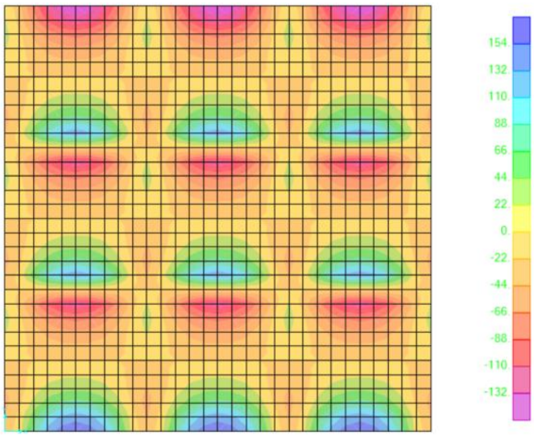

FIGURE 28BOTTOM SLAB VY

The same cover of 50mm is used for the bottom slab as for the top slab as it is enough as per the provisions in the hydraulic design manual [6].

$$As = \frac{1.7 * M}{z * fy} = \frac{1.7(180 * 10^6\ (Nmm))}{558(\text{mm}) * 500\ (\text{N}/mm^2)} = 1096 \text{ mm2}$$

TABLE 32 FLEXURAL STEEL AREA FOR BOTTOM SLAB

	c (mm)	d (mm)	z (mm)	Moments	Area

				(KNm)	(mm^2)
Mvx	50	637.5	558	180	1096
Msx	50	637.5	558	120	731
Mvy	50	637.5	558	180	1096
Msy	50	637.5	558	120	731

The bottom slab also needs to be checked for water tightness and for that the Lohmeijer [5] crack width criterion was used as mentioned in the section 4.2 of the report.

TABLE 33 WATER TIGHTNESS CHECK BOTTOM SLAB

Height of Liquid	8.5	m
Slab Thickness	0.7	m
Hl/Thickness	12.1429	
W critical	0.095	mm
Wmo	0.06	mm
N	361.25	KN
Sigma ct	0.51607	KN/m2
Es/Ec	6.18	
fccm	43	Mpa
fctm	3.15	Mpa
Sigma cr	2.3625	Mpa
Sigma s at cr	117.12	MPa
As	3084.44	mm2/m

TABLE 34 REINFORCING BARS AND SPACING BOTTOM SLAB

Area (mm^2)	Roh (%)	Roh Min (%)	Roh Provided (%)	Bars
3084	0.48	0.21	0.48	ϕ 25 @ 150
3084	0.48	0.21	0.48	ϕ 25 @ 150
3084	0.48	0.21	0.48	ϕ 25 @ 150
3084	0.48	0.21	0.48	ϕ 25 @ 150

The steel per unit volume of concrete is calculated as follows.

TABLE 35REINFORCEMENT RATIO OF BOTTOM SLAB

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	25	25	25	25
Spacing	150	150	150	150
L1 (m)	42.7	42.7	72.7	72.7
L2 (m)	72.7	72.7	42.7	42.7
Number of Bars	485.0	485.0	285.0	285.0
Bar Length (m)	48.0	48.0	81.8	81.8
Mass (kg)	92063.6	92063.6	92108.0	92108.0
Total Steel (kg)	368343.3	Kg		
Ratio (kg/m3)	169.5	Kg/m3		

4.4.2 Shear Check

The same procedure and equations were used as mentioned in the section top slab to check the shear resistance of the bottom slab against the applied loads.

$$v_{rd} = C_{rd} * k * (100 * \rho * fck)^{\frac{1}{3}}$$

$$v_{min} = 0.035 * k^{\frac{2}{3}} * \sqrt{fck}$$

$$v_{Ed} = \frac{Vx}{b * d}$$

TABLE 36SHEAR CHECK FOR BOTTOM SLAB

	V (KN/m)	VEd(N/m/mm2)	VRd(N/m/mm2)	Vmin(N/m/mm2)	Status
Vx	152	0.238	0.479	0.278	OK
Vy	152	0.238	0.479	0.278	OK

4.5 Static Stability – Towing Journey

The floating body has to go through two towing journeys and the static stability of the floating body is to be ensured for both the project phases so that the wave action does not cause the overturning of the body. For that the metacentric height check was applied as mentioned in the hydraulic design manual [6].

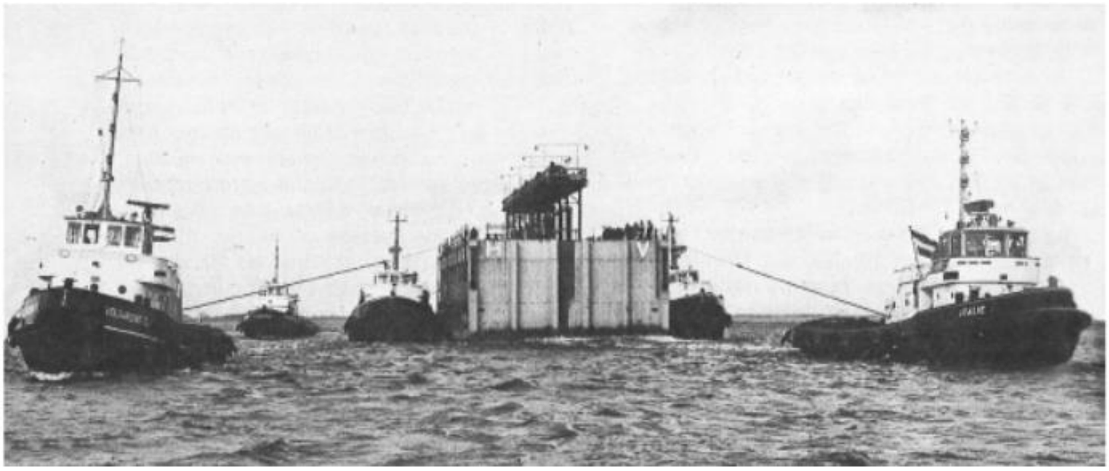

FIGURE 29 TOWING OF CAISSON [7]

To calculate the metacentric height following procedure was used as mentioned in [7] section 4.4.5

1- Definition of a reference level – K
2- The distance between the centre of gravity of the body and the reference level was calculated which is called $\overline{KG}$

$$\overline{KG} = \frac{\sum_{i=1}^{n} V_i . e_i . \gamma_i}{\sum_{i=1}^{n} V_i . \gamma_i}$$

- Vi = Volume of the element
- ei = distance between the centre of gravity of element and the line K
- ϒi = Unit weight of the element

3- The distance between the centre of bouncy of the element and the reference line K

$$\overline{KB} = \frac{Draught}{2} = \frac{\text{Weight of the tank (varies per phase)}}{\text{2*Length of tank*Width of Tank*}\gamma_{\text{water}}}$$

4- The weaker direction of moment of inertia for the floating body, which in our case is

$$\mathrm{I}_{\text{weaker}} = \frac{Length * Width^3}{12}$$

5- The displaced volume of the water by the floating body is computed and is called V

6- The $\overline{BM}$ is computed using

$$\overline{BM} = \frac{I_{weak}}{V}$$

7- The metacentric height is computed using

$$h_m = \overline{KB} + \overline{BM} - \overline{KG}$$

The same procedure was repeated for all the considered alternatives and for both the construction phases and the metacentric height was checked against 0.50 to ensure static stability of the body.

The calculations are shown for one of the alternatives for both the cases in the following table.

TABLE 37 HYDRAULIC STABILITY CHECK

	Towing from the dock to quay side	Towing from quay side to the final location
1	K is the base of floating body	K is the base of floating body
2	$\overline{KG} = \frac{(V_{bs}.e_{bs} + V_{walls}.e_{walls}) * \gamma_{conc}}{V_{total}.\gamma_{conc}} = 2.90\text{m}$	$\overline{KG} = \frac{(V_{fb}.e_{fb}.\gamma_{conc} + V_{evap}.e_{evap}.\gamma_{evap} + V_{st}.e_{st}.\gamma_{st} + V_t.e_t.\gamma_s)}{V_{fb}.\gamma_{conc} + V_{evap}.\gamma_{evap} + V_{st}.\gamma_{st} + V_t.\gamma_s} = 5.3m$
3	$\overline{KB} = \frac{Draught}{2} = \frac{4.77}{2} = 1.7\,\text{m}$	$\overline{KB} = \frac{Draught}{2} = \frac{5.64}{2} = 2.81\,\text{m}$
4	$\mathrm{I}_{\text{weaker}} = \frac{L * W^3}{12} = \frac{70.7 * 40.7^3}{12} = 397211 m^4$	$\mathrm{I}_{\text{weaker}} = \frac{L * W^3}{12} = 397211 m^4$
5	$V = L * B * d_1 = 70.7 * 40.7 * 4.77 = 10047 m^3$	$V = L * B * d_2 = 70.7 * 40.7 * 5.64 = 16222 m^3$
6	$\overline{BM} = \frac{I_{weak}}{V} = \frac{397211}{10047} = 39.5m$	$\overline{BM} = \frac{I_{weak}}{V} = \frac{397211}{16222} = 24.5m$
7	$h_m = \overline{KB} + \overline{BM} - \overline{KG} = 1.7 + 39.5 - 2.90 = 38.4m$	$h_m = \overline{KB} + \overline{BM} - \overline{KG} = 2.81 + 24.5 - 5.30 = 22.0m$

As seen from the table, in both the cases the h_m is greater than 0.5m, it is ensured that the body is stable in both the floating phases.

4.6 Sliding Check at the Final Location

As at the final location the floating body would have equal amount of water on all side, under normal conditions it can be assumed that the body would be stable against sliding but nonetheless we apply the check for the worst possible case that the wave from one side is at its trough and from the other at crust so we have a horizontal load equivalent to 3m of height. The check is applied as follows [7]

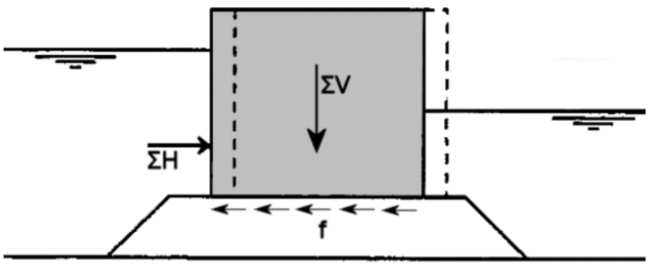

FIGURE 30SLIDING CHECK

$$f.\sum V = \sum H$$

$\sum V$ = Sum of all the vertical forces = $240892KN$

f = Frictional Coefficient between body and the soil=0.50

$\sum H$ = Sum of all the horizontal forces = $3*\gamma_w = 3(10.2) = 30.6KN$

As seen and expected, the caisson is safe against sliding.

4.7 Outer Wall

The outer walls of the floating body do not only have to act as a structural element but also as a separating membrane between fresh water and sea water, therefore the consideration of water tightness of the structure is of immense importance. Hence, when the floating body is ready, the water tightness check should be done in accordance with the provisions of ACI-350R [8].

The forces on the wall caused by the hydrostatic water pressure were computed using $\Upsilon_{seawater}$ which is 10.21 KN/m^3.

$$P = \frac{1}{2}\Upsilon h^2$$

To get more accurate results and include the two-way action of the outer walls, the structure was also modelled in SAP2000 as top and bottom slab. The resulting moments were used in the design procedure to calculate the total reinforcement required. A side check was also applied on the strength of outer walls for the time when the top slab in being casted on the structure and it was found to be within the limits. The results for main loading are shown below.

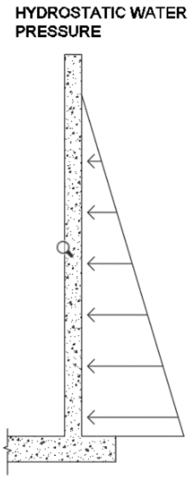

FIGURE 31 WATER PRESSURE ON WALLS

4.7.1 Moments

The moments obtained from SAP analysis and the required reinforcements to resist the calculated moments have been summarised below.

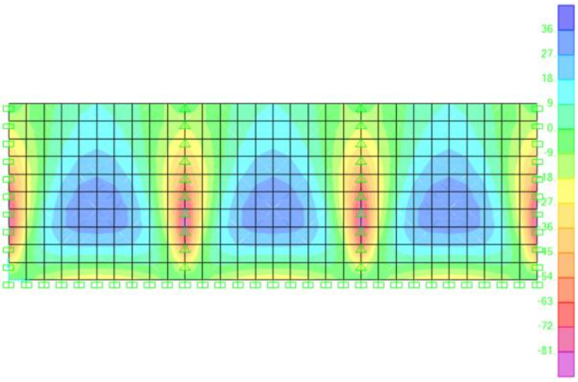

FIGURE 32 WALL MXX

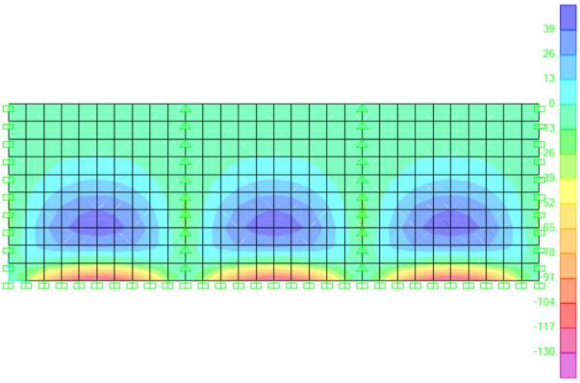

FIGURE 33 WALL MYY

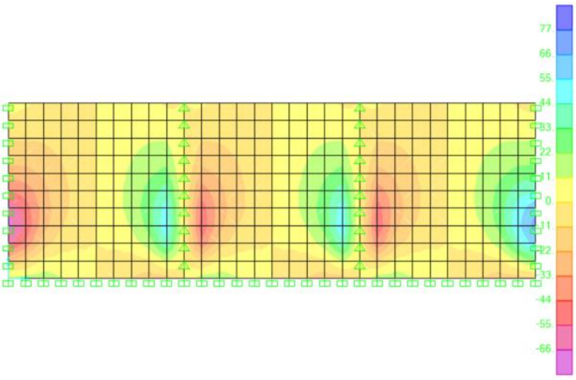

FIGURE 34 WALL VX

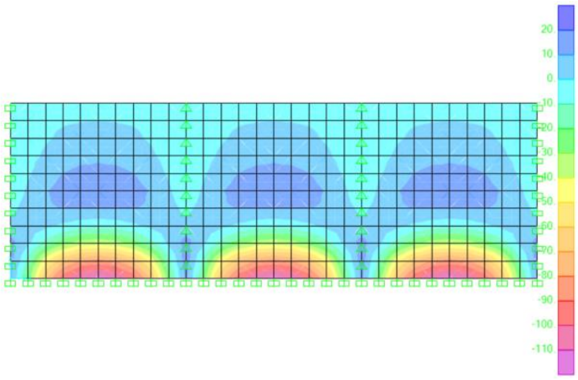

FIGURE 35 WALL VY

For the calculation of required reinforcements in the walls the same procedure was used as mentioned in top and bottom slab calculations. The results of the analysis are given below in parts for flexural and water tightness requirements.

TABLE 38 FLEXURAL STEEL CALCULATION OUTER WALL

	c (mm)	d (mm)	z (mm)	Moments (KNm)	Area (mm^2)
Mvx	50	440	385	130	1148
Msx	50	440	385	96	883
Mvy	50	440	385	130	1148
Msy	50	440	385	96	883

For walls a lower height of liquid was assumed for water tightness because at the final location our permanent height of liquid is 6 and we also have water pressure from the other side of the wall which actually help reduce the tensile forces on the other side, hence is beneficial in terms of crack width.

TABLE 39 WATER TIGHTNESS OF OUTER WALL

Height of Liquid	6	m
Slab Thickness	0.5	m
Hl/Thickness	12	
W critical	0.07	mm
W_{mo}	0.04	mm
N	180	KN
Sigma ct	0.36	KN/m2
Es/Ec	6.18	
fc_{cm}	43	Mpa
fc_{tm}	3.15	Mpa
Sigma cr	2.3625	Mpa
Sigma s at cr	108.974	MPa
As	1651.77	mm2/m

As the area required due to water tightness check is governing, this area was provided and is summarised in the table.

TABLE 40 STEEL PROVIDED OUTER WALL

Area (mm^2)	Roh (%)	Roh Min (%)	Roh Provided (%)	Bars
2486	0.38	0.21	0.38	ϕ 20 @ 175
2486	0.38	0.21	0.38	ϕ 20 @ 175
2486	0.38	0.21	0.38	ϕ 20 @ 175
2486	0.38	0.21	0.38	ϕ 20 @ 175

4.7.2 Shear Check

Following the same procedure and outer walls were also checked against shear.

$$v_{rd} = C_{rd} * k * (100 * \rho * fck)^{\frac{1}{3}}$$

$$v_{min} = 0.035 * k^{\frac{2}{3}} * \sqrt{fck}$$

$$v_{Ed} = \frac{Vx}{b * d}$$

TABLE 41SHEAR CHECK FOR OUTER WALL

	V (KN/m)	VEd(N/m/mm2)	VRd(N/m/mm2)	Vmin(N/m/mm2)	Status
Vx	72	0.162	0.476	0.292	OK
Vy	114	0.259	0.492	0.292	OK

4.7.3 Steel Quantities

The calculation for the amount of steel in the wall is mentioned in the following table.

TABLE 42REINFORCEMENT RATIO OF OUTER WALL IN LENGTH

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	20	20	20	20
Spacing	175	175	175	175
L1 (m)	40.7	40.7	8.9	8.9
L2 (m)	8.9	8.9	40.7	40.7
Number of Bars	51.0	51.0	233.0	233.0
Bar Length (m)	45.8	45.8	10.0	10.0
Mass (kg)	5905.6	5905.6	5899.9	5899.9
Total Steel (kg)	23611.0			
Ratio (kg/m3)	130.4			

TABLE 43REINFORCEMENT RATIO OF OUTER WALL IN WIDTH

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	20	20	20	20
Spacing	175	175	175	175
L1 (m)	70.7	70.7	8.9	8.9
L2 (m)	8.9	8.9	70.7	70.7
Number of Bars	51.0	51.0	404.0	404.0
Bar Length (m)	79.5	79.5	10.0	10.0
Mass (kg)	10258.6	10258.6	10229.9	10229.9
Total Steel (kg)	40976.9			
Ratio (kg/m3)	130.2			

4.8 Inner Walls

To allow for the lighter reinforcement in the inner walls, we have provided openings in all the chambers so the water level rises equally in all – without causing lateral forces, as the force from one side will be balanced with the forces from the other side. The only loading on the inner walls is the ballast loads, as the ballast is only in the edge compartment so the walls between the edge and inner compartment has to resist the lateral forces caused by ballast.

4.8.1 Moments

The mentioned forces were modelled in SAP2000 and the resulting moments were checked against the minimum reinforcement for flexural members i.e. 0.21%. As per the given design the moments caused by these lateral loads required reinforcement which was less than the minimum reinforcement requirement so the later could be provided in the inner walls. However, we were also required to check the one compartment failure of the floating body and ensure its safety in case of such an event, therefore it was assumed that if outer wall fails in a certain compartment, the water will flow in equal to the draft of the body. The load in such a scenario was modelled in SAP2000 and the results are tabulated and shown.

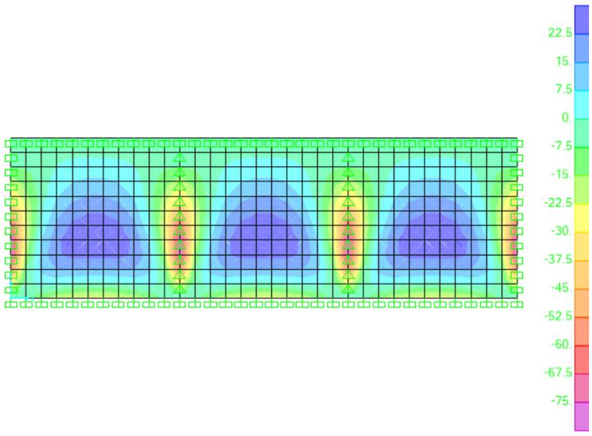

FIGURE 36 INNER WALL MXX

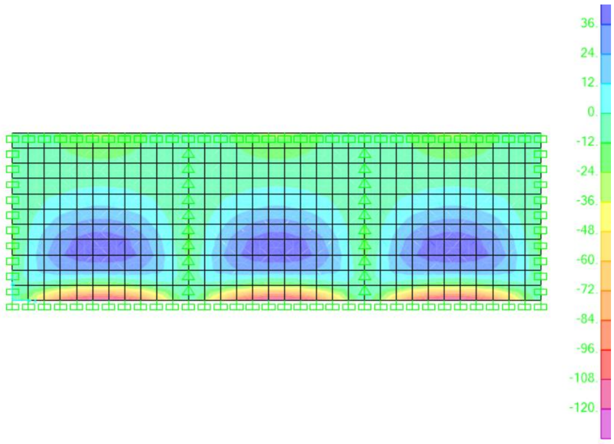

FIGURE 37 INNER WALL MYY

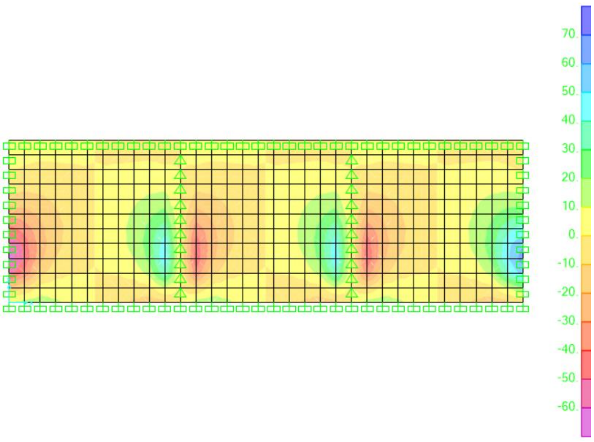

FIGURE 38INNER WALL VX

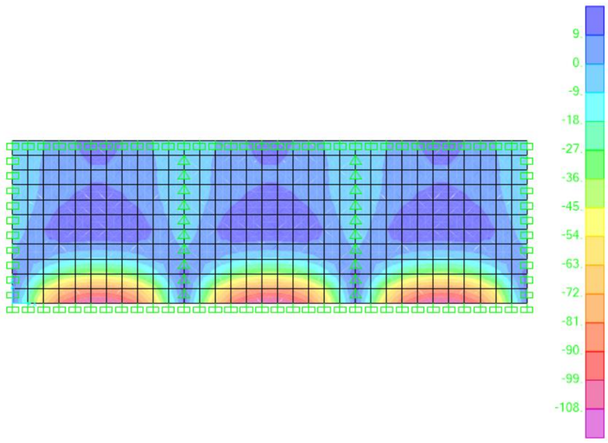

FIGURE 39INNER WALL VY

TABLE 44REINFORCEMENT RATIO OF INNER WALL IN LENGTH

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	16	16	16	16
Spacing	225	225	225	225
L1 (m)	70.7	70.7	8.9	8.9
L2 (m)	8.9	8.9	70.7	70.7
Number of Bars	40.0	40.0	314.0	314.0
Bar Length (m)	79.5	79.5	10.0	10.0
Mass (kg)	5149.4	5149.4	5088.6	5088.6
Total Steel (kg)	20476.0			
Ratio (kg/m3)	108.5			

TABLE 45 REINFORCEMENT RATIO OF INNER WALL IN WIDTH

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	16	16	16	16
Spacing	225	225	225	225
L1 (m)	40.7	40.7	8.9	8.9
L2 (m)	8.9	8.9	40.7	40.7
Number of Bars	40.0	40.0	181.0	181.0
Bar Length (m)	45.8	45.8	10.0	10.0
Mass (kg)	2964.4	2964.4	2933.2	2933.2
Total Steel (kg)	11795.2			
Ratio (kg/m3)	108.5			

4.8.2 Shear

TABLE 46 SHEAR CHECK FOR INNER WALLS

	V (KN/m)	VEd(N/m/mm2)	VRd(N/m/mm2)	Vmin(N/m/mm2)	Status
Vx	60	0.247	0.499	0.319	OK
Vy	96	0.397	0.499	0.319	OK

4.9 One Compartment Failure

As mentioned in the technical specifications, the floating body also need to be checked in terms of hydraulic stability for one compartment failure so, in order to do that the following procedure was used.

- Note the draught of the alternative during towing
- Assume the failure of the outer wall which would result in the water level inside the compartment equal to the water level outside which is equal to draught
- Calculate the extra weight on the structure due to this damage
- Calculate the net moment caused by this loads
- Compare this with the resisting uplift moment
- If uplift is greater than the load, than the structure is safe.

TABLE 47 ONE COMPARTMENT FAILURE CHECK

Draught	5.64m
Extra Weight Moment (L)	$A_c.d.\gamma_w.r = 9.7*9.7*5.64*10*29.5 = 156546 KNm$
Resisting Moment (L)	Uplift.r = $89339*35.4 = 3162600 KNm$
Extra Weight Moment (B)	$A_c.d.\gamma_w.r = 9.7*9.7*5.64*10*14.5 = 76912 KNm$
Resisting Moment (B)	Uplift.r = $89339*20.35 = 1818062 KNm$

4.10 Summary

In the preliminary cost analysis, we ignored the actual effect on the steel ratios to compare the costs, we do this here. For better understanding, the results of calculated steel ratios are summarized in this section. These values are used in the final cost analysis of the two selected variants.

TABLE 48 SUMMARY OF STEEL RATIOS - ALTERNATIVE A

	Steel (kg)	Volume (m3)	Ratio (kg/m3)
Top Slab	141077	1151	123
Outer Walls - L	81954	629	130
Outer Walls - B	47222	362	130
Inner Walls - L	61428	566	108
Inner Walls - B	70771	652	109
Bottom Slab	368343	2173	170
Total	770795	5534	139

5 Structural Design – Alternative F

Alternative F is designed in the same way as Alternative A – the order of chapters and figures is kept same for the ease of understanding of the reader.

5.1 Top Slab

The loadings on the top slab are same as mentioned in the previous chapter because the super structures are the same. The moments however, would be higher because of higher spans. The results are shown in the sections of the report to come.

5.1.1 Moments

The analysis results of SAP 2000 are attached in the figures 41-44.

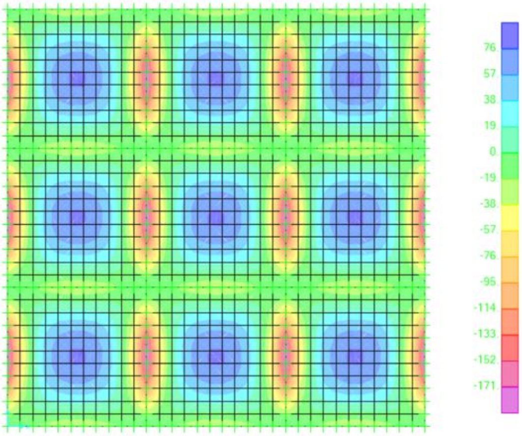

FIGURE 40 TOP SLAB MXX

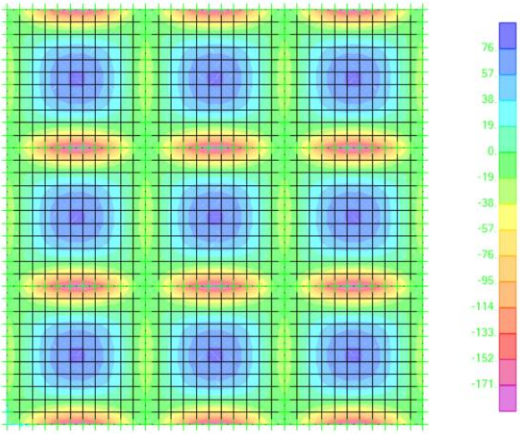

FIGURE 41 TOP SLAB MYY

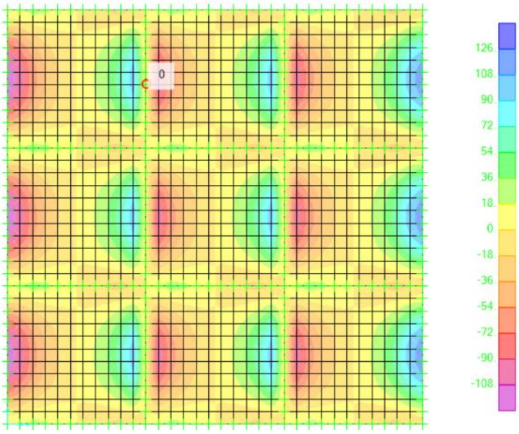

FIGURE 42 TOP SLAB VX

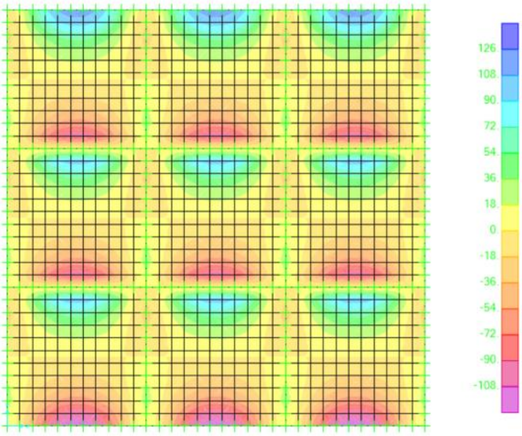

FIGURE 43 TOP SLAB VY

TABLE 49 MOMENTS AND STEEL AREA FOR TOP SLAB

	c (mm)	d (mm)	z (mm)	Moments (KNm)	Area (mm^2)
Mvx	50	352	306	171	1836
Msx	50	352	306	76	816
Mvy	50	352	306	171	1836
Msy	50	352	306	76	816

The spacing and bar provided are calculated as mentioned in the table below.

TABLE 50 REINFORCING BARS AND SPACING TOP SLAB

Area (mm^2)	Roh (%)	Roh Min (%)	Roh Provided (%)	Bars
1836	0.50	0.21	0.50	ϕ 16 @ 110
816	0.225	0.21	0.225	ϕ 16 @ 200
1836	0.50	0.21	0.50	ϕ 16 @ 110
816	0.225	0.21	0.225	ϕ 16 @ 200

The steel ratio is calculated below

TABLE 51REINFORCEMENT RATIO OF TOP SLAB

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	16	16	16	16
Spacing	110	200	110	200
L1 (m)	33.7	33.7	77.7	77.7
L2 (m)	77.7	77.7	33.7	33.7
Number of Bars	707	390	307	170
Bar Length (m)	37.9	37.9	87.4	87.4
Mass (kg)	43383.8	23931.7	43434.8	24051.8
Total Steel (kg)	134802.1			
Ratio (kg/m3)	128.7			

5.1.2 Shear Check

The shear force obtained from SAP2000 was checked as mentioned in the table 52.

TABLE 52 SHEAR CHECK FOR TOP SLAB

	V (KN/m)	VEd(N/m/mm2)	VRd(N/m/mm2)	Vmin(N/m/mm2)	Status
Vx	126	0.368	0.55	0.30	OK
Vy	126	0.368	0.421	0.30	OK

5.1.3 Deflection

- Case 1: Fresh concrete on 200mm thick lattice girder

$$w = \frac{5.q.l^4}{384.EI} = \frac{5*5*10700^4}{384*(34000*\frac{1}{12}*1000*200^3)} = 37.64mm$$

So, propping is required while pouring the top slab, as expected because of larger spans.

- Case 2: Final deflection when loaded with super structure

$$w = \frac{5.q.l^4}{384.EI} = \frac{5*25.8*10700^4}{384*(34000*\frac{1}{12}*1000*400^3)} = 24.28mm$$

The calculated value was checked against 42.8mm (l/250). Again this is a very conservative value, this will not be the actual deflection at the final stage.

5.2 Bottom Slab

5.2.1 Moments

- Loading Case 1: Floating

$$UDL = -(Draught + 1.5)\gamma_w + Thichness * \gamma_c = -(5.77 + 1.5) * 10.2 + 0.70 * 25 = -56.65KN / m^2$$

- Loading Case 2: Ballasted but Empty

$$UDL = -BP + Thichness * \gamma_c + \frac{W_{Ballast}}{Area_{Ballast}} = -13.25 + 0.7 * 25 + \frac{36215}{1832} = 24.02KN / m^2$$

- Loading Case 3: Operational (Water + Ballast)

$$UDL = -BP + Thickness * \gamma_c + \frac{W_{ballast}}{Area_{ballast}} + \frac{W_{water}}{Area_{water}} = -77.60 + 0.7 * 25 + \frac{36215}{1832} + \frac{200000}{2404.2} = 42.86KN / m^2$$

The analysis results of SAP2000 are shown in the figures 45-48 below.

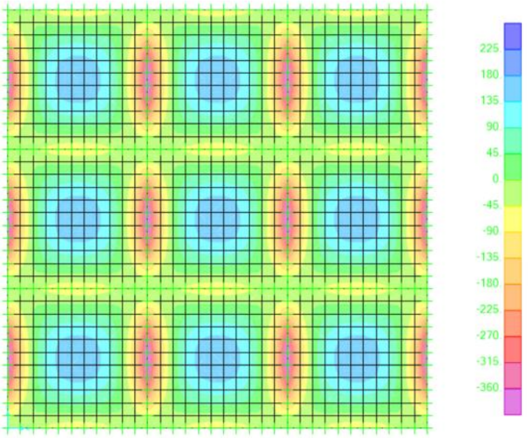

FIGURE 44 BOTTOM SLAB MXX

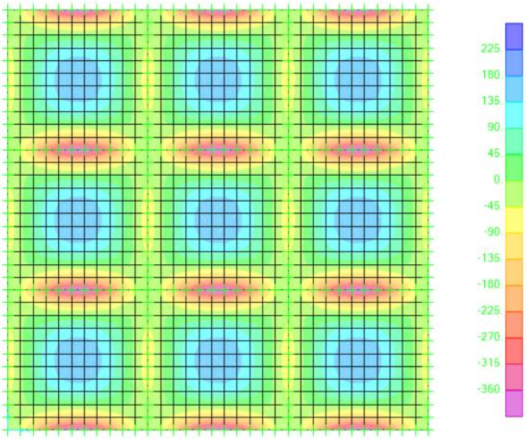

FIGURE 45BOTTOM SLAB MYY

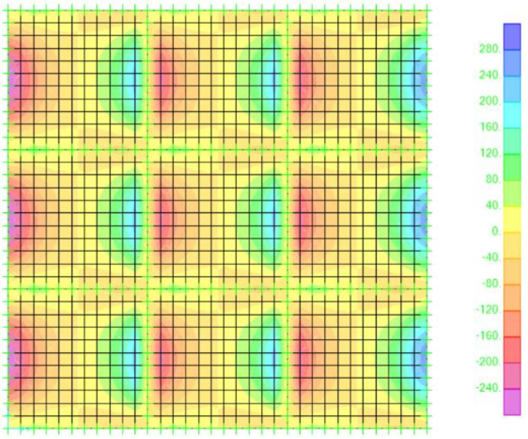

FIGURE 46 BOTTOM SLAB VX

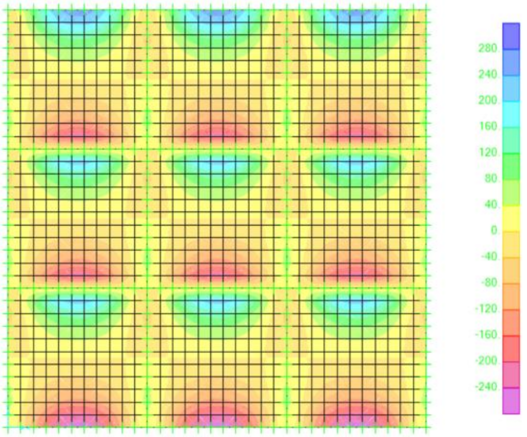

FIGURE 47 BOTTOM SLAB VY

TABLE 53 MOMENTS AND STEEL AREA FOR TOP SLAB

	c (mm)	d (mm)	z (mm)	Moments (KNm)	Area (mm^2)
Mvx	50	637.5	558	360	2194
Msx	50	637.5	558	225	1371
Mvy	50	637.5	558	360	2194
Msy	50	637.5	558	225	1371

The bottom slab also needs to be checked for water tightness and for that the Lohmeijer [5] crack width criterion was used as mentioned in the section 4.2 of the report.

TABLE 54 WATER TIGHTNESS FOR BOTTOM SLAB

Height of Liquid	9	m
Slab Thickness	0.7	m
Hl/Thickness	12.8571	
W critical	0.09	mm
Wmo	0.05	mm
N	405	KN
Sigma ct	0.57857	KN/m2
Es/Ec	6.18	
fccm	43	Mpa
fctm	3.15	Mpa
Sigma cr	2.3625	Mpa
Sigma s at cr	126.895	MPa
As	3191.62	mm2/m

TABLE 55 REINFORCING BARS AND SPACING BOTTOM SLAB

Area (mm^2)	Roh (%)	Roh Min (%)	Roh Provided (%)	Bars
3191	0.50	0.21	0.50	ϕ 20 @ 100
3191	0.50	0.21	0.50	ϕ 20 @ 100
3191	0.50	0.21	0.50	ϕ 20 @ 100
3191	0.50	0.21	0.50	ϕ 20 @ 100

TABLE 56REINFORCEMENT RATIO OF BOTTOM SLAB

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	20	20	20	20
Spacing	100	100	100	100
L1 (m)	35.7	35.7	79.7	79.7
L2 (m)	79.7	79.7	35.7	35.7
Number of Bars	797	797	357	357
Bar Length (m)	40.2	40.2	89.7	89.7
Mass (kg)	80951.5	80951.5	80951.5	80951.5
Total Steel (kg)	323805.9			
Ratio (kg/m3)	162.6			

5.2.3 Shear Check

TABLE 57SHEAR CHECK FOR BOTTOM SLAB

	V (KN/m)	VEd(N/m/mm2)	VRd(N/m/mm2)	Vmin(N/m/mm2)	Status
Vx	280	0.44	0.48	0.27	OK
Vy	280	0.44	0.48	0.27	OK

5.3 Static Stability – Towing Journey

As done for alternative A, the metacentric height was checked for Alternative F.

TABLE 58 HYDRAULIC STABILITY CHECK

	Towing from the dock to quay side	Towing from quay side to the final location
1	K is the base of floating body	K is the base of floating body
2	$\overline{KG} = \frac{(V_{bs}.e_{bs} + V_{walls}.e_{walls})*\gamma_{conc}}{V_{total}.\gamma_{conc}} = 4.60\text{m}$	$\overline{KG} = \frac{(V_{fb}.e_{fb}.\gamma_{conc} + V_{evap}.e_{evap}.\gamma_{evap} + V_{st}.e_{st}.\gamma_{st} + V_t.e_t.\gamma_s)}{V_{fb}.\gamma_{conc} + V_{evap}.\gamma_{evap} + V_{st}.\gamma_{st} + V_t.\gamma_s} = 6.4m$
3	$\overline{KB} = \frac{Draught}{2} = \frac{3.82}{2} = 1.9\text{ m}$	$\overline{KB} = \frac{Draught}{2} = \frac{5.64}{2} = 2.90\text{ m}$
4	$\text{I}_{\text{weaker}} = \frac{L*W^3}{12} = \frac{70.7*40.7^3}{12} = 247816m^4$	$\text{I}_{\text{weaker}} = \frac{L*W^3}{12} = 247816m^4$
5	$V = L*B*d_1 = 70.7*40.7*3.82 = 10013m^3$	$V = L*B*d_2 = 70.7*40.7*5.64 = 15107m^3$
6	$\overline{BM} = \frac{I_{weak}}{V} = \frac{397211}{10013} = 24.7m$	$\overline{BM} = \frac{I_{weak}}{V} = \frac{397211}{16222} = 16.4m$
7	$h_m = \overline{KB} + \overline{BM} - \overline{KG} = 1.9 + 24.7 - 46 = 22.1m$	$h_m = \overline{KB} + \overline{BM} - \overline{KG} = 2.81 + 24.5 - 5.30 = 12.9m$

As seen from the table, in both the cases the h_m is greater than 0.5m, it is ensured that the body is stable in both the floating phases.

5.4 Sliding Check at the Final Location

$$f.\sum V = \sum H$$

$$\sum V = \text{Sum of all the vertical forces} = 237718KN$$

$$f = \text{Frictional Coefficient between body and the soil=0.50}$$

$$\sum H = \text{Sum of all the horizontal forces} = 3*\gamma_w = 3(10.2) = 30.6KN$$

As seen and expected, the caisson is safe against sliding.

5.5 Outer Wall

5.5.1 Moments

The moments obtained from SAP analysis and the required reinforcements to resist the calculated moments have been summarised below.

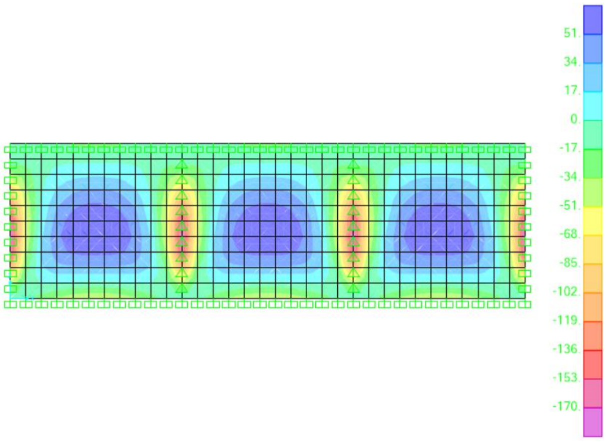

FIGURE 48 OUTER WALL MXX

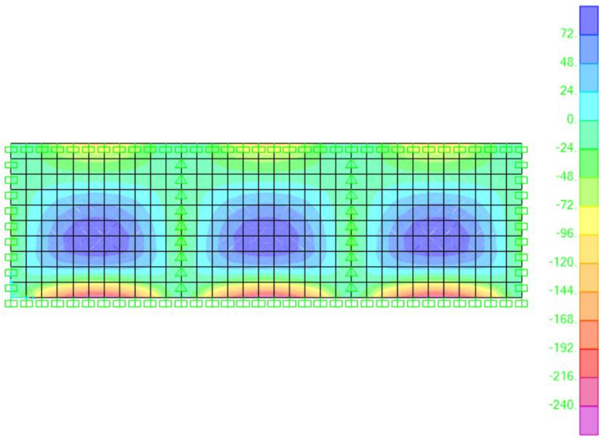

FIGURE 49OUTER WALL MYY

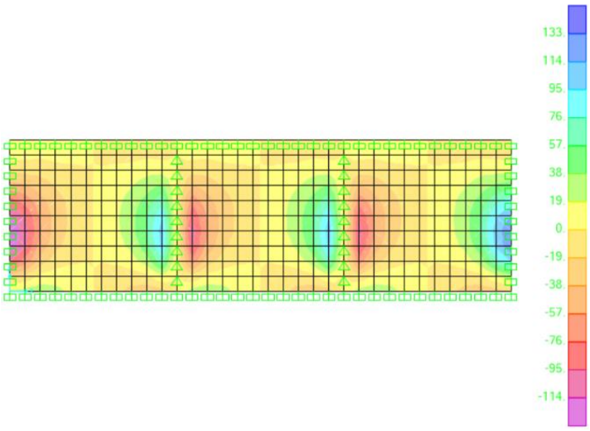

FIGURE 50OUTER WALL VX

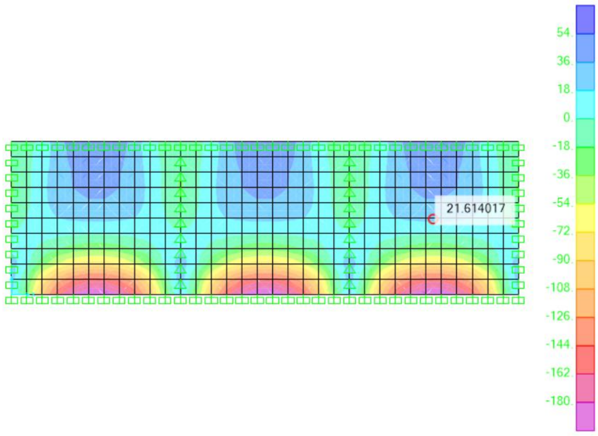

FIGURE 51OUTER WALL VY

The area of steel required is as mentioned in table.

TABLE 59FLEXURAL STEEL CALCULATION OUTER WALL

	c (mm)	d (mm)	z (mm)	Moments (KNm)	Area (mm^2)
Mvx	50	440	385	170	1501
Msx	50	440	385	52	459
Mvy	50	440	385	240	2119
Msy	50	440	385	72	635

5.5.2 Water Tightness

TABLE 60 WATER TIGHTNESS OUTER WALL

Height of Liquid	6	m
Slab Thickness	0.5	m
Hl/Thickness	12	
W critical	0.07	mm
Wmo	0.04	mm
N	180	KN
Sigma ct	0.36	KN/m2
Es/Ec	6.18	
fccm	43	Mpa
fctm	3.15	Mpa
Sigma cr	2.3625	Mpa
Sigma s at cr	108.974	MPa
As	1651.77	mm2/m

As the area required due to water tightness check is governing, this area was provided and is summarised in the table.

TABLE 61 STEEL ARES PROVIDED OUTER WALL

Area (mm^2)	Roh (%)	Roh Min (%)	Roh Provided (%)	Bars
1652	0.38	0.21	0.38	ϕ 20 @ 175
1652	0.38	0.21	0.38	ϕ 20 @ 175
2486	0.48	0.21	0.48	ϕ 20 @ 150
1652	0.38	0.21	0.38	ϕ 20 @ 175

TABLE 62REINFORCEMENT RATIO OF OUTER WALL IN LENGTH

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	20	20	20	20
Spacing	175	175	150	175
L1 (m)	33.7	33.7	9.4	9.4
L2 (m)	9.4	9.4	33.7	33.7
Number of Bars	54	54	225	193
Bar Length (m)	37.9	37.9	10.6	10.6
Mass (kg)	5177.5	5177.5	6017.4	5161.6
Total Steel (kg)	21534.0			
Ratio (kg/m3)	136.0			

TABLE 63REINFORCEMENT RATIO OF OUTER WALL IN WIDTH

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	20	20	20	20
Spacing	175	175	150	175
L1 (m)	77.7	77.7	9.4	9.4
L2 (m)	9.4	9.4	77.7	77.7
Number of Bars	54	54	518	444
Bar Length (m)	87.4	87.4	10.6	10.6
Mass (kg)	11937.5	11937.5	13853.4	11874.3
Total Steel (kg)	49602.7			
Ratio (kg/m3)	135.8			

5.2.3 Shear Check

TABLE 64SHEAR CHECK FOR OUTER WALL

	V (KN/m)	VEd(N/m/mm2)	VRd(N/m/mm2)	Vmin(N/m/mm2)	Status
Vx	133	0.303	0.476	0.291	OK
Vy	180	0.410	0.476	0.291	OK

5.3 Inner Walls

5.3.1 Moments

The moments in case of one compartment failure were used to calculate the steel reinforcement required in the inner walls.

TABLE 65REINFORCEMENT RATIO OF INNER WALL IN LENGTH

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	16	16	16	16
Spacing	200	200	200	200
L1 (m)	77.7	77.7	9.4	9.4
L2 (m)	9.4	9.4	77.7	77.7
Number of Bars	47.0	47.0	389.0	389.0
Bar Length (m)	87.4	87.4	10.6	10.6
Mass (kg)	6649.6	6649.6	6658.2	6658.2
Total Steel (kg)	26615.6			
Ratio (kg/m3)	121.5			

TABLE 66REINFORCEMENT RATIO OF INNER WALL IN WIDTH

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	16	16	16	16
Spacing	200	200	200	200
L1 (m)	33.7	33.7	9.4	9.4
L2 (m)	9.4	9.4	33.7	33.7
Number of Bars	47.0	47.0	169.0	169.0
Bar Length (m)	37.9	37.9	10.6	10.6
Mass (kg)	2884.1	2884.1	2892.6	2892.6
Total Steel (kg)	11553.4			
Ratio (kg/m3)	121.6			

5.3.2 Shear Check

TABLE 67 SHEAR CHECK FOR INNER WALLS

	V (KN/m)	VEd(N/m/mm2)	VRd(N/m/mm2)	Vmin(N/m/mm2)	Status
Vx	66	0.272	0.520	0.319	OK
Vy	102	0.421	0.520	0.319	OK

5.4 One Compartment Failure Stability

TABLE 68 ONE COMPARTMENT FAILURE CHECK

Draught	5.77m
Extra Weight Moment (L)	$A_c.d.\gamma_w.r = 10.7*10.7*5.77*10*32.5 = 214677 KNm$
Resisting Moment (L)	Uplift.r = $83808*38.9 = 3255731 KNm$
Extra Weight Moment (B)	$A_c.d.\gamma_w.r = 10.7*10.7*5.77*10*10.5 = 69357 KNm$
Resisting Moment (B)	Uplift.r = $83802*16.85 = 1412074 KNm$

5.5 Summary

The total steel and concrete calculation for Alternative F is tabulated below.

TABLE 69 SUMMARY OF STEEL RATION OF ALTERNATIVE F

	Steel (kg)	Volume (m3)	Ratio (kg/m3)
Top Slab	134802	1047	129
Outer Walls - L	99205	730	136
Outer Walls - B	43068	317	136
Inner Walls - L	53231	438	121
Inner Walls - B	69320	570	122
Bottom Slab	323806	1951	166
Total	723433	5054	143

Chapter 6 Selection and Optimization

6.1 Selection of the most economically attractive alternative

As expected, Alternative F has a higher steel to concrete ratio of 143 kg/m^3 compared to the steel-concrete ratio of Alternative A i.e. 139 kg/m^3. Nevertheless, the ratio of Alternative F is well within the acceptable limits and keeping in mind the difference in cost, risk and the execution time alternative F seems to be the most attractive but as seen from the plan of alternative F, the working space on the sides of the evaporators is too low – only 1.85m. The detailed cost analysis of Alternative F is attached in the annex.

At this level the comparison is made with more details with respect to steel and in that we use the actual obtained ratios of steel reinforcement.

$$Cost = (\text{Cost}_i - \text{Steel Cost}_i) + \text{Actual Steel Cost} + \text{Risk}$$

$$\text{Cost}_i = \text{The cost calculated for initial comparison}$$

$$\text{Steel Cost}_i = \text{The steel cost in initial comparison}$$

Actual Steel Cost = The steel cost after detailed design

TABLE 70 COST WITH ACTUAL STEEL

Alternative	Ci	Steel Cost$_i$	Actual Steel Cost	Cost	Time	Relative Cost
A	6475777	584735	580558	6471600	15.7	6471600
F	6471165	615428	550038	6405775	15.2	6280775

As seen from the table, the comparative cost of Alternative F is the least so it is most attractive economically. In the following chapter, the details for the execution of Alternative F are provided.

6.2 Optimization

The structural optimization was done for all the alternatives in the initial phases, we started the design of body assuming 500mm thick top slab and 750mm thick bottom slab, which were reduced to 400mm and 700mm respectively, as per the loading and applied moments. The dimensions of outer and inner walls were however not changed from the initial starting value because they seemed to fulfil all the requirements in a close to optimal way.

For the final alternative chosen, we tried to do detailing for the inner compartment inner walls separately as they need not be strong in flexure. In case of one compartment failure only the walls of outer compartment can be exposed to the lateral load of water but not the inner compartment walls, so we provide minimum reinforcement there. The highlighted walls in the structure can be provided with minimum reinforcement. The weights of providing minimum reinforcement is listed below.

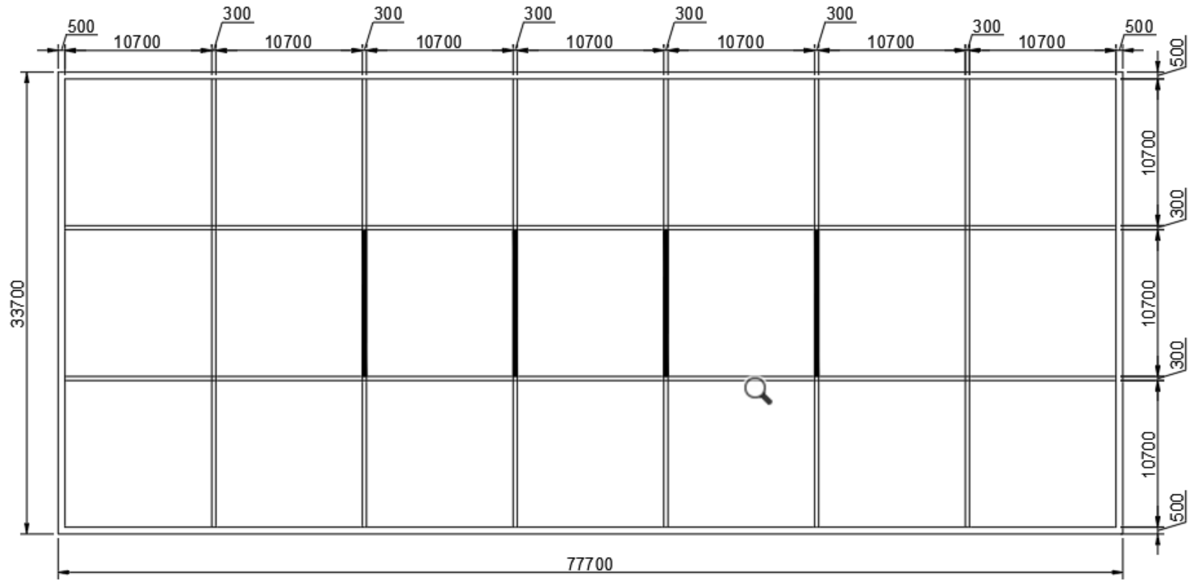

FIGURE 52 WALLS WITH MINIMUM REINFORCECMENT

TABLE 71 STEEL RAITO OF INNER WALLS

	Mxx Top	Mxx Bottom	Myy Top	Myy Bottom
Bar Diameter (mm)	12	12	12	12
Spacing	175	175	175	175
L1 (m)	10.7	10.7	9.4	9.4
L2 (m)	9.4	9.4	10.7	10.7
Number of Bars	54.0	54.0	61.0	61.0
Bar Length (m)	12.0	12.0	10.6	10.6
Mass (kg)	591.8	591.8	587.3	587.3
Total Steel (kg)	2358.2			
Ratio (kg/m3)	78.2			

The minimum reinforcement can be provided in three walls and the resulting total steel weights and ratios are listed below.

TABLE 72 FINAL STEEL RATIOS OF ELEMENTS

	Steel (kg)	Volume (m3)	Ratio (kg/m3)
Top Slab	134802	1047	129
Outer Walls - L	99205	730	136
Outer Walls - B	43068	317	136
Inner Walls - L	53231	438	121
Inner Walls - B	52618	433	122
Inner Wall - Min	9433	137	69
Bottom Slab	323806	1951	166
Total	716163	5054	142

This results in further decreasing the mass of steel required for the project hence optimises the cost.

7 Design and Detailing of the structure to sustain the mooring force

The design of the structure with detail to transfer the mooring force is described in this section. The provision for this structure is given in the bottom slab and to ensure that the structure has minimum shear forces a buoy is used, as mentioned in the towing journey section.

7.1 Design of the structure

7.1.1 Structure for attaching ropes

To design the structure to which the hooks can be attached for the transfer of mooring forces an FEM software – Diana – was used and the obtained results are displayed in the following figures.

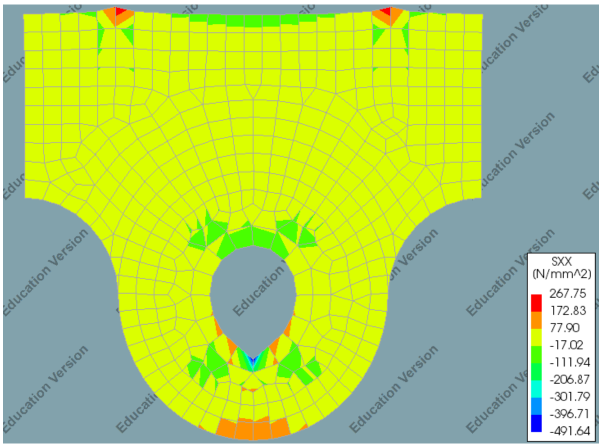

FIGURE 53 SXX OF MOORING ELEMENT

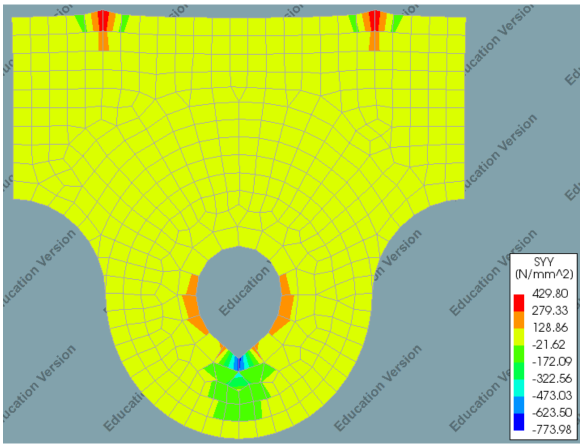

FIGURE 54 SYY OF MOORING ELEMENT

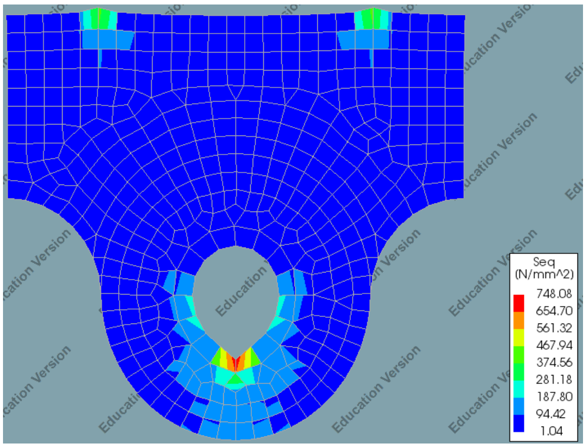

FIGURE 55 VON MISES CHECK FOR MOORING ELEMENT

As seen from the figures, the value obtained is greater than 500 MPa, which is the yield stress for the attaching structure, but this high value is only because of the limitations of Finite Element Method. To get a feeling of the actual stresses, the stress at the closest nodes to the singularity was read and found to be 210 MPa so it was considered safe to assume that the actual stresses are below the yield stress value. To compare the stresses with the yield value we can choose various checks from structural mechanics but for this particular case we use the Von Mises Criterion to check the stresses.

7.1.2 Anchors to transfer Loads to concrete

Hexagonal headed bolt of grade 6.8 are used as anchors to transfer the forces from the steel to concrete. To calculated the required length of the anchors the provisions of ACI-318 [9] appendix D are used [10]. The equations for the calculation of anchor bolt length and required reinforcement are mentioned below. At the end of the chapter the calculation table is also attached.

Steel Strength of anchor in tension

$$A_{se} = \frac{\pi}{4} * (d_0 - \frac{0.9743}{n_t})^2$$

where n_t is the number of threads per mm

A_{se} is net tensile area

$$N_{sa} = n.A_{se}.f_{ut}$$

f_{ut} is the ultimate tensile strength of bolt

N_{sa} is the tensile capacity of bolt

Steel Strength of anchor in shear

$$V_{sa} = n * 0.6 * A_{se} * f_{ut}$$

V_{sa} is the shear capacity of tha anchor bolt

n is the number of anchors

Concrete breakout strength in tension

$$\psi_n = 1.25$$

$$A_{nc} / A_{nc0} = 2$$

$$Nb = 10 * \sqrt{f_{ck}} * h_{ef}^{1.5}$$

$$N_{cbg} = (A_{nc} / A_{nc0}).\psi_n.N_b$$

ψ_n is a factor for headed anchor bolts

N_{cbg} is concrete breakout strength in tension

h_{ef} is the effective length of the anchor bolt

Concrete side face blowout strength for bolts in tension

$$C_{a1} > 0.4 * h_{ef}$$

C_{a1} is the edge distance

Concrete Pull-out Strength

$\psi_{cp} = 1.4$ for anchor bolts in region where there is no cracking at service load levels

$$N_{pn} = \psi_p * 8 * A_{brg} * f_{ck}$$

A_{brg} = Area of the bolt headed anchor

N_{pn} is the concrete pull-out strength

Number of Reinforcing Bars to transfer load to concrete

$$Number = \frac{N_{ua}}{f_{y,rebar} * A_{sb}}$$

A_{sb} in the are of rebar

Development Length of Rebar before failure surface

$$l_d = \frac{f_{y,rebar} * (\psi_t . \psi_e . \lambda)}{25 * \sqrt{f_{ck}}} * d_b \text{ or 300mm}$$

ψ_e, ψ_t and λ is 1.0

d_b is the diameter of bar

The input values are mentioned in the following table.

TABLE 73 INPUT TABLE FOR ANCHOR DESIGN

f_y	480	Mpa
f_u	600	MPa
n_t	1.5	
d_o	25	mm
n_b	4	
h_{ef}	400	mm
f_{ck}	35	MPa
C1	235	mm
N_u	250	KN
A_{bar}	314	mm2
D_{bar}	20	mm
$Fy_{,rebar}$	500	MPa

The results of the calculations as per the equations mentioned above are presented below.

TABLE 74 OUTPUT OF ANCHOR DESIGN

Head Dia	32.5	mm	
Ab	465.7	mm2	
Nt	1117.7	KN	OK
Nv	558.8	KN	
Anc	25920000	mm2	
Anco	12960000	mm2	
Anc/Anco	2		
Nb	473.3	KN	
Ncbg	1183	KN	OK
Abrg	830	mm2	
Np	232.3	KN	
Npn	325.2	KN	OK
0.4hef	160		OK
Length	800	mm	
Minimum Edge Distance	150		OK
Number of bars	1.6		
Ld	68	mm	
Ld provided	300	mm	

7.2 Detailing of the structure

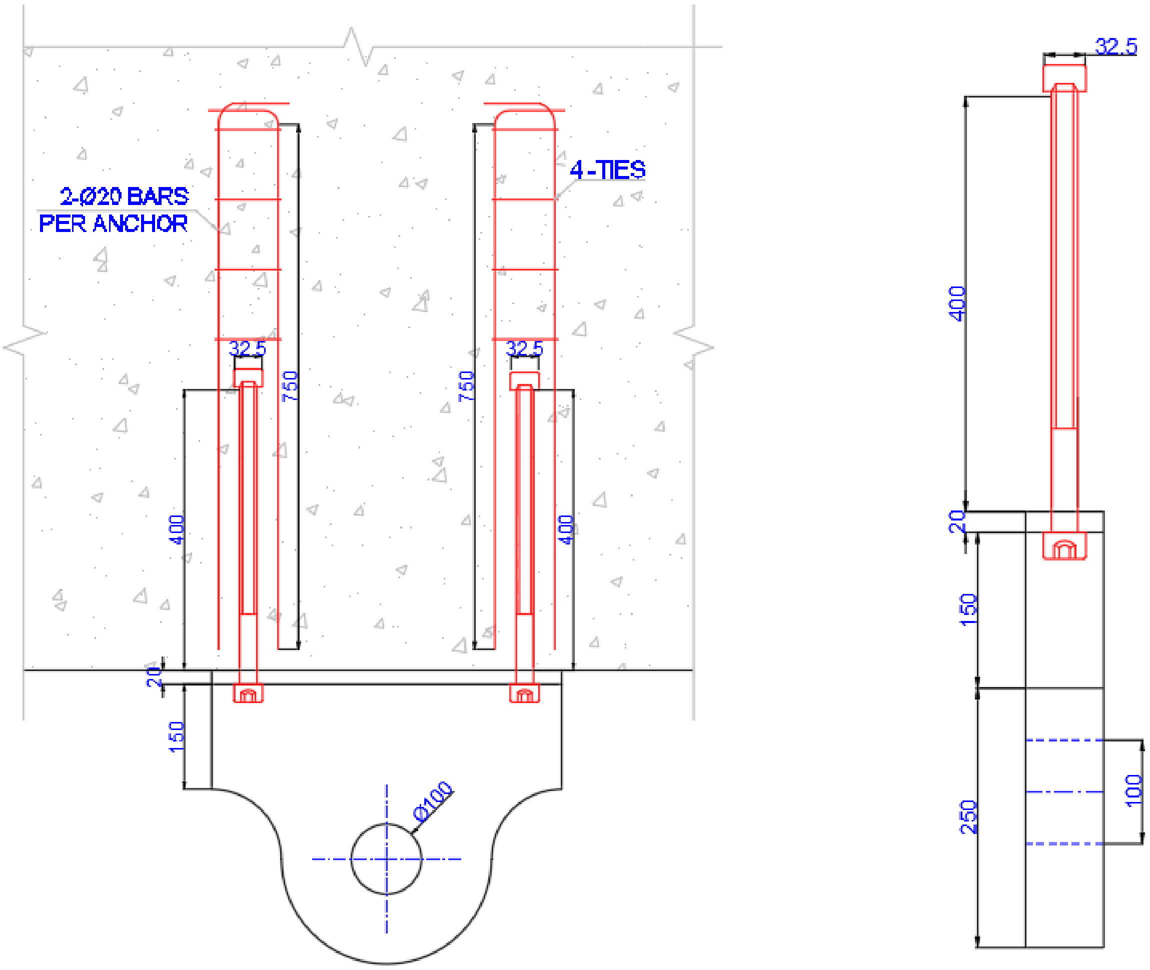

FIGURE 56 PLAN AND SECTION OF MOORING STRUCTURE

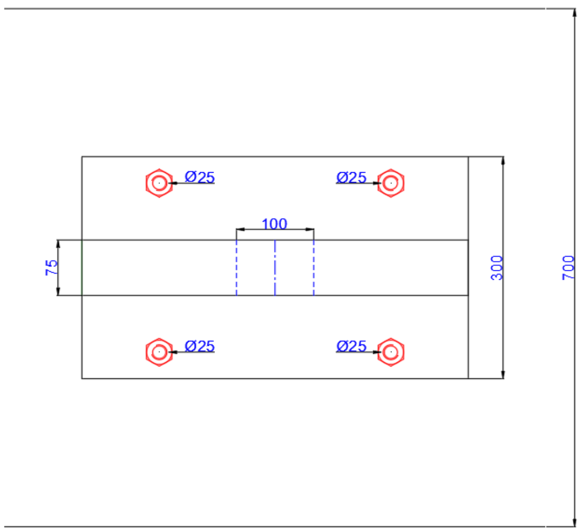

FIGURE 57 ELEVATION OF MOORING STRUCTURE

The provisions for such structures are provided two in the bottom slab and two in the corner walls on either side of the floating body.

Chapter 8 Execution/Method Statement

8.1 Graving Dock

To select between the choice of building the floating body in the graving dock or in the floating dry dock, we need to first build a graving dock and look into the time and cost that it requires. The total cost of building the graving dock and the structure will be compared with the rent of the dry dock and structure cost with associated risks. To make the decision between the bentonite-slurry cut off wall and sheet piles, the cost analysis was done for both the alternatives for an area of 4000 m^2 as shown in table 75.

TABLE 75 COST ANALYSIS FOR BENTONITE WALL AND SHEET PILES

	Area – m^2	Volume – m^3	Weight - ton	Cost - €
Sheet Pile	4000	40	322	563500
Cut-Off Wall	4000	2000	-	630000

In accordance with the cost analysis for Bentonite-Cement cut-off wall and sheet piles, sheet piles were used to allow for the dewatering of the building pit for construction usage. The important parameters for graving dock cost are the length, width and height of the excavated soil and the method of isolating the building pit from the surrounding ground water. As we just need this pit to construct the floating body, it is designed in such a way that it will be destroyed once the structure is complete – this helps avoid extra cost of constructing a permanent graving dock.

The building of graving dock is divided in to following steps:

- Excavation of the land – the length of the excavated land is kept 20m more than the length of the floating body to allow for adequate work space for formwork and other construction equipment including the crane and concrete pump, the width however is only 10m more than the width of the floating body for formwork and working space.
- Building the dikes on three side – The earth excavated will be used to build the dikes on three sides of the graving dock. The forth side will be used to allow for the access of construction machinery at site.
- Installing the bentonite clay layer – To allow for removal of water from the building pit, a bentonite clay layer was required up to the depth of water tight layer at -12.00m. This was done on all four sides of the graving dock. On the dike free side this will be done up to the ground level initially and the slurry layer will be continued once the forth dike is being constructed.
- Dewatering the building pit – After installation of the water tight layers on the side of the graving dock, the water inside will be pumped out of the pit in to the river. The ground water will be lowered an extra 0.5m from the final excavated level of soil in the pit to allow for adequate utilization of the building pit.
- Laying the gravel bed – A gravel bed of 500mm depth will be installed where the bottom slab of the floating body is to be casted. This gravel bed will allow for the development of water pressure necessary for lifting up the floating body, once the graving dock is flooded.

- Closing the forth dike – After the construction of the floating body, the forth dike will be built once the construction machinery has left the graving dock.

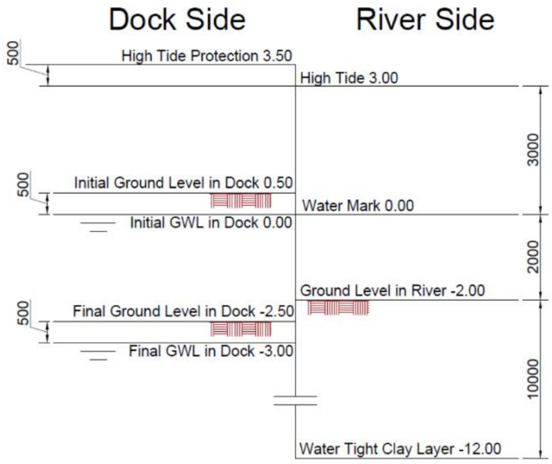

FIGURE 58 GROUND AND WATER LEVEL DATA FOR GRAVING DOCK

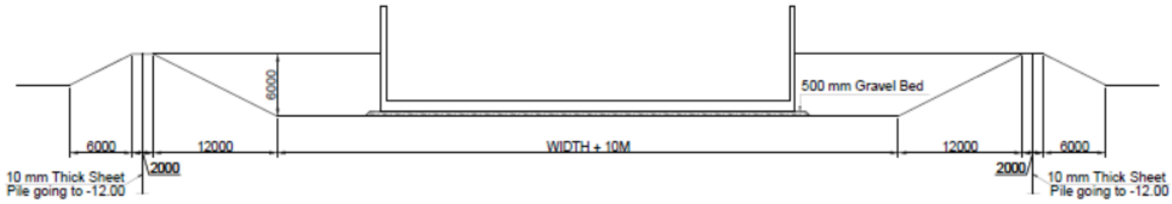

FIGURE 59 CROSS SECTION OF GRAVING DOCK

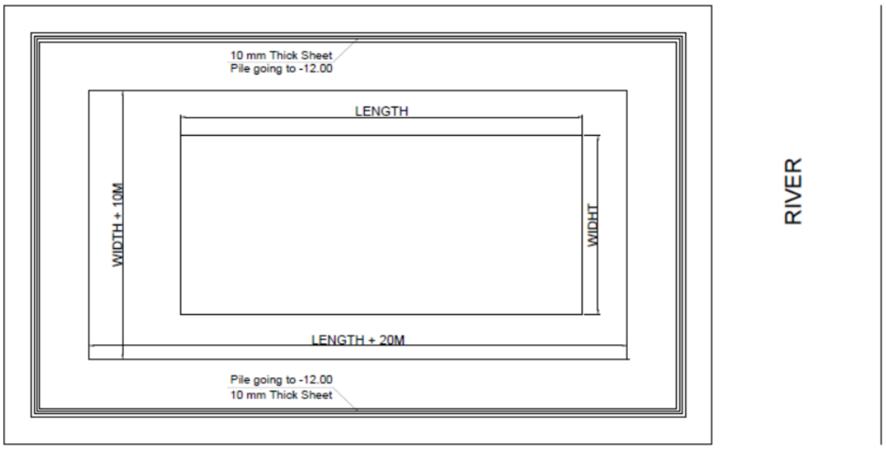

FIGURE 60 PLAN OF THE PROPOSED GRAVING DOCK

8.2 Dry Dock

A floating dry dock with the dimension 46 x 250 m is available for the construction of the floating body. To use this as a construction site, we provided a 0.5m thick gravel layer at the base of the dock to allow for built up of water pressure which would help us lift the body, this process takes around 5 days. The important parameter in this case is the time for which we need to rent the dry dock as it includes the cost of operation too. The floating dock costs 110000 Euros/week compared to only 20000 Euros/week rent of the quay site so we decided to leave the dry dock as soon as possible – which is after construction of the bottom slab and walls only. The top slab of the floating body will be casted once we have our floating body at the quay side and the empty fuel tank has been placed inside of it.

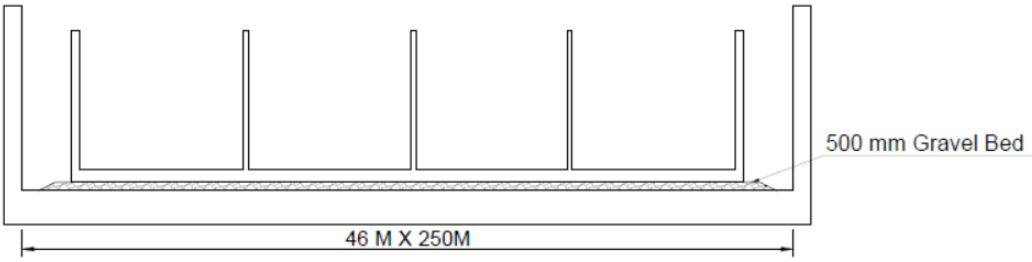

FIGURE 61 CROSS SECTION OF DRY DOCK

For construction in the dry dock, initially the building material is transported to the dry dock using road transportation and then through barges. The on-site concrete mixer is then used to make and concrete and other equipment used to bend steel bars as per the requirements. The crane available helps place the vertical formwork for walls, as we need to move the formwork in segments using the available crane. Once the construction body is complete, it is checked for water tightness as stated in section 8.2.4 of the report. After the test, the steel ropes from the boats are attached to the mooring force transfer details of the floating body. The chambers in the bottom of the dry dock are then opened allowing the water in, which results in sinking of the dry dock. Once the floating body is afloat with help of the gravel layer placed under the floating body, it is towed to the quay site by tug boats.

8.2.1 Bottom Slab

On the top of the 0.5m thick gravel layer, which would help build up the pressure for the floating body to float, we would use plywood formwork to cast bottom slab. As this formwork cannot be retrieved until the floating body is afloat, so it has a repetition factor of 1. For the sides of the bottom slab, smalls panel of 700mm height are used which also have a repetition factor of 1. Along with the bottom slab the bottom 400mm of all the walls is also casted including chamfers so that the construction joint is not at the base. The formwork for this has a repetition factor of 2 as we use the same formwork for the top slab, once at the quay side. The water-stop is also installed as explained in 8.3. The choice between steel and plywood formwork was easy in this case because of no repetition and plywood is much cheaper in terms of initial cost. The plywood formwork would be placed on the gravel bed and concrete would be poured. As per the volume of concrete in the bottom slab, we need 3 days to complete the process and during execution it will be ensured that the construction joints are directly under the walls so we do not have any water tightness problems [11] in the operational phase. The concreting would be done for 3 consecutive days ensuring that the continuation bars and water-stop are placed as detailed in the drawings. For continuation bars couplers are used as by having continuous bars we would require to have formwork with holes which can allow the sticking out of bars from the casted side – this practice is generally avoided because of complex detail and execution.

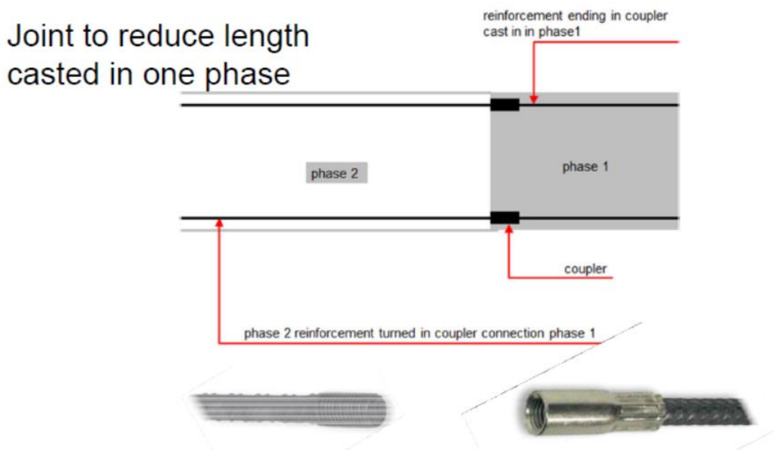

FIGURE 62 COUPLERS IN BOTTOM SLAB

While fixing the reinforcement for bottom slab, provision for the anchor and anchor reinforcement should also be given as mentioned in chapter 7. A similar connection is also provided in the corner walls to tow the structure and also to act as a safety structure if the bottom rope fails while towing.

8.2.2 Walls

As mentioned in formwork section of chapter 2, we decided to use plywood formwork because of the low repetition factor. For the walls the formwork for entire outline was fixed and the concrete was poured in, keeping in mind the 800 kg/m^3 casting capacities. This gives us a repetition factor of six for the formwork and requires 6 weeks of casting as a one-week cycle was used as mentioned during the lectures. The formwork will be prepared and reinforcement placed (also de-shuttering and sliding the formwork up) from Monday to Thursday and concreting will be done on Friday – the concrete would be left to develop some strength and the cycle repeated from Monday. To erect the formwork in the required position and to slide it upwards the crane available at the dry dock would be used. All the compartments also have a 1m diameter opening in the walls so curved plywood needs to be placed in the formwork during the fixing period. These opening will be used by the worker to get in to compartments while removing the formwork for the next cycle because otherwise only vertical access would be left, which is an execution problem. All the joints in the walls should also be equipped with the mentioned water-stop to ensure water tightness.

Another limiting factor realised here was the area for placing the formwork, as to ensure that the formwork and reinforcement is placed in 4 days we had restriction on the total area of formwork we can place – effecting the height of the wall that can be casted per week. A one-week cycle was preferred in this case because it allows the utilization of weekends for development of concrete strength and the working days are not wasted waiting for the concrete to harden so the formwork can be moved vertically. This restriction allowed us to cast 1.50m of concrete every week and resulted in a repetition factor of 6 for the formwork, so per week we cast around 330 m^3 of concrete for walls.

To ensure the stability of formwork against fresh concrete pressure, diagonal propping and ties in between the formwork panels should be used. During the casting of the walls, the provisions for the attachment of the mooring force transfer structure also needs to be provided as mentioned in the drawings and details. In the edge compartments the monkey ladder is also provided so that the compartment can be accessed for repair and maintenance.

The steel bars of monkey ladder should be epoxy coated to avoid corrosion and polluting the fresh water.

8.2.3 Compaction

The concrete in the walls should be properly compacted using an immersion vibrator. Extra care should be taken to avoid the collision of the vibrator with the formwork panel as it might destroy the formwork and in case of excessive damage, the portion of the wall might need to be done again. Similar precautions should be used to avoid contact of vibrator with the reinforcing bars.

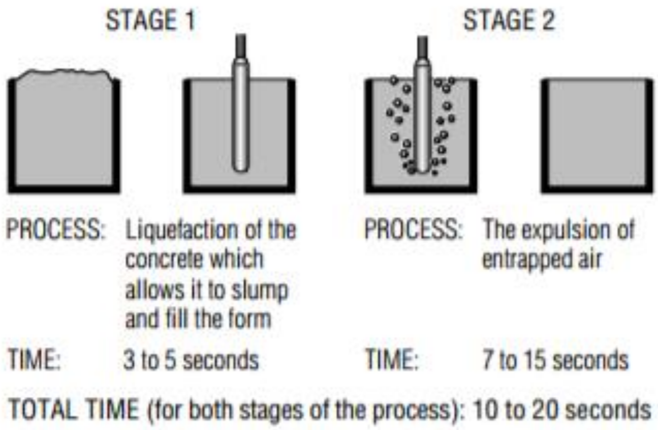

FIGURE 63 COMPACTION OF CONCRETE [12]

For thick members, like the bottom slab in our case, the vibrator should be put up to at least 300mm of the following layer of fresh concrete in order to ensure good compaction.

8.2.4 Floating Body

Once the floating body has been constructed in the dry dock, it will be checked for water tightness as per the provisions of ACI-350R – Code for Environmental Engineering Concrete Structures [8]. Basically, three extreme loading conditions are defined for the check of tanks in PCA note [13] but as we have a floating concrete body so, we only need to apply one of the checks.

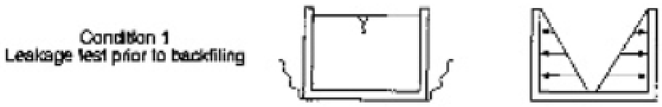

FIGURE 64 WATER TIGHTNESS CHECK [13]

Fill the floating body with the maximum water depth that it has to resist in its service life while in dry dock and check for leakages. This also ensures that the concrete body is actually fulfilling the serviceability and strength criterions. In our case, this test is also the worst possible loading case the floating body has to go through because in operation phase it has water on both sides so the forces are being balanced, hence lower moments. This check also ensures safety of the structure in normal operating conditions.

8.3 Construction Joints

All the construction joint should have a W10U-250 [14] internal rubber-steel water-stop to ensure that the joints are water tight. The concrete around the water stop should be properly compacted to ensure that there are no voids in the hardened concrete as it would affect the durability of concrete structure.

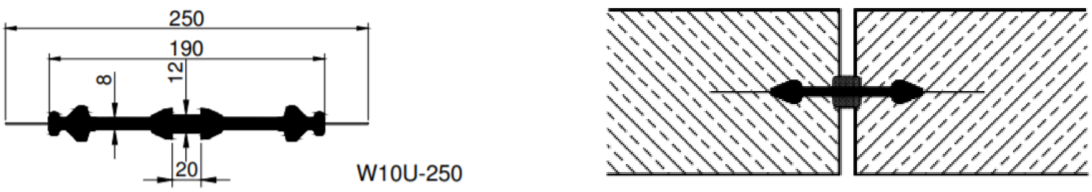

FIGURE 65 DETAILS OF WATER STOP [14]

8.4 Towing to the Quay Side

Once the bottom slab and walls are casted and the water tightness check is applied – also confirms strength of walls, we will attach the towing rope with the mooring fixtures on the floating body and flood the dry dock by letting water in the base compartments, the bottom slab formwork will be retrieved and the body will float.

The body will then be towed to the quay site, the hydraulic and structural checks for the journey are applied in the section for Alternative F – Structural Design. At the quay side the floating body will be made stable with the help of mooring ropes and large bollards at the quay side.

The connection details as provided in chapter 7 should be used to attach the rope to the floating body.

8.5 Quay Side

At the quay side, we need to install the fuel tank inside the body, the top slab and the superstructure. Here the limiting factors are the reach of the crane, the capacity of the quay crane and the available draught. The checks for draught are mentioned in the earlier chapters and the execution details of the task are mentioned here.

8.5.1 Top Slab

Before the casting of top slab, the empty fuel tank will be lowered down in the compartment highlighted in the plan of Alternative F. For the top slab lattice girder floor system is used. This has an advantage over the cast in-situ concrete floor, in terms of formwork. However, the joint detailing is more complex in lattice girder floors. As the span of the planks required is relatively large, we ordered 200mm thick prefabricated planks of the regular 2.16m width and 10.8m length. Pekso [15] Precast, a company that operates in The Netherlands can provide such dimensions as mentioned in their referenced brochure. We also checked the allowable transport size and weights in The Netherlands and the plank floor is well within the limits as per the provisions of IABSE report [16].

The planks will be ordered with the required openings and will be placed on the top of the floating body with help of the quay crane. To estimate the time required to place all the planks on the floating body we assumed a crane speed of 30 fpm (0.15m/s) [17]. Each plank weighs 11.6 tonnes and hence can be placed on top of the floating body using the available crane as it has a capacity of 37.5 tonnes at 40m reach. The placing of planks on the body would take around 5 days, including the propping. As the remaining volume of the concrete is lower than 800 kg/m^3, the entire top slab will be casted in one go. But as the span of the plank is relatively long, we still provide propping to ensure safe casting operation with minimal deflections. These propping will then be removed through the opening in all the top slabs panels as shown in the figure. Some of these opening will permanently be closed by the super structure in later phases of the project. The formwork used at this stage is the same 400mm plywood sheets that were used for the wall continuation at the bottom slab.

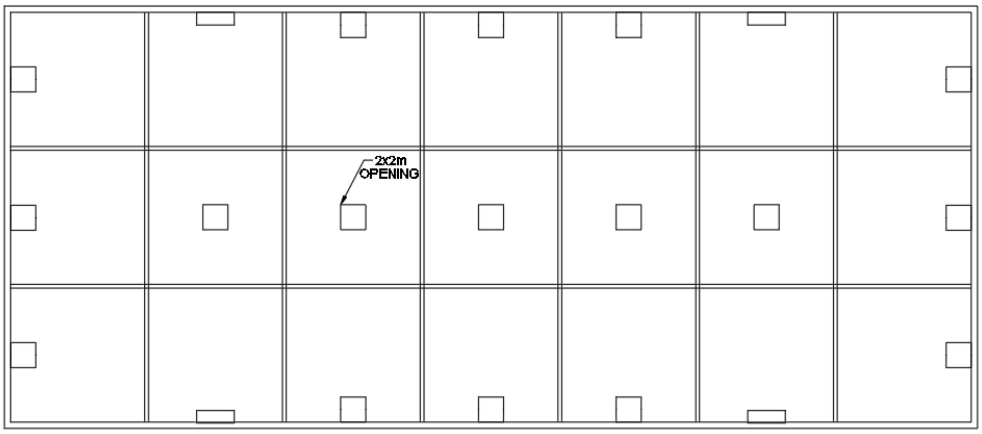

FIGURE 66 OPENINGS IN THE TOP SLAB

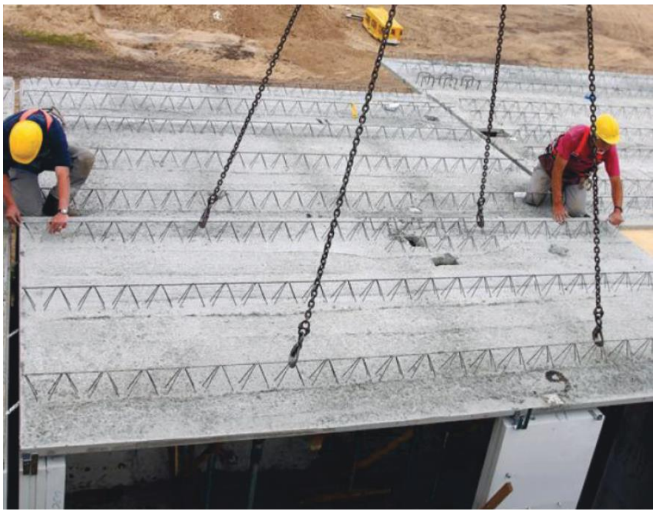

FIGURE 67 LATTICE GIRDER FLOOR SYSTEM [20]

8.5.2 Reinforcement details around the sleeves in the inner walls and manholes in the top slab

All the inner walls have been provided with a circular sleeve of 1m to allow for the water in all the compartments to rise equally – this helps us provide minimum reinforcement in the inner walls because the loading form one side is cancelled out by the loading from the other. However, this opening in the wall causes some reinforcement to be cut so extra reinforcing bars are required in the area around the openings as per the details in the attached figure. Similarly, all the top slabs have openings to allow for the filling in of the ballast at the final location so we can sink the floating body and ensure a factor of safety of 1.20 against uplifting. The reinforcement details around such openings are also attached.

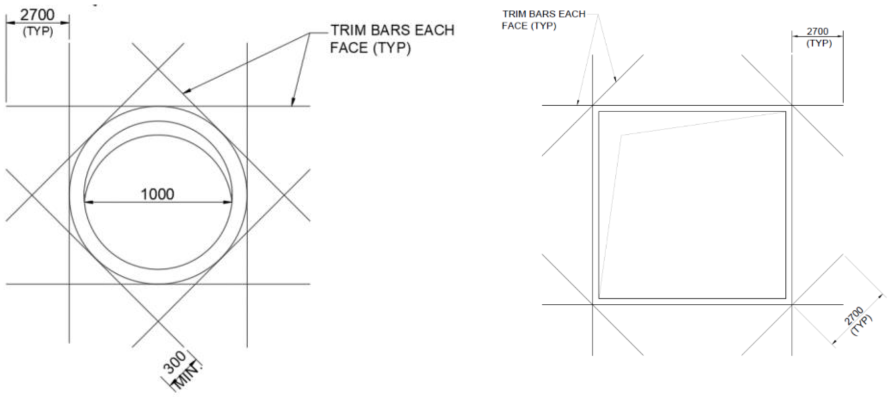

FIGURE 68REINFORCEMENT DETAIL AROUND SLEEVES AND MANHOLES

8.5.3 Installation of Superstructure

We will allow the top slab to harden for 14 days so almost 90% of the total compressive strength is developed. After that we will start to install the superstructures on the top slab including the evaporators and buildings. The evaporators will be installed in such a way that the entire structure does not tilt too much in one direction and hydraulic stability is maintained. To estimate the total time required for super structure installation we assumed the same speed and the minimum capacity of 37.5 tonnes and calculated the number of cycles required by the crane to complete the task. This will require 5 days to complete.

Before the installation work, a 3m high guard rail is also installed around the perimeter of the floating body, in line with safety and health regulations.

8.6 Towing journey from quay site to final location

After the installation of the superstructure the floating body with all the installations will be towed to its final location using tug boats. The hydraulic and structural stability is checked in the structural design chapter for alternative F. The final draught for this alternative is 5.77m so we have almost 1m clearance from the bed as recommended [18]. At the final location we have also planned the dredging till 6.5m of depth giving us 730mm of clearance from the bed so the body can safely reach the final location.

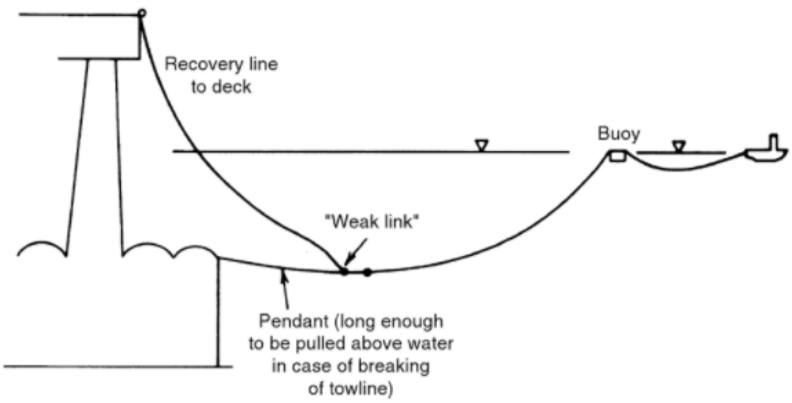

FIGURE 69 TOWING CONFIGURATION [19]

As shown in the figure 69 the attachment for the rope is given at the bottom slab and a safety rope is attached to the top slab in case the bottom rope fails – this reduces the risk while towing the floating body.

8.7 Final Location

8.7.1 Dredging and Platform

As mentioned in the earlier sections of the report, we could opt for either having an offshore structure with an extended platform or having a structure close to land with a lot of dredging. We compared the costs of the two and decided to opt for dredging as it was cheaper compared to having a platform as it required a lot of construction including pile installation as vehicular passage is also required. Following this, we decided to have the minimum possible length of the platform and have the body as close to land as possible.

For the final alternative, we need a 32.5m platform maintaining the safe slope of 1:5 for the seabed – as we have 10.5m of height and 4m has to be free board. The detail is shown in the figure below.

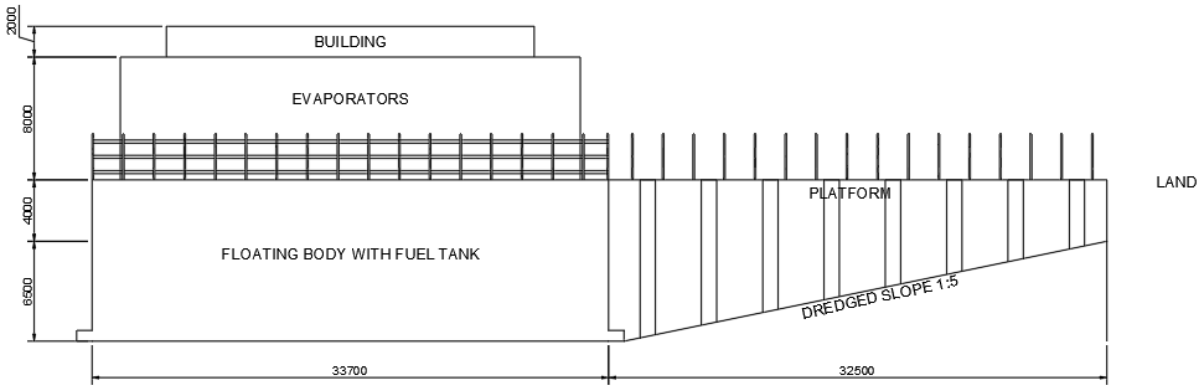

FIGURE 70 SIDE ELEVATION OF THE DESALINATION PLANT AT FINAL LOCATION

After dredging the bottom of the seabed, where the floating body has to rest should be levelled very carefully so no torsional stresses are generated in the bottom slab.

8.7.2 Immersion of the floating body - Ballasting

Once at the final location the floating body needs to be immersed. The weight of the ballast added should be enough that even in the case of having zero fresh water, the structure has a 1.20 factor of safety against uplifting. We had the choice of adding either 18 KN/m^3 or 30 KN/m^3 ballast and we opted for 30KN/m^3 because it was economically more attractive. The ballast will be lowered down in the floating body using the 2x2m openings in the top slab. We analysed some of the possible distributions in the earlier chapters where we argued about the various possible ballast space and fresh water space distribution, but with this alternative the working space on the sides of the evaporators is relatively low to put in the ballast. This required to come up with another possible distribution with globally the same effect so we chose the one in the following figure.

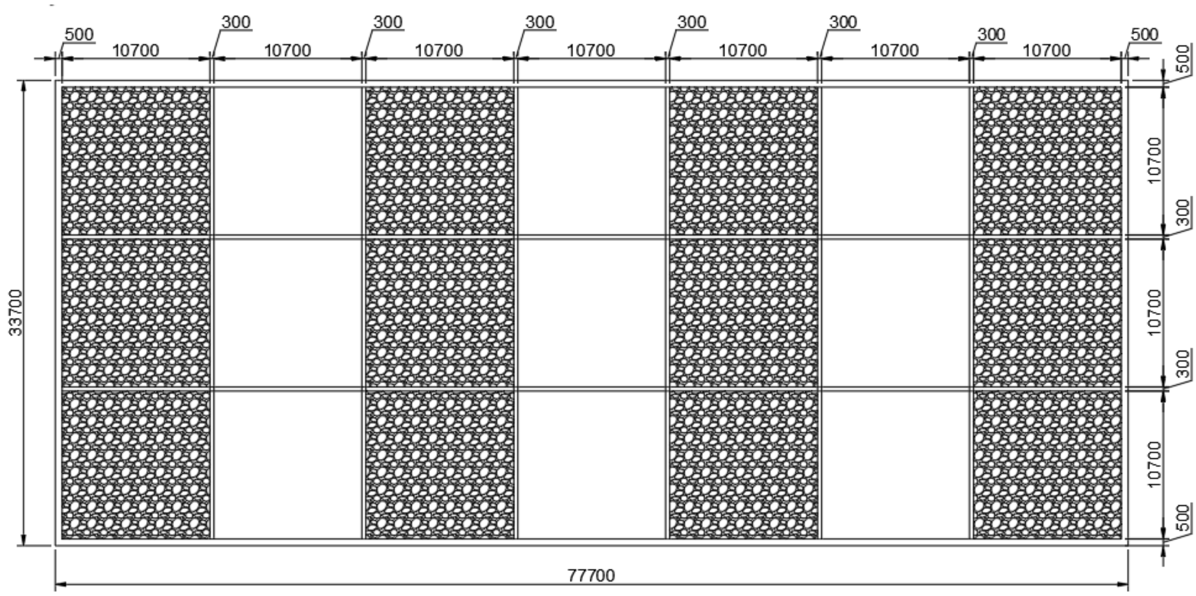

FIGURE 71 BALLAST DISTRIBUTION FOR ALTERNATIVE F

This distribution has the same area available for ballasting as if only the edge compartments were filled so the height of ballast and the structural behaviour remains the same. Ballast should also be uniformly placed in the body, avoiding extra tilting in one direction and damaging the floating body – the cost of such an event is however calculated and included as a risk factor. For this alternative we just need bottom 880mm for the ballast layers and the rest of the volume can be used for water storage.

8.7.3 Scour Protection

After immersing the structure with ballast, we also need to ensure that the bed it rests on is safe from erosion and scouring so we add sea bed protection at a slope of 1:5 around the structure as shown in the figure.

The shore protection was placed in marine environment which helps avoid erosion damages with time to the floating body.

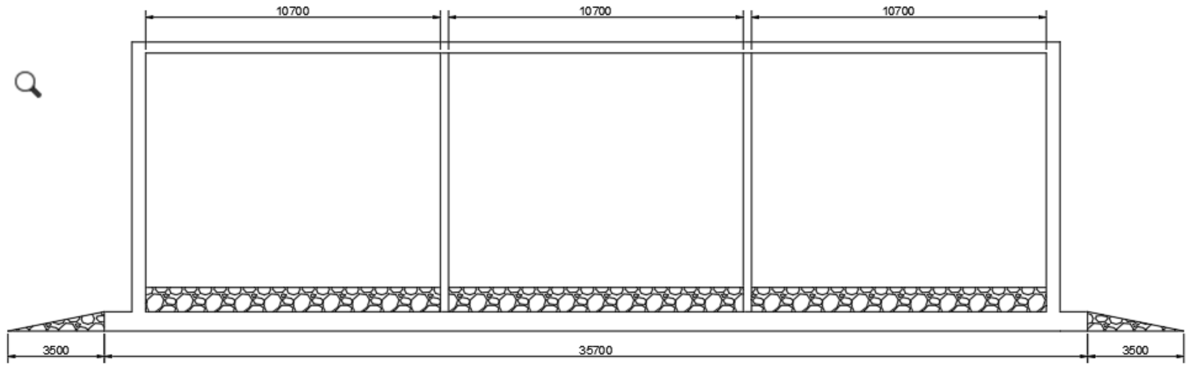

FIGURE 72 SCOUR PROTECTION AT SIDES WITH BALLAST INSIDE

8.8 Detail Drawings

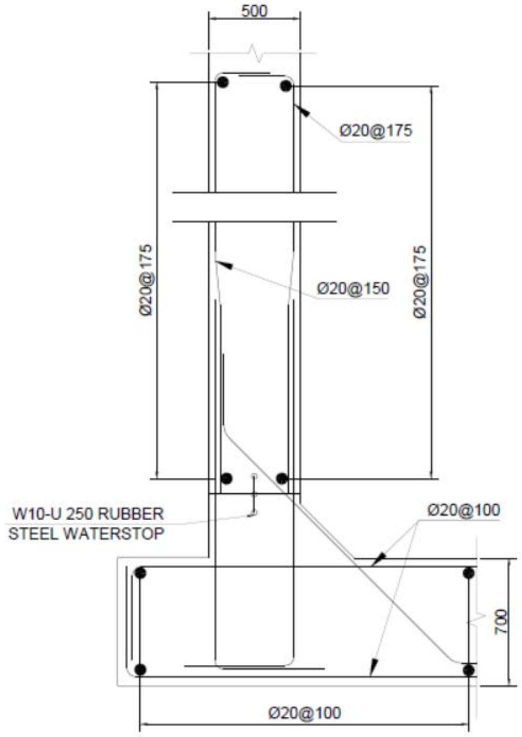

FIGURE 73 TYPICAL X SEC OF OUTER WALL

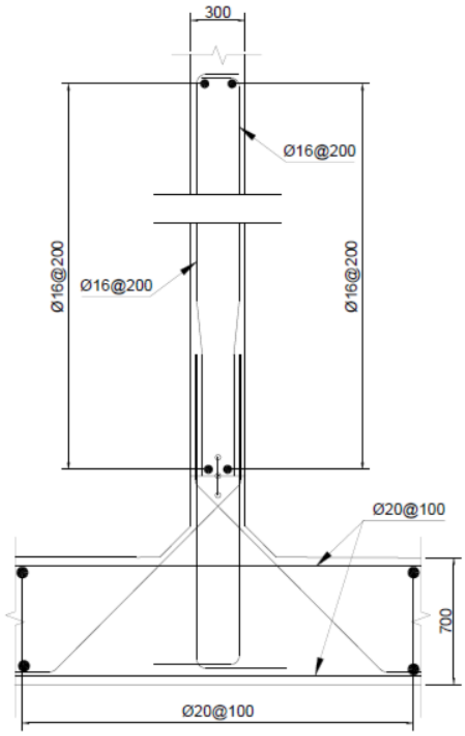

FIGURE 74 TYPICAL X SEC OF INNER WALL

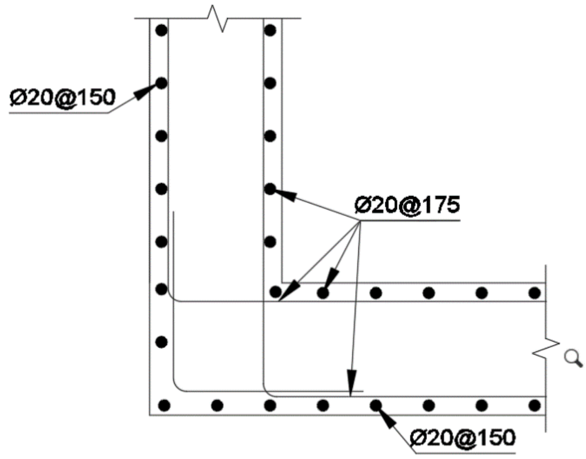

FIGURE 75PLAN OF CORNER REINFORCEMENT

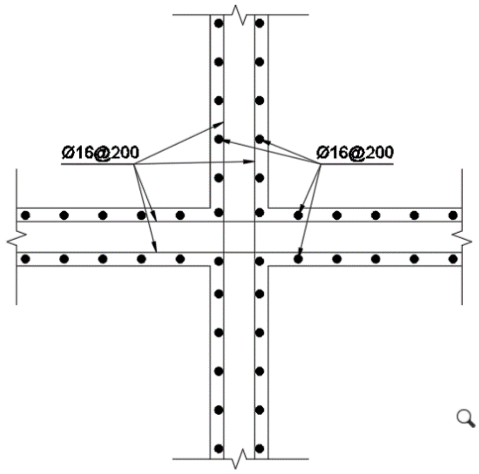

FIGURE 76PLAN OF INNER WALL CONNECTIONS

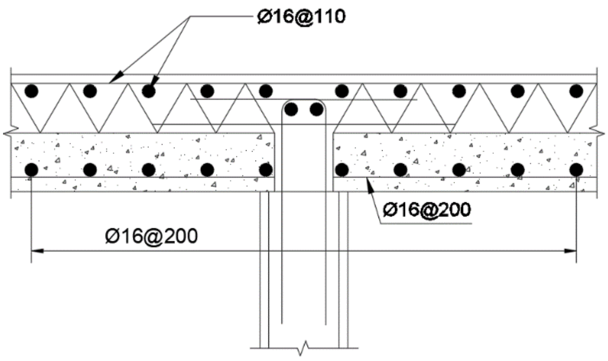

FIGURE 77TOP SLAB REINFORCEMENT

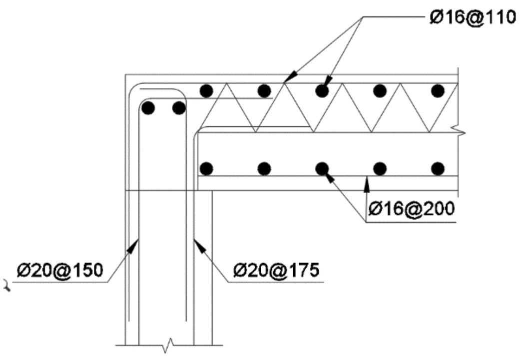

FIGURE 78TOP SLAB CORNER REINFORCEMENT

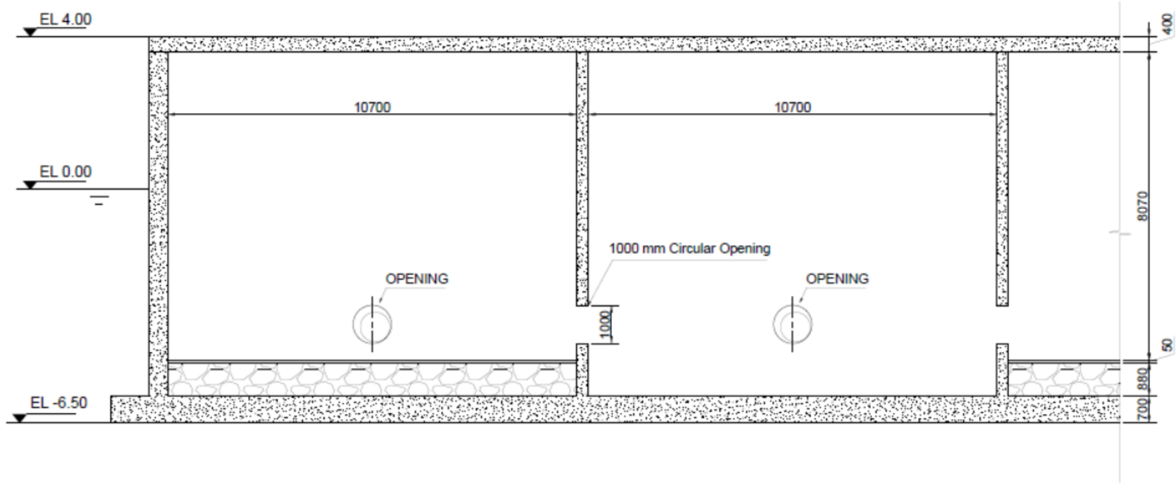

FIGURE 79 X SEC OF BODY IN WIDTH

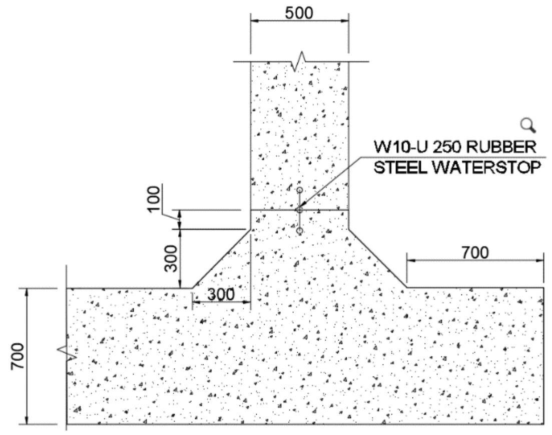

FIGURE 80CORNER CONCRETE OUTLINE

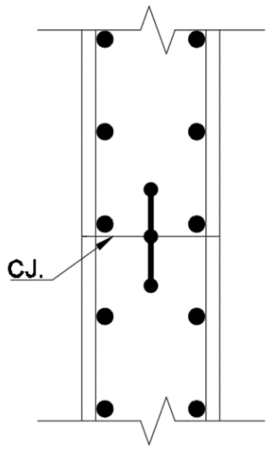

FIGURE 81 CONSTRUCTION JOINT DETAIL

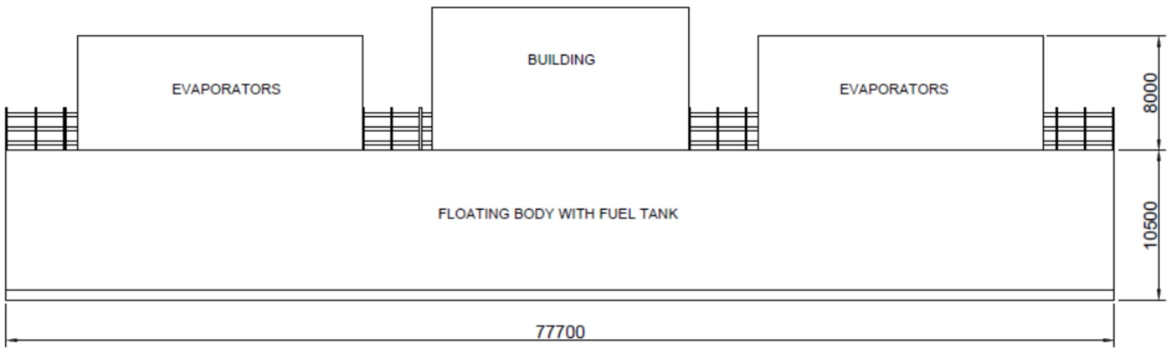

FIGURE 82 ELEVATION OF WATER DESALINATION PLANT

8.9 Scheduling

The scheduling was done for the most economically attractive alternative using Microsoft Project. Finish to start relationship was mostly used for various activities of the project. The independent activities like dredging at final location and construction of the platform however were started as soon as the project commenced so that we can have theses structured ready once the floating body is completed. For wall concreting one-week cycle was used and we assumed a total of 40 workers on site. The Gantt chart for the scheduling is attached in the annex.

To calculate the time required for placing of the prefabricated top slab on the floating body a crane speed of 30 fps (0.15m/s) [20] was used.

The schedule of the project is attached as Annex C.

9 Recommendations

In this chapter we focus on the problems we faced during the project and tried come up with the possible analysis and design suggestions using which we can improve the quality and the reliability of the calculations we made and cost/risks we estimated.

1. Instead of performing analysis on individual elements, as we have done for the assignment, build a full 3 dimensional model with all the applied loads and read of moments and shear forces from that. This will help us visualise and include the effect of the third dimensional forces in our structure and a more accurate and optimized design can be made.
2. For the mooring structure also consider several alternatives to find the one with the most attractive risk/cost ratio. For example having bollards on top slab, having the same hooked connections in the side walls, using L or J type anchor instead of hexagonal head stud.
3. Perform a more detailed risk analysis in which all the small risks are also included which are neglected in this report due to lack of experience and knowledge.
4. In the cot analysis some of the costs are ignored, like extra cost for making the formwork climbing or the cost of anchor bolts for the mooring structure. Some indirect costs were also overlooked. Considering all the costs would give a much better estimation of the actual cost of the structure.
5. Consider making the top slab completely pre-fabricated so the time required for the curing of the top slab can be utilised for the progress of the project, joint however would need extra attention.
6. Use superplasticizers to help improve the compaction of the concrete – self-compacting concrete systems.
7. Use admixtures which speed up the strength gain of concrete and can help save some extra time like accelerators or rapid hardening cementitious systems. Compare the cost/risk of using the admixtures and earning the time value aspect to the cost of the project with traditional normal strength concrete.
8. Calculate the actual laps required in the steel bars so the actual length and weight of steel can be computed and the ratio of steel to concrete can be calculated – this would give a much more realistic value for cost.
9. Consider prestressing the structure – this would help us improve the water tightness of the structure and may also lead to comparatively thinner sections because of the prestressing load.
10. Compare the cost of installing a retaining structure and making a vertical cut to bring the structure on shore with having the 1:5 slope with a small length of platform.
11. In the schedule include the resources (financial and human) in addition to the time required to complete each task.

ANNEX A

TABLE 76 RISK ANALYSIS FOR ALTERNATIVE A

	Category	Cause	Event	Consequence	Prob.	Impact		Risk	Event Cost (€)	Risk Cost (€)
						Cost	Time			
1	**Monetary**	Economic Situation of the country	Inflation	Increase in net cost	M	VL	-	0.025	83701	2093
2		Economic Situation of the country	Government Failure	Suspension of the project	VL	VH	-	0.08	858474	68678
3	**Environmental**	Extensive dewatering	Possible subsidence of the soil	Damage of the surrounding areas	VL	VH	VH	0.16	24469	3915
4		New Environmental Regulations	Changes in the environmental permit	Delay of the construction activities as new design required	M	L	M	0.125	500000	62500
5		Inexperienced environmental assessors	Inaccurate Environmental analysis	Temporally suspension of construction	VL	L	M	0.025	500000	12500
6	**Technical**	Flood during construction	Water in the dry dock	Delay in Construction	H	L	L	0.14	200000	28000
7		Unforeseen event during excavation of graving dock	Alternative plans to solve the incident	Extra time to solve the problem and rent of new technology	VL	H	H	0.08	150000	12000
8		Failure in formwork	Non-uniform concrete surface	Re-do the wall of the floating body	L	M	L	0.006	24039	144
9		Tugging failure during transportation	Instability of the floating body	Extra time and effort required to solve the issue	M	L	M	0.125	50000	6250
10		Human error	Improper compaction of concrete	High porosity of concrete - less durability	L	H	H	0.08	501201	40096
11		Tugging failure during positioning of the floating body	Wrong placement of the floating body	Un-Ballasting the floating body and then repositioning	VL	H	H	0.08	59212	4737
12		Improper soil study	Bearing capacity failure	Extra time and cost to study/mitigate effects	VL	VH	VH	0.16	429237	68678
13		Miscalculation of the amount of ballast needed	FoS 1.20 not ensured	Excessive settlement in case of accidental loads	L	L	L	0.06	505648	30339
14		Dynamic stability failure - not checked	Failure of the floating body	Method to cater that, If possible or redesign	L	H	H	0.08	1287711	103017
15		The dewatering pump malfunction in graving dock	Insufficient lowering of water table	Extra time and cost for adjustments	L	L	L	0.06	48939	2936
16		Insufficient provision of gravel layer under the floating body	Problems in lifting the floating body	Measures and time to lift	VL	M	L	0.03	6811	204
17		Water tightness failure	Check as per ACI 350-R fails	Repair and redesign the floating body	VL	M	M	0.04	4292	172

18		Not enough clearance from the sea bed	Damage to the floating body	Repair of the floating body	L	VH	VH	0.48	241709	116020
19		Seabed not even	Generation of torsional stresses in the bottom slab	Structural checks to check strength	L	H	H	0.24	48342	11602
20		Strong waves during construction	Dry dock unstable	Delay in construction	M	H	H	0.4	50000	20000
21		Strong waves during transport	Instability of the floating body	Delay in transport	M	H	H	0.4	50000	20000
22		Higher high tides than usual	Failure of dykes in the graving dock	Repaired cost and time	L	L	M	0.09	36725	3305
23	**Project**	Improper sheet pile depth in graving dock	Water seepage in the graving dock	Temporary suspension of construction activities	VL	M	H	0.06	150000	9000
24		Excessive rain	Delay in Concreting	Waste of time	M	L	M	0.15	50000	7500
25		Dysfunctional construction and transport equipment	Damage of the equipment	Delay in transport and construction activities	L	M	H	0.18	50000	9000
26		Lack of acceptance by the investor of design proposals	Delays in the approval	Increase of costs due to extra time	L	L	M	0.09	250000	22500
27		Lack of co-ordination between consultant, contractor and client	Project team conflicts	Extra time and cost due to improper communication	VL	L	L	0.02	250000	5000
28		Inexperienced Captain of boat	Difficulties to maneuver the boats	Delay in transport of the floating body	VL	L	L	0.02	50000	1000
29		Inexperienced workforce and staff turnover	The constructed floating body does not fulfil one of the boundary condition	Delays due to repair	VL	H	H	0.08	952672	76214
30		Insufficient Budget	Improper Cost Analysis	Suspension of construction until new budget approved	M	M	H	0.3	500000	150000
31		Insufficient Marking of areas	Proximity of unauthorized personnel	Safety or security violation	L	VL	VL	0.03	50000	1500
32		Delayed supply of materials	Transportation or communicational problems	Difficulties to meet the set deadline	M	L	H	0.25	250000	62500
33		Material supplied not meeting the set requirements	Communicational problems	Difficulties to meet the set deadline	L	M	H	0.18	976211	175718
34	**Human**	Human Error	Errors in the design	Extra cost and time to make corrections	L	M	H	0.18	251344	45242
35	**Social**	Community disagreement on the construction of the project	Public objections	Project Suspension	L	H	H	0.24	858474	206034
36	**Mechanical**	Malfunction in the valves of dry dock	Mechanical error	Delay in project for repair of the dry dock	M	M	H	0.3	150000	45000
37		Malfunctioning of the dredger	Mechanical Problems	Delay in project for repair of the dredger	L	M	M	0.12	100000	12000

38		Malfunctioning of the crane at quay site	Mechanical Problems	Delay in project for repair of the crane	L	M	H	0.18	150000	27000

TABLE 77 RISK ANALYSIS FOR ALTERNATIVE F

	Category	Cause	Event	Consequence	Prob.	Impact		Risk	Event Cost (€)	Risk Cost (€)
						Cost	Time			
1	**Monetary**									
2		Economic Situation of the country	Inflation	Increase in net cost	M	VL	-	0.025	99137	2478
4		Economic Situation of the country	Government Failure	Suspension of the project	VL	VH	-	0.08	1016792	81343
6	**Environmental**	Extensive dewatering	Possible subsidence of the soil	Damage of the surrounding areas	VL	VH	VH	0.16	363901	58224
7		New Environmental Regulations	Changes in the environmental permit	Delay of the construction activities as new design required	M	L	M	0.125	500000	62500
8		Inexperienced environmental assessors	Inaccurate Environmental analysis	Temporally suspension of construction	VL	L	M	0.025	500000	12500
9	**Technical**	Flood during construction	Water in the dry dock	Delay in Construction	H	L	L	0.14	200000	28000
10		Unforeseen event during excavation of graving dock	Alternative plans to solve the incident	Extra time to solve the problem and rent of new technology	VL	H	H	0.08	150000	12000
11		Failure in formwork	Non-uniform concrete surface	Re-do the wall of the floating body	L	M	L	0.006	24039	144
12		Tugging failure during transportation	Instability of the floating body	Extra time and effort required to solve the issue	M	L	M	0.125	50000	6250
13		Human error	Improper compaction of concrete	High porosity of concrete - less durability	L	H	H	0.08	461571	36926
14		Tugging failure during positioning of the floating body	Wrong placement of the floating body	Un-Ballasting the floating body and then repositioning	VL	H	H	0.08	548288	43863
15		Improper soil study	Bearing capacity failure	Extra time and cost to study/mitigate effects	VL	VH	VH	0.16	435616	69699
16		Miscalculation of the amount of ballast needed	FoS 1.20 not ensured	Excessive settlement in case of accidental loads	L	L	L	0.06	510811	30649
17		Dynamic stability failure - not checked	Failure of the floating body	Method to cater that, If possible or redesign	L	H	H	0.08	1306848	104548
18		The dewatering pump malfunction in graving dock	Insufficient lowering of water table	Extra time and cost for adjustments	L	L	L	0.06	46438	2786
19		Insufficient provision of gravel layer under the floating body	Problems in lifting the floating body	Measures and time to lift	VL	M	L	0.03	24400	732
20		Water tightness failure	Check as per ACI 350-R fails	Repair and redesign the floating body	VL	M	M	0.04	4356	174

21		Not enough clearance from the sea bed	Damage to the floating body	Repair of the floating body	L	VH	VH	0.48	219953	105578
22		Seabed not even	Generation of torsional stresses in the bottom slab	Structural checks to check strength	L	H	H	0.24	43991	10558
23		Strong waves during construction	Dry dock unstable	Delay in construction	M	H	H	0.4	50000	20000
24		Strong waves during transport	Instability of the floating body	Delay in transport	M	H	H	0.4	50000	20000
25		Higher tides than usual	Failure of dykes in the graving dock	Repair, cost and time	L	L	M	0.09	3193	287
26	**Project**	Improper sheet pile depth in graving dock	Water seepage in the graving dock	Temporary suspension of construction activities	VL	M	H	0.06	150000	9000
27		Excessive rain	Delay in Concreting	Waste of time	M	L	M	0.15	50000	7500
28		Dysfunctional construction and transport equipment	Damage of the equipment	Delay in transport and construction activities	L	M	H	0.18	50000	9000
29		Lack of acceptance by the investor of design proposals	Delays in the approval	Increase of costs due to extra time	L	L	M	0.09	250000	22500
30		Lack of co-ordination between consultant, contractor and client	Project team conflicts	Extra time and cost due to improper communication	VL	L	L	0.02	250000	5000
31		Inexperienced Captain of boat	Difficulties to maneuver the boats	Delay in transport of the floating body	VL	L	L	0.02	50000	1000
32		Inexperienced workforce and staff turnover	The constructed floating body does not fulfill one of the boundary condition	Delays due to repair	VL	H	H	0.08	938129	75050
33		Insufficient Budget	Improper Cost Analysis	Suspension of construction until new budget approved	M	M	H	0.3	500000	150000
34		Insufficient Marking of areas	Proximity of unauthorized personnel	Safety or security violation	L	VL	VL	0.03	50000	1500
35		Delayed supply of materials	Transportation or communicational problems	Difficulties to meet the set deadline	M	L	H	0.25	250000	62500
36		Material supplied not meeting the set requirements	Communicational problems	Difficulties to meet the set deadline	L	M	H	0.18	461571	83083
37	**Human**	Human Error	Errors in the design	Extra cost and time to make corrections	L	M	H	0.18	254198	45756
38	**Social**	Community disagreement on the construction of the project	Public objections	Project Suspension	L	H	H	0.24	871232	209096
39	**Mechanical**	Malfunction in the valves of dry dock	Mechanical error	Delay in project for repair of the dry dock	M	M	H	0.3	150000	45000

40		Malfunctioning of the dredger	Mechanical Problems	Delay in project for repair of the dredger	L	M	M	0.12	100000	12000
41		Malfunctioning of the crane at quay site	Mechanical Problems	Delay in project for repair of the crane	L	M	H	0.18	150000	27000

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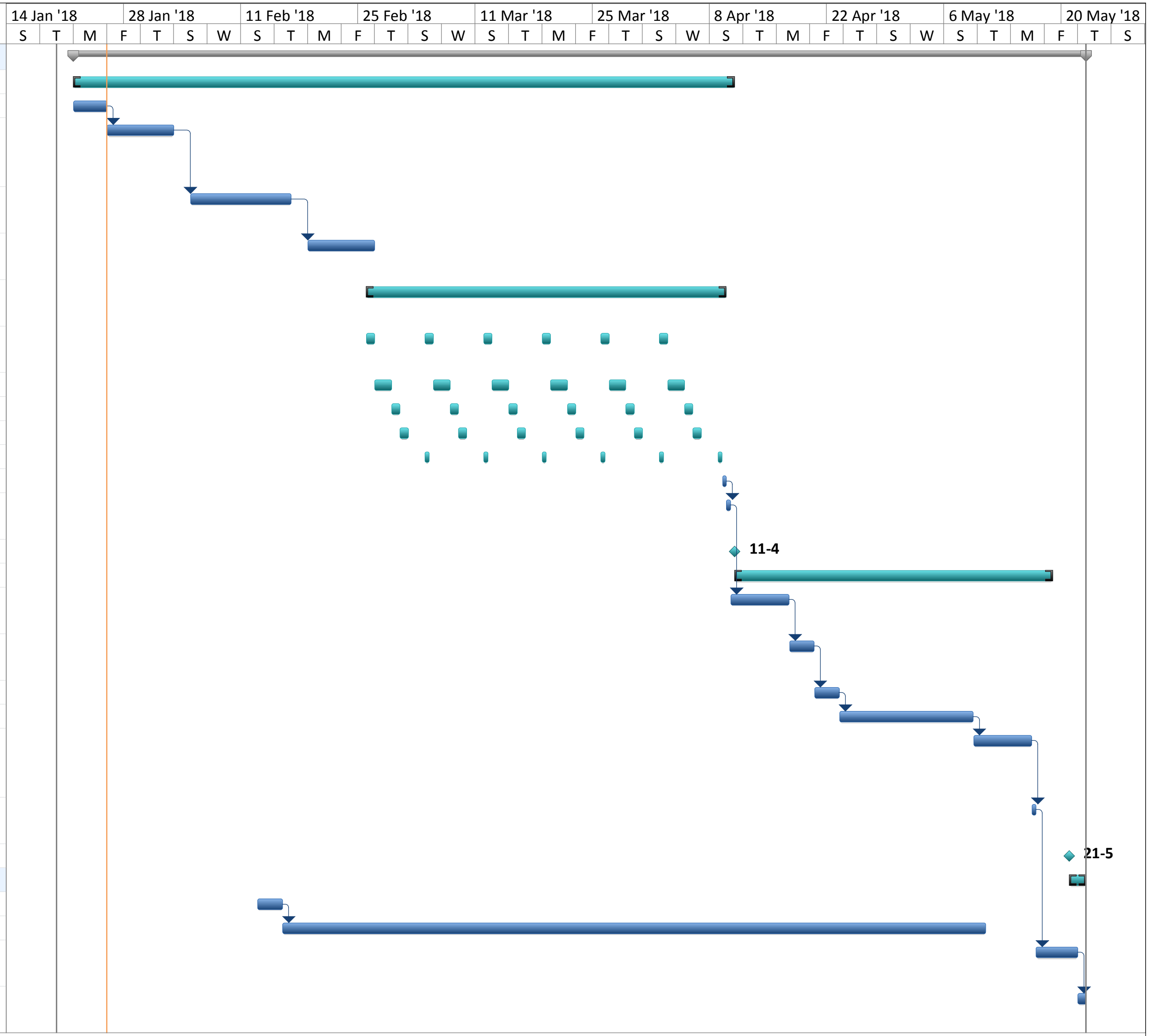
ANNEX B

TABLE 78 FINAL SUMMARY

D-W-L	10.5x33.7x77.7	m
Extensions for Bottom Slab	1	m
Draught during towing journey over sea	5.77	m
Compartment Substructure		
Number of Cells	21	
Cell Dimensions	10.7x10.7	m
Inner Wall Thickness	0.3	m
Outer Wall Thickness	0.5	m
Deck Thickness	0.4	m
Bottom Slab Thickness	0.7	m
Chamfers	0.3	m
Bottom Slab		
m3 of Concrete	1991	m3
Kg of reinforcing steel	323806	kg
m2 of formwork including the bottom wall projections and chamfers	3595	m2
Repetition factor	1	
Compartment Walls		
m3 of Concrete	2065	m3
Kg of reinforcing steel	257555	kg
m2 of formwork	1682	m2
Repetition factor	6	
Deck		
m3 of Concrete	1047	m3
Kg of reinforcing steel	134802	kg
m2 of formwork (also side formwork for bottom slab)	89	m2
Repetition factor	2	
Entire Structure		
m3 of Concrete	5103.00	m3
Kg of reinforcing steel	716163	kg
m2 of formwork	5277	m2
Total Concrete Cost (concrete, reinforcement and formwork with repetition without manhour)	1406030	€
Construction Time	5 weeks 1 day	
Type of Building Dock	Dry Dock	
Solution at final location	Platform + Dredging	
Length of platform	32.5	m
Length of dredging trench	617.49	m
m3 of ballast sand	1207	m3
Cost Summary		
Total Concrete	2080787	€
Building Dock	968000	€
At site location	1406506	€
Ballast Sand	48288	€
Total	4503581	€

ID	Task Mode	Task Name	Duration	Start	Finish
0		**Scheduling**	**87 days**	**Mon 22-1-18**	Tue 22-5-18
1		Dry Dock	57 days	Mon 22-1-18	Tue 10-4-18
2		Laying of Gravel Layer	4 days	Mon 22-1-18	Thu 25-1-18
3		Placing plywood formwork for bottom slab	48 hrs	Fri 26-1-18	Fri 2-2-18
4		Reinforcement fixing of bottom slab	10 days	Mon 5-2-18	Fri 16-2-18
5		Concreting the bottom slab	6 days	Mon 19-2-18	Mon 26-2-18
6		Ordering Planks for top slab	31 days	**Mon 26-2-18**	Mon 9-4-18
7		**Placing Wall Formwork**	**26 days**	**Mon 26-2-18**	**Mon 2-4-18**
14		**Fixing Wall Reinforcem**	**27 days**	**Tue 27-2-18**	**Wed 4-4-18**
21		**Placing Formwork**	**26 days**	**Thu 1-3-18**	**Thu 5-4-18**
28		**Concreting of walls**	**26 days**	**Fri 2-3-18**	**Fri 6-4-18**
35		**De-Shuttering**	**25,5 days**	**Mon 5-3-18**	**Mon 9-4-18**
42		Flooding the dry dock	4 hrs	Mon 9-4-18	Mon 9-4-18
43		Towing the body to quay side	4 hrs	Tue 10-4-18	Tue 10-4-18
44		Quay side	0 days	Wed 11-4-18	Wed 11-4-18
45		Quay side	28 days	Wed 11-4-18	Fri 18-5-18
46		Placing precast planks on the body	5 days	Tue 10-4-18	Tue 17-4-18
47		Fixing reinforcement for top slab	3 days	Tue 17-4-18	Fri 20-4-18
48		Concreting top slab	1 day	Fri 20-4-18	Mon 23-4-18
49		Curing of top slab	12 days	Mon 23-4-18	Wed 9-5-18
50		Installation of superstructure on top slab	5 days	Wed 9-5-18	Wed 16-5-18
51		Towing the floating boady to final location	4 hrs	Wed 16-5-18	Wed 16-5-18
52		Final Location	0 days	Mon 21-5-18	Mon 21-5-18
53		Final Location	2 days	Mon 21-5-18	Tue 22-5-18
54		Dredging	3 days	Tue 13-2-18	Thu 15-2-18
55		Platform Construction	60 days	Fri 16-2-18	Thu 10-5-18
56		Ballasting the body at final location	3 days	Thu 17-5-18	Mon 21-5-18
57		Scour Protection around body	1 day	Tue 22-5-18	Tue 22-5-18



Project: Scheduling
Date: Fri 26-1-18

Task	Project Summary	Inactive Milestone	Manual Summary Rollup	Deadline
Split	External Tasks	Inactive Summary	Manual Summary	Progress
Milestone	External Milestone	Manual Task	Start-only	
Summary	Inactive Task	Duration-only	Finish-only	