“Analysis and Design of an Onshore Structure”

A PROJECT

Submitted in partial fulfillment of the requirements for the award of the degree of

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

Under the supervision of

Mr. Chandra Pal Gautam

(Assistant Professor)

By

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To

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT SOLAN – 173 234

HIMACHAL PRADESH INDIA

June, 2016

CERTIFICATE

This is to certify that the work which is being presented in the project title **"Analysis And Design Of An Onshore Structure"** in partial fulfillment of the requirements for the award of the degree of Bachelor of Technology and submitted in Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by **Rajat Rana (121659) and Suraj Rai (121626)** during a period from July 2015 to June 2016 under the supervision of **Mr. Chandra Pal Gautam** Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology Waknaghat.

The above statement is correct to the best of my knowledge.

Date: - ………………………

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The topic **"Analysis and Design of an Onshore Structure"** was very helpful to us in giving the necessary background information and inspiration in choosing this topic for the project. Our sincere thanks to our Project Guide **Mr. Chandra Pal Gautam,** Assistant Prof. for having supported the work related to this project. Their contributions and technical support in preparing this report are greatly acknowledged.

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Abstract

Marine and onshore structures are constructed worldwide for variety of functions and in variety of water depths and environmental conditions. Offshore construction is the installation of structures and facilities in a marine environment, usually for the production and transmission of electricity, oil, gas and other resources. Construction and pre-commissioning is typically performed as much as possible onshore. To optimize the costs and risks of installing large offshore platforms, different construction strategies have been developed .

As the population density of cities like Mumbai, Chennai is increasing; land is going scarce day by day. And very soon the time will come when there will be no land to construct anything. So aim of this project is to push our structures to the coastline taking advantage of huge coastline of India (7,515 km) as new exile area for construction practices. Since the right selection of equipment, types of platform and also right planning, design, transportation, installation considering the water depth, tidal heights, and environmental conditions are very important, this project presents the general overview of all these aspects.

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List of Symbols

Fck	=	Grade of Concrete
Fy	=	Grade of Steel
b	=	Width
d	=	Effective depth
D	=	Overall Depth
Mu	=	Moment of Resistance
Ast	=	Area of steel
β	=	dimensionless shaft friction factor
p_o	=	effective overburden pressure at the depth
K	=	coefficient of lateral earth pressure
Δ	=	friction angle between the soil and pile wall
N_q	=	dimensionless bearing capacity factor
k_h	=	Modulus of subgrade reaction
I_p	=	Moment of Inertia of pile cross section area
E_p	=	Elastic modulus of pile
E_s	=	Elastic modulus of soil
u	=	Poisson ratio of soil

Chapter 1: Introduction

1.1 General

Offshore/onshore is generally a term used in Oil and Gas Industry. Offshore is an oil rig / structure stationed in the middle of the sea (like those you see at the North Sea in Europe). Onshore means on land or having a land base.

A building or is a man-made structure with a roof and walls standing more or less permanently in one place, such as a house or factory. Buildings come in a variety of shapes, sizes and functions, and have been adapted throughout history for a wide number of factors, from building materials available, to weather conditions, to land prices, ground conditions, specific uses and aesthetic reasons. Buildings serve several needs of society – primarily as shelter from weather, security, living space, privacy, to store belongings, and to comfortably live and work.

A Multi-storey is a building that has multiple floors above ground in the building. Multi-storey buildings aim to increase the floor area of the building without increasing the area of the land the building is built on, hence saving land and, in most cases, money (depending on material used and land prices in the area). The building with the most stories is the Burj Khalifa, with 162.

According to the building code of Hyderabad, India, a high-rise building is one with four floors or more, or one 15 meters or more in height.

1.2 Need of Study

As the population density of cities like Mumbai, Chennai is increasing land is going scarce day by day. And very soon the time will come when there will be no land to construct anything. So aim of this project is to push our structures to the coastline taking advantage of huge coastline of India (7,515 km) as new exile area for construction practices.

1.3 Scope

With the current project as design of a building in coastal region (onshore structure) using berth and pile foundation there can be an increase in land space which is the need for the development of region and a new alternative when all the cities are dying for land in present world.

1.4 Project Methodology

Site Selection

↓

Making Plan

↓

Modelling on staad and applying loading conditions

↓

Analysis of structure and getting design reactions

↓

Design of Slab, beams, columns, berth slab.

↓

Analysis and Design of piles

CHAPTER 2: LITERATURE REVIEW

2.1 General

A good literature review demonstrates that you know the field. This means more than reporting what you've read and understood. Instead, you need to read it critically and to write in such a way that shows you have a feel for the area; you know what the most important issues are and their relevance to your work, you know the controversies, you know what's neglected, you have the anticipation of where it's being taken. All this would allow you to map the field and position your research within the context. It justifies the reason for your research. This is closely connected with demonstrating that you know the field. It is the knowledge of your field which allows you to identify the gap which your research could fill. However, it is not enough to find a gap. You have also to be able to convince your reader that what you are doing is important and needs to be done. It allows you to establish your theoretical framework and methodological focus. Even if you are proposing a new theory or a new method, you are doing so in relation to what has been done.

2.2 Paper Description

Mather Bryant (1964), "Effects of Sea on Concrete" ***International Journal of Engineering.paper no 6-690.***

Concrete exposed to sea water is wetted by a solution of salts--principally sodium chloride and magnesium sulfate. Damage to concrete, if it occurs, usually results from failure to use good practices in concrete construction, and often is the result of freezing and thawing or wetting and drying as much as or more than the results of the effects of sea water as such. Magnesium sulfate may attack most, if not all, of the constituents of hardened portland cement paste, especially the aluminate constituent; chlorides may promote corrosion of steel; and alkalies may participate in alkali-aggregate reaction. Thus, concrete exposed to sea water should be made with cement of controlled aluminate content and with nonreactive aggregate; embedded steel should be well covered by concrete of low permeability; and good construction practices should be followed.

Conclusion:

(1) Reduction in amount of free calcium hydroxide, by reaction with pozzolan, thus reducing the degree to which the reaction of sulfates and calcium hydroxide can occur.

(2) Increased solubility of hydrated calcium aluminates with decreased concentration of calcium hydroxide, and hence greater likelihood that the sulfate-aluminate reaction will take place through solution rather than in the solid state and thus produce less expansion.

(3) Decreased tendency of the low-sulfate calcium aluminum sulfate to convert to the high-sulfate form (ettringite) as the concentration of calcium hydroxide in solution decreases.

(4) Decreased permeability of the concrete with reduced rate of entry of' sulfate solution.

(5) Formation of lime-pozzolan reaction product films that protect the hydrated calcium aluminate.

(6) Expansive Cement can be used.

(7) Aluminous cement also stands well but sea water should not be used as mixing water.

(8) Use of asphalt impregnated piles to improve resistance to sea-water exposure.

Shill S. T. , (2014). "Chloride Penetration into Concrete Structures Exposed to the Marine Atmosphere" ***Engineering Structures 42 142–153.***

Reinforced concrete has been used in marine applications to construct piers, bridges, jetties, ships, floating platforms, as well as many other coastal and offshore structures. The marine environment is very aggressive towards the reinforcement steel used to strengthen these concrete structures, in terms of corrosion. Seawater contains a high amount of chloride ions, which will penetrate through concrete to attack the reinforcement steel and initiate corrosion. Once the corrosion process has begun, the effects can be detrimental to the entire structure. Due to the harsh conditions of the marine environment, protection and minimization of corrosion is of utmost importance to maintain the structural integrity of marine reinforced concrete structures.

Conclusion:

Concrete structures exposed to the marine atmosphere are prone to degradation and corrosion via marine aerosols containing chlorides. The chlorides are deposited on the concrete surface which may penetrate into the concrete and pose a threat to the structural integrity of the concrete if the chloride content reaches the reinforcing steel exceeds the chloride threshold and corrosion initiates. Lower water to cement ratio, 20% fly ash + 8% silica fumes Or slag had lower rate of accumulation of chloride.

Kavitha P. E. , Narayanan K. P. , Sudheer C. B. , (2008) "Software Development for the Analysis and Design of Ship Berthing Structures"

Software BESTDESIGN has been developed using the computer language Visual Basic and the Database MSAccess for the analysis and design of ship berthing structures. The software can be used for the analysis and design of new berthing structures and can also be used for obtaining the design aspects while reconstructing an existing structure. The software developed was tested with the requirements at Cochin Port. Cochin Port Trust is the authority of a number of ship berthing structures. Some of them are to be reconstructed and there are new projects involving the construction of new ship berths or the extension of existing berthing structures.

Muthukkumaran K (2013) , "Behaviour of Berthing Structure under Changing Slope in Seismic Condition." ***Civil Engineering Dimension, Vol. 11, No. 1.***

In the morning hours at 8:46 am on the day of 26th January 2001, a killer earthquake of the magnitude of 6.9 on the Richter scale lasting for about 90 seconds rocked Gujarat, resulting in the tremendous loss of human life and property. The Kandla port, a major port of Indian situated on West coast in Gujarat, suffered the maximum damages due to the devastating earthquake. The actual design slope is 1V: 3H, over a period of time due to siltation the design slope is changed to 1V:1.5H. In the present study, both design slopes are analysed and the extra lateral load coming to the pile due to unstable slope are determinated.

Conclusion:

(i) The earth force due to the failure of the unstable slope as the main reason for the damage to the berthing structure.

(ii) Earthquake forces have only initiated the failure of slope along critical slip surface.

(iii)The analysis result for structure with design conditions of slope 1V:3H, shows that the behind raker piles are taking the maximum forces as compared to the front vertical piles.

(iv)The slope underneath the structural changes to an unstable slope of 1V:1.5H, the front vertical piles are subjected to maximum lateral FORCES, WHICH LEAD TO THE FAILURE.

Seismic Analysis of RCC Building with and Without Shear Wall

-by P. P. Chandurkar, Dr. P. S. Pajgade

Shear wall are one of the excellent means of providing earthquake resistance to multistoried reinforced concrete building. The structure is still damaged due to some or the other reason during earthquakes. Behavior of structure during earthquake motion depends on distribution of weight, stiffness and strength in both horizontal and planes of building. To reduce the effect of earthquake reinforced concrete shear walls are used in the building. These can be used for improving seismic response of buildings. Structural design of buildings for seismic loading is primarily concerned with structural safety during major Earthquakes, in tall buildings, it is very important to ensure adequate lateral stiffness to resist lateral load. The provision of shear wall in building to achieve rigidity has been found effective and economical. When buildings are tall, beam, column sizes are quite heavy and steel required is large. So there is lot of congestion at these joint and it is difficult to place and vibrate concrete at these place and displacement is quite heavy. Shear wallsare usually used in tall building to avoid collapse of buildings. When shear wall are situated in advantageous positions in the building, they can form an efficient lateral force resisting system.

Conclusion:

It is observed that in 10 story building, constructing building with shear wall in short span at corner is economical as compared with other models. From this it can be concluded that large dimension of shear wall is not effective in 10 stories or below 10 stories buildings. It is observed that the shear wall is economical and effective in high rise building.

Also observed that

1. Changing the position of shear wall will affect the attraction of forces, so that wall must be in proper position.

2. If the dimensions of shear wall are large then major amount of horizontal forces are taken by shear wall.

3. Providing shear walls at adequate locations substantially reduces the displacements due to earthquake.

CHAPTER 3: SITE SELECTION

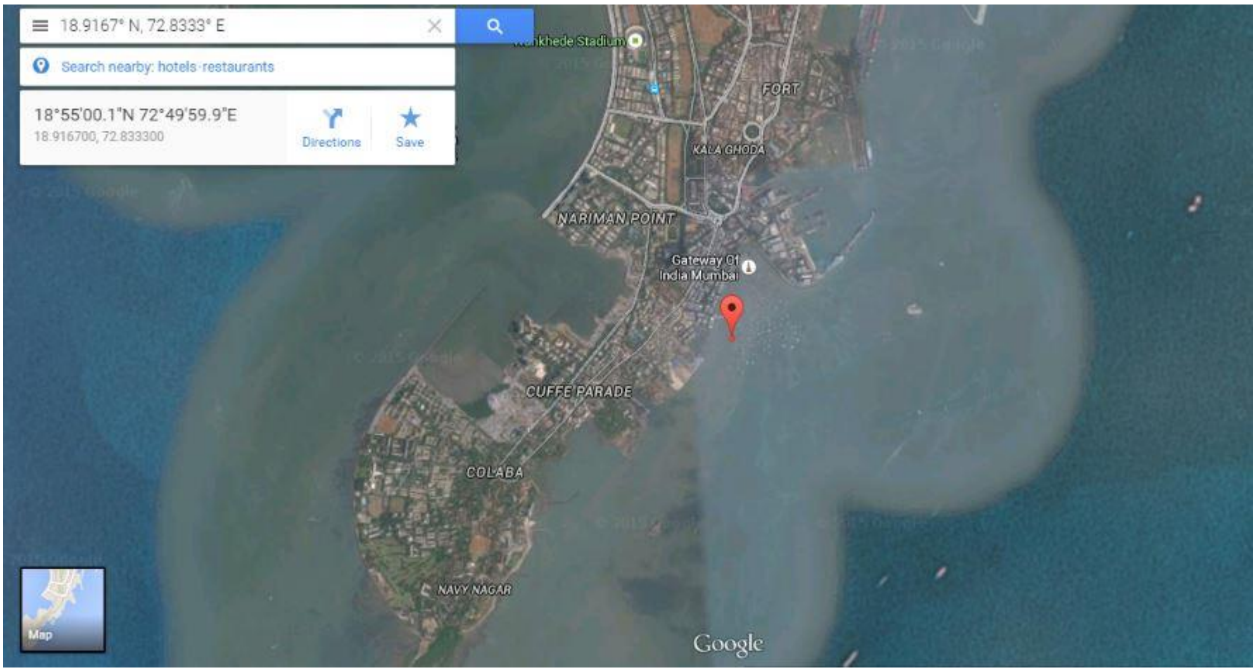


Fig 3.1: Site for gathering data

Mumbai (18.916 N, 72.8333E) near Apollo Bandar is selected for data collection.

- Maximum tide height = 5.0m (based on 4 year tidal data by Maharashtra Maritime Board Mumbai)
- Basic Wind Speed = 44 ms^{-1} (based on 50 year return period Fig. 1 IS 875 Part III)
- Terrain Category : Category 1 (Clause 5.3.2 IS 875 PartIII)
- Design Wind Pressure = $0.6(V_z)^2$ (Clause 5.4 IS 875 PartIII)

CHAPTER 4: STRUCTURAL PLAN

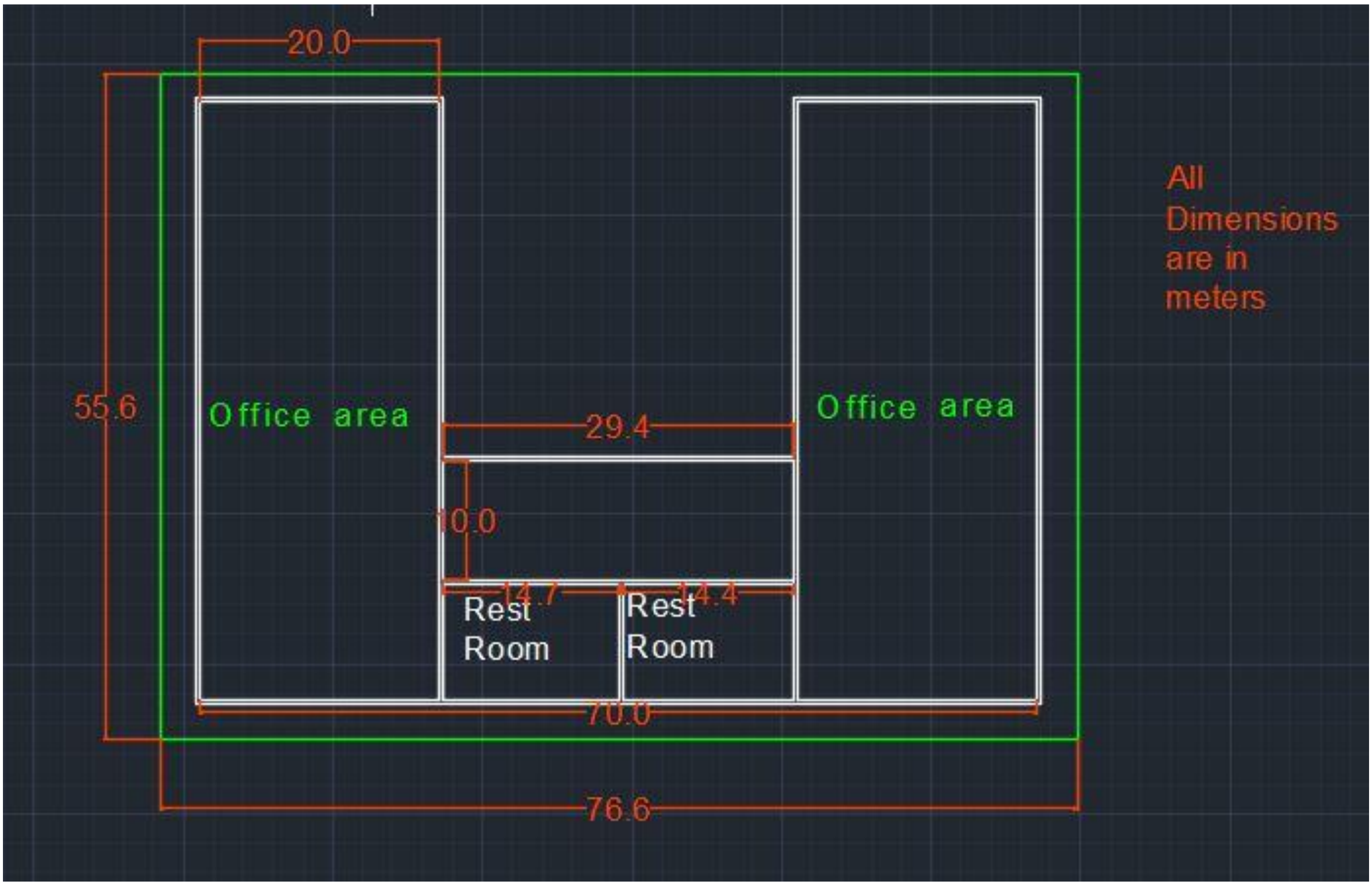


Fig 4.1: Structure Plan of Building.

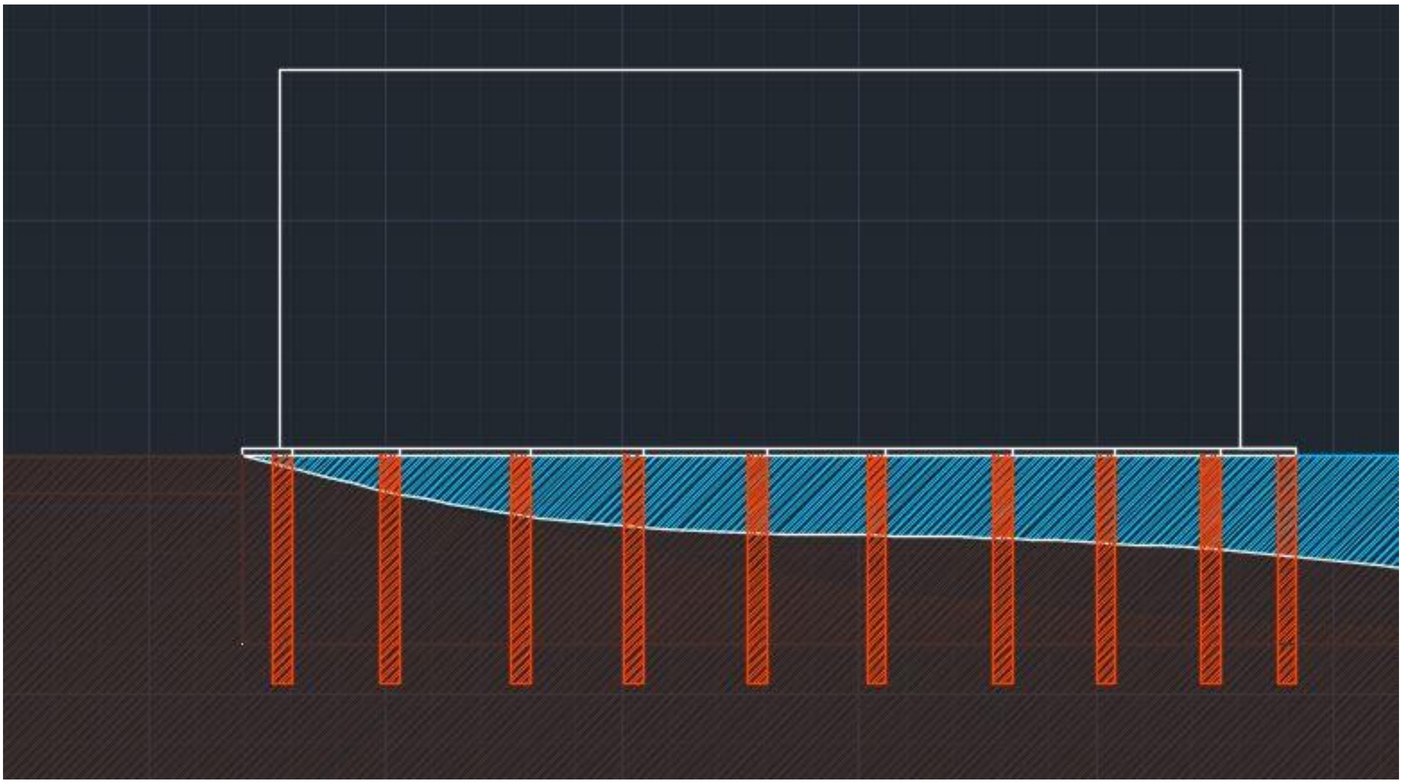


Fig 4.2: Elevation showing berth and piles. (Note to Scale)

CHAPTER 5: STAAD MODEL

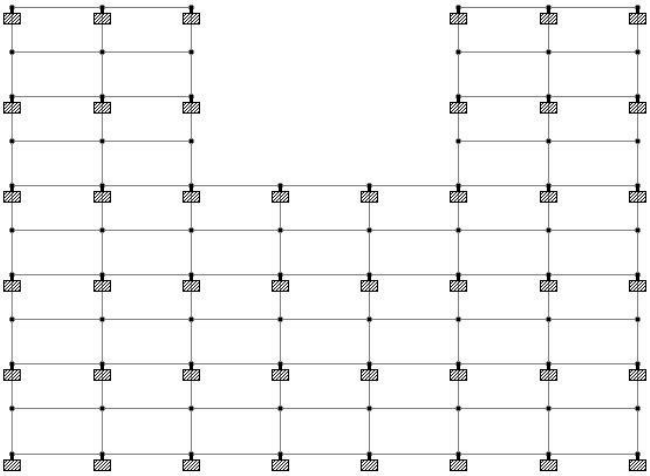


Fig 5.1: Plan of the Structure

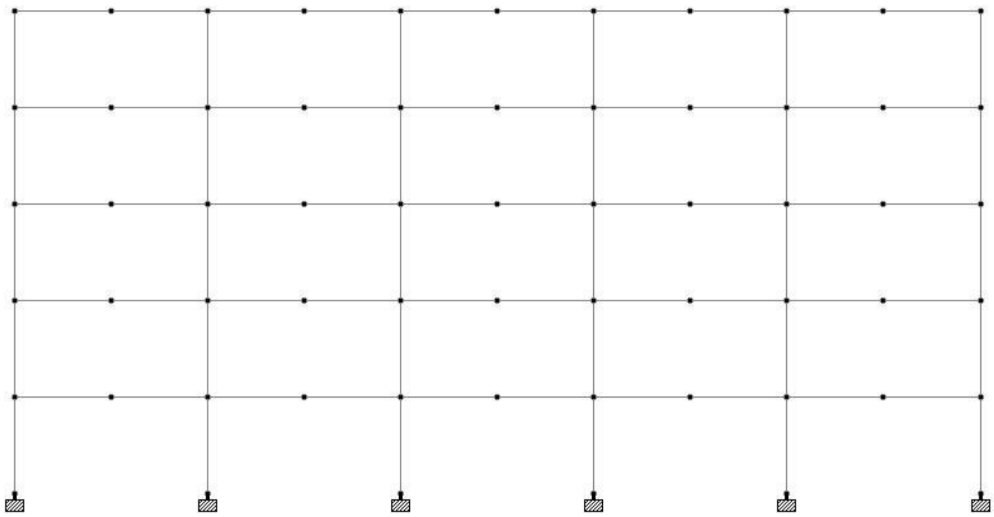


Fig 5.2: Side View of the Structure

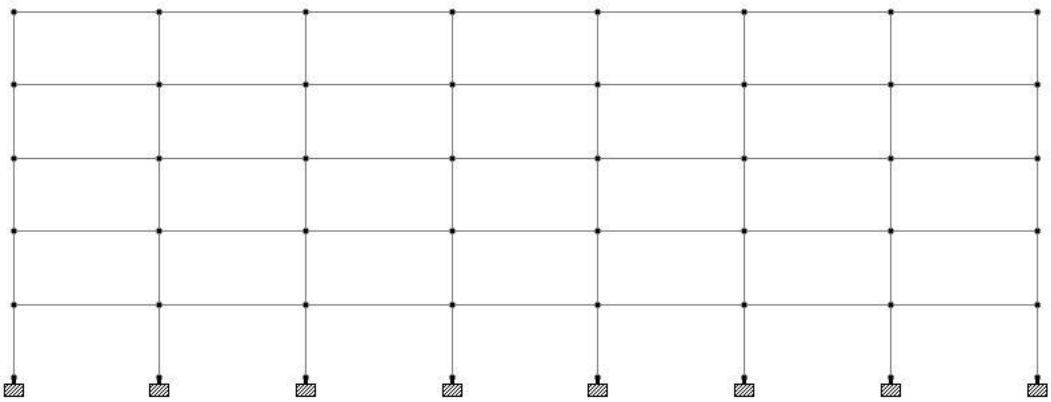


Fig 5.3: Elevation of the Structure

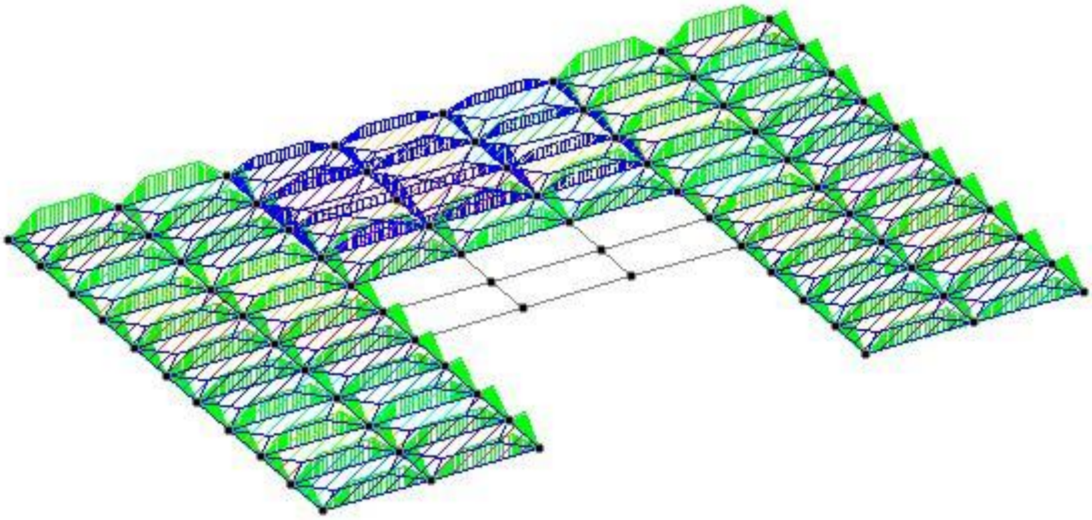


Fig 5.4: Loading on beams

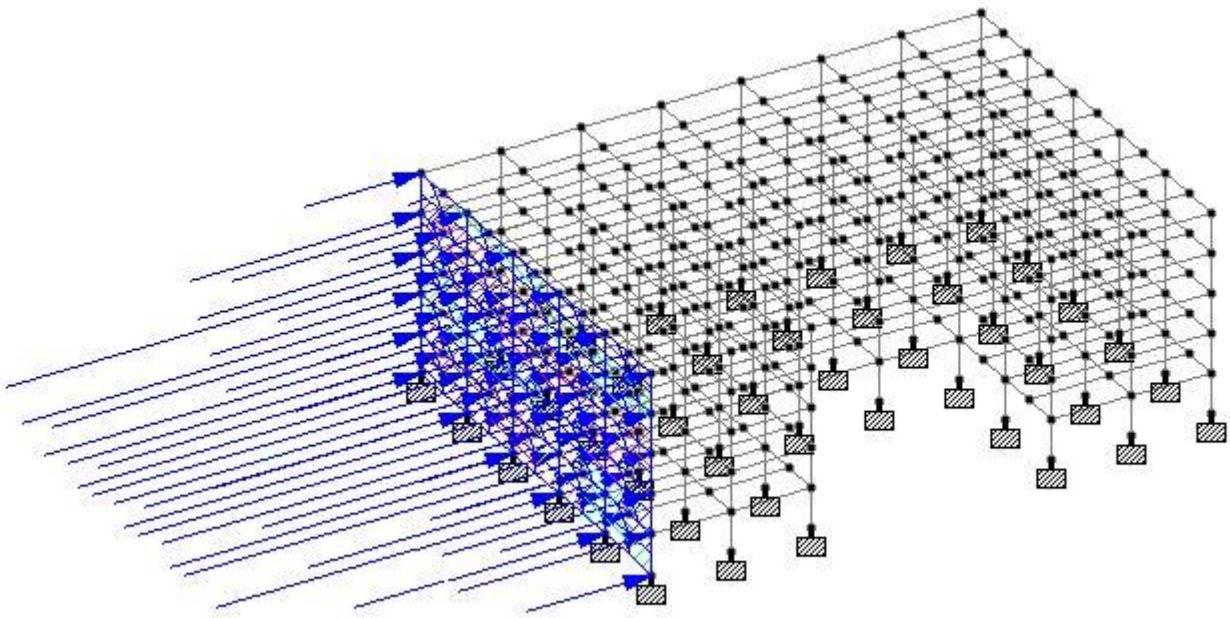


Fig 5.5: Wind Load on Structure (As According to IS875 Part 3)

CHAPTER 6: DESIGN

6.1 SLAB:

panel	Effective Span	boundary conditions
S1	10m * 5m	one short edge discontinuous
S2	10m * 5m	one long edge discontinuous
S3	10m * 5m	2 adjacent edge discontinuous
S4	10m * 5m	all 4 edges continuous

Table 1 : Boundary Conditions for slab

Fig.6.1.1: Floor slab system

fck	30 N/mm^2	fy	500 N/mm^2	
Lx	5000	mm		
Dx	128.2051	mm		
Depth	200	mm		
Clear Cover	20	mm		
Dia	8	mm	50.24	mm^2
Effective Depth	dx	176	mm	

	dy	168	mm
Effective Span	lx	5000	mm
	ly	10000	mm
ly/lx	2	S1 and S4	
ly/lx	2	S2 and S3	

Loading

Self Weight	5	kN/m^2
Live Load	4	kN/m^2
Total Factor load	13.5	kN/m^2

άx,άy	0.037	0.028	S1
	0.056	0.028	S2
	0.06	0.035	S3
	0.034	0.024	S4

ά- 1.33 ά+

Mu= $\alpha W l^2$

	panel	S1	S2	S3	S4
	load	13.5	13.5	13.5	13.5
	span lx	5000	5000	5000	5000
short span Moment	Mux+	12.4875	18.9	20.25	11.475
short span Moment	Mux-	16.60838	25.137	26.9325	15.26175

long span moment	Muy+	9.45	9.45	11.8125	8.1
long span moment	Muy-	12.5685	12.5685	15.71063	10.773

Flexure Reinforcement Requirement							
		b	1000	mm			spacing
panel S1	Dx	176	dy	168			Provided
short span	Mux+	12.4875	Ast	165.7918501	Spacing	303.0306	300
	Mux-	16.608375	Ast	221.6955163	Spacing	226.6171	227
long span	Muy+	9.45	Ast	131.0795484	Spacing	383.2787	300
	Muy-	12.5685	Ast	175.1107926	Spacing	286.9041	287
panel S2	Dx	176	dy	168			
short span	Mux+	18.9	Ast	253.0526161	Spacing	198.5358	199
	Mux-	25.137	Ast	339.4034773	Spacing	148.0244	148
long span	Muy+	9.45	Ast	131.0795484	Spacing	383.2787	300
	Muy-	12.5685	Ast	175.1107926	Spacing	286.9041	287
panel S3	Dx	176	dy	168			
short span	Mux+	20.25	Ast	271.6170274	spacing	184.9663	185
	Mux-	26.9325	Ast	364.5432562	spacing	137.8163	138
long span	Muy+	11.8125	Ast	164.4000369	spacing	305.596	300
	Muy-	15.71063	Ast	219.8824641	spacing	228.4857	228
panel S4	Dx	176	dy	168			
short span	Mux+	11.475	Ast	152.1495732	spacing	330.2014	300
	Mux-	15.26175	Ast	203.3595262	spacing	247.0501	247
long span	Muy+	8.1	Ast	112.1404241	spacing	448.0097	300
	Muy-	15.71063	Ast	219.8824641	spacing	228.4857	228

Check : Deflection Control

	Max Ast	364	mm2	

	Pt	0.20681818		
	Fs	285		
	k	1.55	from fig.3 IS 456:2000	
	l/d	26*1.55	40.3	
	(l/d)provided	5000/176	28.40909	DESIGN OK

Table2 : Flexure reinforcement for slab

Provide 8mm diameter @ 120c/c (for short span)

Provide 8mm diameter @ 250c/c (for longer span)

Torsion reinforcement

Torsion reinforcement shall be provided at any corner where the slab is simply supported on both edges meeting at that comer. It shall consist of top and bottom reinforcement, each with layers of bars placed parallel to the sides of the slab and extending from the edges a minimum distance of one-fifth of the shorter span. The area of reinforcement in each of these four layers shall be three-quarters of the area required for the maximum mid-span moment in the slab.

Calculation for corner torsion reinforcement:

Corner reinforcement has to be provided over a distance of $l_x/5$ = 1000mm in both directions in meshes at top and bottom (4 layers) each layer comprising $0.75A_{st}$. The bars are provided as U shaped (i.e. with the mesh extending top and bottom)

Spacing of 8mm diameter bars = 120/0.75 = 160 c/c

Hence provide 8mm diameter bars at top and bottom at each corner over an area of 1000mm * 1000mm, U shaped in 2 directions.

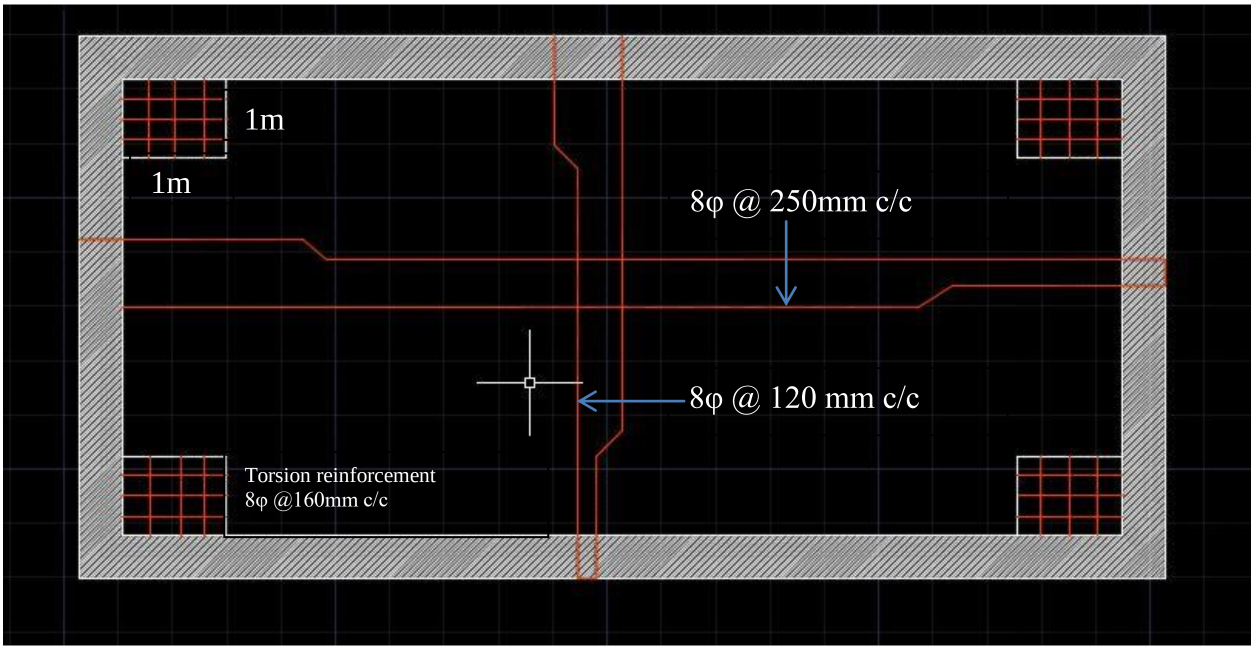


Fig.6.1.2: Detailing of the slab

6.2 COLUMNS

Column dimensions = 500mm * 500mm

Fe500, and M30 grade of concrete

Cover provided = 50mm

Staad Pro V8 Design for one of the column :

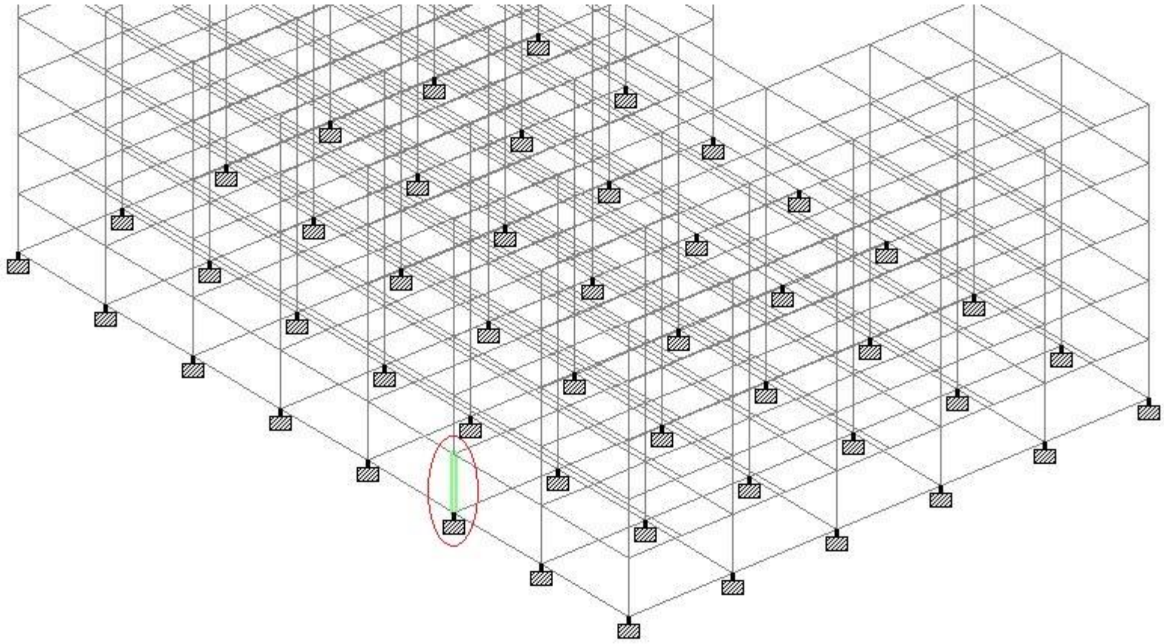


Fig 6.2.1 : location of column in staad pro

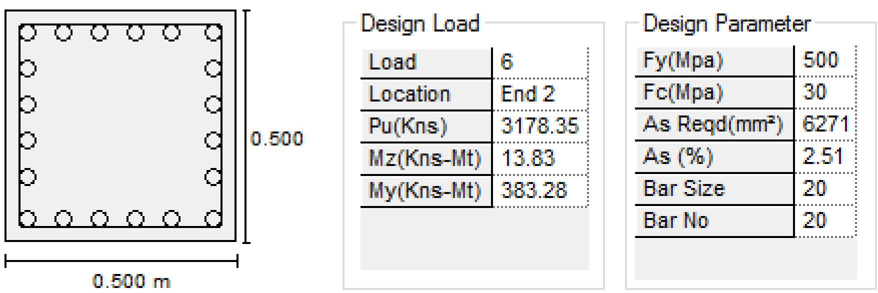


Fig 6.2.2: Staad pro V8 design result for the column

6.3 BEAM

Beam Dimensions = 500mm * 700mm

Fe500 M30 grade of concrete

Cover = 50mm

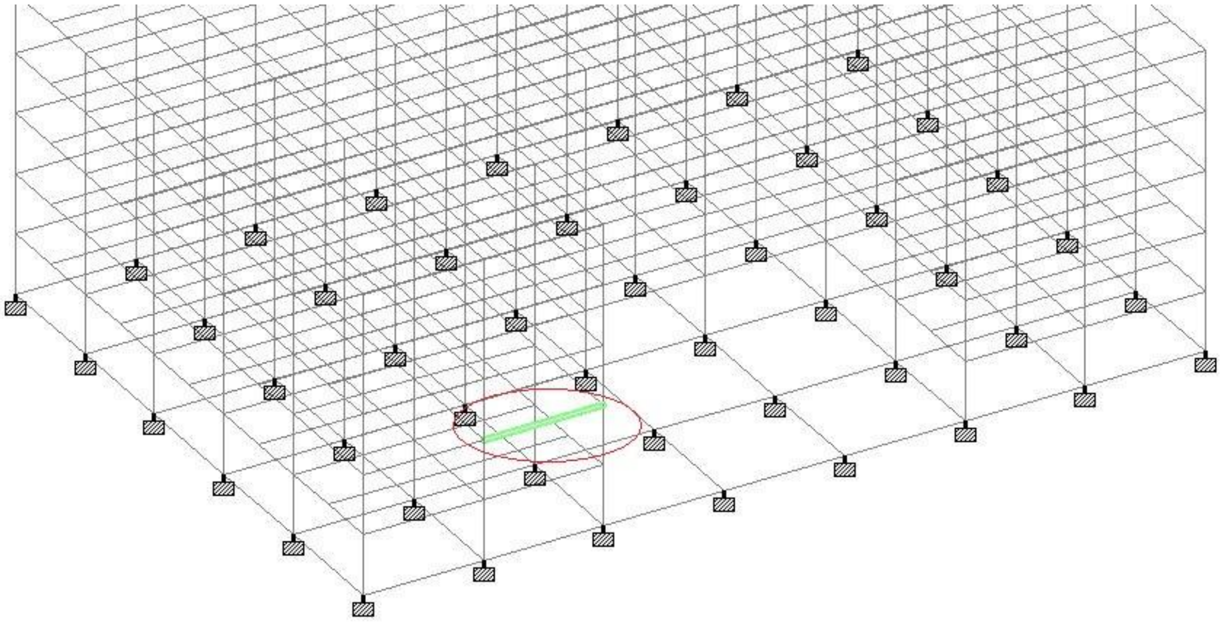
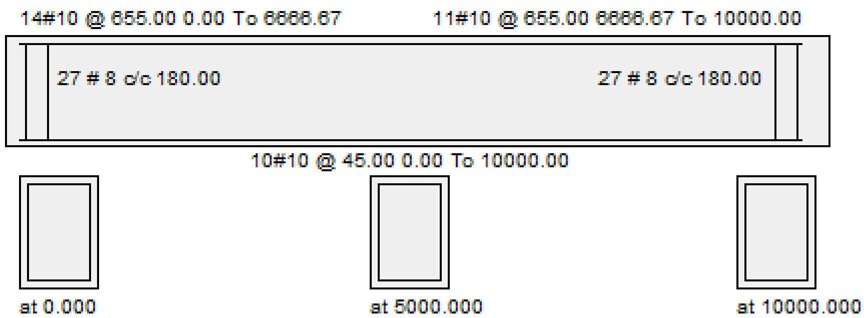


Fig 6.3.1 : location of beam in staad pro



Design Load

Mz Kn Met	Dist. Met	Load
167.96	5	6
-258.1	0	6
-191.81	10	5

Design Parameter

Fy(Mpa)	500
Fc(Mpa)	30
Depth(m)	0.699999988
Width(m)	0.5
Length(m)	10

Fig 6.3.2 : Design result for the beam shown in above figure

Loads from Super-Structure

(Reactions evaluated by staad pro v8 anaylsis)

Type	L/C	Name
Primary	1	LIVELOAD
Primary	2	DEADLOAD
Primary	3	WINDLOADX
Primary	4	WINDLOADZ
Combination	5	1.2(DL+LL+WL)
Combination	6	(DL+LL)*1.5
Combination	7	1.5 * (DL+WL)

Fig 6.3.3 : Load Combinations Used

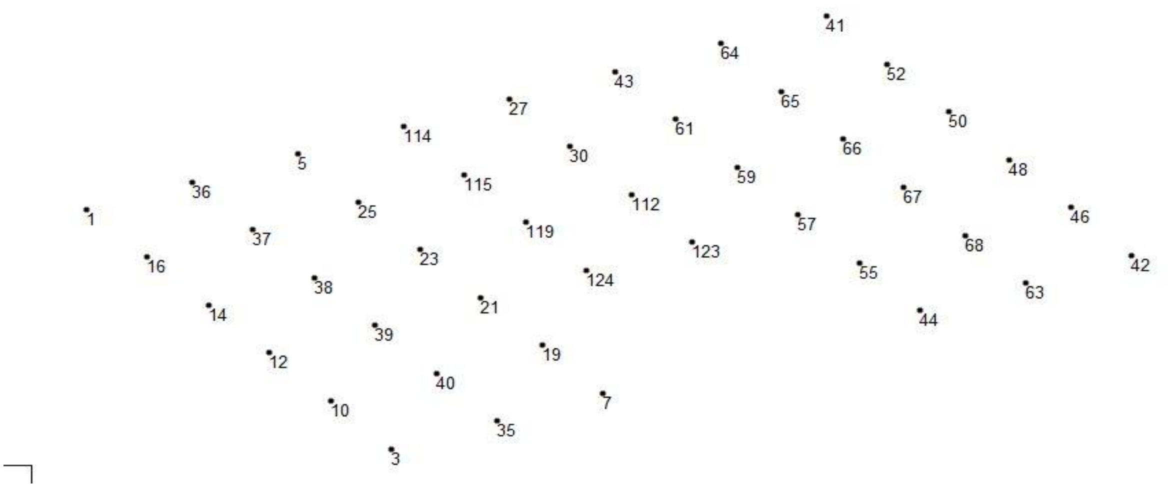


Fig 6.3.4 : Support reaction node layout

COLUMN	FORCES			Moments		
Node	Fx (kN)	Fy (kN)	Fz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
1	15	1196	1	-23	0	-23
3	7	1306	-75	-168	1	0
5	-10	2520	33	39	0	32
7	-51	1375	-82	-188	1	108
10	18	2603	-35	-101	0	-27
12	24	2548	-40	-109	0	-44
14	26	2555	-38	-106	0	-48
16	27	2645	-45	-117	0	-52
19	-75	2643	-41	-120	0	151
21	-45	3553	-66	-160	0	94
23	-21	4763	-52	-139	0	54
25	-24	4983	-64	-158	-1	57
27	-23	2597	37	40	-1	53

30	-19	5985	-66	-166	0	49
35	-23	2926	-122	-250	1	61
36	-18	2629	24	29	0	45
37	-22	5631	-52	-133	0	54
38	-24	5560	-39	-112	0	58
39	-22	5706	-43	-120	0	57
40	-26	5875	-36	-108	0	70
41	-45	1237	4	-15	-1	89
42	-43	1352	-73	-160	-1	86
43	-9	2272	13	9	-1	29
44	22	1316	-80	-183	0	-22
46	-66	2661	-31	-91	0	124
48	-71	2603	-36	-100	0	138
50	-71	2598	-34	-96	0	137
52	-70	2679	-41	-108	-1	133
55	42	2588	-39	-114	0	-55
57	7	3567	-66	-158	0	8
59	-10	4823	-47	-128	-1	36
61	-10	5028	-51	-133	-1	34
63	-14	2921	-120	-244	-1	37
64	-11	2706	29	40	-1	33
65	-12	5729	-49	-126	-1	39
66	-14	5616	-39	-109	0	43
67	-16	5690	-40	-112	0	47
68	-14	5875	-33	-101	0	38
112	-24	4591	-75	-181	0	59
114	-10	3329	147	222	0	32
115	-13	1385	-16	-67	1	40
119	-14	5078	-136	-282	0	43
123	-20	1731	-90	-205	-1	53
124	-16	1660	-81	-191	0	47

Table 3 : Support reactions of the structure

6.4 BERTH

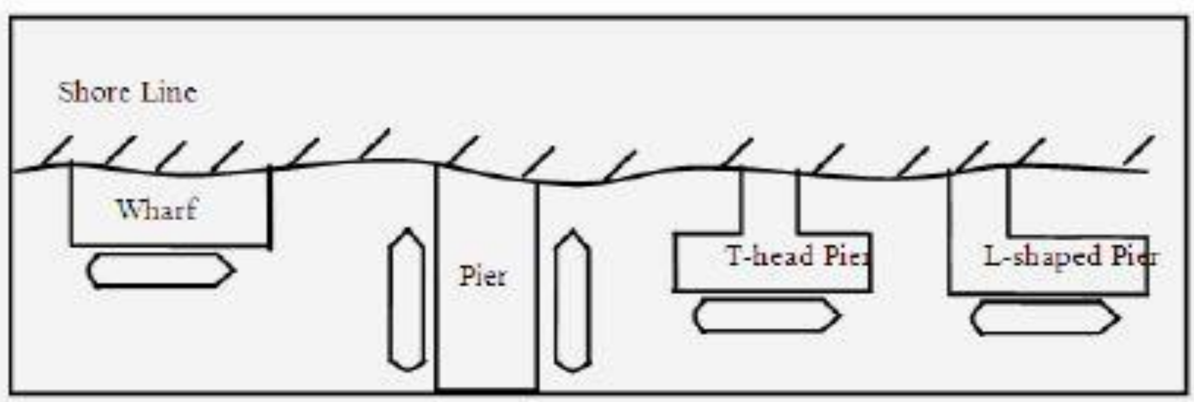
The berthing structures are constructed for the berthing and mooring of vessels to enable loading and unloading of cargo and for embarking and disembarking of passengers, vehicles etc.

Classification

Berthing structures can be classified as wharfs and piers.

1) Wharf - A wharf is a berthing structure parallel to the shore. It is generally contiguous with the shore, but may not necessarily be so.

2) Pier - A pier is a berthing structure which projects out into water. A pier does not necessarily need to run perpendicular to the shoreline but may project under any angle. It may also be connected with the shore by a trestle and may thus be T or L shaped.



Different Types of Berthing Structures

Design of berth slab

- The slabs are designed to carry a live load of $50kNm^{-2}$.
- They are designed as simply supported on all the four edges and spanning two directions.
- M30 concrete (design mix) and HYSD bars are used.

Panel	Boundary conditions
S1	one short edge discontinuous
s2	one long edge discontinuous
s3	2 adjacent edge discontinuous
s4	all 4 edges continuous

Lx	5000	mm
Dx	130	mm
Depth	300	mm
Clear Cover	50	mm
Dia	12	mm

Loading

Self Weight	7.5	kN/m^2
Live Load	50	kN/m^2
Total Factored Load	86.25	kN/m^2

	panel	S1	S2	S3	S4
	Load(kN/m^2)	86.25	86.25	86.25	86.25
	span lx	5000mm	5000mm	5000mm	5000mm
short span Moment	Mux+ (kNm)	79.78125	120.75	129.375	73.3125
short span Moment	Mux- (kNm)	106.1091	160.5975	172.0688	97.50563
long span moment	Muy+ (kNm)	60.375	60.375	75.46875	51.75
long span moment	Muy- (kNm)	80.29875	80.29875	100.3734	68.8275

Reinforcement calculations for berth slab :

panel S1	Moment	kNm		mm²		mm
short span	Mux+	80	Ast	795	Spacing	140
	Mux-	106	Ast	1080	Spacing	100
long span	Muy+	60	Ast	627	Spacing	180
	Muy-	80	Ast	848	Spacing	130
panel S2						
short span	Mux+	121	Ast	1244	Spacing	90
	Mux-	161	Ast	1715	Spacing	65
long span	Muy+	60	Ast	627	Spacing	180
	Muy-	80	Ast	848	Spacing	130
panel S3						
short span	Mux+	129	Ast	1343	spacing	80
	Mux-	172	Ast	1858	spacing	60
long span	Muy+	75	Ast	793	spacing	140
	Muy-	100	Ast	1079	spacing	100
panel S4						
short span	Mux+	73	Ast	727	spacing	155
	Mux-	98	Ast	985	spacing	110
long span	Muy+	52	Ast	533	spacing	210
	Muy-	69	Ast	720	spacing	155

Table 4: Flexure reinforcement Calculation for berth

Provide 12mm diameter @ 50c/c (for short span)

Provide 12mm diameter @ 100c/c (for longer span)

Torsion reinforcement

Torsion reinforcement shall be provided at any corner where the slab is simply supported on both edges meeting at that comer. It shall consist of top and bottom reinforcement, each with layers of bars placed parallel to the sides of the slab and extending from the edges a minimum distance of one-fifth of the shorter span. The area of reinforcement in each of these four layers shall be three-quarters of the area required for the maximum mid-span moment in the slab.

Calculation for corner torsion reinforcement:

Corner reinforcement has to be provided over a distance of $l_x/5$ = 1000mm in both directions in meshes at top and bottom (4 layers) each layer comprising $0.75A_{st}$. The bars are provided as U shaped (i.e. with the mesh extending top and bottom)

Spacing of 12mm diameter bars = 50/0.75 = 66.67 c/c. Provide approx. 60c/c.

Hence provide 12mm diameter bars at top and bottom at each corner over an area of 1000mm * 1000mm, U shaped in 2 directions.

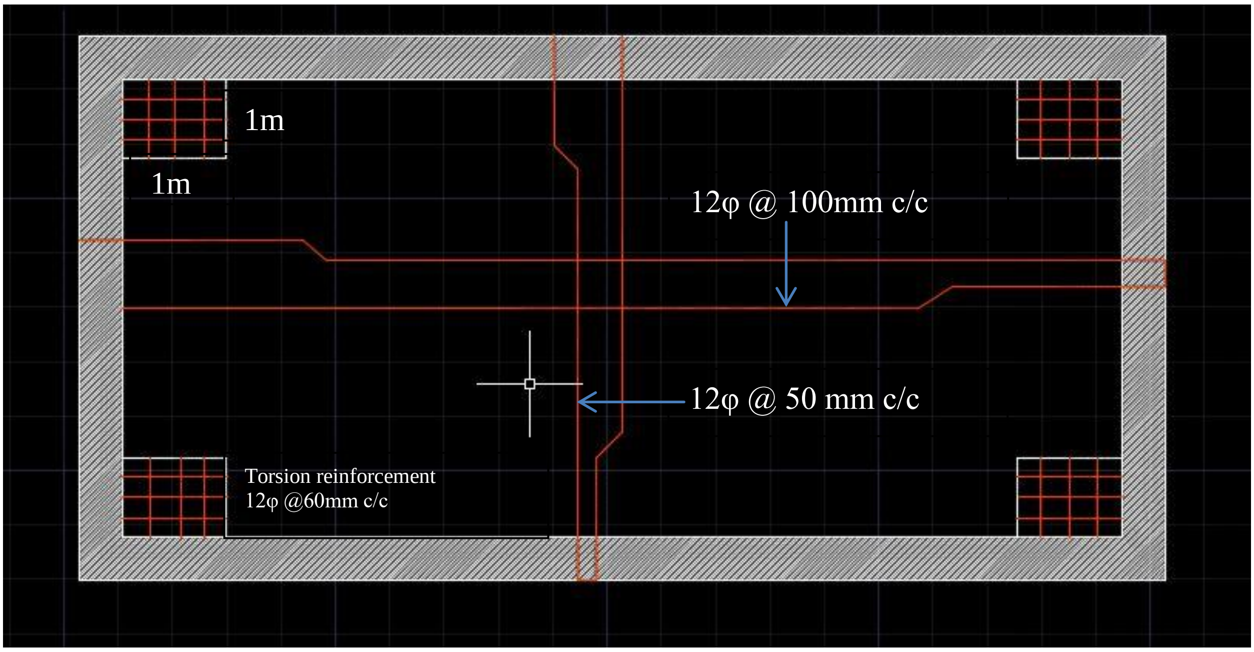


Fig.6.4.1: Detailing of the Berth slab

6.5 Berth Beams:

- Beam Cross-section = 1300mm*700mm
- Clear Cover = 50mm
- M30 Concrete, Fe500 Grade of Steel
- Span / depth ratio provided = 8.33 < 26

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	2500.0 mm	5000.0 mm	7500.0 mm	10000.0 mm
TOP REINF.	12541.43 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	12541.43 (Sq. mm)
BOTTOM REINF.	117.98 (Sq. mm)	1813.67 (Sq. mm)	6641.94 (Sq. mm)	1813.67 (Sq. mm)	117.98 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION	0.0 mm	2500.0 mm	5000.0 mm	7500.0 mm	10000.0 mm
TOP REINF.	10-40í 2 layer(s)	2-40í 1 layer(s)	2-40í 1 layer(s)	2-40í 1 layer(s)	10-40í 2 layer(s)
BOTTOM REINF.	5-25í 1 layer(s)	5-25í 1 layer(s)	14-25í 2 layer(s)	5-25í 1 layer(s)	5-25í 1 layer(s)
SHEAR REINF.	2 legged 12í @ 290 mm c/c	2 legged 12í @ 80 mm c/c	2 legged 12í @ 290 mm c/c	2 legged 12í @ 80 mm c/c	2 legged 12í @ 290 mm c/c

Mooring Load:

The maximum mooring loads are due to the wind forces on exposed area on the broad side of the ship in light conditions.

F = Cw.Aw.P

= 1.475*1.5*1640

= 3628.5kN

Where, F = force due to wind in KN.,

Cw = shape factor,

P = windage pressure in kg/m2

Aw= Windage area for the design ship.

Fenders for the berth:

- To prevent damage to both vessel and berth
- Absorb kinetic energy from berthing impact.
- Design life of fenders is only 10 years.

Type of fender system:

- **Rubber fender Units** made from vulcanized rubber (often with encapsulated steel plates).
- **Pneumatic fender Units** comprising fabric reinforced rubber bags filled with air under pressure and that absorbs energy
- **Foam fender Units** comprising a closed cell foam inner core with reinforced polymer outer skin that absorbs energy by virtue of the work done in compressing the foam.
- **Steel Panel,** structural steel frame designed to distribute the forces generated during rubber fender compression.

BERTHING LOAD:

The Kinetic Energy E, imparted to a fendering system, by a vessel moving with velocity V m/s is given by

$(WD \times V^2 \times Cm \times Ce \times Cs) / 2g$

Where WD = 100 TONS = displacement tonnage (DT) of the design vessel in tonnes,

v = 0.1 m/s = velocity of vessel in m/s, normal to berthing,

g = 9.81 = acceleration due to gravity in m/s^2,

Cm = 1.6 = mass coefficient,

Ce = .51 = eccentricity coefficient,

Cs = .95 = softness coefficient

Current Load:

The force per square metre of area produced by sea water impinging on the side of a ship may be computed from the formula:

$F = w\, v^2/2g$

$= 9.81 * 2.54 * 2.54 / (9.81 * 2)$

$= 3.3$ kN/m2

F= force in kN/m2,

v= velocity of current in m/s = 2.54 ,

w = unit weight of sea water and

g = 9.81m/s2.

6.6 PILES

A deep **foundation** is a type of **foundation** which transfers building loads to the earth farther down from the surface than a shallow **foundation** does, to a subsurface layer or a range of depths. A **pile** is a vertical structural element of a deep **foundation**, driven deep into the ground at the building site.

A pile is basically a long cylinder of a strong material such as concrete/steel/wood that is pushed into the ground to act as a steady support for structures built on top of it.

Pile foundations are used in the following situations:

1. When there is a layer of weak soil at the surface. This layer cannot support the weight of the building, so the loads of the building have to bypass this layer and be transferred to the layer of stronger soil or rock that is below the weak layer.
2. When a building has very heavy, concentrated loads, such as in a high rise structure, bridge, or water tank.
3. Pile foundations are capable of taking higher loads than spread footings.

There are two types of pile foundations, each of which works in its own way.

End Bearing Piles

In end bearing piles, the bottom end of the pile rests on a layer of especially strong soil or rock. The load of the building is transferred through the pile onto the strong layer. In a sense, this pile acts like a column. The key principle is that the bottom end rests on the surface which is the intersection of a weak and strong layer. The load therefore bypasses the weak layer and is safely transferred to the strong layer.

Friction Piles

Friction piles work on a different principle. The pile transfers the load of the building to the soil across the full height of the pile, by friction. In other words, the entire surface of the pile, which is cylindrical in shape, works to transfer the forces to the soil.

To visualize how this works, imagine you are pushing a solid metal rod of say 4mm diameter into a tub of frozen ice cream. Once you have pushed it in, it is strong enough to support some load. The greater the embedment depth in the ice cream, the more load it can support. This is very similar to how a friction pile works. In a friction pile, the amount of load a pile can support is directly proportionate to its length.

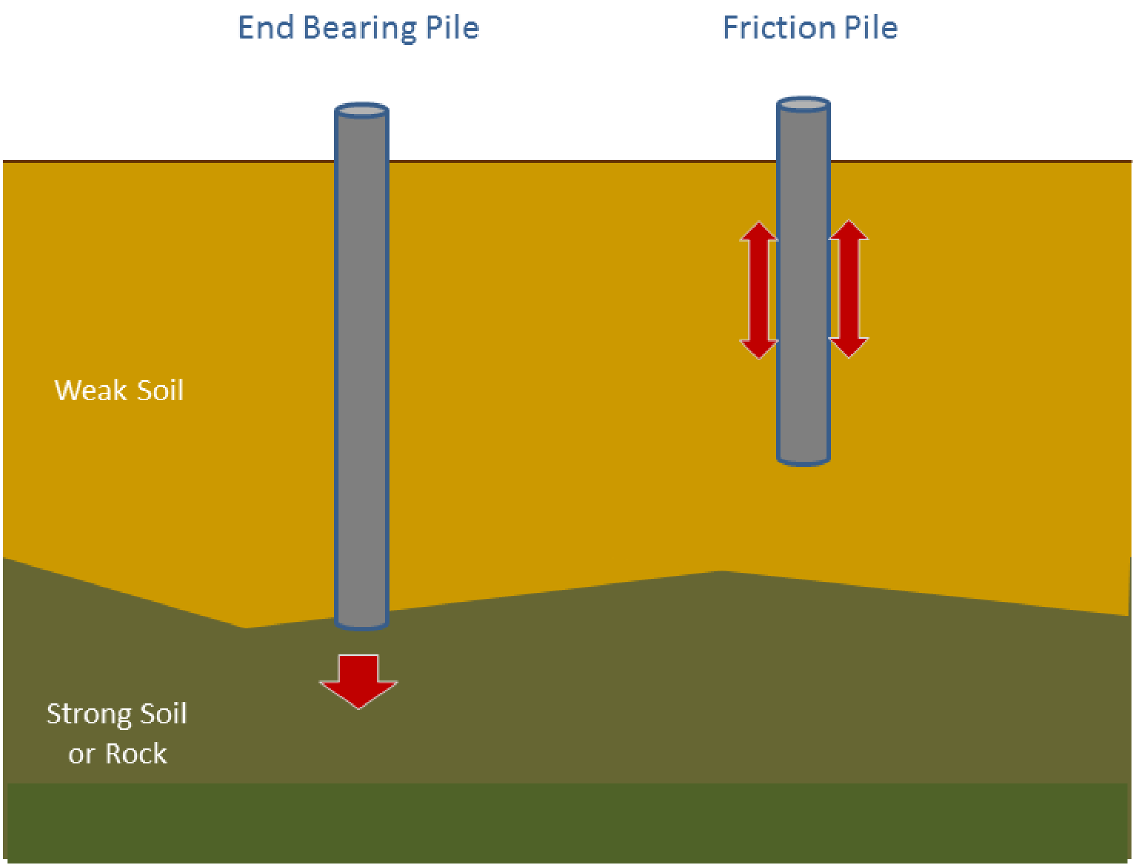


Fig. 6.6.1 : Two types of piles

WHAT ARE PILES MADE OF?

Piles can be made of wood, concrete, or steel.

In traditional construction, wooden piles were used to support buildings in areas with weak soil. Wood piles are still used to make jetties. For this one needs trees with exceptionally straight trunks. The pile length is limited to the length of a single tree, about 20m, since one cannot join together two tree trunks. The entire city of Venice in Italy is famous for being built on wooden piles over the sea water.

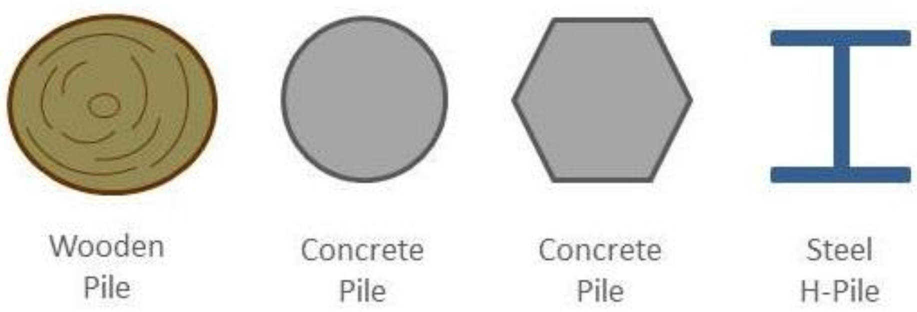


Fig. 6.6.2 Cross sections of various pile foundations

Concrete piles are precast, that is, made at ground level, and then driven into the ground by hammering - more on that later. Steel H-piles can also be driven into the ground. These can take very heavy loads, and save time during construction, as the pile casting process is eliminated. No protective coating is given to the steel, as during driving, this would be scraped away by the soil. In areas with corrosive soil, concrete piles should be used.

Steel Piles:

Steel/ Iron piles are suitable for handling and driving in long lengths. Their relatively small cross-sectional area combined with their high strength makes penetration easier in firm soil. They can be easily cut off or joined by welding. If the pile is driven into a soil with low pH value, then there is a risk of corrosion, but risk of corrosion is not as great as one might think. Although tar coating or cathodic protection can be employed in permanent works. It is common to allow for an amount of corrosion in design by simply over dimensioning the cross-sectional area of the steel pile. In this way the corrosion process can be prolonged up to 50 years. Normally the speed of corrosion is 0.2-0.5 mm/year and, in design, this value can be taken as 1mm/year.

Advantages of Steel Piles:

1. The piles are easy to handle and can easily be cut to desire length.
2. Can be driven through dense layers. The lateral displacement of the soil during driving is low (steel section H or I section piles) can be relatively easily spliced or bolted.
3. Can be driven hard and in very long lengths.
4. Can carry heavy loads.
5. Can be successfully anchored in sloping rock.

HOW PILES ARE USED

As pile foundations carry a lot of load, they must be designed very carefully. A good engineer will study the soil the piles are placed in to ensure that the soil is not overloaded beyond its bearing capacity.

Every pile has a zone of influence on the soil around it. Care must be taken to space the piles far enough apart so that loads are distributed evenly over the entire bulb of soil that carries them, and not concentrated into a few areas.

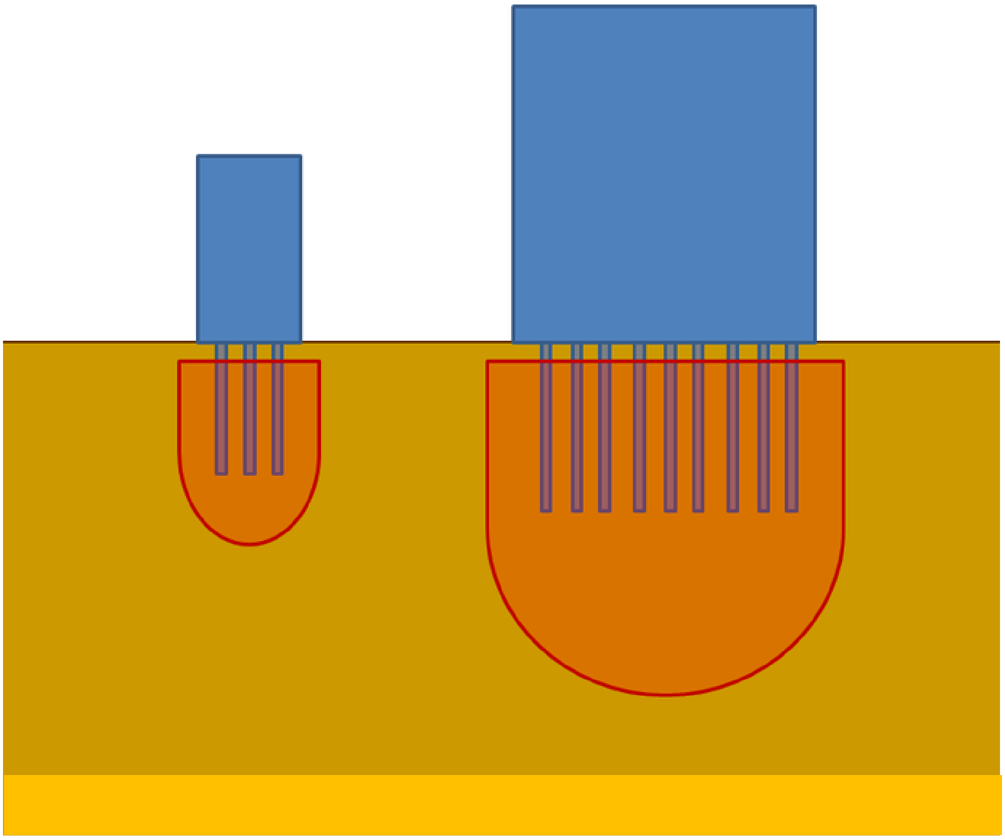


Fig. 6.6.3: The load pattern of the piles on the soil surrounding them. This is also called a zone of influence.

Engineers will usually group a few piles together, and top them with a pile cap. A pile cap is a very thick cap of concrete that extends over a small group of piles, and serves as a base on which a column can be constructed. The load of this column is then distributed to all the piles in the group.

HOW PILES ARE CONSTRUCTED

Fig. 6.6.4: Pile driver

Piles are first cast at ground level and then hammered or driven into the ground using a pile driver. This is a machine that holds the pile perfectly vertical, and then hammers it into the ground blow by blow. Each blow is struck by lifting a heavy weight and dropping it on the top of the pile - the pile is temporarily covered with a steel cap to prevent it from disintegrating. The pile driver thus performs two functions - first, it acts as a crane, and lifts the pile from a horizontal position on the ground and rotates it into the correct vertical position, and second, it hammers the pile down into the ground.

Piles should be hammered into the ground till refusal, at which point they cannot be driven any further into the soil.

ANALYSIS AND DESIGN OF PILE

Assumed site conditions:

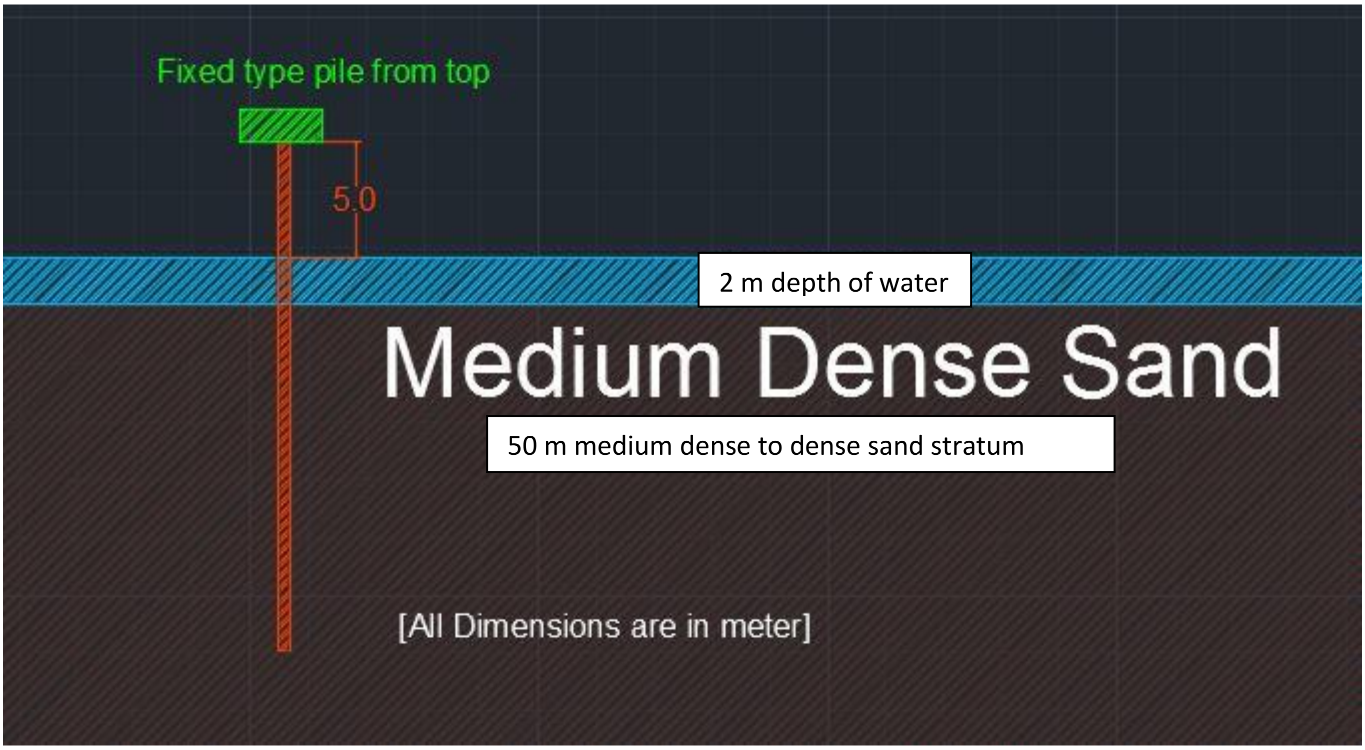


Fig. 6.6.5.: Soil Conditions

Axial Capacity of driven steel pipe piles:

Driven steel pipe piles can be considered as partial displacement piles. The axial resistance of the soil along the outer and inner surface of the steel-soil interface contributes to the resistance. In addition, the bearing resistance contributed by the end area shall also be taken into account

The estimation of overall axial capacity involves the following:

1. Estimation of skin friction resistance between steel pipe and soil (sand or clay)
2. Estimation of bearing resistance (sand or clay)
3. Determination of plug formation (plugged or unplugged)
4. Estimation the axial capacity using the pile dimensions and penetration.
5. Application Factor of safety

1. Estimation of skin friction

Beta method – for cohesion-less soils

The skin friction between steel pipe piles and sandy soil can be estimated using the following relationship as per API RP 2A earlier versions

$F = K\ p_o \tan\delta$

However due to uncertainty in the estimation of soil pile friction angle (δ) API RP 2A has recommended the use of β factor as tabulated in 6.4.3.1. The skin friction between pipe piles and sandy soil can be estimated using the following relationship as per API RP 2A latest version.

$F = \beta\ p_o$

Where

β = dimensionless shaft friction factor

p_o = effective overburden pressure at the depth

K = coefficient of lateral earth pressure

Δ = friction angle between the soil and pile wall

Description	φ	δ	K	K tan (δ)	β
	deg	deg	–	–	–
Very Loose to Loose	20	15	0.8	0.21	-
Medium Dense	25	20	0.8	0.29	0.29
Medium Dense to Dense	30	25	0.8	0.37	0.37
Dense to Very Dense	35	30	0.8	0.46	0.46
Very Dense	40	35	0.8	0.56	0.56

Note : API RP 2A does not recommend friction factor for Very Loose to Loose sand

Fig. 6.6.6: Comparison of Beta values and K tan (d)

2. **Estimation of unit end bearing**

Cohesion-less Soils:

The end bearing in sandy soils for steel pipe piles can be estimated using the following relationship

$Q = N_q\ p_o$

Where

N_q = dimensionless bearing capacity factor

p_o = effective overburden pressure at the depth

3. **Factor of Safety**

Factor of safety against the computed ultimate axial capacity depends on the type of pile and soil conditions. Similar to IS 2911 for bored concrete piles require a FOS of minimum 2.5 where as API RP 2A suggests otherwise. API RP 2A suggests a minimum FOS for each loading condition for offshore platforms.

Load Conditons	Tension	Compression
Operating Conditions	2.0	2.0
Storm Conditions	1.5	1.5
Seismic Conditions	1.2	1.2

Table 5 : FOS according to API RP 2A

4. **Weight of pile**

The weight of pile below the seabed shall be treated as the part of the load. This shall either be added to the the load or deducted from the computed compression capacity.

However, this can be taken as advantage for the tension capacity by adding to the capacity as it acts agains the pull out load.

5. **Weight of soil plug**

Weight of soil plug shall be added to the load or deducted from the compression capacity if the pile is plugged.

For the tension capacity, the plug weight can be added if the pile plugged.

ALLOWABLE PILE CAPACITY

Allowable pile capacity is required as the design method using the working stress design and to allow for the uncertainties' in the estimation of the axial capacity, a suitable FOS is applied on the ultimate capacity.

$$Q_{allow,compression} = \text{Minimum}\left[\frac{Q_{cp} - W_{plug} - W_{pile}}{FOS}, \frac{Q_{cp} - W_{plug}}{FOS}\right]$$

$$Q_{allow,tension} = \text{Minimum}\left[\frac{Q_{tp} + W_{plug} + W_{pile}}{FOS}, \frac{Q_{cp} + W_{plug}}{FOS}\right]$$

Lateral Capacity of piles

The offshore structures are subjected to large lateral loads from wave, current and wind in the addition to the gravity loads. Hence the lateral loads needs to be transmitted to the foundation soil through the pile foundations.

The estimate of the soil lateral load carrying capacity needs to be obtained and related to the soil response in terms of displacement.

This is required to assess the lateral displacement of piles and supported structure on it since these lateral displacements may hinder the operational aspects of the platform.

In order to satisfy the operational aspects of the platform, the lateral displacement of the structure shall be limited to certain serviceability limit. Hence the establishment of the load displacement relationship is essential to the whole process.

For fixed head long pile

Limiting solution for linearly varying modulus of subgrade

Lateral Deflection, $y_0 = \frac{F\beta}{k_h D}$

Where

y_0 = deflection in mm

F = Lateral force

D = diameter of pile

β = Constant = $[\frac{k_h D}{4 E_p I_p}]^{\frac{1}{4}}$

$k_h = \frac{0.65}{D}[\frac{E_s D^4}{E_p I_p}]^{\frac{1}{12}} \frac{E_s}{(1-u^2)}$ { Linear Spring Stiffness Method (Vesic Method – 1961)}

k_h = Modulus of subgrade reaction

I_p = Moment of Inertia of pile cross section area

E_p = Elastic modulus of pile

E_s = Elastic modulus of soil

u = poisson ratio of soil

CALCULATIONS:

Pile Specification

Inner dia of pile	508	mm	FOS	1.2	
Outer dia of pile	522.27	mm	Area of Section	0.214	m^2
Moment of Inertia	382895376	mm^4	Circumference	1.640	m

Soil Conditions

Medium Dense Soil

Angle of friction	25	degrees	Unit Weight (Soil)	20	kN/m^3
Coeff. Of earth pressure	0.39		Unit Weight (Water)	9.81	kN/m^3
Soil pile friction angle	16.67	degrees	Submerged Unit Wt.	10.19	kN/m^3
Friction Coefficient	0.37		N_q	20	
Weight of pile	4.05	kN			
Weight of soil plug	371.57	kN	h	50	m

Penetration Length(m)	F_s (kN/m^2)	E_b(kN/m^2)	Q (kN)	**Allowable Q(kN)**	**Tensile Capacity(kN)**
45	169.66	9171	14484	**7054.35**	**6448.12**

Lateral Capacity

Ep	200000	N/mm^2	Poisson Ratio	0.4
Es	50000	N/mm^2		

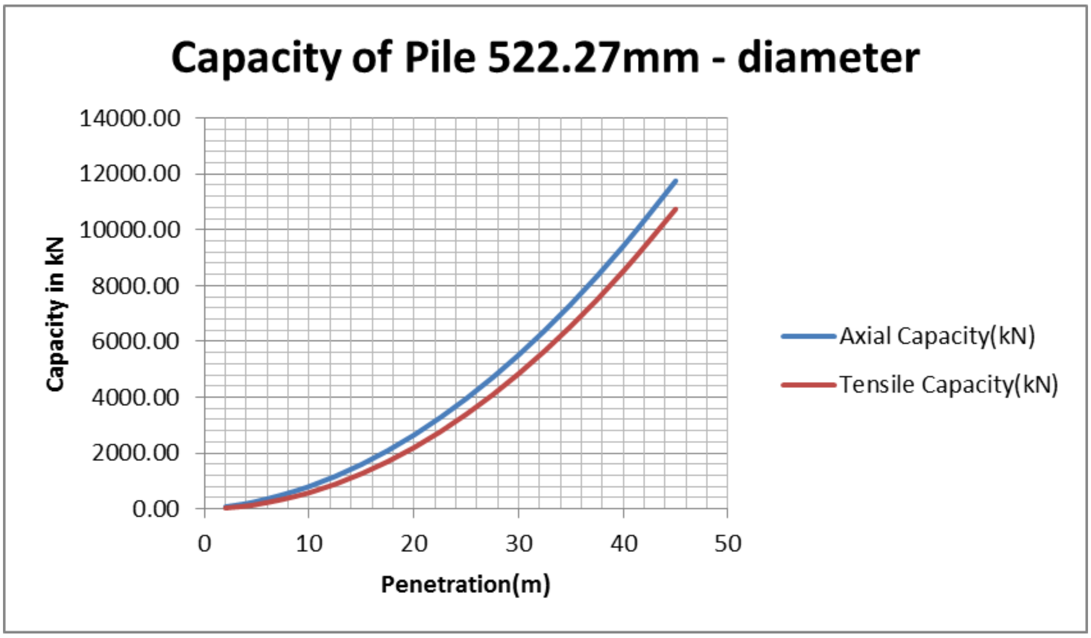
Modulus Of Subgrade	375592.77	N/m^3	Deflection Allowed	50mm
β	3.5			

Lateral Capacity 2670 kN

[which is quite sufficient to handle all the lateral forces due to waves and moment.]

Penetration Length(m)	Axial Capacity(kN)	Tensile Capacity(kN)
45	11757.25	10746.87
40	9420.33	8522.32
35	7341.04	6555.39
30	5519.38	4846.1
25	3955.34	3394.42
20	2648.93	2200.38
15	1600.15	1263.96
10	808.99	585.16
5	275.45	164
2	78.99	34.96

Table 6 : Pile capacity Vs Penetration



Typical Pile Capacity Curve

Pile depth penetration for column loads (Table 7)

Column Load (kN)	Penetration Required (m)
1196	15
1306	15
2520	20
1375	15

2603	20
2548	20
2555	20
2645	20
2643	20
3553	25
4763	30
4983	30
2597	20
5985	35
2926	25
2629	20
5631	35
5560	35
5706	35
5875	35
1237	15
1352	15
2272	20
1316	15
2661	25
2603	20
2598	20
2679	25
2588	20
3567	25
4823	30
5028	30
2921	25
2706	25
5729	35
5616	35
5690	35
5875	35
4591	30

3329	25
1385	15
5078	30
1731	20
1660	20

PILE COLUMN CONNECTTION:

Column reinforcement is directly going to be inserted into the hollow steel pile as diameter of pile and column dimensions are same of 500mm. Cover bars can be used to for properly placing reinforcement in the pile.

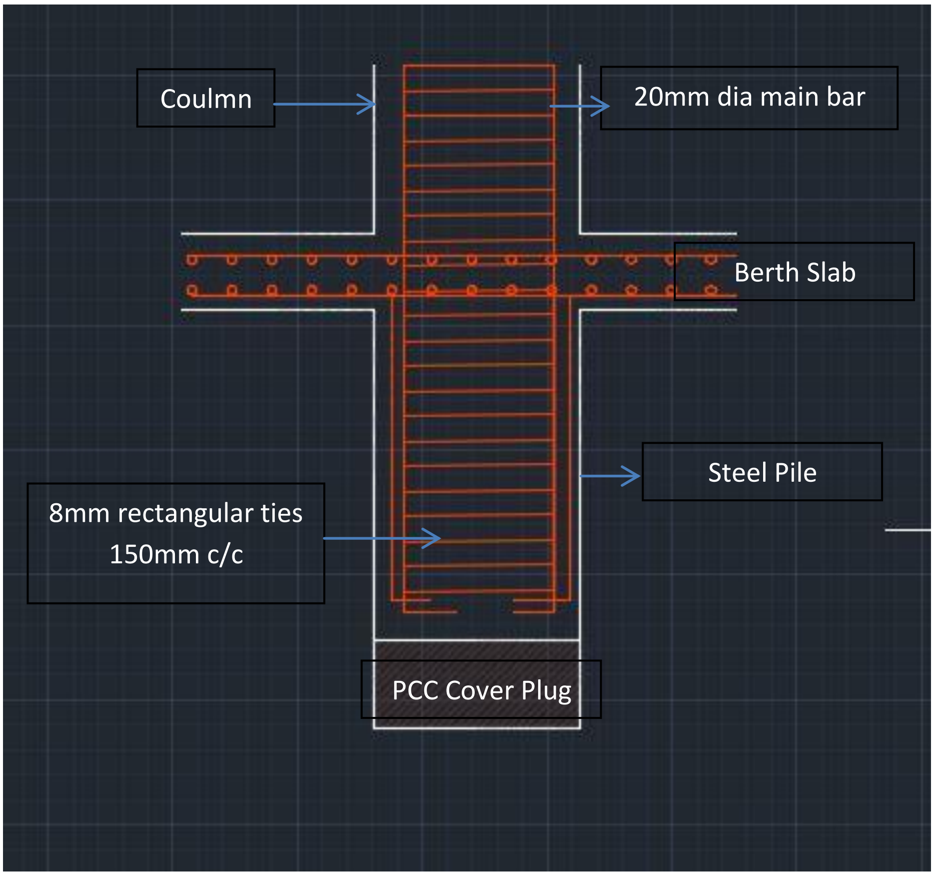


Fig.6.6.7: Cross-sectional view of pile column connection

CHAPTER 7: RESULTS

Design results for the structure:

Beams:

All beams used are of same dimensions to avoid complexities during site execution.

Dimensions of the beam = 500mm * 700mm

Fe500 M30 Grade of Concrete, Clear Cover = 50mm

Columns:

Dimensions of the column = 500mm * 500mm

Fe500 M30 Grade of Concrete, Clear Cover = 50mm

Beams to Support Berth and Structure:

Dimensions = 1300mm * 700mm

Fe500 M30 Grade of Concrete, Clear Cover = 50mm

Berth Slab:

Depth of slab = 300mm

Clear cover = 50mm

Structural Slab:

Depth of Slab = 200mm

Clear Cover = 30mm

Pile:

Steel Piles @ 522.27mm of outer diameter and thickness of 7.35mm

Pile Weight = 90 N/m

Maximum Pile Penetration = 35m

Minimum Pile Penetration = 15m

CHAPTER 8: CONCLUSION

Some of the piles are penetrating up-to the depth of 35m as the assumed sand stratum is composed of medium dense sand which is not sufficient to provide required axial capacity for lower depths.

Lateral Capacity is calculated by assuming every pile as a long pile and behaving like a beam supported on the springs (soil).

Steel piles are used as these piles are easy to handle and can easily be cut to desire length.

They can be driven through dense layers. The lateral displacement of the soil during driving is low (steel section H or I section piles) can be relatively easily spliced or bolted. They can be driven hard and in very long lengths can carry heavy loads and can be successfully anchored in sloping rock.

Beams supporting berth are of larger dimensions 700mm*1300mm because the large live load of $50kN/m^2$ is used as prescribed by the IS 4561 : Design code for ports and harbours. Due to of which large moments are developed on the beams supporting berth and structure.

All columns are of same size 500mm * 500mm to avoid complexity during site execution. But area of steel has been revised for every floor level to economise the design.

Safety factors are used while calculating the reactions for design of piles. And an additional factor of safety of 1.2 is also used while analyzing pile load carrying capacity, which is a seismic factor as suggested by API RP 2A : 2000. Due to of which structural design is completely safe and can also bear seismic activity as foundation is made of steel providing necessary ductility for the structure.

CHAPTER 8: FUTURE SCOPE

Design of beams, columns can be further economized by giving individual designs for each story of the building, keeping in mind about the site execution and formwork.

As the piles are driven deep inside the sand stratum upto the depth of 35m. A pile drivability analysis can be performed to check the feasilbility of pile driving into to the stratum.

Settlement for each pile can be calculated to ensure the permissible settlement ranges for the proper serviceability of the project.

DCPT method should be employed to properly simulate the actual environmental conditions.

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