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Large Diameter Piles in Versova Bandra Sea Link Project - A Case Study

Mr. Madhanraj Murugesan[1], Dr. (Mrs.) Pratibha M. Alankar[2]
[1]Master of Engineering, Civil Engineering (Construction & Management)
[2]Professor, Department of Civil Engineering, RMD Sinhgad School of Engineering
RMD Sinhgad Technical Institutes Campus, Warje, Pune

Abstract: The construction of bridges in marine environments poses unique challenges that demand innovative solutions to save the ecosystem along with the time and construction cost. Monopiles have emerged as a pivotal component in such conditions and have much reduced impact on marine environment and ecosystem. Monopile foundation is studied to find a good idea for design. It primarily illustrates why a large diameter pile foundation is preferable than a group pile bridge foundation. The motivation behind this study is to analyze the performance of monopile foundation over group pile foundation in marine conditions. This study explores the challenges, and advancements, scheduling in utilizing monopiles for such critical infrastructure projects. Through case studies and technical discussions, the paper aims to provide valuable insights for those who are involved in marine and bridge engineering projects.

I. INTRODUCTION

Monopile foundation consisting of a single, generally large-diameter structural element that supports the entire load of the above-surface structure. This system is followed because the time consumption is less compared to other foundation techniques. Monopile is a hard engineering method of construction in marine conditions where the wave impact is more, and climate is not helpful for the construction works. The resilience of monopile towards lateral load is very much important considering the impact of waves and swell on the foundation. Monopile foundation is a very good alternative over the grouped pile which takes more construction time and requires heavy movements in marine. The pile foundation undergoes very high combined loading (vertical & lateral), so it is necessary to test the pile for both vertical loads to understand the bearing capacity and for lateral loads to understand the flexural capacity. we will look deeper into the challenges in monopile construction, technologies used in construction, case studies of successful projects, and outline future research directions.

Through this study, we aim to provide a comprehensive understanding of Monopiles and their pivotal role in marine bridge construction. During the service life of the bridge, the foundation structure is subjected to several extreme conditions, including weather conditions (such as waves, currents, wind), earthquakes, tsunamis. The principal difference between an offshore and onshore construction is the combination of wind and wave loads on the structure, which implies a higher level of constraints, especially for extreme weather conditions. The choice of the type of foundation and the design process is a complex task because it strongly depends on the geographical location and soil properties.

A. Aim And Objective

1) Aim

To study the benefits of Monopiles over pile group in offshore bridges with case study.

2) Objective

- To study the geotechnical parameters considered in the design of large diameter piles.
- To study the design parameters of Monopiles and group piles.
- To detail out the construction time cycle of Monopiles and group piles.

B. Methodology

- Through detailed literature survey
- Through case study (Actual site visit and data collection)

C. Advantages And Disadvantages

The Monopile construction has several advantages over the group pile system in a marine construction project.

- The construction time can be reduced to large extent (the average construction time of monopile is approximately 5 days whereas it takes more than 10 days to construct a 4-pile group).
- The cost-effective technique compared to the group pile system.
- This is effective in location where space constraints are there.
- Large diameter pile is effective in location with multiple utilities.
- Minimal disruption to the marine ecosystem as one location is drilled instead of multiple drilling.
- Reduced barge movements and positioning which in turn reduced the carbon footprints.
- No necessity for pile cap construction and its design.
- Reduced overall cost of the project.

There are some disadvantages in the construction of Monopiles like,

- The preliminary design and its costing are more.
- Expert involvement is required which in turn increase the cost.
- High precision is required in design and execution.
- Increased geotechnical investigations.
- Not economical for small scale bridges.

II. LITERATURE REVIEW

In this section, we will go through the literatures on the models on the monopile design, constructions, and the factors to be considered while designing and executing the monopile foundation. This will give an idea on the factors that govern the monopile execution and design.

1) "High-Capacity Piles Confirming the Viability of Cost-Efficient Bridge Designs" Brian A. Liebich- Oct 2003.

The objective of the research was to verify pile capacities on individual projects and to provide information about load-deflection behavior characteristics that could enhance designer knowledge and confidence in static design procedures for high-capacity piles. The research results encourage designers to consider high-capacity piles whenever the economic benefits of use outweigh the cost of testing.

The paper presents the review of the understanding how the California Department of Transportation (Caltrans) has achieved substantial cost savings and has reduced construction time by using large-diameter, high-capacity piles. On the two Santa Clara River bridge projects, the pile testing permitted use of high-capacity piles, with an estimated savings of $14 million. In addition, high- capacity pile installation realized significant time savings in the construction schedule, compared with the alternative methods. Generally, use of high-capacity piles results in fewer piles and little or no redundancy; verification of capacity, therefore, is essential. The information on the behavior of high-capacity piles was not definitive, but the potential for significant cost savings encouraged further investigation.

2) "Experimental Testing of Monopiles in sand subjected to one-way long-term cyclic lateral loading" Roesen H.R., Ibsen L.B., Andersen L.V – Aalborg University, Denmark-2013.

This paper presents a description of a 1g laboratory small scale test setup for modelling laterally long-term cyclic loading of a stiff pile in saturated dense sand. A static loading test and four one-way cyclic loading tests with maximum moment loading equal to 18% to 36% of the maximum static capacity are presented.

The purpose of the tests is to evaluate the influence of the number of load cycles on the accumulated rocking rotation of the pile at seabed during long-term cyclic loading. In addition, the effect of the cyclic loading on the static lateral capacity is evaluated by means of static loading tests conducted post cyclic loading. The purpose of the cyclic tests is to evaluate the influence of the number of load cycles, on the accumulated rocking rotation of the pile at seabed, under long-term cyclic loading with constant frequency but different loading amplitude and mean loading level.

3) *"High-Speed Railway Bridge and Pile Foundation: A Review" by Brian Bachinilla, Ana Evangelista, Milind Siddhpura, Assed N. Haddad, Bruno B. F. da Costa – 2022.*

This paper revisits the published studies on HSR bridges and pile foundations through a bibliometric review by using the VOS viewer software using the keywords "Monopile foundation" and the combination of "pile shaft" and "bridge". All the results indicate the need for further research to fill the knowledge gap related to the soil–structure interaction phenomenon considering the performance-based seismic design of a high-speed railway bridge with the innovation of monopile foundation in areas with liquefiable soil because of lateral spreading during earthquakes. Moreover, the bibliometric map shows monopile foundations applied more in offshore structures than in bridges. More studies therefore need to be published on monopile foundations that support bridges, especially HSR bridges. Another factor affecting the HSR bridge design is the innovation of pile foundations. Instead of designing multiple piles of small diameter, the HSR bridge pier is on the top of a monopile foundation. Lessons learned from bridge projects highlighted that a monopile foundation is preferable when working in limited spaces and congested areas such as busy streets. It could reduce conflicts with existing utilities and minimizes the risk of foundation damage caused by an active fault line passing directly underneath the pier. Moreover, a large-diameter monopile foundation can be a practical choice of HSR bridge foundation for areas subjected to a lateral spreading caused by liquefaction, because the monopile has greater stiffness and relative strength than multiple small-diameter piles.

4) *"Structural design optimisation of single piles" Jorge Eduardo Perez Loaiza Matr – 2017/2018.*

This thesis deals with the optimisation of three different materials for single piles: steel pile, concrete pile, and steel fibre reinforced concrete pile. A program has been developed with the use of interior point algorithm implemented in MATLAB and an objective function based on the cost for each of the pile materials. As single piles have to be optimized, a first study of the literature has been done to proceed with the best design approach that can be applied to the optimisation problem. The study also evaluates the influence of the variation in soil parameters and loads applied, demonstrating that in some cases FRC piles would represent a better option from the point of view of economics. This thesis aims at optimizing the design of single pile foundation subjected to vertical and lateral loads with three different pile materials: steel, RC and FRC. Through the development of MATLAB codes for the optimisation and review of the existing literature, a study of the optimisation in single piles is presented.

5) *"Soil-Structure Interaction Study on Group pile over Monopile Foundation" Maddela Jyothi Kiran, Gomasa Ramesh, Dr. Annamalai Rangasamy Prakash – 2021.*

This paper discusses about the economical aspect of Monopile foundations over group pile foundations. Monopile foundations are simple and easy to construct. Compared to the group pile foundation, monopile foundations are very good and suitable for any type of soil. It is suitable for the stability of structures and improves performance. By using a monopile foundation we can minimize the overall cost of the structure compared to the group pile foundation and also decrease the number of piles. So, it is economical compared to group pile structures. in this response of monopile foundations are determined. All the results are good and acceptable in conditions. In this monopile foundations are used in bridges with a deep-water condition.

6) *"A Review Study on Aspects of Monopile as Bridge Foundation" Smita K. Badole, Dr. Valsson Varghese – 2022.*

This thesis deals with the optimisation of works involved in a project. It also states that with a monopile foundation we can reduce the structure compared to the group pile foundation and also decreases the number of piles. It may be possible to achieve optimum performance by positioning a relatively small number of piles in the right place rather than using more heaps or increasing the raft thickness. In monopile case, the vertical load reduces the maximum bending torque as well as the lateral deformation when subjected to single rod lateral load. Safety against a bearing capacity failure, average settlement and different settlement are the quantities to be controlled by monopile foundation. Monopile foundations are suitable for the stability of structures and improve the performance.

7) *"Monopile Implementation in Offshore Bridge" Prof. Tarranum khan, Yogesh Sudam Gamre, Affan Ahmed Chowdary, Ahad Sharif Sayyad, Asad Abbas Azadar Saiyyed - 2024.*

This paper presents an in-depth analysis of the implementation of monopiles in offshore and overwater bridge construction. Monopiles, cylindrical steel structures driven into the seabed or riverbed, have gained prominence as a foundation solution due to their cost-effectiveness, ease of installation, and environmental benefits. A review of Literature revealed that, by using a monopile foundation we can reduce the structure compared to the group pile foundation and also decreases the number of piles.

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8) "Local scour at offshore windfarm monopile foundations: A review" Da-Wei Guan, Yu-xuan Xie, Zi-shun Yao, Yee-Meng Chiew, Ji-sheng Zhang, Jin-Hai Zheng – 2021.

This review gives a comprehensive assessment of the study of local scour at monopile foundations. It includes the summary of scour mechanisms, predictions, numerical modeling, and field observation. Finally, future study hotspots are provided in the conclusion after considering the results of previous studies under various hydrodynamic conditions. This aims to extend the limitations of the scour prediction methods in existing guidelines of scour design at offshore windfarm monopile foundations. Despite the periodic changes in the current velocity magnitude and direction, the water depth of a tidal current experiences cyclic changes in reality. Given that the physical experimental technologies at present are difficult to control these two variables simultaneously, the scour development under a specific tidal current condition is obtained using a qualitative analysis. However, the scour development data under a specific tidal current condition cannot be acquired using a physical simulation.

9) "Determining the Embedded Pile Length for Large-Diameter Monopiles" - Victor D.Krolis, Gerrit L. van der Zwaag, Wybren Everen de Vries – 2010.

This paper evaluates the applicability of current foundation design criteria for large diameter monopiles. Emphasis will be on the vertical tangent criterion as suggested by Germanischer Lloyd and the "zero-toe-kick" criterion for determining the required embedded pile length under static loading conditions. The lateral behavior of a total of 40 different design cases of monopile support structures in water depths ranging from 15 to 35 m has been studied. The soil conditions ranged from loose to very dense sand, which is typical for the North Sea. It has been concluded that the vertical tangent and the "zero-toe-kick" criteria leads to overly conservative embedded pile lengths.

A preliminary design approach is presented, which is based on the knowledge that shortening the embedded pile length will decrease the natural frequency of the support structure. The results from this preliminary design approach study have been compared with the current monopile design practice, and it was concluded that the embedded monopile length can be reduced while achieving both lateral stability and maintaining small values of deflection at mudline and the pile toe.

10) "Design and Construction Approach of Marine Rock Socketted Piles for An Oil Jetty Project" Seema Gurnani, K. R.Vinjamuri, A.Usmani C. Singh – 2018.

This paper discusses about the methodology followed for pile fixity and pile capacity calculation is discussed. Rock socket length is computed for allowable socket pressure which depends on UCS of rock layer. Rock socket length is designed as 3 times pile diameter below the start of mod. weathered rock level (i.e. RQD>25%). The rock socket length is sometimes increased up to 5 times pile diameter in case very poor quality of rock is encountered in the entire socket length during field execution. Also, Large unsupported length of marine piles causes huge lateral loads, huge moments, and large pile head deflections. Rock socket length is designed based on the required axial capacity and allowable socket pressure. Virtual fixity depth of pile can be considered at middle of rock socket below highly weathered rock and pile termination level.

11) "A Case History of Super-Large Scale Bridge Pile Foundation in Soft Soil" Chao Yang, Guoliang Dai, Weiming Gong – 2013.

This paper discusses about the test of bearing capacity of super-long, large-diameter bored pile was determined by combination of O-cell test method and anchor piles, which provides a reliable basis for the project design and a valuable analysis data for scientific research. According to the test results, the super-long large-diameter bored pile of Sutong Bridge is floating piles, and it is difficult for the resistance at pile bottom to play. The compression of super-long bored pile should not be overlooked. The compression of pile after grouting account total displacement at pile top for 30~40%. The pile-cap-soil interaction was analyzed by centrifugal model test and numerical simulation, load distribution of pile top was obtained, both of which have guiding significance for design in future.

III. CASE STUDY

The infrastructure network in India is getting bigger and bigger with the introduction of new technologies and techniques which is reducing the construction time and cost of the projects. Marine infrastructure projects in India are getting more importance as the land acquisition prices are more than the construction cost in marine. Therefore, the government is opting for the infrastructure projects in the marine (wherever possible) in order to reduce the raise in city traffic. The case study details about the Versova Bandra Sea link project.

A. Versova Bandra Sea link Project

The Versova Bandra sea-link project is in progress. The initial design is totally with group pile system. The Contractor executed the group pile system in the beginning of the project. Once the construction of pile-cap work started, the contractor found it difficult to execute the pile cap at the design level. The contractor then proposed the use of bent piles to replace the pile group. The client, consultant and the contractor discussed the possibilities on several meetings and finally agreed with all parties. With the introduction of monopiles and bent piles in the project, the contractor predicted a considerable savings in the project timeline. The pile bent method is already executed in many locations throughout the project and in this method, instead of 4 pile groups, only two large diameter piles were constructed and followed by a pier cap connecting both the piles at the top.

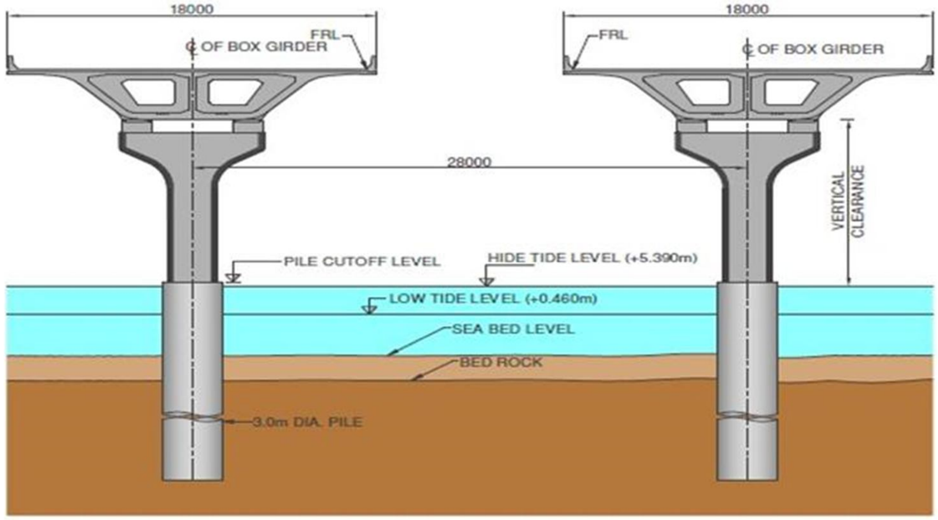

Fig:1 – Bridge Cross-sectional image (with Monopile)

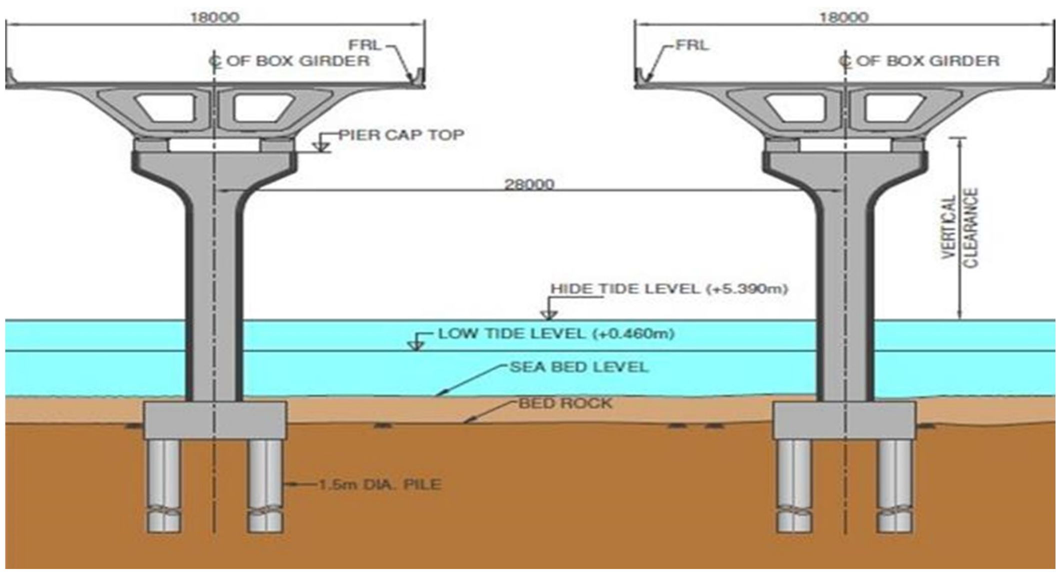

Fig:2 – Bridge Cross-sectional image (with group pile)

1) Original Scope

Item	No. of footings	No. of piles
1.20m dia pile	54	224
1.50m dia pile	744	3157
2.00m dia pile	52	218
2.40m dia pile	NA	300

2) Scope change with Pile Bent System

Item	No. of footings	No. of piles
1.20m dia pile	54	224
1.50m dia pile	278	1183
2.00m dia pile	52	218
2.40m dia pile	NA	1132

After the introduction of pile bent system in the project, the total number of piles reduced from 3899 nos to 2757 nos which will help the contractor by reduction in construction time and saves the time and money. After finding it really valuable, the contractor proposed the monopile system in the change of scope works in Juhu and Versova Connector. The client agreed with the proposal and the contractor is going to execute the Monopile foundations in the change of scope works.

3) Original Scope

Item	No. of footings	No. of piles
Juhu Connector	28	128
Versova Connector	37	162

4) Scope change with Monopile

Item	No. of footings	No. of piles
Juhu Connector	NA	31
Versova Connector	NA	38

With the introduction of monopiles of Dia 2.400m, the number of piles reduced from 290 nos to 69 nos. this reduction in number of piles will save a lot of time in pile construction. Also, there is no need of pile cap construction which further saves time and material.

5) Probable Saving items and Additional Costs Incurred

S. No	Probable Saving Items	Probable additional cost Items
1	Number of piles	Design and Analysis
2	Reinforcement	Construction Expertise
3	Concrete (Pile cap)	Drilling Equipment (RCD)
4	Cofferdams (For pile cap)	Additional costs

IV. PILE DESIGN

The of monopiles includes (1) Geotechnical design and (2) Structural design. Geotechnical Investigations are very much important for the design of Monopiles. Unlike group piles, we need the GTI for each and every Monopile. The GTI included Ground characterization and detailed analysis of the geological data in each pile location. The same has been carried out for the design of Monopiles. The structural design includes the modelling and analysis of the loads from the structure above the piles.

A. Geotechnical Design

For each monopile location the following will be carried out:

1) Develop geological profile
2) Derive characteristic values of geotechnical parameters appropriate for each design limit state, the design ground model will be presented in updates of the GIR for each module using the site wide and monopile specific ground investigation information.
3) Produce pile load bearing capacities based on the design standards.
4) Identify critical load cases for the determination of the rock socket length based on compression and lateral load capacity
5) Determine the monopile rock socket length based on the lateral pile capacity calculation.
6) Check the rock socket length determined.
7) Use L-Pile to derive P-Y curves to model the stiffness of the ground supporting the monopole in the Midas model which combines the behaviour of the monopiles and superstructure.

B. Ground Characterization:

The ground characterization includes the following things,
Soil description
Soil strength and Stiffness
Rock Quality Designation

C. Detailed Analysis

The following things are done in the detailed analysis,
Soil Models
Testing
Geotechnical Factual report
Geotechnical interpretative report.

The guidelines for large diameter pile design includes the calculation of,
Rock mass modulus
Lateral Stiffness
Pile socket length
Shaft Bearing Resistance
Pile Settlement
Lateral resistance.

D. Rock Mass Modulus

The rock mass modulus is defined as the stability or deformability of a rock structure. It is a vital parameter for numerical modelling and analysis of rock mass behaviour. It is calculated using the following formula,

Reese suggests the correlation given by Bieniawski (1978) between rock mass modulus, modulus of intact rock core, and RQD, given as –

$$\textbf{For } RQD < 70\text{: } E_M = E_R\left(\frac{RQD}{350}\right)$$

$$\textbf{For } RQD > 70\text{: } E_M = E_R[0.2 + (RQD - 70)/37.5]$$

Where,
E_R = Intact rock modulus
E_M = Equivalent rock mass modulus

E. Lateral Stiffness

Lateral stiffness is defined as the resistance from the soil at a point along the depth of the pile divided by the horizontal deflection of the pile at that point. The load-displacement curves are typically developed with little to no knowledge of how stiff the foundation is; thus, considering only pile-soil interaction. Therefore, it is essential to account for the effect of foundation rotational stiffness on the lateral stiffness of fixed-head piles, i.e., the pile-foundation interaction, when evaluating the response to lateral loads. It is calculated using the following formula,

$$K_i = \frac{1.0\, ED}{(1-\nu^2)D_{ref}} \left[\frac{ED^4}{E_p I_p}\right]^{1/12}$$

Where,

Reference pile diameter, D_{ref} = 1.0 m

E_pI_p = flexural rigidity of the piles or drilled shafts

K_i = Lateral Stiffness

E = Deformation modulus of rock

D = Diameter of pile

F. Pile Socket Length

The depth up to which a pile-end needs to be penetrated into sound and rock or soil layer is called pile socket length. The socket length is calculated mainly by the load transfer through skin friction and end bearing depends on soil/rock profile and grade. Here the socket length is arrived by using the caltrans approach. The actual socket length is increased by 20% as a measure of safety which ended up in the range of 1.2 times of critical socket length as per the design/model.

FHWA (Caltrans) method for Pier P15 (Arm-3)-3 m dia. Pile

S.No	Length of socketing (Ls) considering scour depth as 1.0 m	Total length of Pile (m) = 1.2Ls	Deflection (mm)	Relative variation (%)
1	6	7.2	46.183	
2	7	8.4	31.841	31.06
3	8	9.6	28.056	11.89
4	**9**	**10.8**	**26.824**	**4.40**
5	10	12	26.480	1.29
6	11	13.2	26.406	0.28

Elevation of pile	Deflection (mm)
-11	26.406
-10	26.480
-9	26.824
-8	28.056
-7	31.841
-6	46.183

Pile length below GL as per Caltrans method including 1.0 m scour = **10.8 m**
≈ **11 m**

G. Shaft Bearing Resistance

The ultimate bearing resistance of a pile foundation may be calculated from static pile formulae using values of ground parameters obtained from field or laboratory tests on soil and rock. As per AASHTO LRFD 2017, the nominal axial compression resistance of a shaft is derived from the pile tip or from from the shaft resistance using,

$$R_R = \phi . R_N = \phi_{qp} . R_p + \phi_{qs} . R_s$$

in which:

$$R_p = q_p . A_p$$

$$R_s = q_s . A_s$$

where:

R_p = nominal shaft tip resistance

R_s = nominal shaft side resistance

ϕ_{qp} = resistance factor for tip resistance,

ϕ_{qs} = resistance factor for shaft resistance

q_p = unit tip resistance

q_s = unit side resistance

A_p = area of shaft tip

A_s = area of shaft side surface

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H. Settlement Check

The settlement criteria of a monopile is calculated using is IS14593:1998, clause 6.6.1, in which pile settlement is defined as,

$$S = S_i + S_p$$

$$S_p = \frac{PL}{A_p E_p} \quad \& \quad S_i = \left[\frac{2\,PI_r}{E_d\, l_s}\right]$$

Where,

S = Total settlement of the pile

A_p = Cross-section area of the pile

S_p = Elastic compression of pile

S_i = Immediate settlement of pile

P = Design load on pile

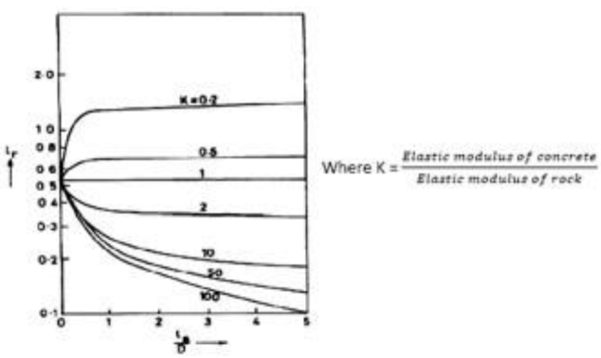
E_p = Modulus of elasticity of pile

I_r = influence factor as given in figure

E_d = Modulus of deformation of rock mass

l_s= length of socket

L = total pile length



I. Lateral Stiffness (For Structural Design)

Lateral stiffness is calculated based on nonlinear P-y curve using ALP software. Using MIDAS software the shear force and Bending moment is calculated. All these values will be used for further structural design of reinforcement and sections.

Lateral resistance check will be done using AASHTO LRFD:2017, lateral capacity checks are performed for extreme case with maximum moments and c orresponding shear with 0.8 factor and strength case with 0.67 factor.

Limit State	Axial/Lateral Resistance	Resistance Factors as per AASHTO LRFD 2017	Resistance Factors as per GEC-10	Adopted Resistance Factors in Design
Strength Limit State	Geotechnical Axial Side Resistance	0.44	0.5	0.44
	Geotechnical Axial Tip Resistance	0.4	0.5	0.4
	Geotechnical Lateral Resistance	0.8	0.67	0.67
Extreme Limit State	Geotechnical Axial Resistance	0.8	1	0.8
	Geotechnical Lateral Resistance	1	0.8	0.8
Service Limit State		1	1	1

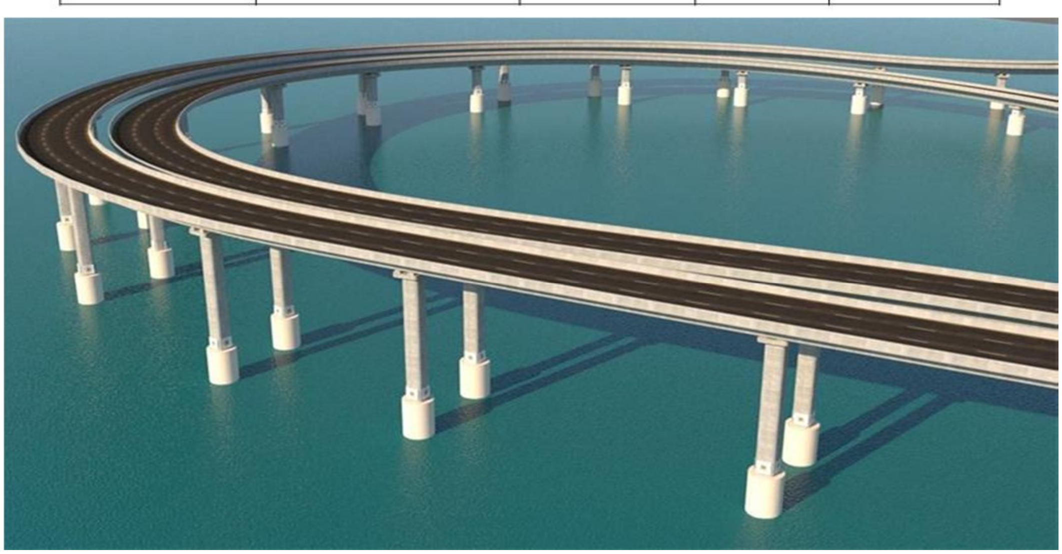


Fig (3): Aesthetic view with Monopile Foundation.

Fig (4): Aesthetic view with Group pile Foundation.

B. Structural Design

1) Design Loads

The following loads are considered in the structural design of the monopiles.

a) Dead Loads

Unit weights of materials are defined in IRC: 6-2017, Cl. 203, and ERs and will be as following: -

Material & Unit Weight
Plain Cement Concrete - 25 kN/cu.m
Reinforced Cement Concrete (RCC) - 25 kN/cu.m
Pre-stressed concrete - 25 kN/cu.m
Structural steel - 78 kN/cu.m
Earth compacted - 20 kN/cu.m
Asphaltic concrete in wearing coat - 22 kN/cu.m

In addition to the above, unit weight of mass concrete and green concrete will be taken as 24 kN/m3 and 26kN/m3 respectively.

b) Superimposed Dead Loads

A load of 5.0 kN/m is considered or utility services which is proposed to be carried through the inside of main line box girder (load to be confirmed based on MEP requirements). For interchange arms, no additional load will be considered for utilities. A load of 0.9 t/m is considered for weight of each crash barrier. Noise barrier loading is to be considered for those stretches where it is required, and load will be considered as per the cross section of noise barrier.

c) Water Current Forces

The portion of bridge which may be submerged in running water shall be designed to sustain the horizontal pressure due to force of water current as per the stipulations of Cl. 210 of IRC:6-2017. Maximum mean velocity of water current shall be taken as 1.5 m/s in any direction for design purpose based on the current speed data.

d) Buoyancy Force

100% buoyancy will be considered while checking stability of foundation irrespective of their resting on soil/weathered rock/or hard rock. However, the maximum base pressures shall also be checked under an additional condition with 50% buoyancy in cases where foundations are embedded into hard rock.

e) Wave and abnormal wave Forces

The effect of waves on the structure will be considered in accordance with Cl. 5.7 of IS 4651 Part 3 -1974. For abnormal wave loads, for the design wave heights (taken as 1.89 m as per NIO report). In addition, recommendation from Coastal engineering manual will be followed for computation of wave forces. Based on chapter 3 of NIO report, the maximum amplitude of tsunami wave is only 0.73 m and hence it is considered as less critical.

f) Wind Loads on Structure

The basic wind speed to be adopted in the wind load determination is defined as per Cl. 209 of IRC: 6-2017.The basic wind speed at 10 m height in Mumbai, for 50-years return period, is 44 m/s. As per Cl. 209.2 of IRC:6-2017, the wind force is based on hourly mean wind speed (Vz), and horizontal pressure (Pz). The corresponding values to a basic wind speed of 33 m/s, return period of 100 year, for bridges in flat topography is presented in Table 12 of IRC:6-2017.

g) Differential Settlement

Differential settlement of supports is considered as a permanent load. The piles are socketed in rock and there might not be any differential settlement as such. However, for design purpose, 6mm differential settlement between the supports will be assumed as long-term loading.

h) Earthquake (Seismic)

The seismic design is based on the IRC:6-2017, being the design spectrum defined as per the assuming:

- Seismic Zone III
- Zone factor, Z for zone III is 0.16
- Importance factor, I, of 1.5.

i) Barge Impact on Bridges

The Barge impact force and kinetic energy shall be calculated as per Cl.220 IRC:6- 2017. All the piers in the navigational spans of the Main Viaduct shall be protected by providing suitable fender type protection system. IRC:6-2017 states that the bridge should be designed to minimise the risk of structural failure of a bridge component due to collision with a barge. The risk of damage to the barge should also be minimised.

j) Scour Analysis

The purpose of the scour analysis is to estimate the total scour depth at the base of the bridge piers structural supports. The main purpose of this analysis is to evaluate the depth and extent of scour during design working life operation of the bridge, so that its potential effect on the design capacity of the bridge foundations can be assessed. For this project, the substructure has been proposed as monopile, thus scour calculation for a single pile has been carried out.

The following are the standards used for the design of monopiles.

S. No	Design Standards
1	AASHTO LRFD Bridge Design Specifications (2017)
2	GEC-09: FHWA Design, Analysis, and Testing of Laterally Loaded Deep Foundations that support Transportation Facilities design Methods
3	GEC-10: FHWA-NHI 18-024: Drilled Shafts - Construction Procedures and Design Methods
4	IRC – 078 (2014): Standard specification and code of practice for road bridges, Foundations and Substructure.

5	IS 2911-1-2 (2010): Design and Construction of Pile foundations – code of practice, Part 1 Concrete Piles: Section 2 Bored cast in-situ concrete piles.
6	IS 1892 (1979): Code of practice for subsurface investigation for foundations.
7	IS 12070 (1987): Code of Practice for Design and Construction of Shallow Foundations on Rock.
8	IS 14593 (1988): Design and construction of bored cast in situ piles founded on rocks – Guidelines.
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V. TIMECYCLE FOR CONSTRUCTION

A. Time-cycle for monopiles

The construction time of monopiles is much less compared to the pile group system.

The monopile construction normally takes 5-7 days depends on the pile depth and geology of the pile location. The piles are constructed using the Reverse Circulation Drilling technique which considerably reduced the construction time. The following table gives a simple insight of the activities associated with monopile construction.

ACTIVITY	DAYS
Guide frame positioning and Liner & RCD Installation	1
Drilling	2-4
Rebar cage lowering (In parts)	1
Concrete Pour	1
TOTAL DAYS FOR 1 PILE (On average)	5 – 7 days

B. Time-cycle for group piles

ACTIVITY	DAYS
Guide frame positioning & Liner Installation	0.5
Drilling	1-2
Rebar cage lowering (In parts)	0.5
Concrete Pour	0.5
TOTAL DAYS FOR 1 PILE (On average)	2.5 - 3.5 days

The construction of a 4-pile group will take approximately will take a minimum of 10 days. Once the piles are constructed, pile cap construction begins. The timeline of pile cap construction is given below.

ACTIVITY	DAYS
Base slab installation	2
Reinforcement installation	3
Formwork and Alignment	1
Concrete pour	1
Formwork removal	1

On average the pile cap construction will take about 7-8 days which is not necessary in case of large diameter pile construction.

In short, the time taken to construct a large diameter pile up to pier starter will take maximum 7 days whereas in group pile system it will take minimum 20 days which is comparatively more time consuming.

On an average 60% of time is saved by adopting the monopile and large diameter pile technique.

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VI. MONOPILE CONSTRUCTION

Monopile construction was done using Reverse Circulation Drilling (RCD) technology. The construction of Monopile includes the following steps.

A. *Liner Pitching and Driving*

The pre-fabricated liner with bottom shoe is shifted to the location and is lifted with the help of the Service crane. The liner is then placed in the piling platform and lowered gently till it reaches the seabed level. After that, the position of the liner (verticality) is checked. Once it is found within the limits, the liner is driven into the seabed with the help of vibro-hammer (Fig-4) of suitable capacity (12.5 MT capacity used). During the driving, care should be taken to avoid the liner deformation at the bottom and the liner position is checked after the completion of driving.

Fig-5: Liner driving with Vibro-hammer

B. *RCD mounting and Drilling*

The mast of RCD is lifted with the crane and is mounted on the pile casing. Once positioned on the casing top, the mount is arrested with the help of clamping unit. The drill string unit or Bottom Hole Assembly (BHA) is lifted and installed inside the liner and locked with the drilling unit on the mast. This is followed by the installation of drill pipe (normally 3m long and variable Dia) and the swivel assembly and drill string are fixed (Fig-5). The air hose is then connected with compressor and the outlet delivery hose is connected to the collection bin/tank. Water (salt/fresh) will be pumped inside the bore through a pump attached with RCD or through a separate pump. The water will enhance smooth drilling operation. Add the drill pipes and add the Drill Rod Stabilizer (DRS will be added for every 3no's drill rod installed). The cutters can be changed based on the boring strata. The addition of drilling pipes will continue till the borehole reaches the design toe level. Once the drilling works are done up to the design level, bore flushing will be done.

Fig-6: Jack up barge Drilling by RCD

C. Rebar cage Installation

The rebar cages are fabricated in the yard and later shifted to the pile location. The CHSL tubes should be tied strongly with the inner reinforcement to avoid any deformation. The cages are shifted in parts (as per the crane capacity and boom length) and care must be taken that position of lapping/couplers must be the same. Any mismatch in position will be time-consuming during installation and is difficult to fix the couplers. The installation of the steel cage inside the excavated shaft will proceed by using the service crane (Fig-6). The cage will be lifted by the wire ropes of the crane tied at different levels with help of spreader beam/cage lifter unit to reduce the bending stresses and deformation during lifting. The connection between two pieces of the cage will be done by couplers. After installing all the elements, the rebar cage is rested in its design level with the help of cage hangers fixed on the casing top. All the CHSL tubes should be water-tight to prevent any slurry entry during the concrete pour.

Fig-7: Rebar Cage installation

D. Tremie Installation and Airlifting

Airlifting is done with water/stabilization fluid. The aim is to remove the slush/debris from the bottom of the borehole. Airlifting is done with the help of an air compressor by blowing in the air to the bottom of the borehole and the pressure shift cleans the bottom surface and the slush/water will come up through the pipes (200mm Dia). Water will be poured from top of the borehole and the airlifting should continue till the entire borehole is cleaned. At the end of tremie installation, make sure that the minimum space (normally 300mm) is maintained between the tremie bottom and borehole bottom.

E. Concrete Pour

Concrete can be either transported to the location with a Transit Mixer. A concrete pump with hose supporting boom will be provided to supply the concrete to the tremie hopper during casting. To prevent contamination and segregation of the concrete during it falling inside the tremie pipes, a “sacrificial buffer” made of lightweight material (such as vermiculite or similar material or ball) will be introduced inside the tremie hopper prior to the start of concreting operations. A charge plug will be paced at the bottom of the hopper/tremie neck, and the hopper is filled with concrete and is charged with the help of a service crane. Once started, there should not be any delays in concrete supply as the choking chances are higher. Make sure that the tremie pipe bottom is always inside the concrete (maintain at least 2m to 3m inside the concrete).Remove the tremies as per the volume of concrete filled inside the bore and concrete should be stopped either after overflow and 1m above cut-off level. The hopper can be replaced with a smaller one during the first set of tremie removal.

Fig-8: Pile Concrete pour with Floating batching plant

VII. CONCLUSION

The case studies indicates that the use of large diameter pile technique will reduce the construction time and cost to a considerable extent. Also, there is no need to design the pile cap and pier as the bent pile will continue till the pier head soffit. The design and execution of such piles requires expertise and careful designing that include detailed analysis and proper geotechnical investigations and testing. This method also helps the contractor to achieve the timeline of the project. The large diameter piles also help in protecting the environment as it requires lesser barge movements compared to a group pile. The seabed disruption is also minimal as the number of piles will be cut down by 50% approximately. For the projects with stringent timeline, large diameter piles can really help the construction team to achieve the target on time.

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