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QUALITY EVALUATION OF MONOPILE HIGH STRENGTH CONCRETE IN MARINE BRIDGE FOUNDATION

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ABSTRACT

Monopile is the most common form of foundation employed under offshore or Marine works. These foundations are subjected to millions of repeated load cycles from the wind and waves of varying magnitude leading to accumulated displacements and changes in soil-pile stiffness. The purpose of this study was to investigate the behavior of Quality Evaluation of Monopile High strength Concrete in Marine bridge foundation.

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. This paper explores the design considerations, construction methodologies, challenges, and advancements in utilizing monopiles for such critical infrastructure projects. Through case studies and technical discussions, the paper aims to provide valuable insights for engineers, researchers, and policymakers involved in marine and bridge engineering projects. The investigation was divided into two parts: the first part is studying the long-term behavior through the reduced scale model test, and the second to analyse the short-term behavior using the Finite Element analysis. The findings of three monotonic and seventeen cyclic load tests performed in the laboratory on medium dense sand and dense sand were presented in this thesis. The experimental investigations discussed the effect of asymmetric two-way cyclic loads on the rate of accumulated displacements and changes in soil-pile stiffness. The overall conclusion of this research was that the monopile foundation experiences a higher reversal of accumulated displacement at relatively low load amplitude with an increasing number of cycles. As the monopile was subjected to irrecoverable displacement at the initial cycles and recoverable displacement with an increasing cycles on both serviceability and fatigue loading conditions. Under asymmetric two way cyclic loading with $\xi c = -1.3$, the reversal of accumulated displacement was 49% higher than the symmetric two-way cyclic loading with $\xi c = -1$ at around cycle number $N = 2150$. It is observed that a more severe problem occurs under asymmetric two-way loading conditions. The non-linear response was observed for both test series, first lateral cyclic secant stiffness increases with a higher rate, and then the rate of increasing got decreased with an increasing number of cycles, but it did not get stabilized. This indicates a gradual increase in soil-monopile system stiffness in each cycle owing to sand densification. The linear regression analysis was also performed to fit the conventional degradation parameter using the minimum number of critical constraints that includes the loading conditions and the flexibility parameter of soil-pile system. In this study, an attempt has been made to examine the influence of embedded length on monopile behavior using finite element analysis. The centrifuge test carried out on a monopile embedded in sandy soil was used to validate the constituent model (Hardening soil model with small-strain stiffness). The numerical studies were performed on a 6m diameter monopile by varying the load amplitude and embedded length ratios (L/D = 4, 5, and 6). The monopile was subjected to two-way symmetric lateral cyclic loading with an amplitude of 30%, 40%, and 50% of the monotonic ultimate capacity of the pile. The difference between the measured displacement of numerical analysis and the centrifuge test varies by 27%. The similar trend irrespective of the values, and the monopile response under cyclic loading was observed from the load displacement curve, which indicates that the measured accumulated displacement increases drastically for the first load cycle. For a given embedded length, the lateral displacement was observed to increase with an increase in load amplitude. Also, the load amplitude was observed to cause a linear increase in the accumulated displacements.

1. INTRODUCTION

The construction of bridges spanning over waterbodies or in offshore environments poses unique challenges that demand innovative solutions. Among these solutions, monopiles have emerged as a pivotal component, offering a versatile and efficient foundation system for such projects. As our infrastructure needs continue to expand, understanding the capabilities and applications of monopiles becomes increasingly crucial. This paper provides a comprehensive overview of monopiles and delves into their applications in offshore and over waterbodies bridge construction. By examining the design considerations, construction methodologies, challenges, advancements, and

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case studies, this paper aims to shed light on the significance of monopiles in addressing the complex demands of marine and bridge engineering.

Monopiles, essentially cylindrical steel structures driven into the seabed or riverbed, have gained prominence for their adaptability and cost-effectiveness. Their utilization as foundation solutions offers several advantages, including simplified installation processes, reduced environmental impact, and enhanced structural stability. Understanding the intricacies of monopiles and their integration into bridge construction projects is paramount for ensuring the longevity and resilience of our infrastructure in marine environments. methodologies, challenges, advancements, and case studies, we seek to contribute to the body of knowledge in marine and bridge engineering. By doing so, we hope to facilitate informed decision-making and foster the development of sustainable and resilient infrastructure solutions for the challenges posed by waterbodies and offshore environments. In the subsequent sections, we will delve deeper into the design intricacies of monopiles, explore the various construction methodologies employed, discuss the challenges faced, highlight recent advancements and innovations, present case studies of successful projects, and outline future research directions. Through this comprehensive examination, we aim to provide a holistic understanding of monopiles and their pivotal role in offshore and over waterbodies bridge construction.

Through this study, we aim to provide engineers, researchers, and policymakers with valuable insights into the effective implementation of monopiles. By exploring the design considerations, construction

Municipal Corporation of greater Mumbai proposed developing a coastal road project (MCRP) from Princess Street flyover to Kandivali junction over about 29km to ease the traffic congestion in Mumbai with recreational spaces.
This project is being implemented in 2 phases namely South and North. The South phase starts at Princess Street flyover and ends at Worli end of Bandra Worli sea link (BWSL).

This phase is divided into 3 packages as mentioned below:

Package 4: Princess Street flyover to Priyadarshini park (CH km 1+970 to CH km 5+900)
Package 1: Priyadarshini park to Baroda palace (CH km 5+900 to CH km 9+720)
Package 2: Baroda palace to Worli end of BWSL (CH km 9+720 to CH km 12+470).

Figure 1.1- Google image showing alignment of MCRP package -1

The monopile technique was adopted by the contractor to reduce construction time, and 33 marine modules with monopile foundations were executed in place of group piles.

Two varied sizes of monopiles are executed in the project i.e., 2500mm dia and 3200mm dia and the concrete grade for all the pile is M60.

The reinforcement used for monopile construction corresponds to Fe-550D1.2

1.3 Monopile Foundation:

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A marine monopile is a type of foundation structure used in offshore construction projects, particularly in the offshore wind energy sector and other marine engineering applications. It is essentially a large, cylindrical steel structure that is driven or installed into the seabed or riverbed to support several types of offshore infrastructure, such as wind turbines, offshore platforms, bridges, and marine terminals. Marine monopiles typically consist of a single steel pile, hence the name "monopile," although variations with multiple piles exist. Monopiles are designed to withstand the significant forces exerted by waves, currents, and other environmental factors in offshore and coastal areas. The monopile serves as a sturdy foundation upon which further structures can be built or attached. The installation process of marine monopiles involves specialized equipment such as pile drivers or drilling rigs, depending on the seabed conditions and project requirements. Once installed, the monopile is securely anchored into the seabed, providing stability and support for the superstructure above water. In offshore wind energy projects, marine monopiles are commonly used to support wind turbine towers. The monopile is driven into the seabed, and the wind turbine tower is then mounted onto the top of the monopile, creating a stable foundation for the turbine to operate efficiently even in harsh marine environments. Overall, marine monopiles play a vital role in offshore construction projects, providing a robust and reliable foundation solution for various marine structures, including wind turbines, bridges, platforms, and marine terminals, contributing to the development of offshore infrastructure and renewable energy production.

2. SCOPE OF PRESENT STUDY

- The scope includes 6 test piles and 99 working piles in AGI (Amarsons Garden Interchange), HAI (Haji Ali Interchange), MLB(Main Line Bridge) locations with a diameter of 2500mm and 3200mm. The depth of monopiles varies between 12m to 41m.
- The monopile design follows the AASTHO and IRC standards (as per the DBR) along with the L-pile analysis and WALLAP software.
- Amarsons garden interchange monopiles have the shorter shaft length (starts from 4m) due the basaltic rock outcrop with high UCS values whereas Haji Ali interchange has the deepest pile shafts (upto 41m deep) due to marine deposits and clay on seabed levels.

There are 32 monopiles of 2500mm dia in Amarsons garden interchange, 50 monopiles of 2500mm & 3200mm dia in Haji Ali interchange and 17 monopiles of 3200mm dia in Main line bridge.

Working scope of Monopile in study:

S.No	Location	ARM	Diameter (mm)	Scope
1	AGI	1	2500	16
2	AGI	2	2500	16
3	HAI	1	2500	13
4	HAI	2	2500	13
5	HAI	2	3200	10
6	HAI	4	3200	05
7	HAI	8	3200	09
8	MLB	LHS	3200	09
9	MLB	RHS	3200	08

3. MATERIAL AND METHODS

Selection of material used in concrete mix and their specification

1. Coarse Aggregates

Specification of used coarse aggregates table 3.2.1

Particle Size : Sieve Analysis	IS:383-2016/ IS 2386 Part 1-1963	Table 1000-1 of MORTH (5th Revision) for Mximum nominal size =20mm (Clause 1.4.1.3 (3) of Sec 3 Volume 5 Construction specification)

Flakiness Index & Elongation Index	IS:383-2016/ IS 2386 Part 1-1963	< 35 % (Flakiness only)As per MoRTH
Deleterious Material	IS:383-2016/ IS 2386 Part 2-1963	Table-2 of IS:383-2016 (Max. 2% for total constituents)
Sp. Gravity	IS:383-2016/ IS 2386 Part 3-1963 Amdt-1(Reaffirm-2016)	Not specified
Water Absorption	IS:383-2016/ IS 2386 Part 3-1963	Not specified
Aggregate Crushing Value	IS:383-2016/ IS 2386 Part 4-1963	< 45% for Concrete work/IS 383-1970
Aggregate Impact Value	IS:383-2016/ IS 2386 Part 4-1963	< 45% for Concrete work/IS 383-1970
Los-Angeles Abrasion Value	IS:383-2016/ IS 2386 Part 4-1963	< 50% for Concrete work/IS 383-1971
Soundness	IS:383-2016/ IS 2386 Part 5-1963	Sodium Sulphate < 12% & Magnesium Sulphate < 18% - IS 383 1970
Alkali Reactivity	IS:383-2016/ IS 2386 Part 7-1963	Innocuous Aggregates Fig.6 of IS:2386 Part-VII ,
Petrographic Examination	IS:383-2016/ IS 2386 Part 8-1963	Identification of Rock as Innocuous as per IS:2386 Part-VIII
Chloride & Sulphate Content	BS 812/IS 2720 Part 26/BS EN 1744	Chloride - Max 0.01% , Sulphate - Max 0.4%(Cl 1.4.2.5 Sec 3 Volume 5)
Moisture Content	IS:383-2016/ IS 2386 Part 3-1963	Actual value

2. Fine Aggregates

Specification of used fine aggregates

Particle Size : Sieve Analysis	IS:383-2016/ IS 2386 Part 1-1963	Crushed Rock Sand gradation confirming to Table 1000-2 of MORTH (5th Revision) with permissible limits of max 20% on 150microns Sieve(, Fineness Modulus : between 2.0 to 3.5
Deleterious Materials	IS:383-2016/ IS 2386 Part 2-1963	Table 2 of IS:383 Max. 2% for total constituents
Materials finer than 75micron IS Sieve	IS 383-2016/ IS 2386 Part 1-1963(R2016)	Table 1 of IS:383 Max.15% for Crushed sand
Specific Gravity & Water absorption	IS:383-2016/ IS 2386 Part 3-1963 Amdt-	Not specified

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	1(Reaffirm-2016)	
Soundness	IS:383-2016/ IS 2386 Part 5-1963	Sodium sulphate < 10% MgSO4<15%/IS 383 1970
Alkali Reactivity	IS:383-2016/ IS 2386 Part 7-1963	Innocuous Aggregates Fig. 6 of IS:2386 Part VII
Chloride & Sulphate Content	BS 812/IS 2720 Part 26/BS EN 1744	Chloride - Max 0.01% , Sulphate - Max 0.4%(Cl 1.4.2.5 Sec 3 Volume 5)
Moisture Content	IS 383-2016/ IS 2386 Part 3-1963(R2016)	Actual value

3. water

To neutralise 100 ml sample of water, using mixed indicator	IS-456-2000(R2016) & IS-3025	It should not require more than25 ml of 0.02 normal H2SO4 (IS 456)
To neutralize 100 ml sample of water, using phenolphthalein as an indicator,		It should not require more than 5ml 0.02normal NaOH (IS 456)
Total Solid Contents		
Organic		200 mg/lit
Inorganic		3000 mg/lit
Sulphates (SO4)		400 mg/lit
Chlorides (Cl)		500 mg/lit
Suspended matter		2000 mg/lit
pH		> 6.0

4.Cement OPC 53 Grade

4.1 Chemical Properties

Loss on ignition	IS:4032-1985(R2019)/ MoRTH (5th Rev) / IS 269 : 2015	Not more than 4% , IS:269:2015
Insoluble residue		Not more than 5% , IS:269:2015
Alumina Iron Oxide Ratio		Not less than 0.66% , IS:269:2015
Lime Saturation factor		Not more than 1.02% & Not less than 0.8%, IS:269:2015
Magnesia		Not more than 6% , IS:269:2015
Sulphuric Anhydride		Not more than 3.5% , IS:269:2015
Alkalis content (Na2O)		Max 0.6%, IS:269:2015
Chlorides Content		Max 0.05%, IS:269:2015

4.2Physical Properties

Normal Consistency	IS:4031-(P1-P14)/IS:269:2015	Actual Value
Initial Setting Time		>30 minute
Final Setting time		<600 minute
Compressive strength		>27 MPA (3d), >37MPa (7 day) and >53 Mpa (28 days)
Fineness by blain air permeability method		>225 sqm/Kg
Soundness of Cement		Expansion should not be more than 10mm

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5. Chemical Admixtures

Chloride content	IS-9103:1999 (R2018) and IS 6925:1973 (R2018), MoRTH rev. 5 (clause 2.7.5, Sec. 2, Vol. 5 Construction Specification)	Max 0.2% MoRT&H clause 1012.3.2
Dry Material Content		To be within 3% and 5 % of liquid and solid admixtures respectively of the value stated by the manufacturer (MoRTH)
Ash Content		To be within 1 % of the value stated by the manufacturer (MoRTH)
Relative density		To be within 2% of the value stated by the manufacturer (MoRTH)
pH		> 6

6.Ground Granulated Blast Furnace Slag

Manganese oxide	IS-4032-1985(R2019)/ IS:12089-1987(R2018)/ BS 6699	Max 5.5 %, IS:12089-1987
Magnesium oxide		Max 17 %, IS:12089-1987
Sulphide sulphur		Max 2 %, IS:12089-1987
Ratio (CaO+ MgO+ l/3 Al2O3)/ (SiO,+2/3 Al2O3)		>= 1.0 IS:12089-1987
(CaO+MgO+Al2O3)/ SiO2)		>=-1.0 IS:12089-1987
(CaO + CaS+-l/2 MgO + Al2O3) (SiO2+MnO)		>= 1.5 IS:12089-1987
Insoluble residue		Should not be more than 5%
Glass Content		Min 85%
Fineness by blain air permeability method		Min 275 m2/kg
Moisture content		Max 1 %
Chloride content		Max 0.1%
Compressive Strength 7 days		Not less than 12 N/mm2
Compressive Strength 28 days		Not less than 32.5 N/mm2

7. Microsilica

% Retained in 45 Micron	IS-15388-2003 (R2017)/ ASTM C 1240/IS 1727-1967 (R2017)	Max 10% IS-15388-2003
Moisture Content, Percentage by Mass		Max 3 % IS-15388-2003
SiO2 Percentage by Mass		Min 85% IS-15388-2003
Alkalies as Na2O, Percentage by Mass		Max 1.5% IS-15388-2003
Loss on Ignition, Percentage by Mass		Max 4% IS-15388-2003
Compressive Strength N/mm2, 7days (% of control sample)		Min 85% IS-15388-2003

Specific surface , m2/gm		Min 15
Particle Size Analysis		
Bulk Density		As per Manufacturer
Particle size (d95)		Less than 11microns
Mean particle size (d50)		Not greater than 6 microns
Activity Index (7days)		105% Min

8. Corrosion Inhibitor Admixture

Chloride content	IS-9103:1999 (R2018)	Within 10 % of the value or within 0.2% whichever is greater as stated by the manufacturer.
Dry Material Content		To be within 3% and 5 % of liquid and solid admixtures respectively of the value stated by the manufacturer
Ash Content		To be within 5% of the value stated by the manufacturer
Relative density		To be within 2% of the value stated by the manufacturer
pH		> 6
Specific Gravity	IS 1448	Min 1
Viscosity	IS 3944-1982 (R2015)	Min 10 second
Accelerated Corrosion Test	JIS -Z-1535 (Japanese Standard)	No spots of Corrossion
Active Solid contents	ASTM-1582	greater than 20%
Compressive & Flexural Strength of concrete	ASTM-1582	98% of control concrete strength
Residue by Oven drying	ASTM-1582	Negligible
Setting Time	ASTM-1582	No Change than control concrete
Effect of chemical admixtures on corrosion of metals	ASTM G 180	

9. Concrete & Concreting

Sample of Cube (A sample means as set of 3 Cubes) for 28 days test with additional 3 cubes for 7 days & any additional cubes for early days as per site request	Table 2.7.8 Sec 2 Volume 5 Construction Specification, IS 1199	Qty. of Concrete (Cum) - No. of samples : (1-5) - 1, (6-15) - 2, (16-30) -3, (31-50) - 5, (51 & above) -4 + One additional for each 50 Cum
Workability (Slump)	(IS : 10262-2019, IS : 456-2000(R2016), IRC:21, IS 1199-1959 (R2018)	As stated in Mix Design for specific grade, structural component and method of concreting
Temperature of concrete at placing location	IS 1199-1959 (R2018)/ MORTH /Employer's Requirement Volume 5 Section 2 Cl 2.7.9	Temperature at placement point preferably should not be more than 32 degree Celcious (Between 5°C to 32°C), in no case more than 40^0

Density of Concrete	IS 516-1959 (R2018) / IS 1199-1959 (R2018)/ MoRTH	Where minimum density of hardened concrete is specified, the mean of any four consecutive samples shall not be less than the specified value and any individual sample result shall not be less than 97.5 per cent of the specified value.
Compressive strength of concrete in Trial Mix	IS 516-1959 (R2018) / MoRTH	(i) Mean strength from any group of four consecutive samples should be greater than Target Mean Strength (ii) Individual sample is not less than Target Mean Strength – 3.5 Mpa (iii) Difference between average and max / min = ± 15% (MoRTH Section 1700)
Compressive strength of concrete	IS 516-1959 (R2018) / MoRTH	(i) Mean strength from any group of four consecutive samples should be greater than Characteristic strength (ii) Individual sample is not less than Characteristic str – 3 Mpa (iii) Difference between average and max / min = ± 15% (MoRTH Section 1700)
RCPT	ASTM C1202	< 1000 Coulombs
Water Permeability Test	DIN 1048 Part 5 1991	< 10mm
Chloride migration Coefficient test	NT Build 492	< 2×10^{-12} m2/s for all structures except super structures (2×10^{-12} to 8×10^{-12} m2/s
Initial Surface Absorption Test	BS-1881Part 208	Max 0.25 ml/M2/sec for all structures except super structures (0.25 to 0.5 ml/m2/sec)
Total Chloride content	Employer's Requirement - Volume 5 Section 2 Cl 2.7.4.2 & 2.7.4.4	Max 0.1% by weight of cement (Prestress Structures), 0.2% by wt of cement (RCC Structures)
Total Soluble Sulphate Content (SO3)	Employer's Requirement - Volume 5 Section 2 Cl 2.7.4.2 & 2.7.4.4	Max 4%

10. Reinforcement Steel (Fe 550 D)

Carbon	IS - 1786:2008 (R2018)	Max 0.25%
Sulphur		Max 0.040%
Phosphorus		Max 0.040%
Sulphur + Phosphorus		Max 0.075%
Ultimate tensile strength		Should not be less than 600N/mm2
0.2 % proof stress		Min 550 N/mm2
Percentage elongation		Min 14.5%
Bend and Rebend		Should pass

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Mass per meter run (kg)		As per Table 1 of IS 1786

11. Rebar Coupler (Fe 550D)

Carbon	IS-16172:2014, Employer's Requirement Volume 5, Section 2 Cluase 2.6.6	0.40-0.45%
Sulphur		0.70-0.90%
Phosphorus		0.05% max
Sulphur		0.05% max
Silicon		0.10-0.40%
Total Slip		0.1% max
Ultimate tensile strength/Static tensile strength		min 125% Strength of Characteristic Strength of corresponding Reinforcement
Total Elongation at maximum force		min 3%

12. Concrete cover blocks

Compresive strength	IS 516-1959 (R2018) / MoRTH	Equal to the grade of concrete when used in particuar structure
RCPT	ASTM C1202	< 1000 Coulombs
Chloride migration Coefficient test	NT Build 492	< 2 x 10^{-12} m2/s for all structures except super structures (2 x 10^{-12} to 8 x 10^{-12} m2/s
Water Permeability Test	DIN 1048 Part 5 1991	< 10mm
Initial Surface Absorption Test	BS-1881Part 208	Max 0.25 ml/M2/sec for all structures except super structures (0.25 to 0.5 ml/m2/sec)
Total Chloride content	BS-1881	Max 0.1% by weight of cement (Prestress Structures), 0.2% to 0.3 % by wt of cement for RCC
Total Sulphate Content	BS-1881	Max 4%

13. Binding wire (MS ANNEALED WIRE 18 BWG)

Diameter	IS: 280 -1978, Employer's Requirement Volume 5 Section 2 Clause 2.6.5	Should not be less than 1 mm
Ultimate tensile strength		30-50 kg/mm^2
Elongation		Not specified

3. Using concrete mix for different part of monopile

Description	Cement Content Kg/ per m3	GGBS Content Kg/ per m3
Ready Mix Concrete M15	140	160
Ready Mix Concrete M30	230	260
Permanent Works (Approved Mix)		
Ready Mix Concrete M15	140	160
Ready Mix Concrete M30-Jetty Work	230	260
Ready Mix Concrete M45-Civil	200	330
Ready Mix Concrete M45-Pile Cap	220	355

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Ready Mix Concrete M45-Permanent Pile	215	350
Ready Mix Concrete M55 Pier	250	320
Ready Mix Concrete M55 Pier Cap	250	320

3.1 Concrete mix design for M-60 monopile

A-1 STIPULATIONS FOR PROPORTIONING			
(a)	Grade of concrete	:	M-60 Monopile
(b)	Type of Cement	:	OPC-53 Grade
(c)	Maximum nominal size of aggregates	:	20 mm
(d)	Cement content	:	Minimum - 400 Kg/m^3 (Table 2.7.2 Section 2, Volume-5), Minimum - 360 Kg/m^3 (Table 5, IS 456)
(e)	Proposed cementitious content	:	700 Kg/m^3
(f)	Water /Cementitious content Ratio	:	Maximum - 0.4 (Table 2.7.3 Section 2, Volume-5), Maximum - 0.4 (Table 5 IS 456)
(g)	Adopted water Cementitious content Ratio	:	0.24
(h)	Workability (MORT&H Sec1700,Clause1704.1 Table-4, IS 516 & project specification Table 2.7.4)	:	500mm - 600 mm flow

Design Mix Proportions (kg) for 1 m^3												
Cement		GGBS	MS	Water	20	10	C Sand	Washed Sand	Admx	CI	W/C ratio
SSD	320	350	30	168	464	461	195	454	7.70	0.125	0.24
WA					1.98	1.99	3.04	2.96			
Dry	320	350	30	206	455	452	189	440.56	7.70	0.125	0.24
Ambuja		JSW	Elkem	CP-30 Plant	Ulwe Hill, CIDCO NMIA (IGBK)			MS Infra	Master GleniumSky 8535	Conchem Sepocem TCCI	

3.2 Slump Test Result of Different Mix Design

Mix	Sump mm
M1	630
M2	620
M3	635
M4	640
M5	630

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3.3 **composition of mix M60 for monopile**

S.no.	Ingredient	Quantity kg/cum
1	OPC (Type V)	320
2	C Sand (150-400 micron)	195
3	Washed sand	454
4	GGBS	350
5	MS	30
6	Admixture	7.70
7	Water	168
8	20 mm agg.	464
9	10 mm agg.	461
10	CI	0.125

4. RESULTS AND DISCUSSION

3 Load test

Applied Load and result

Loading Stage	Percentage of Test Load	Bidirectional Load (Tons)	Unidirectional Load applied on Jack Assembly (Tons)	Minimum Holding Time (min)
0	0%	0	0	0
1	5%	250	125	30
2	10%	500	250	30
3	15%	750	375	30
4	20%	1000	500	30
5	25%	1250	625	30
6	30%	1500	750	30
7	35%	1750	875	30
8	40%	2000	1000	30
9	45%	2250	1125	30
10	50%	2500	1250	30
11	55%	2750	1375	30
12	60%	3000	1500	30
13	65%	3250	1625	30
14	70%	3500	1750	30
15	75%	3750	1875	30
16	80%	4000	2000	30
17	85%	4250	2125	30
18	90%	4500	2250	30
19	95%	4750	2375	30
20	100%	5000	2500	360
21	90%	4500	2250	10
22	80%	4000	2000	10
23	70%	3500	1750	10
24	60%	3000	1500	10
25	50%	2500	1250	10
26	40%	2000	1000	10
27	30%	1500	750	10
28	20%	1000	500	10
29	10%	500	250	10
30	0%	0	0	10

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Total displacement recorded

Sr. No.	% of Test Load Applied	Target Unidirectional Load (MT)	Applied Unidirectional Load (Ton)	Total movement* at Jack level (mm)
1	0%	0	0	0
2	5%	125	165	0.08
3	10%	250	273	0.26
4	15%	375	393	0.53
5	20%	500	501	0.73
6	25%	625	636	0.96
7	30%	750	758	1.15
8	35%	875	886	1.44
9	40%	1000	1027	1.8
10	45%	1125	1133	2.04
11	50%	1250	1250	2.44
12	55%	1375	1391	2.78
13	60%	1500	1537	3.11
14	65%	1625	1631	3.44
15	70%	1750	1773	3.73
16	75%	1875	1907	4.06
17	80%	2000	2035	4.41
18	85%	2125	2157	4.79
19	90%	2250	2266	5.06
20	95%	2375	2380	5.47
21	100%	2500	2576	6.32
23	90%	2250	2266	5.84
24	80%	2000	2031	5.53
25	70%	1750	1767	5.28
26	60%	1500	1542	4.92
27	50%	1250	1271	4.57
28	40%	1000	1028	4.05
29	30%	750	782	3.54
30	20%	500	566	3.51
31	10%	250	310	1.61
32	0%	0	0	1.4

Recorded displacement

All the three BDSLT (BIDIRECTIONAL STATIC LOAD TEST) test results didn't show much movement on the monopiles and almost all the load dissipated even before reaching the pile toe. In MPV2, the displacements of bearing plates in either direction are less than 5 mm, so load test was stopped after reaching the maximum unidirectional load of 2576 MT (planned 2500 MT).

Since the observed displacements (in either direction), was less than limiting displacement as explained above, extrapolation of the bidirectional plots is not performed and based on available data points, the equivalent top load plot was developed (see Figure A1).

Based on above test results, it may be concluded that for the given rock embedment length in the similar rock geology, the ultimate pile resistance can be considered as higher than the target test load of 5000 MT Strain gauges at level 1 to 4 showedexceedingly small values of microstrain sensing from the beginning of the test. Since these strain gauges were located within the liner level (placed around pile shaft), the sensed microstrain was exceedingly small and may be ignored. At smaller applied unidirectional test loads, microstrain readings of few levels (i.e., Level 11 strain gauges show lower strain than Level 10 strain gauges. However, beyond it, the microstrain readings were in good agreement with applied load and subsequent load distribution. The individual microstrain readings of all the four units (installed at each level/elevation) are uniform and no significant difference is seen between them.

The last strain gauge level was located at RL (Relative Level) – 23.7 m CD (Chart Datum) (i.e., 0.55 above the pile toe level). Hence, load sensed by the last level strain gauges can be load transferred to the end bearing. In other words, the mobilized end bearing resistance may be representative of the load calculated at the Strain gauge level SG13.

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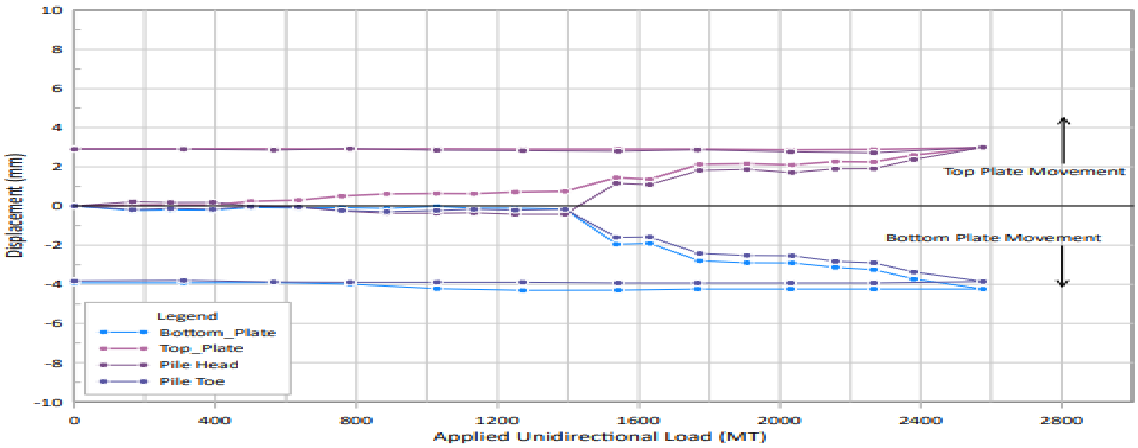
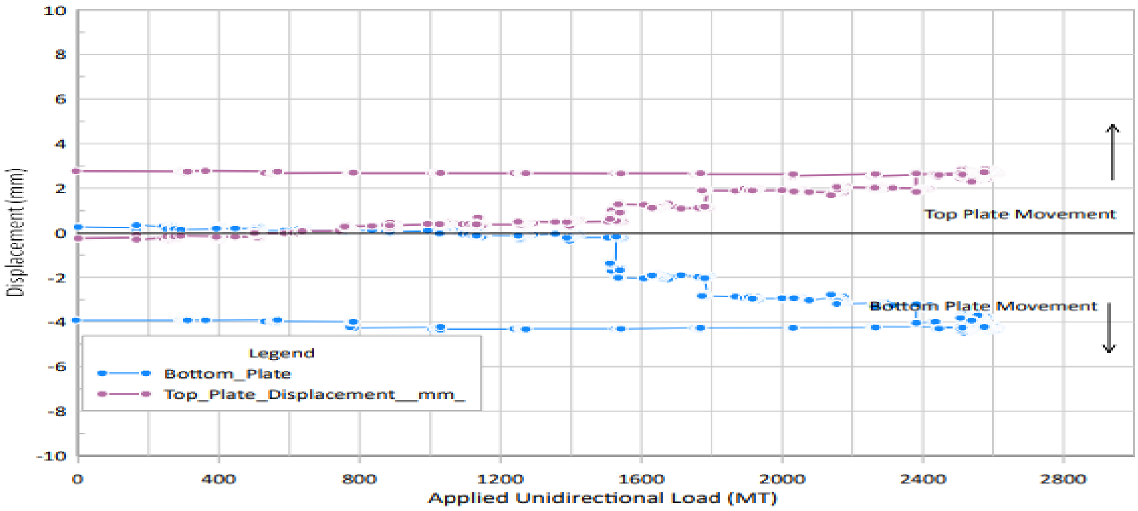
Applied Unidirectional Load		Level 1	Level 2	Level 3	Level 4	Level 5	Level 6	Level 7	Level 8	Level 9	Level 10	Level 11	Level 12	Level 13
MT	kN													
165	1619	-0.06	0.00	0.00	0.00	0.01	-0.03	-0.01	0.01	0.06	0.07	0.06	0.03	0.04
273	2678	-0.10	0.00	-0.01	0.00	0.04	0.00	0.06	0.18	0.31	0.31	0.29	0.17	0.22
393	3855	-0.13	-0.01	0.01	0.01	0.08	0.11	0.23	0.51	0.72	0.72	0.69	0.53	0.72
501	4915	-0.16	0.00	0.02	0.01	0.12	0.23	0.45	0.86	1.11	1.11	1.04	1.06	1.30
636	6239	-0.20	-0.02	0.02	0.02	0.18	0.39	0.70	1.23	1.47	1.50	1.53	1.78	2.10
758	7436	-0.22	0.00	0.04	0.03	0.24	0.52	0.94	1.50	1.76	1.88	2.21	2.67	2.96
886	8692	-0.22	0.00	0.07	0.04	0.31	0.76	1.31	1.90	2.17	2.38	2.99	3.69	4.06
1027	10075	-0.21	0.02	0.12	0.05	0.41	1.11	1.87	2.39	2.62	3.00	4.13	5.19	5.38
1133	11115	-0.19	0.02	0.14	0.07	0.46	1.31	2.25	2.67	2.92	3.41	5.01	6.34	6.38
1250	12263	-0.18	0.01	0.11	0.09	0.59	1.65	2.79	3.18	3.39	4.19	6.57	8.31	8.13
1392	13656	-0.13	0.04	0.27	0.11	0.71	1.92	3.21	3.55	3.77	4.79	8.09	10.06	9.49
1537	15078	-0.13	-0.01	0.16	0.13	0.83	2.15	3.60	3.96	4.17	5.50	9.68	11.90	10.89
1632	16010	-0.07	0.05	0.28	0.15	0.99	2.40	3.98	4.37	4.60	6.29	11.37	14.10	12.38
1773	17393	-0.05	0.07	0.22	0.17	1.15	2.56	4.31	4.69	4.99	7.76	13.04	16.08	13.60
1908	18717	-0.12	0.08	0.25	0.19	1.33	2.81	4.65	5.16	5.44	9.13	15.21	18.40	15.08
2035	19963	-0.04	0.06	0.26	0.22	1.48	3.04	5.01	5.55	5.84	10.11	17.19	20.54	16.48
2157	21160	0.01	0.13	0.35	0.24	1.63	3.25	5.41	5.98	6.35	11.04	19.27	22.69	17.84
2266	22229	0.01	0.09	0.33	0.26	1.73	3.44	5.68	6.29	6.67	11.71	20.56	24.01	18.66
2380	23348	0.02	0.08	0.47	0.27	1.88	3.70	6.02	6.76	7.15	12.70	22.58	25.94	19.82
2576	25271	0.29	-0.07	0.30	0.31	2.16	4.81	6.22	7.37	7.88	14.43	26.55	29.43	21.68

Load & recorded Displacement





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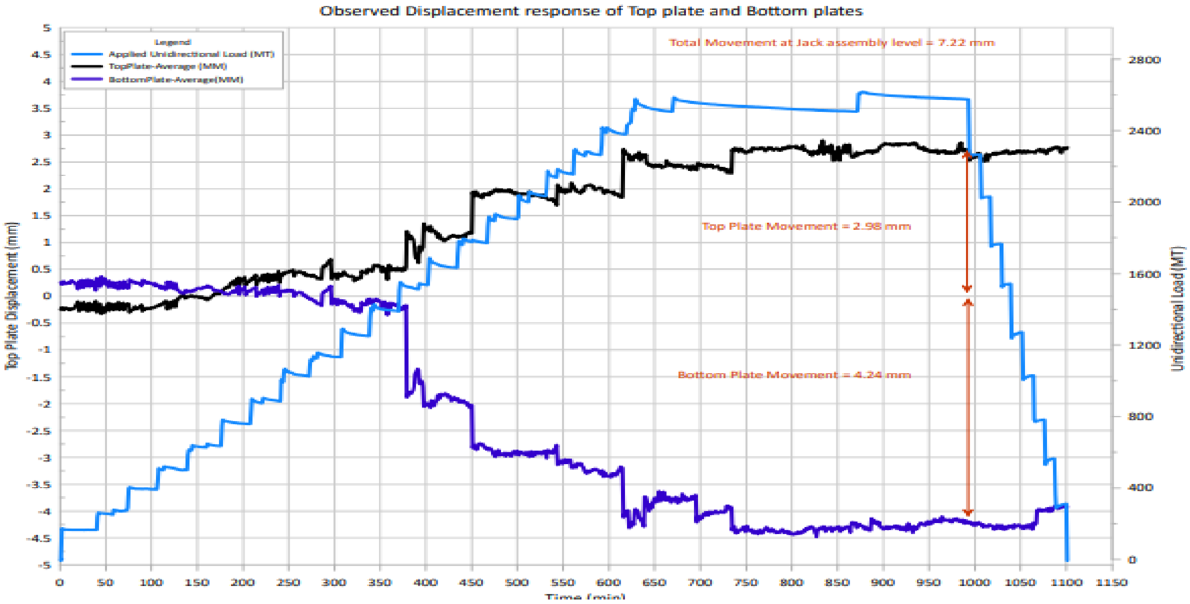
Load vs Displacement chart



Observed Load-Settlement Response during Test chart

During the maximum applied test load, at final stage, the pile above or below the jack did not experience any significant displacement showing mobilization of considerable shaft resistance. This also shows that pile has considerable balance capacity, and the observed shaft resistance is not ultimate shaft resistance (i.e., upon further loading beyond present test load it may reach to higher value further without crossing limiting displacement).

The end bearing load is derived based on the consideration that the load sensed by level 13 strain gauges will be transferred to the toe load/ End bearing (ignoring the shaft friction contribution below strain gauge level SG13). The end bearing stress is then derived considering the full shaft area contact at the toe. At the ultimate test load, the mobilized end bearing is seen to be 3.17 MPa.

Maximum end bearing load of 1558 MT was seen during ultimate test load (as calculated through strain gauge readings). However, it is imperative to note that with microstrain measurements saw during test, calculated mobilized end bearing resistance are not ultimate end bearing resistance and pile may show significant balance end bearing resistance for higher toe movements.

4.4 Lateral Load test:

Three lateral load testing on a test pile of 2500 mm diameter and 3200mm diameter drilled shaft at proposed location in Amarsons, Haji Ali and Main bridge.

The main aim of this load test was to proof-load the test pile to its maximum Lateral test loadof 700 MT todecide the ultimate pile resistance in lateral direction/ capacity and

establish a safe working lateral load on the pile for the proposed construction facilities.

The test pile was a drilled shaftwith a total embedment length below test level of 19.65m

Pile Dimension: 2500 mm

Pile Built- up Level: + 5.899 mCD.

Pile Toe Level (PT): -13.750 mCD

Pile Length (Pile Built-up level to PT): 19.65 m (Specified length below cut off level)

Working Load: 350 MT

Test Load: 700 MT

4.5 TESTING ON WORKING PILES:

4.5.1.CROSS-HOLE SONIC LOGGING:

CHSL testing follows the ASTM D6760 specifications. All the working piles were tested with Cross-Hole Sonic Logging. Generally, the CHSL tests were carried out 7 days after the date of concreting. Cross hole sonic logging tests conducted at Construction Joint Level +3.750m irrespective of the pile built-up level.

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A Cross Hole Analyzer device is connected to one transmitter &one receiver which is lowered in the pile shaft through access/sonic pipes and then pulled at a reasonably constant rate to continuously record the arrival time of pulses between the probes and record the loss of energy during transmission, as the probes are drawn up from the bottom to the top of the pile. This is a quality assurance program to ensure that the pile shaft is formed properly and is of targeted depth.

The probes shall be lowered to the bottom of the piles, brought to the same level and then pulled back. While pulling the probes, the CHA equipment continuously records arrival times versus depth graph. Optionally, data can be collected while lowering the probes from the top. The equipment shows if the probes are not at the same level while pulling or if there is a missing scan in which case the probes shall be adjusted by lowering one of the probes to match each other before the test is continued.

The CSL (Crosshole Sonic Logging) (Cross hole Sonic Logging) procedure inspects the piles' structural integrity, extent and location of defects, if any. At the receiver probe, pulse arrival time and signal strength are affected by the quality of concrete. Uniform concrete yields consistent first arrival times (FAT) with reasonable pulse wave speed and signal strengths. Non-uniformities such as contamination, soft concrete, and honeycombing, voids, or inclusions show delayed arrival times (FAT) with reduced signal strength. Here, the rating of the integrity considers the increases in "first arrival time" (FAT) and the energy reduction compared to the arrival time or energy in a nearby zone of good concrete. The wave speed obtained is a useful tool to evaluate concrete quality.

Wave speed is best found from the test results from the major diagonals. The wave speed is also affected by the age of concrete, plumb of the holes, reinforcement if any between the holes and hence many times the energy is considered a more important parameter in evaluating the results.

Table 4.3 .

Classification	FAT Increase compared to good concrete	Energy reduction compared to good concrete
Satisfactory / Good	0-10%	< 6 dB
Minor Defect	11-20%	< 9 dB
Poor/Flaw	21 to 30%	< 9 to 12 dB
Poor/Defect	>31%	> 12 dB

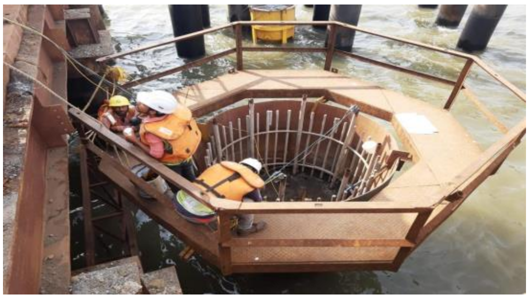

Fig CHSL testing in offshore monopile.

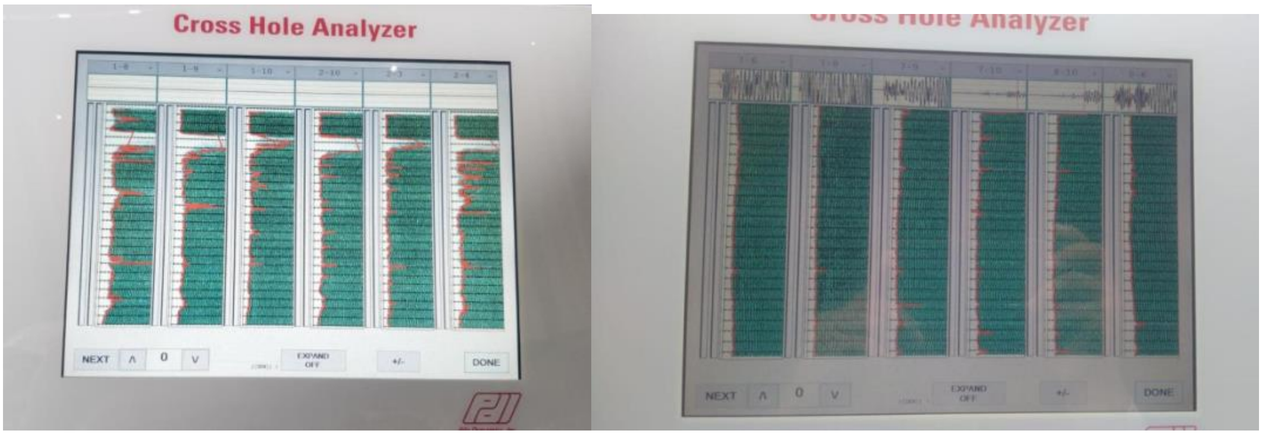

Fig Field report of CHSL showing some hazy signal

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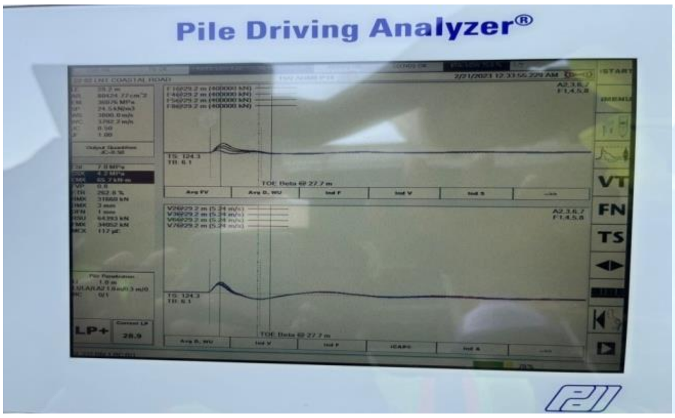
4.8 PILE DYNAMIC ANALYSIS (PDA) PDA test setup

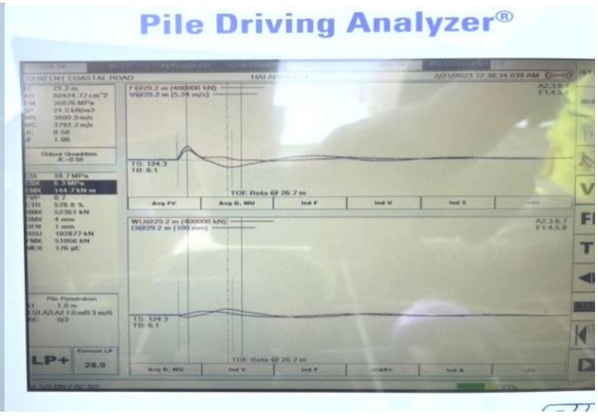
The testing is conducted by impacting the pile with blows of the hammer generally starting with a smaller drop height of 0.5m. This is to ensure the correctness of the data and the setup arrangements. Testing continues by increasing the hammer height by about 0.5m increment till the time the pile set or capacity reaches the required or limiting values.

For each hammer blow, the strain transducers measure strains while accelerations are measured by accelerometers connected on either side of the pile and the settlement/results can be viewed through the monitor with real time readings.




Fig Pile head concrete pou Prepared pile top for PDA. PDA test setup (offshore)fig PDA Sensors fixed at the platform level.





Results with height of fall 0.5m **Results with height of fall 1.0m**.

4.9.1 Liner driving in AGI:

Amarsons garden interchange boasts Basaltic bedrock with highly undulated bed formation created difficulties in liner placement and driving. Therefore, outer liners were used to nullify the effect of waves in monopile liner placing (wherever possible).

Extra beams were welded with the support piles to hold the liner in position and within tolerance. The gap between the outer and inner was filled with plug concrete to reduce the wave impact that disturbed the liner position earlier.

Fig Plug concrete between outer & inner liner to counter the wave impact on liner.

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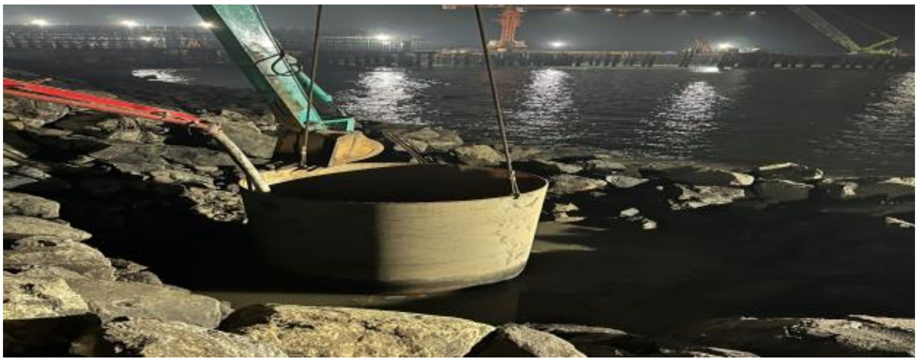


Fig Plug concrete in the outer liner to get a proper surface for inner liner.

Another main issue faced in AGI, is the liner tilt while drilling. Also, severe water loss occurred in piles which is countered byadded water input with the help of 2*75HP pumps. Liner tilt during drilling caused the BHA struck inside the borehole which took almost 30 days to retrieve. Several attempts were made to remove the borehole assembly, but everything ended up in vain. Finally, Hydraulic jacks were used to pull out the BHA assembly.

All the above challenges delayed the production rate drastically and almost all the piles in AGI ended up in excess concrete consumption (some piles consumed more than 150% of theoretical concrete consumption). Concrete loss saw at liner tip location almost all the piles. This pattern of concrete loss was inspected with the diver team, and underwater photos revealed a considerable gap between the liner toe and surrounding bedrock. The tremie pipes were kept immersed in the concrete upto 10metres to avoid the choking due to the sudden loss in the concrete level inside the monopile.

To avoid excessive concrete wastage into the sea, sandbags were placed around the pile liner at the bed rock levels. This in turn reduced the amount of concrete loss. Towards the fag end of the monopile construction, concreting was scheduled in hightide time to avoid the excess concrete loss as the wave pressure arrested the concrete loss considerably.

4.10 Borehole collapse while drilling:

In haji Ali interchange and Mainline bridge, 3 monopiles were collapsed at the time of drilling. The main reason for such a collapse is the pile liner which was not driven upto the design level.

Once the collapse is confirmed (the drilling levels stayed the same even after drilling for 4-5 hours), the RCD unit along BHA was de-mounted and the pile was then left idle for a couple of days and then Liner driving done upto the possible level. Once the collapse stopped, the pile drilled upto the roe level and concreted.During the drilling of a collapsed pile, the slush from the collapsed zone clogged the outlet pipe of cutting drum (BHA's part) and then the entire BHA was removed and dismantled into pieces to clear the outlet pipeline.


Fig RCD/BHA dismantling from Pile.   Fig Slush blocked the drum outlet.

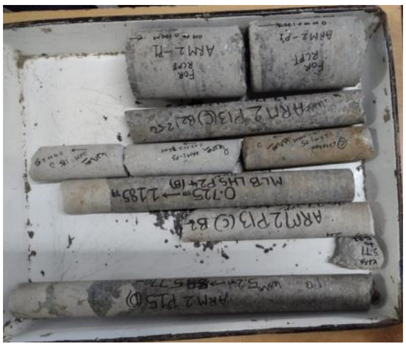

Fig Clogged Drum outlet   **fig Outlet pipeline after slush removal.**

4.11 Anomalies in CHSL test:

Each monopile was tested for CHSL as per the contract specifications. In the first stages of monopile construction, hazy signals were seen in the longer corridors of sonic tubes, while the adjacent tubes showed satisfactory results and were ruled out as debonding between the Concrete and CHSL tube.

As the same pattern continued in successive tests, concrete coring was done. The cores retrieved from the pile were tested for water permeability and visual examination of cored samples. In some pile core samples, there was a continuous breakage in the cores at the same levels of defects seen in the CHSL report.The piles with such defects are then tested for water permeability and optical televiewer & hydro-jetting. The test results showed the presence of weak concrete/honeycombing in the defect levels of CHSL reports.

Epoxy & Microfine grout was used to fill those piles with defective cores and in some piles 32mm dia rebars were inserted into the core holes and then grouted.




4.12 Cost estimation of the work

Table 4.15 abstract

Sl.No	Properties	As per Present ITP	If revised	Qty	Cost (Ind. Rup.)
1	Deleterious material	1/Source	1/3 months	32	14080
2	Crushing value	1/15 days	1/6 months	16	5600
3	Soundness	1/Source	1/6 months	16	16000

4	Petrographic examaination	1/Source	1/Year	6	
5	Alkali Reactivity	1/Source	1/6 months	16	7040
6	Chloride & sulphate content	1/Source	1/3 months	32	16640
7	OPC	1/Source	1/6 months	4	7600
8	Admixture	1/Source	1/3 months	20	37200
9	Microsilica	1/Source	1/6 months	4	5400
10	corrosion inhibitor	1/Source	1/6 months	4	15000
11	Durability	1/ 6 months	1/1000 cum	200	3291000
12	Reinforcement steel	1/1500 MT	1/500 MT	40	20000
13	Strand	1/Source	2/Lot	36	90000
	Total amount of the work is				RS=3525560

5. CONCLUSIONS

The construction of Monopiles delayed from initial schedule due to various issues like heavy downpour, rough sea and construction & Quality related delays. Executing the monopiles in 3 different locations with a totally different geology is itself a biggest challenge as the drilling rate and method in each loaction varies from another.

The test results of 3 piles with maximum defects in AGI,HAI & MLB shows that the pile is capable of taking the loads from the superstructure as the piles where tested with an impact load approximately 10-15% more than the design load. Many of the delays could have been eliminated if the liner driving was done upto the design level and a proper mix design of the concrete.

Though the construction works posted many challenges, the construction of new piles never stopped which added to the further delay in the schedule.

- 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 defor-mation 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 performance.

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