EXPERIENCE FROM A LARGE SCALE DRIVEN PRECAST PILING PROJECT IN INDIA

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A large scale piling work is undertaken in India for an important fast track industrial project. Thousands of solid square precast piles are driven to large depths of around 50m. Soils encountered are of alluvial origin with very soft clays varying from 10 to 18m thickness at the top followed by medium dense to dense and occasionally very dense sand, stiff clay and finally dense to very dense sand layer as the founding layer. Recent site grading with around 4 metres thick hydraulically filled river sand necessitated driving the piles to the bottom sand layer to avoid problems due to negative skin friction and to achieve optimum pile capacity. A process approach suggested by Bell et al (2002) to promote better understanding of the pile behaviour, pile capacity, and parameters assumed at design stage for static capacity and for wave equation analysis is followed with an aim to bring in effective economy in terms of pile length and pile installation speeds. Soil investigation work is also undertaken in stages to cut down pile wastage. A judicious combination of PDA and static pile load tests is followed. PDA tests at end of driving and also at various stages of recovery of pile capacity are undertaken to understand the pore pressure dissipation phenomena. The paper attempts to present some of the valuable experiences that would go to establish precast pile as a very effective alternative to driven cast-in-situ piles and bored piles which are very common in this part of the world despite their limitations in terms of economy, speed and certain geotechnical aspects.

Introduction

Pile foundations play an important role in any major project. They are required based on the subsurface conditions, foundation loads and the design criteria. Precast concrete driven pile is one of the best pile foundation systems in terms of quality, productivity and cost effectiveness. With proper pile design based on adequate soil investigation data, good quality control system for pile casting, deployment of suitable piling equipment for pile installation, and systematic pile testing, a good foundation system can be achieved.

A large scale precast concrete piling work is undertaken in India and this paper presents some of the valuable experiences that would go to establish precast pile as a very effective alternative to driven cast in-situ piles and to some extent bored piles, which are very common in India despite their limitations in terms of economy, speed and certain geotechnical aspects.

The Project

Reliance Industries Limited (RIL), a major industrial house in India, is setting up an Onshore Gas Terminal near Kakinada, Andhra Pradesh on the east coast of India. Natural gas is proposed to be brought from the offshore KGD6 basin via a submarine pipeline to the Onshore Terminal for processing, storage and transportation by a cross country pipeline. Larsen & Toubro Ltd – a major engineering and construction company in India is assigned the task of construction of the project. The work reported in this paper is the result of a seamless teamwork among client, consultant and contractor.

The project is spread over an area of 80Hectares. The original ground level of the site is about 0.7m above Mean Sea Level (MSL), which is about 1.45m above the Chart Datum (CD). As the site is subjected to river floods and tides under intense cyclonic rainfall the ground is raised to a level of 4.2m above MSL by dredging and hydraulic filling of river sand. The site is enveloped by a 3.0km long peripheral bund having a top level of 5.7m above MSL. The project site comprises of the following two areas:

- Onshore Terminal (OT) area with heavily loaded units like slug catchers, TEG Dehydration plants, MEG Regeneration Units, Turbo-generators, Storage Tanks (up to 32 m diameter and 15 m height), pump houses, utility buildings and pipe racks.

- Infrastructure area (Infra) with ware house, pipe shop, workshop, administration building, helipad hanger and other buildings and yards.

Regional Geology

Due to the pronounced easterly drainage of the Indian Peninsula, wide belt of river borne alluvium is formed on the east coast aided by great deltaic deposits at the mouth of Mahanadi, Godavari, Krishna and Kaveri rivers (Karandikar 2006). The project site is on the Kakinada shore, which is in the Godavari river delta formed by 'Recent' river alluvium. There is a large variation in the subsurface stratification and the geotechnical properties. Clayey deposits are found to be sensitive and highly compressible.

Soil Investigation & Subsoil Data

Initial soil investigation was carried out in 2003 before the reclamation work. It comprised of 17 Boreholes, 17 Static Cone Penetration Tests, 4 Plate Load Tests, 4 Block Vibration Tests and 17 Electrical Resistivity Tests. In addition, 18 numbers of confirmatory boreholes were also carried out during the initial stage of the project. The typical subsoil profile is given in Fig. 1. The generalized stratification and the properties of the subsoil are given in Table 1.The properties of very soft to soft clay layer (layer-2) and stiff to very stiff layer (layer-5) are given in Table 2.The ground water label was at the top of the layer-2.

Soil below the peripheral bund was improved by the installation of Prefabricated Vertical Band Drains (PVD) and stage construction of the bund, leading to gain in shear strength and prevention of shallow and deep seated failures. The settlements observed were between 171mm and 970mm with an average value of 482 mm.

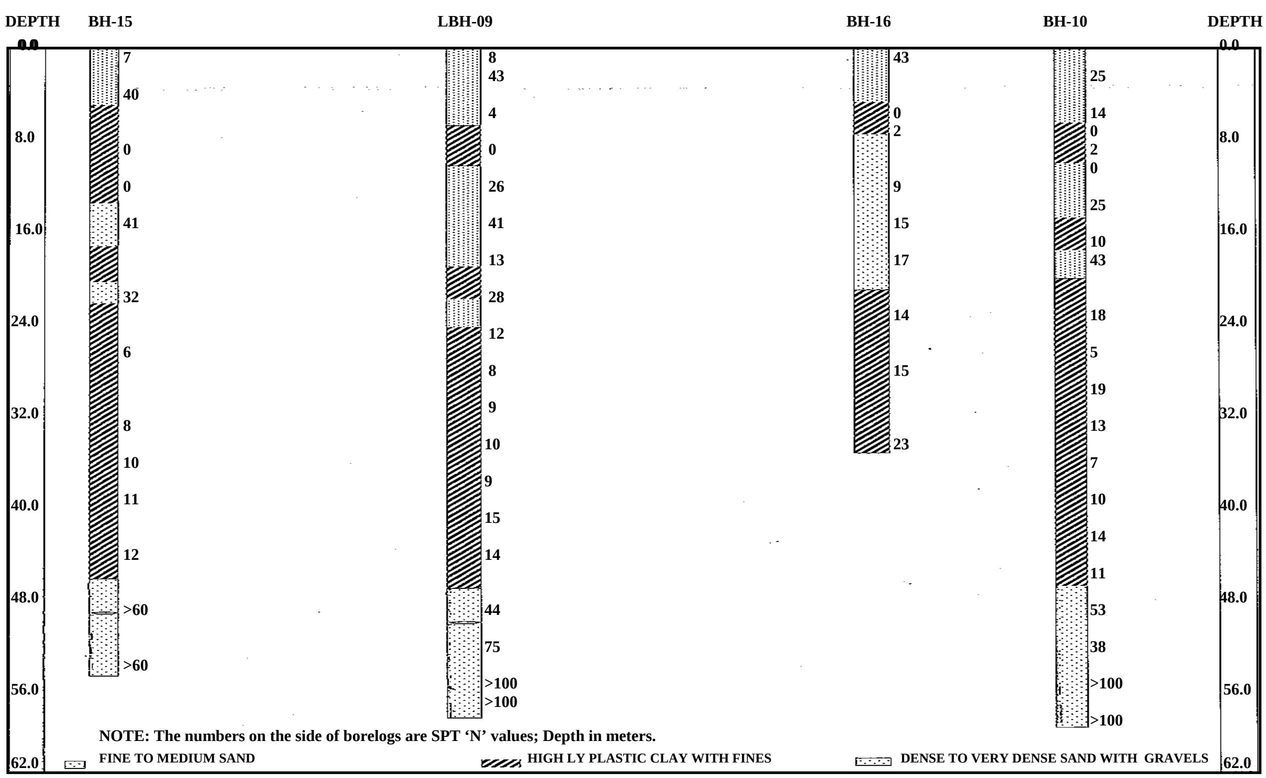

Fig.1 Subsoil Profile

Table 1: Stratification and subsoil Properties

Layer	Soil Type	Thick-ness m	Density kN/m^3	Avg. SPT N	c_U kN/m^2	ϕ Deg
1	Hydraulic River Sand fill	4.0-5.0	16	10	0	31
2	V.Soft to Soft Grey Clay	4.0-10.0	6	2	10	0
3	Loose Silty sand	2.0 - 8.0	6	6	0	29
4	Medium Silty sand	8.0-10.0	8	20*	0	32
5	Stiff to very stiff silty clay	20.0-25.0	7.5	14	70	0
6	Dense to very Dense Silty sand	5.0 – 10.0	10	80	0	36

* In three boreholes N values of 49 to 98 was recorded in Layer-4 indicating very dense sand.

Table 2: Properties of Layer-2 and Layer-5

Sl.No	Properties	Layer-2	Layer-5
1	Submerged density of γ_{sub}, kN/m^3	6.0	7.5
2	Specific Gravity G	2.78	2.78
3	Natural moisture content wc%	67	56
4	% of Clay content	50	58
5	Liquid Limit LL,%	70	70
6	Plasticity Index PI,%	40	40
7	Compressibility Index, CI	0.54	0.54
8	Coeff. of volume compressibility,m_v m^2/kN	0.0014	0.0003
9	Coeff. of Compressibility c_v, m^2/yr	1.0	100

Foundation Systems Studied

Main foundation systems such as shallow footings, rafts and various types of pile foundations were examined. Since the shear strength of the very soft-to-soft clay layer is only 10.0 kPa and its extent is to a considerable depth, shallow footings and raft foundations were not found to be appropriate. The tight construction schedule did not permit ground improvement works prior to the foundation works. In addition, the gain in strength was not expected to be adequate to adopt shallow foundations for majority of the structures involved.

Precast Driven Pile Foundation Systems

After deciding on the requirement of deep foundations for all plant structures, and considering the pros and cons of various piles, precast driven concrete piles were recommended, for the following reasons:

- consistently good quality pile can be produced.
- higher bearing capacity compared with bored pile can be obtained
- negative skin friction can be minimized if not avoided by application of slip coat
- pile cap work can start immediately after driving
- overall, an efficient pile can be installed.

In the case of bored cast in-situ and driven cast in-situ piles, the required pile sizes were comparatively larger and around 500mm to 600mm diameters. In the design, negative skin friction was to be considered. Also, execution of long length bored piles would face issues like achieving design toe resistance, congestion of site due to more number of equipment and constraint in the immediate execution of subsequent activities.

For the driven cast in-situ piles, 48.0m to 50.0m pile length were not feasible due to equipment limitations.

The concrete grade of M35 as per the initial pile design was upgraded to M50 to adopt long pile sections, to facilitate early de-moulding, to have good strength during pile handling, to enable hard driving and to have a flexibility in hammer selection. Factory made mechanical pile connectors (Sure-Lock Pile Splice) were preferred to welded pile joints for reasons of consistent high quality of joint, speed of installation, minimizing equipment standby time,

reduction of workforce, etc. This has lead to a significant improvement in quality and productivity.
Integrated piling rigs with hydraulic impact hammers were mobilized in place of conventional Metco rigs with drop hammers to help the project with speed, flexibility, drivability, and quality of installation.

Process Approach

Evaluation Of Static Pile Capacity

The process approach suggested by Bell et al (2002) has a very sound basis as it systematically follows and utilizes the information obtained from static pile capacity estimates, wave equation analyses, driving of probe piles, pile driving analyzer tests and static load tests. The authors had decided to adopt this approach in the current project with an aim to provide a reliable and cost effective pile foundation.

In the first step theoretical static pile capacity was calculated for pile sizes of 270mm to 400mm square, using the parameters from the soil investigation data and based on Indian Standard Code of Practice (IS:2911 Part1/Sec.3). The minimum specified factor of safety is 2.5. Based on the foundation loads, two pile sizes of 270mm square and 350mm square were selected. Theoretical safe capacities of 1000 kN for 350mm square piles and 550 kN for 270mm square piles were firmed up considering soil shear strength, anticipated negative skin friction of upper soft clay layer and the serviceability limit of settlement under design loads. The pile was designed to terminate in the very dense to dense sand strata and the pile length was initially kept as 50.0m.

Selection of Hammer Based on Preliminary Wave Equation Analysis

In the second step, preliminary wave equation analysis was done to select a suitable hammer and input energy for safe installation of piles to required depth. From this analysis, both compressive and tensile stresses in the piles were checked. Appropriate range of required driving energy was determined so that the piles are not damaged during driving and also to ensure that the blow counts are reasonable. Analyses were performed on BSP 357 hydraulic impact hammers for both 5.0 and 7.0ton ram weights. The input data considered for the preliminary analysis were as follows:

Ram weight: 50 kN and 70 kN
Helmet weight: 10.0kN
Hammer cushion:
    hard wood-150mm thick
    elastic modulus- 5000MPa
Pile cushion:
    plywood- 50mm thick
    elastic modulus 206MPa
Hammer efficiency: 0.85
Soil quake:
    shaft - 2.5mm
    toe - 4.5mm
Soil damping:
    shaft - 0.6 s/m
    toe - 0.5 s/m

Soil set up factors of 2.0 and 1.20 were used for clay and sand respectively to represent soil resistance during driving (SRD) along the shaft. The soil setup factor, f_s, is defined as the ratio of long term capacity over end of driving soil resistance. However, no reduction factor was used for end bearing. Over all, the ratio of SRD to R_{ult} was 0.61. Soil quake, soil damping and set up factors were selected following the guidelines given in GRLWEAP manual.

In the drivability analysis a higher ultimate soil resistance, R_{ult} of 3000kN (equal to 3 times design load) was chosen. Accordingly, equivalent SRD value obtained by applying 0.61 factor to R_{ult} was used in the analysis. Hammers with ram weights of 5.0 ton and 7.0ton were selected for driving 270mm square and 350mm square piles respectively. From the results of the analysis hammer drops of 0.2m for driving the piles through layers 1, 2 and 3, 0.5m to 0.60m for layers 4 and 5 and 0.60m to 0.80m for layer 6 were recommended. For pile termination at depths above 50.0m, a set of for 60mm to 85mm for 10 blows for a hammer drop of 0.8m to 1.0m was recommended. A typical bearing graph between the blow count and the pile capacity is shown in Fig. 2.

Pile Casting

Piles were cast at the project site in 18m, 16m and 16m sections to make up for a 50m total pile length. Bituminous slip coat was applied to the top 16.0m length of piles to minimize the drag down forces on pile due to consolidation of upper soft clay layer under the load imposed by

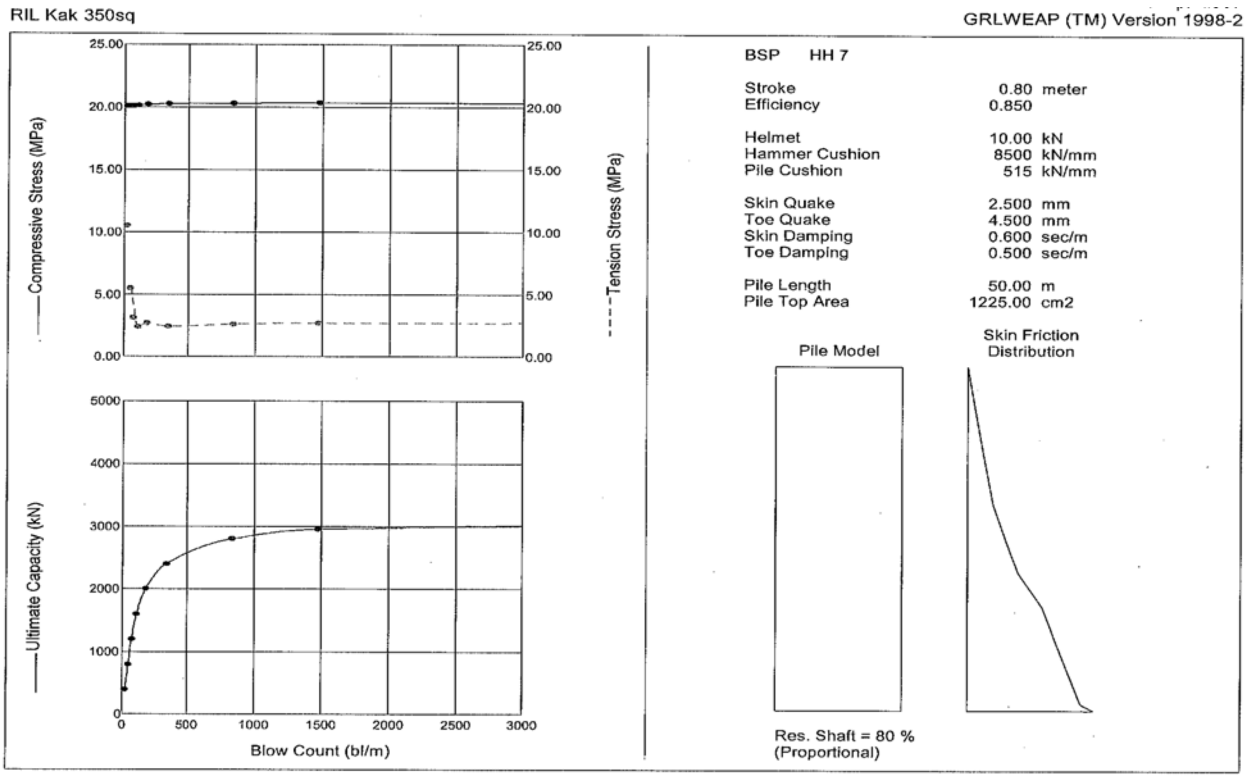

Fig. 2 Typical Bearing Graph from GRL WEAP

the recently filled river sand. Piles were recommended to be driven after 28 days of casting.

Installation of Probe Piles

In the third step installation of probe piles was taken up. Initially 45 probe piles in OT area and 20 probe piles in Infra area were proposed on non-production piles (non-job piles). However, due to the time constraint, the number of probe piles was curtailed to 20 in a 100m grid in the OT area. Some probe piles were also converted to production piles by installing them at the respective locations.

The first objective of the probe piles was to drive piles to the design depth and to establish safe driving criteria, with the mobilized pile driving system. This was systematically observed using Pile Driving Analyzer (PDA) tests on probe piles. Actual driving energy transferred to the pile, and the actual driving stresses in the pile were monitored.

350mm square piles were chosen as probe piles as they comprise about 77% of the total number of production piles. After monitoring the pile drivability, the maximum drop was reduced from 1.0m to 0.80m. The efficiency of the pile driving system was observed to be between 0.55 to 0.85 (Fig.3). It depends on the condition of the hammer and pile cushion and also on the condition of the hammer. Hard wood of 150mm and 100mm thickness were used as hammer cushion and pile cushion, respectively. From PDA tests it was observed that the maximum compressive stress in the pile was less than 28 MPa while the maximum tensile stress was less than 3.3 MPa. These are well within the permissible values of 40.0MPa and 5.0MPa, respectively. A typical stress graph obtained is shown in Fig. 4. and Fig. 5 shows the details of hammer drop adopted and blow count encountered over depth during driving.

Based on the PDA test results on the probe piles, the set criteria from the preliminary drivability study was found to be very conservative and it was modified to 100mm to 150mm for 10blows for 7.0ton ram weight with a drop height of 0.80m.

The second objective of probe piles was to understand the soil resistance to driving as well as the recovery of pile capacity with time. This was achieved by conducting PDA tests at the time of pile driving (EOID) and at various times

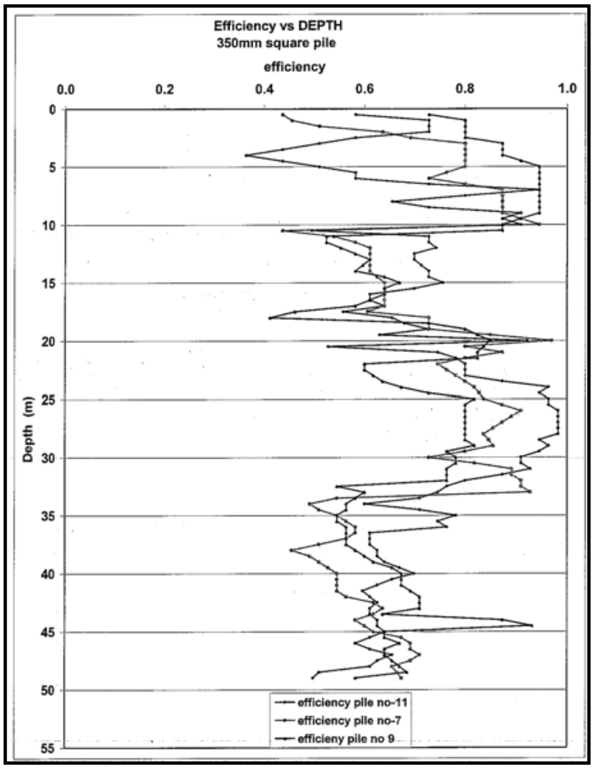

Fig. 3 Efficiency of driving system over depth

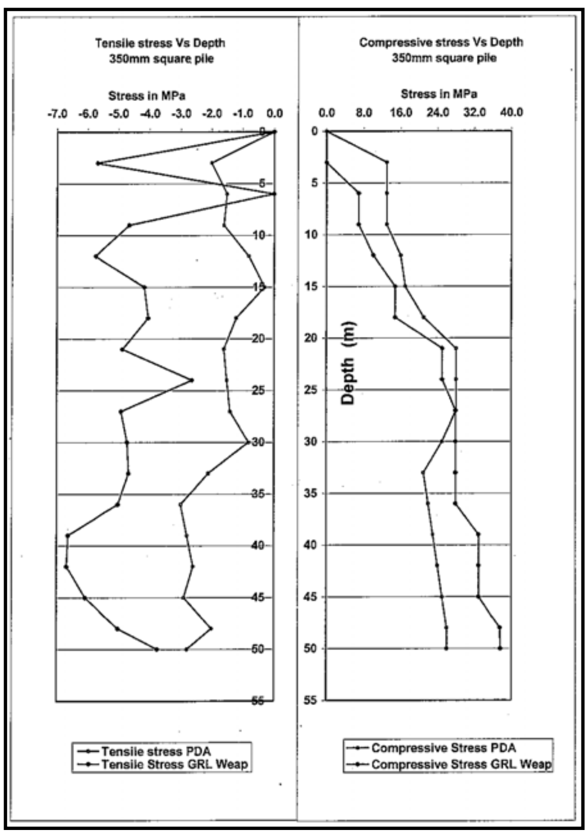

Fig. 4 Stresses in pile from GRL WEAP & PDA

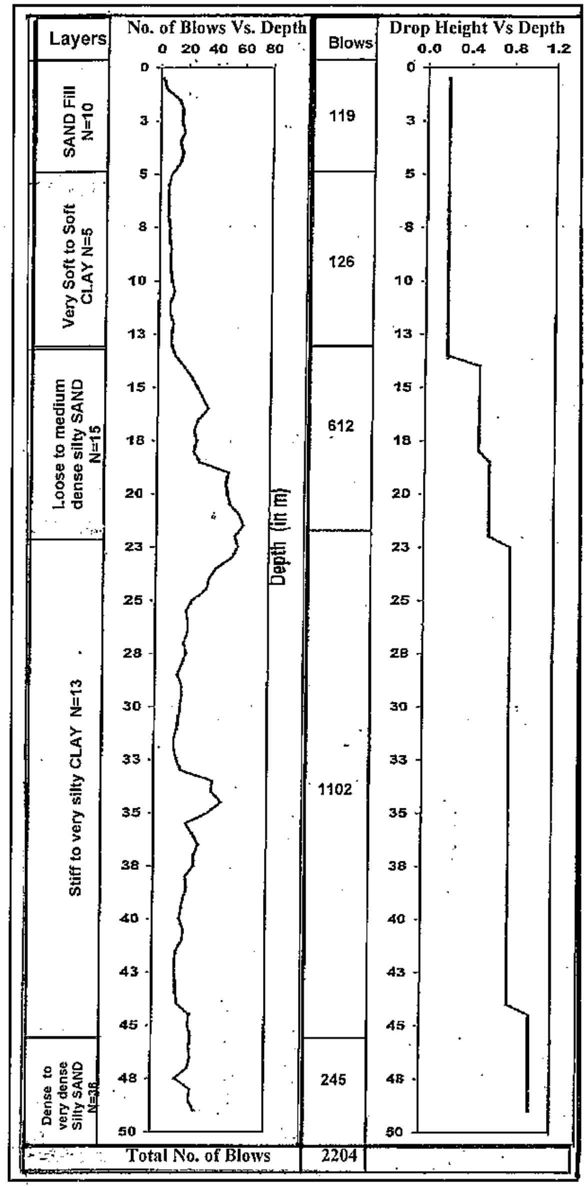

Fig. 5 Typical driving record

by restiking the piles. The restrikes were conducted on 3rd, 7th, 15th and 30th day from the date of pile installation. As this exercise was affecting the project schedule and also resulting in long marching of piling rigs the restrike tests were limited to around 30 numbers. Fig. 6 shows the data of pile capacities obtained from PDA at EOID and at restrikes and Table 3 gives the pile capacities at EOID and at restrikes conducted on various piles. The table also gives the overall setup factors.

The pile capacities obtained from PDA restrike was further investigated by CAPWAP analysis and distribution of shaft capacity over depth and end bearing were obtained.

Table 3: Pile capacities at EOID and at Restrike

Pile No.	Pile Capacity in kN		Restrike time (days)	Overall setup factor
	EOID	Restrike		
WB-3	1520	2082	2	1.37
WB-5	1560	3000	2	1.92
P2V2	1514	1650	3	1.09
P3V1	1484	1775	3	1.20
P4V3	2276	2985	3	1.31
P2V3	1806	2118	4	1.15
P3V3	1528	1849	4	1.21
P2V3	1806	2200	7	1.22
P2V2	1514	1800	8	1.19
WB-13	1260	2798	28	2.22
P2V2	1514	3103	30	2.05
P2V3	1806	3255	31	1.80
4/1-4/V-4	2273	4601	39	2.02

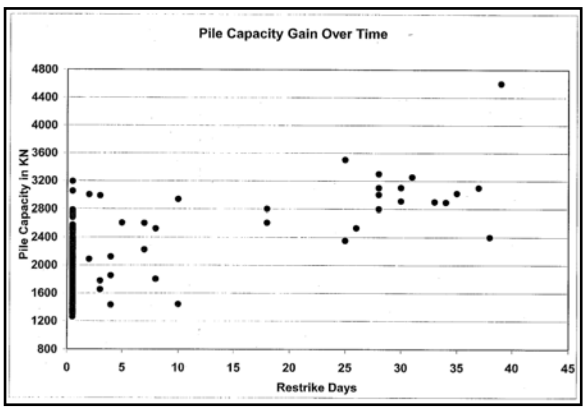

Fig. 6 Pile Capacity gain over time

Static Load Testing

Static load tests were also carried out on selected probe piles on which PDA monitoring and restrike had been done. Test piles were loaded up to three times the design load or up to failure, whichever is earlier. The results indicate that the settlement is of the order of 16.4mm to 28.2mm for a maximum applied load of 3100kN. As per the IS code (IS: 2911-Part 1/ Sec 3) the pile design load is minimum of the following criteria:
1)Two thirds of final load corresponding to 12mm pile settlement.
2)Fifty percent of final load corresponding to a settlement of 10 percent of the pile diameter.
3)Safe load obtained by applying a factor of safety 2.5 to the ultimate capacity.

Accordingly, two thirds of the load corresponding to 12mm pile settlement would be the governing criteria. The load corresponding to 12mm settlement is 1900 kN. Therefore the allowable load on pile is 1167 kN after allowing 100kN for the negative skin friction. Since the failure load was not reached, the ultimate pile capacity was found to be 3500kN to 4250kN by extrapolation.

DISCUSSION OF TEST RESULTS

After the verification of the drivability, the maximum hammer drop of 0.8m was found to be adequate to drive the pile to the design penetration. The set criteria for pile termination was relaxed to a pile penetration of 200mm for 10 blows.

Pile length(s) for the whole project were fine-tuned and the optimum pile length was selected as 48.0m. In view of the large quantity of piles (around 15000 numbers), maximum length criteria (i.e., target penetration) was adopted except where hard driving was encountered at depths less than 48.0m. After gaining sufficient confidence in terms of pile driving according to the site conditions such as pile rundown in the top soft layer (Layer-2) and hard driving through the intermediate sand layer (Layer-3 and Layer-4) and driving the piles to the target penetration, the pile casting lengths were modified to 4 numbers of 12m sections. This had helped the project in many ways, i.e., during casting, lifting, transporting, pitching and driving even though the number of pile connectors (Joints) had increased from 2 to 3 per pile.

The pile capacities measured in PDA at EOID varied widely from 1260kN and 2347kN. The recovery of pile capacity over a period of around 30 days is considerable and generally falls in a narrower range of 2900kN to 3100kN. The gain in pile capacity after 30 days does not seem to be significant based on the limited restrike data (Fig.6). More number of restrikes would probably confirm this observation. Higher uncertainty of EOID based pile capacity were reported by Rausche, et al., (2003) and hence expressed that it must be used with high factor of safety.

From Table 3, it can be seen that the overall setup factor varies from 1.8 to 2.22 over a period of around 30 days. Rausche, et al., (1996) have determined setup factors in a variety of soil types and have recommended 2.0 to clay and 1.2 to sand-silt. It was reported by Saxena, et al., (2004) that pile setup is predominantly derived from skin friction rather than end bearing. The same was observed in this project site

The set values obtained during the PDA were used to interpret the pile capacities using the GRL WEAP and are given in Table 4

Table 4: Pile Capacity using Field driving data and GRL WEAP Bearing Graph

Description		Set for 10 blows	Ram weight in ton	Hammer drop in m	Capacity in kN from GRL WEAP
EOID		180	7	0.8	1500
Restrike	3-4 days	70	7	0.8	2000
	7-8 days	63	7	0.8	2150
	28-30 days	35	7	0.8	2500

The CAPWAP results indicate that the pile shaft frictional resistance is 74% to 87% and the end bearing resistance is 13% to 26% of the pile capacity.

The frictional resistance distribution from CAPWAP was used to back calculate the soil properties. The adhesion factor, α for the layer 2 (very soft to soft clay) was found to be 0.42 to 0.74 and for the layer 5 (stiff to very stiff clay) 0.31 to 0.46.

The simulated static load test results obtained from CAPWAP analysis indicate that the settlement is 31.0mm for 3000kN load on pile. The load for 12.0mm settlement is 1595kN. These values are generally in agreement with the load settlement data obtained from the static load tests. The settlement at test load of 3030 kN is 22mm and the load corresponding to 12mm settlement is 1900kN.

The results summarized in Table 5 indicate that with a safety factor of 2.5 to the ultimate capacity (3000 kN-3800 kN), the safe load on

Table 5:Summary of pile capacity

Sl. No.	Description	Frictional Resistance in kN	End Bearing in kN	Total Pile Capacity in kN	Remarks
1	Theoretical Static capacity	3200	600	3800	Pile capacity (R_{ult})
2	Drivability Analysis SRD	1654	664	2318	$\frac{SRD}{R_{ult}}$ =0.61
3	*PDA-EOID			1773	
	*PDA Restrike				
	3rd day			2082	
	7th day			2501	
	30th day			3038	$\frac{EOID}{Restrike}$ =0.58
4	CAPWAP	2420	580	3000	
5	Static Load Test			3030	Test Load

*Average value

pile would be in the range of 1200kN to 1520 kN. However, two thirds of load corresponds to 12mm pile settlement, which is one of the criteria for safe load on pile is close to the design load of 1000kN.

The ratio of pile capacities measured in PDA at EOID and Restrike (around 30 days) was found to be around 0.58 (Table 5), while the overall ratio of SRD to R_{ult} used in the drivability analysis was 0.61.

INSTALLATION OF PRODUCTION PILES

After the above probe piles and field testing, the installation of production piles was started with better confidence. So far around 7100 piles had been installed. The length of piles was optimized from 50.0m to 48.0m. Every day 2 PDA tests are being conducted out of 100 piles installed per day. So far 6 static pile load tests had been conducted out of 16 tests planned. Further optimization of pile length is possible with more data coming from the tests.

CONCLUSIONS

1.The process approach adopted in this project for the pile design and installation brought the following benefits:

- Economical pile foundation system
- Reduction in the uncertainties of pile driving
- Improved confidence in the pile behaviour
- Enabled smooth operation of piling activities
- Selection of appropriate and safe driving mechanism

2)The ratio of pile capacities measured in PDA at EOID and Restrike after 25 days was found to be around 0.58.

3)The recovery of pile capacity over a period of 25 to 30 days seems to be very significant and nearly complete. However, more data is required to arrive at a definite conclusion.

4)Monitoring of pore pressure by piezometers after driving the pile will enable establishing the correlation between pore pressure dissipation and the gain in pile capacity.

5)By adopting process approach, a very reliable and cost effective pile foundation can be achieved. Hence this foundation system can be adopted with more confidence than bored piles and driven cast-in-situ piles.

6)Through wider application of process approach for the driven precast pile foundation, it is hoped that this approach would be specified in all driven piling codes.

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REFERENCES

GRL WEAP™ (Wave Equation Analysis of Pile Driving), GRL WEAP Program Procedures and Models-1998).

IS: 2911 (Part 1/Sec 3) - 1985 Indian Standard Code of Practice for Design and Construction of Pile Foundations: Concrete piles – Driven Precast Concrete Piles.

KENNETH BELL; JOHN DAVIE and JOSE CLEMENTE; GARLAND LIKINS; 2002. Proven success for driven pile foundation. International Deep Foundation Congress.

Likins, G., Rausche, F., Thendean, G. and Sivikin, M., (1996). CAPWAP Correlation Studies.

Rausche, F., Brent Robinson and Garland Likins .,2003. on the prediction of long term pile capacity from end-of-driving information.

Report on Foundations for Onshore Terminal Structures-May 2006 by KARANDIKAR D. V. and Associates.

Saxena.D.S., Saxena, A. and Rwebyogo, M.F., (1998), Evaluation of Pile Freeze for Driven Piles in Central and Southwest Florida.