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# 9th Conference on Deep Foundation Technologies for Infrastructure Development in India

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Pre-Conference Workshop: Design, Construction, and Applications of Continuous Flight Auger (CFA) Piles in India on November 14, 2019








## Conference Proceedings

# Advances in Deep Foundations, Earth Retention Systems and Ground Improvement

**A Barrier Wall for the New Dam at the Godavari River - Installation with a Monsoon Break**

*Peter E. Banzhaf*

1

**Alternative Solutions to Replace Diaphragm Walls for Shallow Depths**

*Franz-Werner Gerressen and Thomas Vohs*

12

**Back Analysis and Performance of a Cantilever Portion of Excavation in Mixed Soil Profile**

*C. Vimala, Senior Geotechnical Engineer and Madan Kumar Annam, Head of Engineering*

22

**Design of Deep Vertical Circular Diaphragm Wall Using Finite Element Analysis**

*Sharath C. Kudumula, Sr. Geotechnical Engineer and Suresh Kumar Gupta P.*

28

**Effect of Elastic Non-Linearity of Soil and Pile on Uplift Capacity of Transmission Tower Foundation**

*Manish V. Shah, Ph.D, Usha K. Patel, Ph.D and Purav A. Raval, BE*

38

**Effect of Under-Ream Angle and Bulb Spacing on Capacity of Under-Reamed Piles in Marine Clay**

*Minu J., M-tech Student and Benny Mathews Abraham, Ph.D, Professor*

46

**Installation of Large Diameter Bored Pile Socketed in Hard Rock with RCD for MTHL, Mumbai**

*Akshay Sakhuja, Vignesh R., M.Tech and Inki Choi*

55

**Low-Strain Pile Integrity Testing and Analysis**

*Mahesh Hingorani, P.E.*

65

**New Testing Approach for Pile Load Tests in India**

*Vijay Kumar Patidar, Suneet Kaur, Ph.D and Nitin Dindorkar, Ph.D*

75

**Optimization of Project Schedule and Cost Using Available Steel Sections as Foundations for a Deep Seated Structure**

*Biswajit Das, Anup Kumar Mandal, Ph.D, Suman Sen and Shuvranshu Kumar Rout*

82

**Response of a Single Pile Located on Sloping Ground under Dynamic Loading**

*Deendayal Rathod, Ph.D, D. Nigitha, Ph.D and Krishnanunni K. T., Ph.D*

92

**Soil-Structure Interaction for a Tension Pile Pulled with Strand**

*Chandramohan P.V., Senior President (Technical)*

100

# Developments in Safe and Efficient Geo-Construction

**Geohazard Protection of Infrastructure**

*Andreas Brandner, Dipl.-Ing. MSc.*

108

**Innovative Tunnelling Practices Adopted to Mitigate Various Execution Challenges in a Rail Tunnel Construction in India**

*Venkata S. Dronamraju, Ph.D, IRSE*

124

**Lateral Response of Rock Socketed Single Pile Due to the Effect of Overlying Sand Layer**

*Jeya Sonia J., M.E. and Karthigeyan S., M.E., Ph.D*

132

**Reliability on Quality Control & Quality Assurance of Deep Foundations**

*Laxmikanta Tripathy, G.E.*

140

# Improving Current Practice with Case Studies

**A Case Study on Pressuremeter Test in Katni, Madhya Pradesh, India**

*Kuppala Venkata Sivannarayana, Arnab Banerjee, Senior Engineering Consultant and Subhasish Pasupalak, Engineering Consultant*

146

**Comparison of Geophysical and Geotechnical Investigations of a Deep Soil Site in Indo Gangetic Plains**

*Priyanka Sharma, Ph.D, M. L. Sharma, Research Scholar and V. A. Sawant, Ph.D*

156

**Effects of Mixing of Bentonite Suspension in Geotechnical Applications**

*Akhila Manne, Sr. Geotechnical Engineer and Madan Kumar Annam, Head of Engineering*

163

**Forensic Evaluation of Defective Bored Piles of an Industrial Plant**

*Sorabh Gupta, Ravi Sundaram and Sanjay Gupta*

171

**Implication of Site Investigation on Piles Socketted in Weathered Rock**

*Kuppala Venkata Sivannarayana, Associate Chief Consultant, G. Gowthami, Senior Engineering Consultant and E. Suresh, Geologist*

180

**Importance of Collecting Soil Samples in Hard Strata for Soil Classification and Determination of Shear Strength Parameter**

*Kuppala Venkata Sivannarayana, Associate Chief Consultant, Parvathy V. Raj, Senior Engineering Consultant and N. Manikantan, Geologist*

190

**Prediction of Modulus of Elasticity and Its Impact on Settlement**

*Vinay Kumar M., Sr. Geotechncial Engineer and Suresh Kumar Gupta P., Lead Geotechnical Engineer*

200

**Rotary Kelly Drilling for the Bored Pile Foundation of the Hong Kong Zhuhai Macao Bridge**

*Thomas Domanski, Regional Director*

210

# Urban Development/Redevelopment

**Capacity of Piles in Pond Ash Based on Model Pile Load Tests**

*Sreedhar M.V.S., Ph.D and Dheeraj Ram A. V., M.E.*

225

**Challenges Encountered for Foundation of Durgam Cheruvu Extradosed Bridge in Hyderabad - Design and Construction**

*Krishnapriya P. R., M.Tech, Vignesh R. and Subir Das, M.Tech*

232

**Construction of the Foundations of Mega Sea Bridge of India - Mumbai Trans-Harbour Link**

*Ekhlaq A. Khan, Sr. Engineering Manager*

240

**Micropiles and Ground Anchors Increase the Capacity of Existing Bridge Abutments**

*Swetha Chennu, EIT, Anthony C. Barila, P.E. and John R. Mckinnon*

251

**Pile Foundation for Cable Support Structure in Brownfield Site with Underground Obstructions - A Case Study**

*Manos De, Civil Engineer, Vijay Bhatt and Shuvranshu Rout*

260

**Strengthening of Existing Building Foundations by Micropiles**

*Runu Dutta, Anup Kumar Mandal, Ph.D, Biswajit Das and Manos De*

271

# A BARRIER WALL FOR THE NEW DAM AT THE GODAVARI RIVER INSTALLATION WITH MONSOON BREAK

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## ABSTRACT

Bauer Specialized Foundation Contractor India, in a joint venture with L&T Geo, executed the cut-off wall for the Polavaram Irrigation project in Andhra Pradesh, India. The project is envisaging irrigation in the region and offers the generation of hydroelectric power, water supply for industries and drinking water supply to villages and towns near the project. The paper describes the planning and installation of a 1,450 m long plastic concrete cut-off wall for the ECRF Dam with depths up to 94 m. This wall construction passed through the Godavari River, which in the months of July to September - the monsoon season - is flooded by all the water coming from up-stream, and the water level rises by up to 12 m.
Accordingly, the Project was planned to be completed in two seasons, with a stop during the monsoon season. The paper as well describes the planning and preparation for the monsoon break, successfully protecting the partly installed concrete cut-off wall as the monsoon flood was passing over the wall.

**KEYWORDS: plastic concrete, cut-off wall, monsoon break**

## GENERAL

To provide the region of West Godavari District and East Godavari District in Andhra Pradesh in the southeast of India with an improved water storage facility, the Indira Sagar Project or Polavaram Dam is being constructed at the Godavari River. The reservoir of the dam also spreads to parts of Chhattisgarh and Orissa States.
The Polavaram Dam Project is a multi-purpose irrigation project and is situated near the Polavaram village about 42 km upstream of Godavari Barrage. The JV BAUER- L&T Geo was assigned by the Water Resources Department of the Government of Andhra Pradesh to carry out the barrier wall under the dam as plastic concrete cut-off wall using the diaphragm wall method.



**Fig. 1. The area of the Polavaram Dam at the Godavari River prior to start off the works**

## THE PROJECT – BENEFITS FOR THE PUBLIC

With the facility of storing the monsoon floods, the public and particularly the agricultural installation will benefit throughout the year from the water prevented to flow unused into the sea.
The project is envisaging irrigation benefits to 720,000 acres of land. In addition to irrigation, the dam offers:

- Generation of Hydro Electric Power with installed capacity of 960 MW


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- Water supply for industries
- Drinking water supply to villages and towns near the project

**THE TASK FOR BAUER AND JV-PARTNER**

For the successful planning, preparation and implementation, the Contractor (BAUER India with its Partner L&T Geo) was commissioned with a large number of local engineers and employees to accomplish the challenging task. BAUER experts had been involved to choose the adequate equipment and source the required material. From locally available sources, BAUER experts successfully designed and tested slurry and concrete mixes to enable installation of the barrier wall with an innovative and durable Plastic Concrete Mix. In order to determine the best possible "Mix Design", numerous suitability tests have been carried out. The proof tests, to ensure that the earth concrete meets the requirements of the technical specifications, had to be carried out with a high frequency.

The barrier wall built as concrete cut-off wall has a total length of 1450 m of varying depths following the geology. The cut-off wall depth ranged from a few meters at the left and right abutments of the river and reached a depth of almost 94 m at the main river bed. A contractual trenching quantity of 68,000 $m^2$ was executed. The cut-off wall thickness was designed to 1.5 m in the deeper areas of the wall to ensure wall continuity and lasting durability.
The project was executed by the JV between March 2017 to June 2018 in two construction phases with a monsoon break in 2017. At this part of the Godavari River, the monsoon season lasts typically from the months of July to September. The wide river area is flooded within one to two weeks by all the water coming from up-stream as the water level rises by some 12 m.

**THE GEOLOGY**

The prevailing geology is river sand and clay overlaying rock. Intermingled in the sand and clay are pebbles, cobbles and boulders which needed to be excavated by the equipment installing the barrier wall. For boulders, the hydraulic trench cutter is the ideal tool to excavate the trench without unnecessarily disturbing the soil matrix around the boulder.
Three main soil strata are found existing at the Polavaram Project:

- STRATA-I:
  Very Loose to Very Dense Silty Sand with traces of gravel and pebbles in most of the boreholes (except five boreholes (BH-1 to BH-5). The thickness of layer varies from 5m to 83m. The field SPT N values observed in this layer are ranging from 3 to >50.

- STRATA-II:
  Stiff to Very Stiff Clay: sSilty clay generally of intermediate plastic to highly plastic is present. The layer thickness is varying from 0m to 25m. The field SPT N values observed in this layer are ranging from 8 to 38.

- STRATA-III:
  Khondalite / Charnockite bed rock is encountered below the sand strata for the boreholes BH-14 to BH-37 and encountered below the clay strata for the boreholes BH-3 to BH-13. Bed rock exists at top surface for the BH-1 & BH-2. This layer is encountered at 15m to 80m below the existing river bed levels and the layer is revealed up to borehole termination depths. The Core recovery is varying from Nil% to 100% and RQD is varying from Nil to 100%. The UCS values are observed as 55 MPa to 162 MPa and point load index values are observed as 10 MPa to 15 MPa.

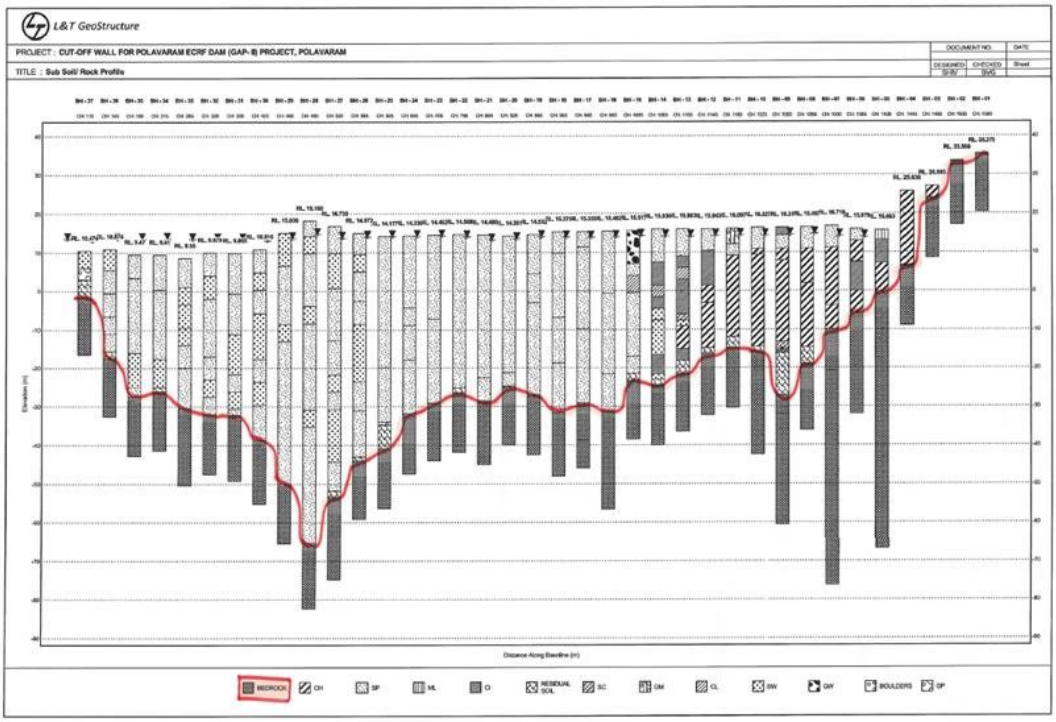



**Fig. 2. *37 Boreholes were conducted for this project**

## THE HYDRAULICS OF THE RIVER

The Godavari River - typically during non-monsoon time - has a width of 400 m and a height of 8.5 m at the location of the future dam. During monsoon between May and October the river rises fast and reaches at peak during July and August a width of 1400 m and a height of 21 m and more. The flow velocity reaches up to 5 m per second. Works on the platforms in the river bed area had to stop in time before the monsoon flood came until near to the platform to allow personnel and equipment to reach higher ground safely. Works had to stop before to allow freshly cast concrete in the diaphragm wall panels to set and reach sufficient strength.

## THE TECHNIQUE INVOLVED

The barrier wall under the Polavaram Dam was designed as a Plastic concrete diaphragm wall. A wall durable for the life time of the dam on the one hand but still ductile enough to accommodate different loading due to monsoon flood filling the reservoir and some impact from seismic events.

The diaphragm wall technique used for the project was excavation of individual panels under slurry by grab / clamshell and hydraulic trench cutter. Primary elements are installed at a defined distance to each other. This defined distance is smaller than the size of the closing secondary panel forming after installation of the continuous wall. The length of Primary elements depended on geology and hydraulic conditions. Either one bite elements (i.e. 2.8 m long) or three bite elements (i.e. 7 m long) are installed following a diligent trench stability design.
The overlapping of the secondary to the primary panel is designed by the experienced specialist Contractor according to the depth of the cut-off wall, the equipment used, the implemented Quality Management Plan and the experience of the specialist contractor's personnel. For the project, the nominal overlap at work platform level varied wall-depth-related between 30 to 40 cm.
The specified minimum wall-thickness (the effective wall-thickness) at depth was of 0.6 m with each panel duly keying in at least 2 m into hard rock.

Crucial for safe working is a stable work platform for the equipment which is kept tidy to also assure safety for the personnel. Prior to the installation of the barrier wall, a guide wall is installed to assure alignment at platform level and prevent damage to the work platform as the slurry level in the open trench under excavation will vary when working with grab or clamshell. The guide wall is used as well to fix the guide frame for the hydraulic trench cutter at the start of cutting at shallow depth. Furthermore the guide wall, is an ideal base to carry safety covers to protect any open or freshly cast trenches preventing staff to fall into the trench.

Should cutting start from a shallow level, i.e. 3 m to 5 m below the work platform level, a pre-excavation by grab or backhoe to that depth is required. For the cutting process, the slurry is used in the trench as transport medium for the cuttings, therefore the slurry filled pre-excavated trench is required.
Depending on geology, excavation starts in easy to excavate soil using a grab to economic depth. In hard soil or rock or at depth where grab excavation is not economical any more, the excavation with the hydraulic trench cutter starts. Hydraulic cutting or milling is a continuous process from start to final depth, typically discontinued only should maintenance of the cutter be required.



**Fig. 3. Hydraulic grab and BAUER Cutter on the work platform**

On site: 2 No of BAUER BC40 and 1 No BC30 cutter as well as a BAUER MC96 grab unit and a C850/BAYAGRAB.

## THE WORKING SEQUENCE

For the execution of this project, the JV mobilized three BAUER trench cutters (MC 128/BC 40, HS 885/BC 40, BG 40/BC 32) and two grabs (BAUER MC 96/hydraulic grab, Baya hydraulic grab unit) were deployed including three BAUER desander units. The working sequence to install the entire barrier wall started at the right abutment continuing to the left abutment. Due to the required relocation of the Godavari River, it was planned from the beginning to install the cut-off wall in two seasons or phases.
To pre-stabilize loose sand layers in the river soil improvement was planned under the entire dam footing. Along the barrier wall the soil improvement was installed prior to the cut-off wall installation to ensure no

adverse effect on the wall by close-by vibro compaction. The vibro compaction was executed from a similar work platform level and had to be interrupted as well during the monsoon season.

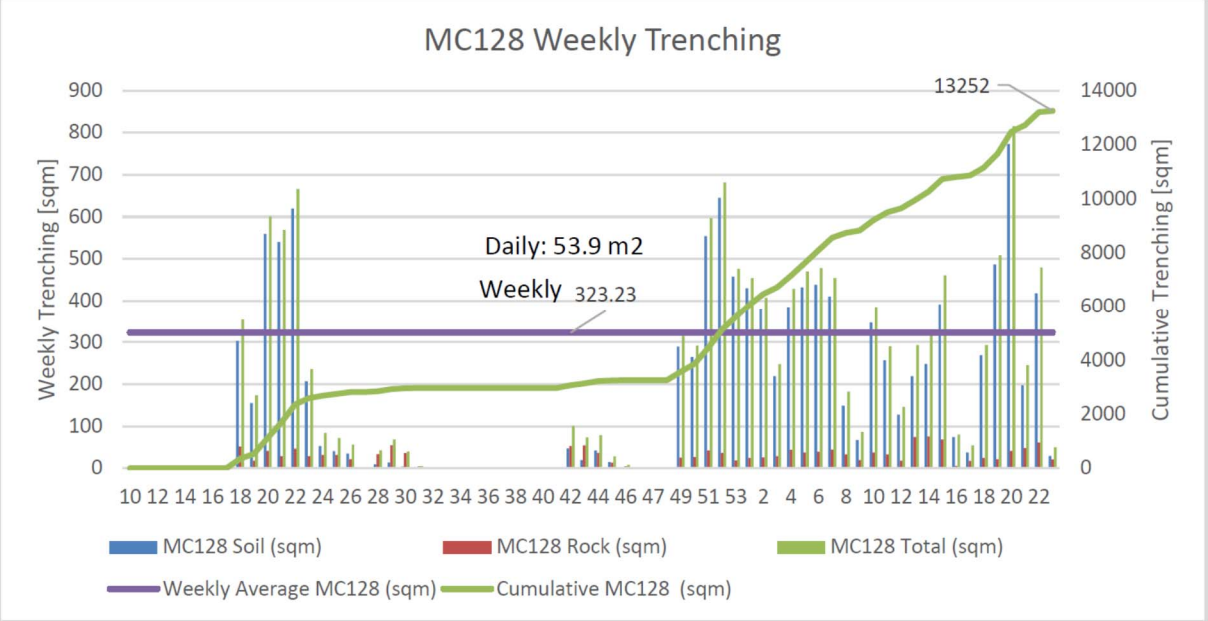



**Fig. 4. BAUER MC128/BC40 – Excerpt of Weekly Trenching Performance**

Across the original river bed, after relocation of the river, a separate and higher work platform had to be built prior to vibro works and the barrier wall installation.
After 15 months, including the 2017 monsoon break, the cut-off wall was successfully closed at the left abutment.

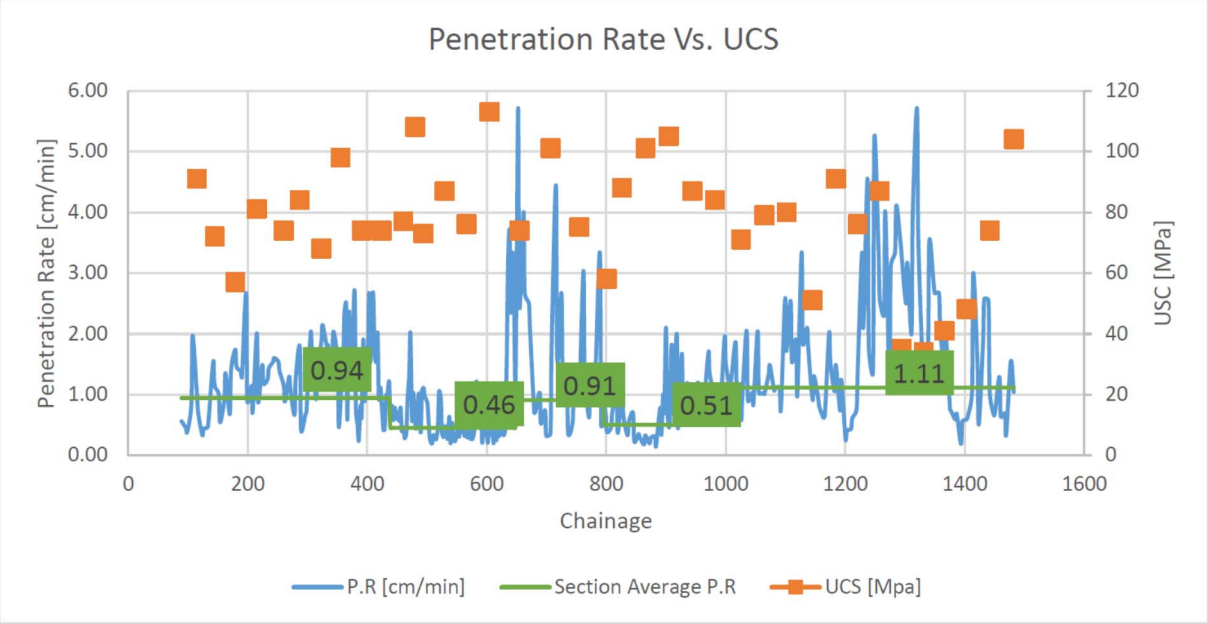



**Fig. 5. Penetration rate in rock versus chainage**

## QUALITY MANAGEMENT FOR A CONCRETE CUT-OFF WALL

Successful installation of the Cut-off Wall depends on three essential requirements:

- proper embedment of the wall toe into sound bedrock
- proper continuity of the entire wall
- integrity of the Cut-off Wall concrete material

The BAUER strategy to insure quality of the Cut-off Wall work deviates from the extensive coring and water testing verification campaign on the completed concrete Cut-off Wall. BAUER's approach relies on the consistent implementation of a highly defined and adequate process to indisputably meet the three essential requirements listed above, together with the strict and accurate continuous quality control of this entire repetitive process, and finally verification of the performance of the wall.

This approach will involve:

- accurately define the wall installation process and its extensive quality control plans
- implement the wall installation process and controls in the Demonstration Section
- proof-test the process and controls, by testing the installed wall as specified in Demonstration section– extensive coring, water testing, investigation of joints
- implement the remaining wall installation with the tested and refined process and controls
- further investigate the installed wall only when and where deviation from the process occurred and/or potential defects were detected by the quality control program.

The proposed Quality Management Control Plan (QMP) addresses the entire process quality and the end-product efficiency. Our approach reflects the overall quality of the Cut-off Wall, while a coring and water testing on the installed wall would only evaluate some local conditions, and would in addition create potential weaknesses in the Cut-off Wall.

## INTEGRITY OF THE CUT-OFF WALL

### BENTONITE SLURRY SUPPORTING FLUID

The bentonite suspension is a primary means of insuring the integrity of the concrete Cut-off Wall. BAUER's past experience on difficult projects has led to the development of the following approach to bentonite slurry:

- Excavation process and concreting operations will be carried out using two different supporting fluids, designated "working" and "concreting" slurry. These will serve different purposes and will be handled at separate facilities on the site.
- Design of each supporting fluid will be modified on site as necessary to address actual site considerations such as soil conditions, equipment, materials and work sequences.

**Table 1. Typical properties of in-trench slurries, to be adjusted on site**

| **"In-trench" Properties**<br>*Main design considerations:* | **Working Slurry**<br>*Soil, Equipment, Sequence* | **Concreting Slurry**<br>*Soil, Equipment, Sequence* |
|---|---|---|
| Density (g/cm³) | < 1,25 | < 1,15 |
| Marsh value (sec) | 32 - 60 | < 50 |
| Fluid Loss (mL) | < 50 | < 40 |
| pH | 7 - 12 | 7 - 12 |
| Sand content (%) | n.a. | < 5% |
| Gel strength (N/m2) | > 15 | n.a. |

A bentonite suspension control plan, part of the overall QMP, will be prepared accordingly and address each suspension type for acceptable properties, tolerances, and test frequencies, as well as remedial treatment requirements for each type of slurry.

The properties of the concreting bentonite will be checked as per the QMP prior to concreting to ensure quality of the slurry. The sand content of concreting bentonite will be no more than 3%, as generally working bentonite will be exchanged by concreting bentonite prior to concreting.
Trials are considered during the trial period only. Compliance tests during the execution period will be according to the accepted QMP.

## STABILITY OF THE OPEN TRENCH

In the Demonstration section initial intermittent KODEN measurements or similar have to prove the trench stability to verify the excavation method in the embankment and document trench stability.
Actual narrowing or necking of the trench would also be detected by KODEN imaging prior to concreting. In the event that trench narrowing occurs, stabilization of the condition will be monitored by several rounds of KODEN imaging prior to and also during concreting operations.



**Fig. 6. Executing the cut-off wall across the Godavari river**

## QUALITY OF JOINTS

All joints will be cleaned with the Trench Cutter, by rotating the Trench Cutter wheels and suctioning slurry all along the trench sides in order to eliminate soil/cake and form a clean serrated joint surface. The use of a dedicated constant "concreting" slurry mix to replace the "working" slurry will limit re-formation of cake or deposits onto the joints surface. In addition, the slurry quality control plan will define a maximum allowable time-frame between joints cleaning and concreting operations (typically 6 hours from finish cleaning to start of concreting.
The bottom of the panel will be cleaned by flushing bentonite slurry through the Trench Cutter-built-in slurry pump for up to 10 minutes to achieve the required interface quality.

## THE PLASTIC CONCRETE

The plastic concrete design used at the Polavaram Dam Project consisted of natural sand (0-4.75mm) and coarse aggregates (4.75mm-12.5mm). The sampling of these aggregates has been carried out at the site. Natural sand was supplied from the riverbed of Godavari River. As for coarse aggregate, crushed aggregates were used. The type of the broken aggregate is called Khondalite. In the tables below and Fig. six above, specific gravity, water absorption and sieve curve analysis of the aggregates are presented.

**Table 2. Water absorption and specific gravity test results of samples**

| Property | Sand Aggregate (0-4.75mm) | Coarse Aggregate (4.75-12.Smm) |
|---|---|---|
| Water Absorption [%] | 0.63 | 0.51 |
| Specific Gravity [g/cm3] | 2.64 | 2.89 |

During the development of the Plastic Concrete (earth concrete) Mix-design, the concrete tests included so-called performance tests, with which the required properties workability (in settling test or settling flow test) permeability and deformability (via elongation at break and modulus of elasticity1) can be proven. At this point, however, it should be pointed out that for the determination of the E-modulus by definition only the elastic deformability of the earth concrete is determined. Especially with low-strength earth concretes the viscous deformation components often predominate. The deformation components imposed synchronously to the "slow" wall deformations (the rule for dams), the relaxation of stresses that occurs during such high stresses, as they are calculated from the modulus of elasticity, do not occur in the first place. This good-natured behaviour has a positive effect on the durability of the earth concrete and could be achieved through the use of a realistic material model, which also lead to a more economical design (Beckhaus et al. 2015).
In addition, the erosion resistance of the earth concrete was also to be investigated as part of the suitability tests. Due to the direct dependence on the compressive strength these separate tests could be dispensed with. As usual, the verification was carried out according to the so-called descriptive concept over a minimum strength, which in this case was well above the required strength range between 1.5 to 2.5 N/mm².

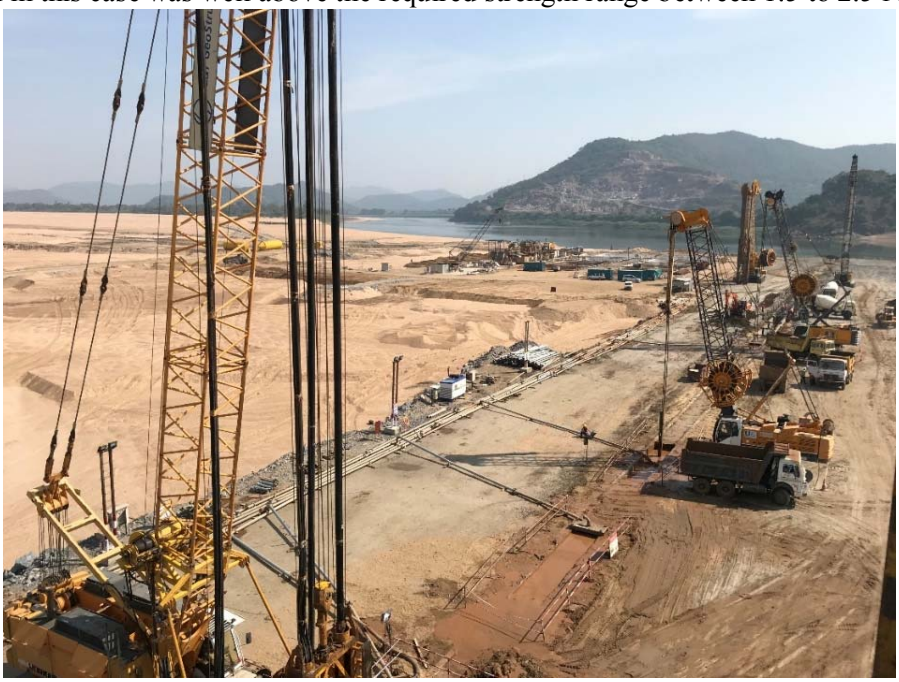


**Fig. 7. Trench excavation and concreting operations during Phase 1**

## QUALITY CONTROL OF PLASTIC CONCRETE

The execution property of the Plastic concrete is based on trial mixes in accordance with the technical specifications. For stability against erosion an adequate hydraulic gradient will be defined and agreed upon. Material Quality Control Tests for hardened Plastic Concrete will be proposed in the QMP.
Concrete Strength for the cut-off wall is defined over a 10 point average, not on individual samples.

**Table 3. Summary of plastic concrete data**

| Mix Design accord. to cement content | Triaxial Compressive Strength $\sigma_c$ = 0.4 MPa [MPa] | Ultimate Elongation in Triaxial Test [%] | Uniaxial Compressive Strength [MPa] | Coefficient of Permeability [m/s] |
|---|---|---|---|---|
| Mix 2: z 180 kg/m³ | 3.5 | 4.9 | 1.8 | 8.0 x 10-9 |

## SOUNDNESS OF THE SOCKETING STRATA

The challenging complex geological nature calls for a pro-active investigation approach. Prior to Cut-off Wall execution, the rock drilling campaign needs to map soil and rock properties, cavities, fissures or gaps in the Cut-off Wall alignment. Doing so, the soundness of the rock strata at the projected cut-off wall toe shall be confirmed prior to its execution.
As excavation proceeds, conditions will be carefully documented on the excavation reports and regularly reported to the site geologist or engineer. The review of the continuously recorded cutting parameters is a valuable tool to assess excavation soil conditions. Visual observation of the material excavated from the base of the wall, such as the absence of "non-rock" materials in the cuttings, will also be used to evaluate the soundness of the rock socket strata.

## THE CHALLENGES AND THE MONSOON BREAK

With experienced personnel and adequate equipment, the challenges to install a concrete cut-off wall in the prevailing ground conditions are limited. A start curve has to be planned to train local personnel in the safety-, quality- and performance level the Client expects from an experienced specialist Contractor.
To receive the plastic concrete in time and in the required quantity at the beginning the local batching plant and concrete trucks reached their limits. Further batch plant capacity was installed by the main contractor to support the barrier wall operation adequately.

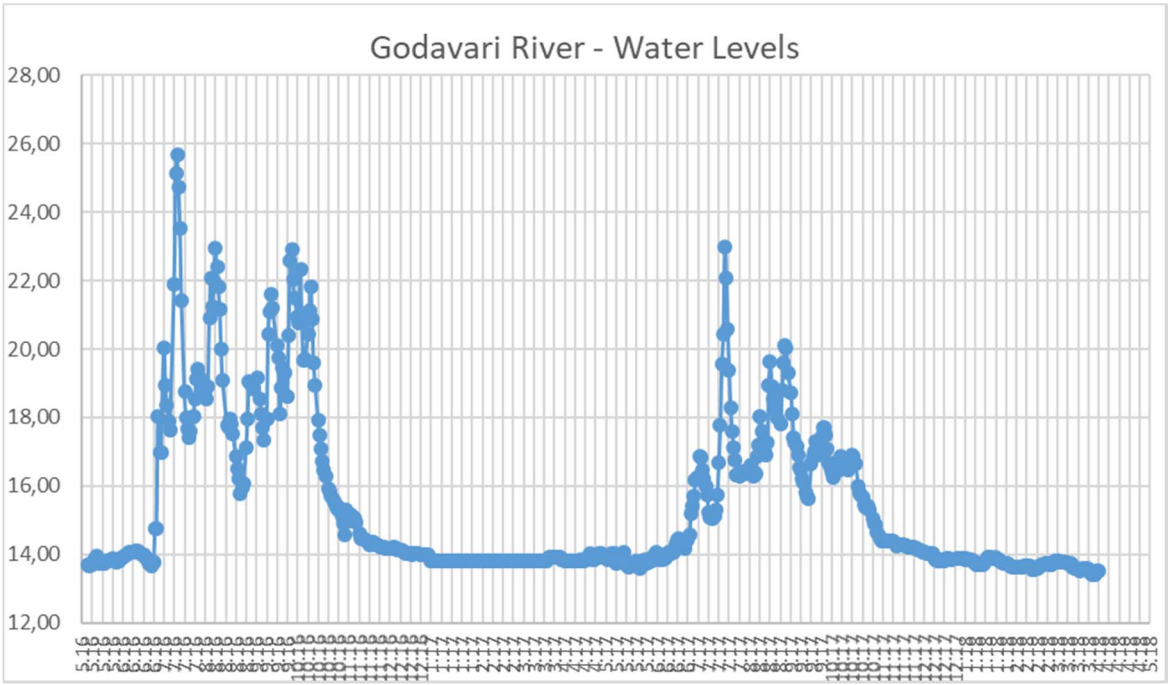



**Fig. 8. Godavari River Water Levels during Monsoon 2016 and 2017**

The monsoon break was known to come and was planned in detail. After long discussions a common work sequence was agreed between the contractor and the Client's Engineer. The work platform needs to be levelled for the large and tall equipment used for the diaphragm wall installation. Therefore, in parts the elevation of the installed cut-off wall was higher than the original river bed.
The Contractor proposed in this area, where the new cut-off wall was installed above the original river bed, that the wall was to be trimmed down below the level of the original river bed and the trench should be covered with similar river sand compacted to similar density and to leave this to the rising river during the monsoon season to flow unhindered over the cut-off wall. Several experts where consulted to finally approve the approach. The new cut-off wall was to be trimmed down approximately 1.5 m below the original river bed and the original river bed reinstated.

At the same time, the Contractor provided as a Plan B a method statement on how the cut-off wall could be repaired should due to the monsoon flood scouring of the wall occur and the wall be damaged. The method statement was submitted and accepted. In a case of shallow damage, the top of the wall would have been excavated, cured and elongated to the required depth with similar type concrete. In case of deeper damage, a parallel plastic concrete wall would have been installed and with perpendicular elements connected to the intact parts of the wall.
At defined locations, the height of the trimmed cut-off walls and the height of the backfill were surveyed and recorded prior to the monsoon flood reaching the site. This documentation assured the re-measurement after the floods ceased, that the top of the wall was not affected.
The original method, which was agreed, has proved to be working. Plan B at no part needed to be used and after the monsoon break, the cut-off wall installation continued as planned.




**Fig. 9. Trimming the head of the Cut-off Wall below River Bed Level before monsoon 2017**




**Fig. 10. Left – From platform +25.0 m looking toward CH0.0 – October 2017 – while water receded**
**Fig. 11. Right - From CH800 looking towards CH1750 - October 2017 - 1 day after water recede**

## THE SUCCESSFUL COMPLETION

With the dedication of all involved, the team was able to finish the cut-off wall works in the second Phase in 2018 in time before the monsoon flood reached the site of the Indira Sagar Project. The interruption by the monsoon 2017 was carried out professionally with success.

## ACKNOWLEDGEMENTS

All participants, Client, Engineer, Partner and site staff, deserve the highest thanks for their individual contribution to successfully completing the challenging barrier wall in the shortest possible time.

## REFERENCES

Maslak, C.C., 2018 and Beckhaus, K. – Polavaram Damm – Qualitätssicherung des Erdbetons für eine 100 m tiefe Schlitzwand – TU Darmstadt, Germany

Beckhaus, K., Lesemann, H., Högl, C. 2015 – Welches Stoffmodell für hochverformbaren Dichtwandbeton ist geeignet für die geotechnische Bemessung im Dammbau? Proceedings of the 10th Austrian Geotechnical Conference, Vienna

# ALTERNATIVE SOLUTIONS TO REPLACE DIAPHRAGM WALLS OF SHALLOW DEPTHS

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## ABSTRACT

Diaphragm walls are well-known in the special foundation industry and suitable for different applications especially of large depths. However, for walls of shallow depth the diaphragm walls method might not always be the most practical regarding the economical solution.

This paper presents alternative methods of diaphragm walls for shallow depths which are commonly used in India. In comparison to the diaphragm wall method, alternatives like secant pile walls or soil mixing walls show time saving and cost effective advantages. This paper focuses on soil mixing methods, which tools with respect to machines are mainly designed for an effective installation of retaining and cut-off walls. The paper discusses the advantages of these soil mixing methods and explains the conditions they can be used the most effectively.

In detail "Soil Mix Wall" (SMW) and "Cutter Soil Mixing" (CSM) are explained and reference jobsites are presented.
The product of both methods are soil mix elements with an oval or rectangular shape which are particularly suitable for the effective installation of walls. Additional advantages of these methods are e.g. the reduced spoil volume due to the usage of the local existing soil as a construction material and the independence from on-time delivery of fresh concrete and therefore the potential cost savings.

**Keywords: Soil Mixing, Soil Improvement, Retaining Wall, Cut-Off Wall, Foundation**

## INTRODUCTION

Diaphragm wall grabs have been used since many years for the production of shoring and sealing walls. However, this method has certain disadvantages in particular when working at limited space conditions or when walls are not too deep. Especially walls with a relative low depth are provide potentials to use alternative methods. Possible alternative methods could be secant pile walls installed by using the CCFA method (Cased Continues Flight Auger) or soil mixing walls by using the CSM (Cutter Soil Mixing) or SMW (Soil Mix Wall) method.

The main advantages of these methods are:

- high productivity,
- low space requirement for the machine and the auxiliary equipment (e.g. no requirement for bentonite storage and desanding equipment),
- easy installation of waterproof joints between the single elements,
- no requirement of pre-mixed concrete deliveries for CSM and SMW jobsite and
- for CSM and SMW the local soil can be used as construction material – hence, less material has to be disposed (important if local soil is contaminated).

### Soil mixing methods as alternative solution to diaphragm wall

By using the soil mixing methodology, the soil is improved by an in-situ mixing process. Therefore, cement, lime or other binders are added and mixed into the soil strata by using a mixing tool. This methodology can


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be used either as wet mixing or as dry mixing.
In the more frequently used wet mixing process, usually a mixture of binder and water with possible addition of additives is injected and mixed with the soil. Depending on the type of soil and binder, a mortar is created which hardens during the hydration process at the end of the mixing process. In the dry mixing process, the binder is directly mixed with the soil while reacting with the existing ground water.
Several versions of mixing tools are available. They can be distinguished by the classification of Bruce 2010 shown in figure 1. Depending on the technique, improvement depths of up to 80 m can be achieved.

While the history of these techniques starts predominantly in Japan (wet mixing) and Scandinavia (dry mixing) in the 1970's, nowadays soil mixing is getting more and more important all around the world.

Soil mixing can be carried out in almost all soil formations. Granular soils are more favorable than cohesive soils, but considering potential limits the use in cohesive soils is also applicable.
When using the CSM Method, due its cutter principle, also the penetration into low strength rock within limits for e.g. socketing purpose, is possible.

Depending on required properties of the soil mix elements and on local conditions, different amounts of cement have to be injected into the soil. In addition, bentonite can be added to the mix. If elements with a relative high strength are required, the cement content has to be increased by decreasing the water-cement ratio. In case there is a need for the use of bentonite for liquefaction, sandy soils will typically require a larger amount of bentonite in the slurry than clays. At some sites with clayey soil conditions, where enhanced resistance to permeability is not required, acceptable properties of the mixed soil can be achieved without the use of bentonite.

## Static System for Retaining Walls Using Soil Mixing Methods

The static system for soil mix walls differs from diaphragm walls. Thus, the approach of static calculation is different. While for diaphragm walls usually a framework of pressure (concrete) and tension rods (rebar) is assumed, arch actions between the reinforcement beams are calculated for soil mix retaining walls. Figure 1 shows the structural behavior for soil mix retaining walls.
For greater excavation depth, the support of the steel beams can be realized by anchors or by temporary "bracings" inside the pit, as shown in figure 2.

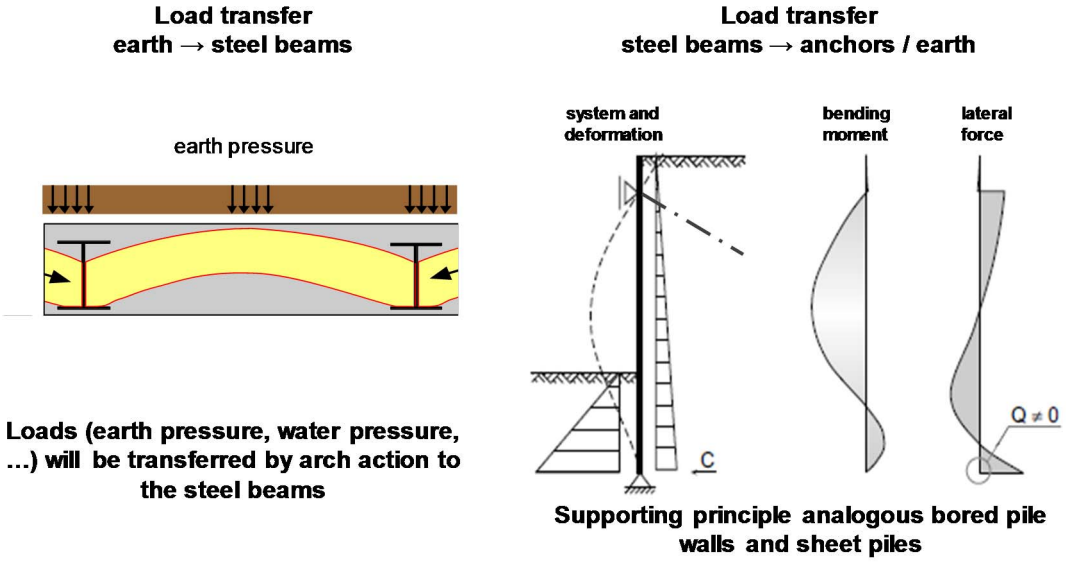



Figure 1: Structural behavior soil wall

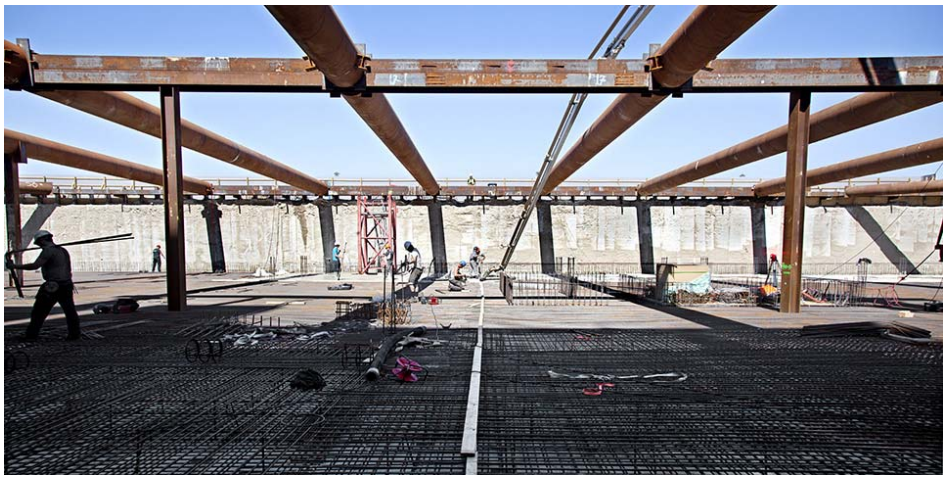


Figure 2: Temporary “bracings” to support soil mix walls

**Productivity factors for CSM and SMW**

Table 1 shows some factors influencing the productivity for the soil mixing methods CSM and SMW.

| | **favourable conditions** | **unfavourable conditions** | **comment** |
|---|---|---|---|
| **soil structure** | homogeneous soil structure | layered soil structure | Operator has to adjust the machine parameters for different soil layers |
| **soil type** | loose to medium dense gravelly sand, silty sand | dense to very dense soil, cobbles and boulders embedded in soil, stiff or hard soil, cohesive or organic soil (reduction of the final strength) | Loose or soft soils can be easier be treated than dense or hard soils |
| **site geometry** | long, straight wall sections | irregular wall layout | more time to set up the machine on the individual panels necessary |
| **wall depth** | wall depth > 6 m | wall depth < 5 m (higher influence of non-productive periods such as moving, setting-up) | |

Table 1: Productivity factors for CSM and SMW

**CUTTER SOIL MIXING (CSM)**

One of the latest developments in the field of wet mixing was the Cutter Soil Mixing (CSM) method, which was developed by a joint venture of BAUER Maschinen GmbH and Bachy Soletanche in 2003/2004. CSM is used mainly for stabilizing soft or loose soils (non-cohesive and cohesive). However, as the development is derived from the cutter technology, application in hard soil strata becomes possible unlike other soil mixing methods. The CSM mixing head was designed to work as a kind of compulsory mixer which leads to very good mixing quality – even in cohesive soils.

**Equipment for CSM**

One of the main differences, in comparison to other soil mixing methods, are the mixing wheels which rotate around the horizontal axis. The design of the mixing head (Figure 3), with so-called reamer plates, provides a very good intermixing of soil with the injected slurry. The principle is comparable to a compulsory mixer.
Two mixing heads of different size and torque are available. The size of individual panels is determined by the type and size of equipment being deployed. Panels can be constructed in lengths ranging from 2.4 m to 2.8 m with wall thicknesses of 0.55 m to 1.525 m. To variate between different wall thicknesses, it is necessary to change the mixing wheels.
With Kelly guided CSM mixing heads depth up to 53 m are achievable. Figure 4 shows a Kelly guided CSM machine with max. mixing depth of 20 m. For depths higher than 53 m (up to 80 m) or jobsites with restricted heights, rope suspended mixing heads as shown in figure 5 can be used. As base carrier for the Kelly guided system, pile drilling rigs and sheet piling rigs can be used. Cranes are used for cable guided CSM mixing units.



Figure 1: Mixing head for CSM



Figure 3: Rope suspended CSM machine for 80 m depth




Figure 2: Kelly guided CSM machine for 20 m depth

The CSM mixing heads are equipped with inclinometers and show deviations in real time to the operators. All required working parameters are presented to the operator in real time and stored for later data evaluation and report preparation.

Additional to the CSM base machine, slurry handling equipment is required. The recommended slurry handling equipment depends on local conditions. For jobsites where the production time per panel is shorter than the initial setting time of the cement (about 2 hours) following slurry equipment is recommended:

- mixing plant with 15 to 40 m³ capacity,
- slurry pump with 400 to 600 l/min capacity, adjustable pump speed by remote control (for the CSM rig operator) is highly recommended, the pump should work with low slurry pulsation (e.g. eccentric screw pump),
- crew conveyor for cement powder, optional one for bentonite powder,
- agitator tank for cement slurry, 2 to 4 m³ and
- slurry hoses with 2" diameter between slurry pump and CSM rig.

The use of an air compressor for flushing air is also recommended for CSM. Air compressors with capacities between 5 and 8 m³/min and 6 to 8 bar are suitable.

**CSM Construction principle**

A typical construction sequence for CSM as shown in figure 6 is as following:

a) Construction of an open guide trench for retaining excess slurry.
b) Liquefaction of the soil strata during penetration to the final depth as an appropriate slurry is simultaneously introduced. Depending on the prevailing conditions, either bentonite slurry is added to the mixing/liquefaction process or cement slurry is introduced into the soil during penetration. The volume of slurry injected is determined by the rate of cutter penetration.
c) During upstroke, the final volume of slurry required for producing the final wall construction material is injected.
d) To utilize the wall as a structural retaining wall, steel beams (UB/IPB sections) are inserted into the freshly mixed wall panels.
e) A continuous wall is formed by the construction of individual panels in an alternating sequence of overlapping primary and secondary panels. Secondary panels can be constructed immediately after completion of primary panels, i.e. "wet-into-wet". The cutter technology does, however, also enable cutting into panels that have already hardened, i.e. "wet-into-hard".

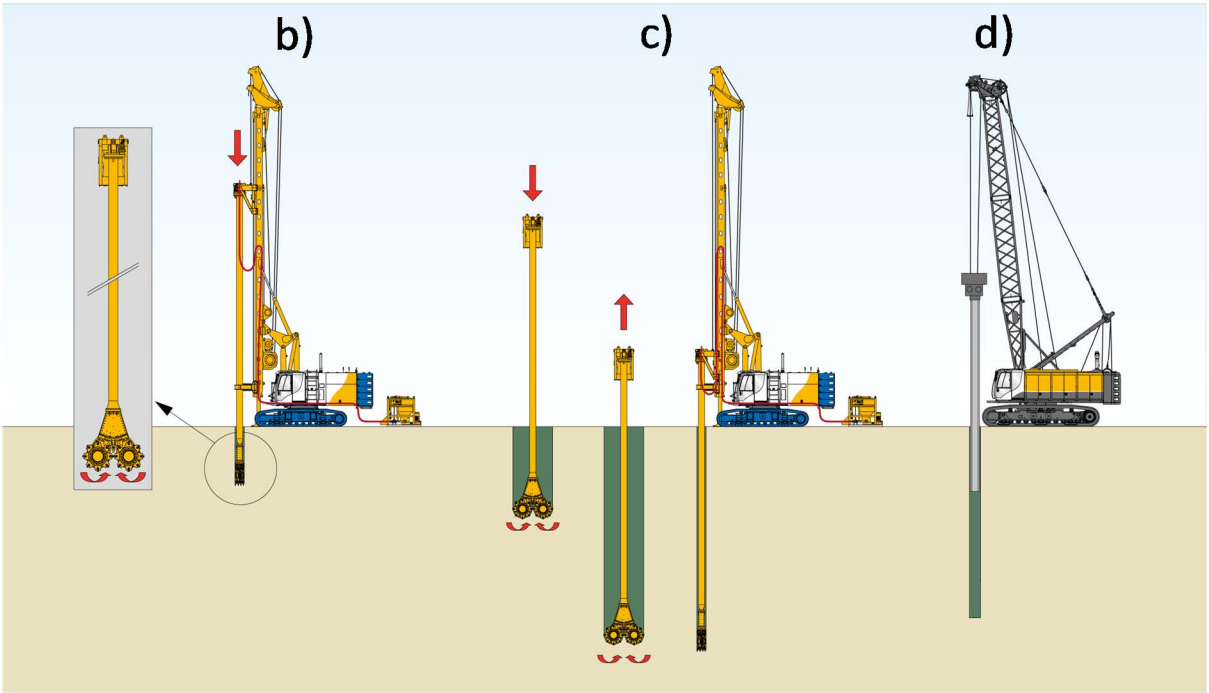



Figure 4: Construction sequence for CSM

## TRIPLE SOIL MIX WALL (SMW)

Another approach is the so called Soil Mix Wall (SMW) method. The SMW is one of the oldest soil mixing methods. In principle, it is a combination of three single column mixing rods to form elongated panels to increase the effectiveness in the production of walls.

### Equipment for SMW

Figure 7 shows the dimensions of typical SMW mixing tools. The slurry is injected by the centered mixing rod. In the outer rods inclinometers are installed. These can be used to measure the inclination and calculate the deviations at the final mixing depth. The base machines are equipped with a production data visualization and storage system. Based on the production data, reports can be prepared.

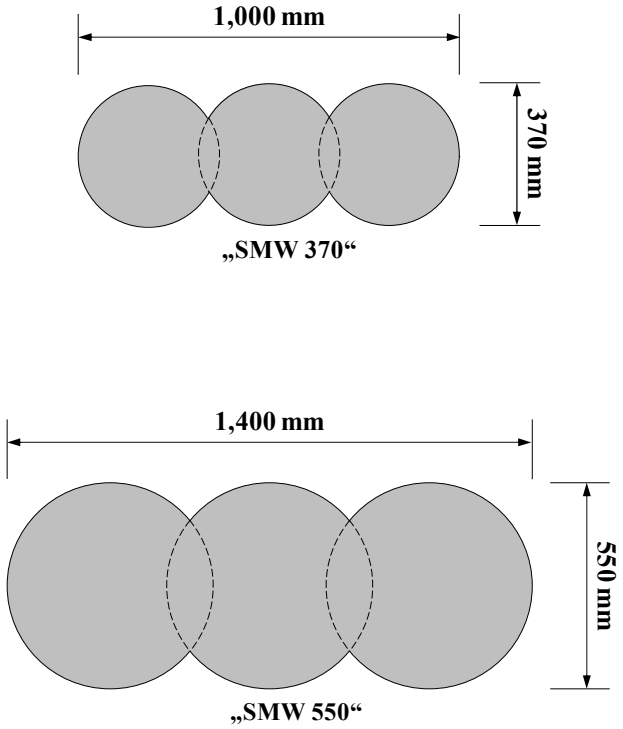



Figure 5: Dimensions of “SMW 550” and “SMW 370” mixing tools



Figure 6: Drill rig with SMW 550

Drill rigs and sheet piling rigs can be used as base carrier. Figure 8 shows a unit with SMW 550 and a bore pile drilling rig. The maximum mixing depth is limited by the mast length of the base unit. Common mixing depths for SMW are up to 23 m.

The additionally required slurry handling equipment is similar to the equipment mentioned for the CSM method.

### SMW Construction principle

The construction sequence for SMW is largely identical to the CSM method. The main difference is, that

for SMW typically the “wet-into-wet” method is used.

## JOBSITE EXAMPLES

As no reasonable references for soil mixing in India exist, the following references will give an idea of the potential use for the Indian market.

### Reference for CSM

For the construction of a seven story residential apartment complex located in Perth, Australia, a 550 mm thick CSM retaining wall was installed. The development included a two-level basement for car parking which covered the full site area. Furthermore, the basement was required to extend under an existing single story heritage building. The site was adjoined by roads on three sides. A single story residential property neighbored the site’s Eastern edge.

For this jobsite a BCM 5 mixing head and a RTG RG 19T (as base carrier) were used.

Several CPTs were performed as part of the ground investigation which indicated sand fill to approximately 2 m depth, subjacent a medium dense to dense sand to approximately 14 m depth, subjacent sandy clay, silty clay and clay. A typical CPT trace is shown in figure 9.

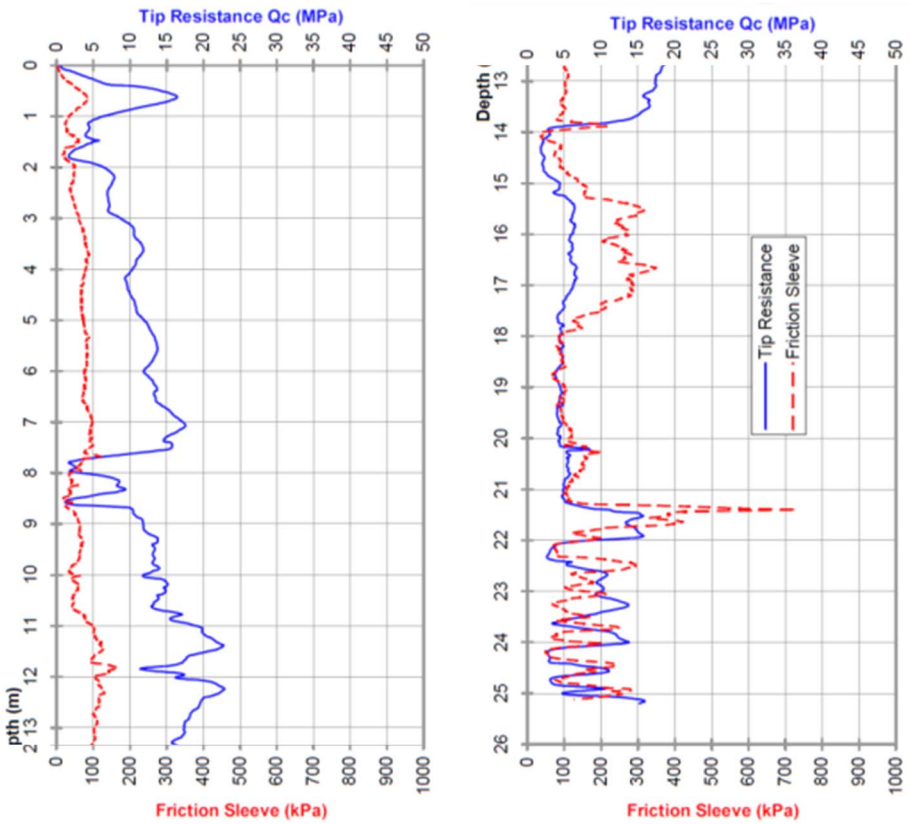



Figure 7: Representative CPT trace (Golder Associates, 2009)

The groundwater level at the time of the ground investigation (Spring) was 1 m below ground level. The design of the retaining wall was changed to a reinforced CSM wall by the construction company. The main

reasons to choose the CSM method were the higher productivity and lower costs of cement compared to concrete.

The design strength of the mixed soil was 5 MPa. The high water table and predominately sand conditions meant dewatering the basement for excavation was likely to be difficult. Consequently, the CSM panels would be installed into the indicated clayey soil layer encountered at approximately 14 - 16 m depth, in an effort to reduce dewatering requirements during basement construction.

An additional challenge was the heritage building which had to be integrated into the new building. To support the building during the construction of the parking garage, a concrete slab was installed gradually under the building. Then 6 underpinning CSM panels were installed, see figure 10. Every underpinning CSM panel was reinforced with 5x 360 UB.



Figure 8: Elevation of underpinned heritage building



Figure 9: Partly excavated pit at the east site with

The owners of the existing structure to the east of the site did not want anchors below their property. This resulted in a different excavation and support methodology for this area of the site. To gain access to the basement level, a temporary ramp was set up parallel to this boundary to provide support whilst the majority of the basement was excavated. The ground floor slab was cast up to the edge of the ramp and was used as a dead man system to prop to wall, see figure 11.

Figure 12 shows the construction site from a bird's eye view.

By using the CSM method a productivity of about 14 m² wall per “mixing hour” was achieved. The spoil volume during the wall production was approx. 50 % of the wall volume. The spoil became hard within hours after the mixing process and could be easily removed and disposed. Only four labours were required for the CSM works.

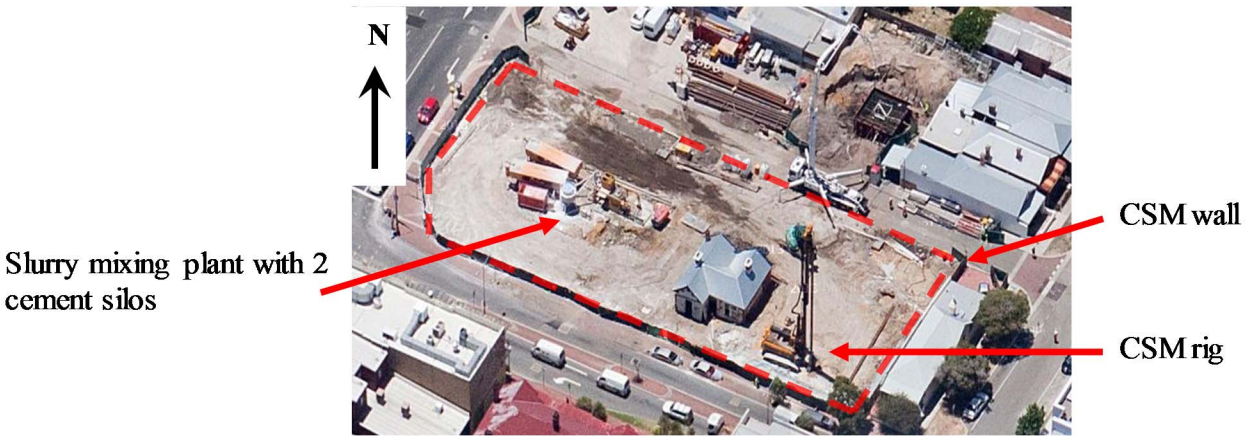



Figure 10: Site layout (Google Maps)

**Reference for SMW**

In Laudenbach, Germany, a soil mix wall with 13 m deep was executed for an undercrossing which is part of a new ring road. The original design for this jobsite was a diaphragm wall which was replaced by SMW.

A RTG RG 19T with SMW 550 was used for this jobsite, see figure 13.



Figure 11: Jobsite equipment, RTG RG 19T with SMW 550, slurry mixing station

The soil investigations showed a soft to medium clay layer in the first 3 meters. The following layers contained mainly medium to dense sand layers (figure 14).

For the execution of the wall, the pilgrim step method ("hit and miss method") was used (wet in wet), see figure 15. The overcut was 0.70 m. The production of each primary panel was completed within 40 - 45 minutes. Due to hard layers the penetration speed (downstroke) was around 0.3 – 0.4 cm/min in average. The average speed during the upstroke was 1.0 - 2.0 m/min. The production time of a secondary panel was

in average 25 minutes. The average penetration speeds varied between 0.6 to 0.7 m/min for the downstroke and 2.0 m/min during the upstroke. For the SMW method, three labours were required to operate the SMW unit, slurry supply equipment and an excavator.

During this jobsite a production rate of about 15 m² per hour could be achieved. The client was satisfied with the results of the SMW method, especially about the limited spoil volume during execution.

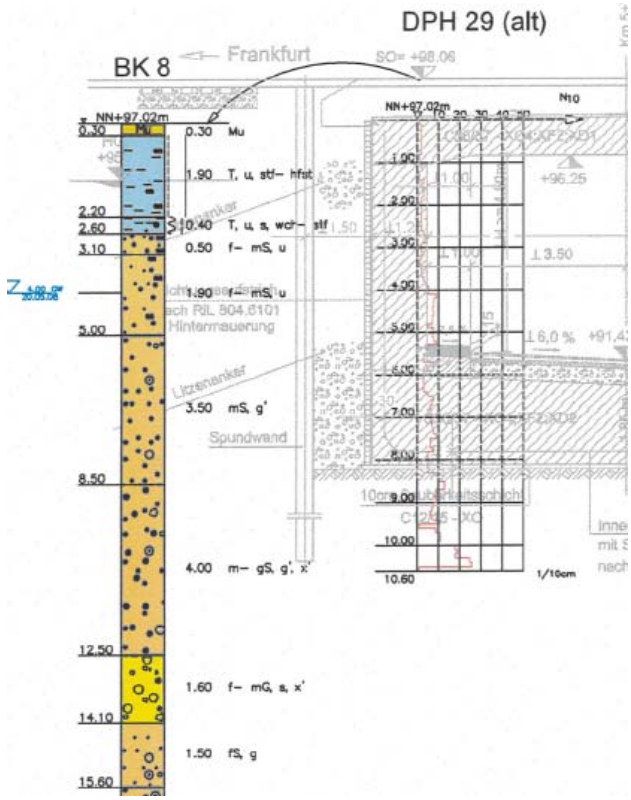



Figure 12: Exemplary borehole

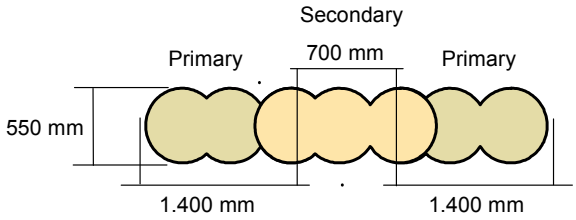



Figure 13: Principal panel layout with overcut

## CONCLUSION

Both soil mixing methods, provide a lot of opportunities to the foundation market. The main advantages compared to grabs are small spoil volume, low space requirement for jobsite equipment, no requirement for in time concrete deliveries and less operational staff.
Therefore, whenever timesaving and a reduction of spoil volume (e.g. when the soil is contaminated, i.e. high disposal costs) is (highly) demanded, one of the soil mixing methods shall be considered.

## REFERENCES

F.-W. Gerressen, October 2017, Soil Improvement Using Soil Mixing – An Alternative Way For the Indian Market, DFI India 2017, Chennai

F.-W. Gerressen, M. Schöpf, E. Stötzer, May 2009, CSM-Cutter Soil Mixing - Worldwide experiences of a young soil mixing method, Deep Mixing 2009, Okinawa

Donald Bruce, Deep Mixing in the United States: Milestones in evolution, Deep Mixing 2015, San Francisco 2015

# BACK ANALYSIS AND PERFORMANCE OF A CANTILEVER PORTION OF EXCAVATION IN MIXED SOIL PROFILE

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## ABSTRACT

A 600mm thick diaphragm wall of more than 16m deep was built as a retaining structure to support 12m deep basement in mixed soil conditions. First level of anchor was installed at 6m and second level was proposed at 10m depth below the existing ground level. About 6.5m excavation was required to install the first level anchor, across the construction site. Intensive monitoring and attention was called for during initial period of excavation due to 6.5m unsupported excavation, to build confidence within the teams. Comprehensive instrumentation using inclinometers, piezometers and total station measurements was in place to observe the wall movements and to monitor ground water table. Deflection patterns of the cantilever portion of the excavated wall at selected locations within the premises is discussed in this paper. Back analysis of the wall deflections has been carried out using WALLAP and findings are presented in this paper.

**Keywords:** Retention System, Diaphragm Wall, Cantilever, Ground Anchors, WALLAP, Back Analysis

## INTRODUCTION

In India, Chennai is one of the fast-developing cities and there is huge demand for underground constructions especially in the heart of the city for parking, retail shops and metro stations. Underground construction provides effective utilization of available land. More than 12.0m deep excavation was proposed for a commercial development in a mixed soil profile, comprising of firm to very stiff silty clay, dense sand and weathered rock. The project site is surrounded by main highway and residential buildings with limited setback. This scenario demands stable retention system for the deep excavation. Diaphragm wall of 600mm thick with 2 level of ground anchors was chosen as an economical solution. First level anchors were installed at 6m below the existing ground level (EGL) due to the structural connection requirements. Hence, it was necessitated to create a 6.5m cantilever wall so that first level of anchor can be installed at 6m below EGL. Intensive real time monitoring system was implemented to measure the wall deflection and ground water levels for entire period of excavation and basement construction. This paper discusses the performance of cantilever portion of the wall in the South and South-East portion of the site and back analysis was performed using WALLAP software to understand behavior of the wall.

## GEOLOGICAL CONDITIONS

Eighteen boreholes up to the hard strata were conducted throughout the site to understand the geological conditions of the proposed site. The subsoil consists of silty clayey soil with varying consistency up to 15m depth followed by dense silty sand till the weathered rock layer at 16m to 20m. Soil samples were collected at different intervals during the confirmatory soil investigation. Laboratory tests were performed to determine index properties of the collected soil samples. The soil idealization and design soil parameters were arrived based on the field and laboratory test data. Ground water table was recorded at about 4.0m below the existing ground level at the time of soil exploration.


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## PREDICTION OF WALL MOVEMENTS

### *Analysis and Design*

Since the proposed site is surrounded by residential buildings and highway within the influence zone, a stable retention system was required. A 600mm thick Diaphragm wall of about 19m deep with 2 level ground anchors was chosen and executed in this project. The construction sequence as considered in the initial analysis is described below.

- Installation of 600mm thick diaphragm wall
- Dewatering followed by excavation till 1st anchor level i.e., 6.5m
- Installation and prestressing of 1st level anchor
- Dewatering followed by excavation till 2nd anchor level i.e., 10.5m
- Installation and prestressing of 2nd level anchor
- Dewatering followed by excavation to final level
- Construction of permanent basement floor and slabs
- Destressing of ground anchors

WALLAP software was used to carry out the retention system analysis. This software uses limit equilibrium analysis to estimate the overall stability of wall and subgrade reaction modulus for predicting wall movements.

This paper concentrates on the performance of the cantilever portion of the wall at South and South East portion of the site. Only four boreholes located near the selected portion of the site are considered for the present study. The design soil parameters used for analysis of retention system is indicated in Table 1 and the idealized soil profile is presented in Fig. 1. Ground water table was recorded at 4.0m below the existing ground level at the time of soil exploration.

**Table 1. Design Soil Parameters**

| SI. No. | Layer thickness, m | Soil Type | SPT N value | Cu, kPa | Φ, deg | E', kPa |
|---|---|---|---|---|---|---|
| 1 | 3.0 | Stiff Silty Clay | 15 | 75 | - | 22,500 |
| 2 | 2.0 | Very Stiff Silty Clay | 18 | 90 | - | 36,000 |
| 3 | 2.0 | Stiff Silty Clay | 12 | 60 | - | 18,000 |
| 4 | 4.0 | Very Stiff Silty Clay | 20 | 100 | - | 40,000 |
| 5 | 3.5 | Firm Silty Clay | 8 | 40 | - | 8,000 |
| 6 | 2.0 | Dense Silty Sand | 60 | - | 37 | 1,20,000 |
| 7 | 3.5 | Jointed Rock | Refusal | 500 | - | 2,50,000 |

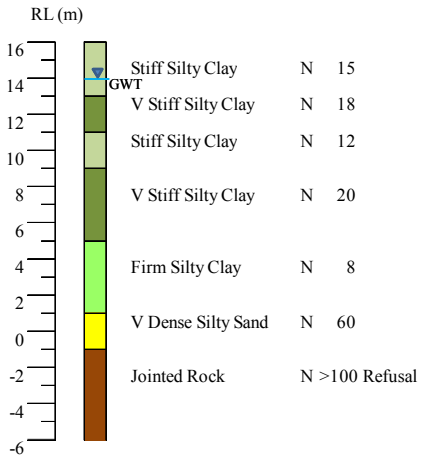



**Fig. 1. Idealized soil profile**

### *Horizontal Wall Movement*

In the initial stage, the subsoil on both sides of wall will be in at rest condition. Soil behind the wall will experience unloading and reloading behavior during excavation process. This will cause the cantilever type of horizontal movements of wall (Suched et.al. 2013). Ground water table was considered at 2m below EGL for performing the analysis. Also, a surcharge of 15kPa was considered to simulate machinery and transportation loads behind the wall for all stages of construction. Typical wall model and predicted horizontal wall deflection with respect to reduced level is presented in Fig. 2 for first stage of excavation. The predicted maximum wall deflection at wall top is about 50mm.

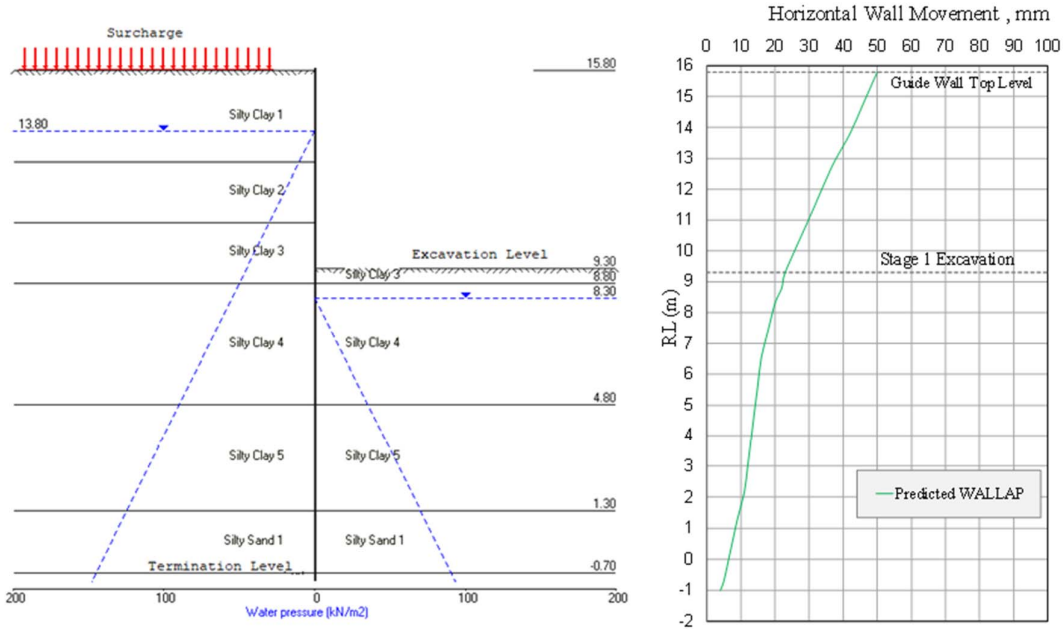



**Fig. 2. WALLAP model and predicted horizontal wall movements**

## INSTRUMENTATION & PERFORMANCE OF WALL

### *Instrumentation*

Deflection monitoring was carried out through four inclinometers and ground water table was monitored through 4 piezometers during the progress of excavation. The location of inclinometers is shown in Fig. 3. The inclinometers are installed approximately middle of the diaphragm wall panel. The position of inclinometer in the diaphragm wall panels is illustrated in Fig. 4.

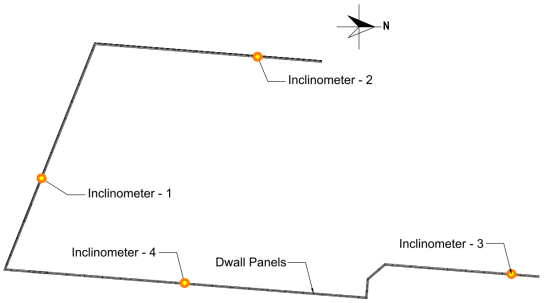



**Fig. 3. Plan showing location of Inclinometers**

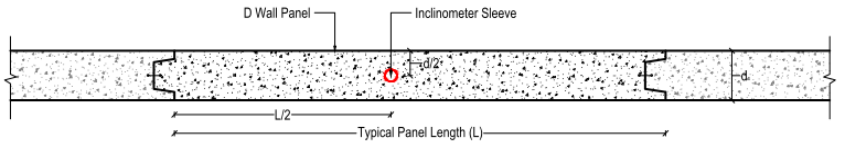



**Fig. 4. Position of inclinometer in diaphragm wall panel**

***Measured Horizontal Wall Movements of 6.5m unsupported excavation***

Wall deflection was monitored continuously during excavation of wall and movements were recorded at regular depth intervals of 0.5m. It was observed that a maximum wall deflection of up to 20mm at top was recorded for the cantilever portion of the wall against predicted wall movement of 50mm. The predicted and observed wall movements are presented in Fig. 5.

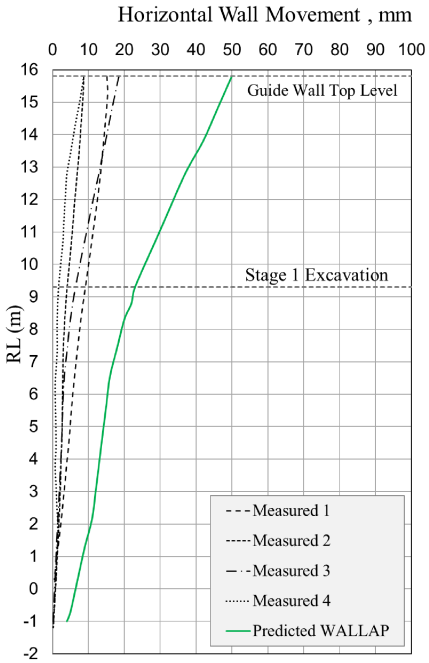



**Fig. 5. Predicted and measured wall movements**

## BACK ANALYSIS

The performance of the cantilever wall was well within the predicted movement. This has given an opportunity to relook into design considerations and to understand the behavior of soil structure interaction. Some of the key considerations such as SPT N correlations with respect to cohesion and soil stiffness, variation in the ground water table, drained and undrained behavior of soil are focused in back analysis.

***Active Earth Pressure for Cantilever (Drained and Undrained)***

Under undrained condition, the cohesion component of the active earth pressure is negative thereby reducing the cumulative earth pressure. The cohesion component reduces during drained condition which leads to increase in active earth pressure from the undrained condition. This allows more lateral pressure on the wall and results in increasing the wall deflection. Undrained conditions are commonly assumed in the temporary excavation supports for the duration not more than six months and clay soil

with low permeability (CIRIA 580). The field measurements will be considerably less than the predicted, in case if the construction is completed in shorter period (Wanchai et.al. 2019). Checking the clayey soil behavior both in undrained and drained condition is the best practice, when the excavation period is expected to be large allowing drainage.

Drained analysis is carried out considering drained soil parameters for the top three clayey layers (up to cantilever portion of excavation). The predicted wall deflection is 70mm at top against 50mm under undrained condition (Refer Fig. 2).

***Influence of Ground water table***

Ground water table of 2m below EGL was considered in the initial analysis. However, the piezometers data reveals that the actual ground water table is at 5m below EGL during construction and excavation period. Since the encountered soil strata is cohesive in nature, the influence of ground water is not significant in case of undrained analysis, as the soil above the water table is close to full saturation. The effect of lowering of ground water table to 5m below EGL in case of drained conditions indicates a wall deflection of 57mm.

***Design Soil Parameters***

The maximum wall movement observed during excavation is in the range of 10mm to 20mm. The water table stood at 5.0m depth. Since the excavation duration was limited to less than three months, the undrained behavior is applicable. The estimated wall displacement was about 50mm based on the average design parameters (Refer Fig. 2). Back analysis is performed using revised design soil parameters, considering the upper limits of the shear strength and stiffness parameters. The soil stiffness which has great influence on the wall movements need to be thoroughly considered (Tan et.al 2001). Various trials with different combination of soil strength and stiffness parameters are made to match the predicted and measured deflections. An attempt was made to increase shear strength parameters to 20% and stiffness parameters to 30% under undrained condition (increase $E_s$ to 400 to 500$C_u$). Initially, cu/N = 5 (Stroud 1974) was considered in the original analysis and the same has been revised to cu/N = 6. The comparison of horizontal wall movements is illustrated in Fig. 6.

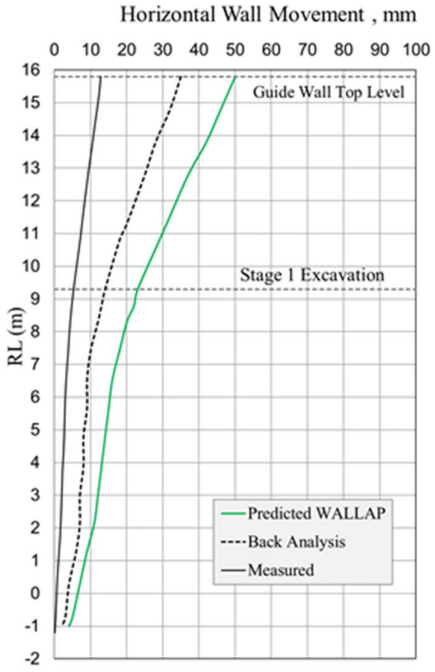



**Fig. 6 Wall movements from back analysis**

Accordingly, increase in soil stiffness was considered in the back analysis. Based on the outcome of the analysis, it is found that the reduction of wall deflection is small. The revised analysis indicates the wall deflection of 35mm against to the originally predicted deflection of 50mm.

***Influence of Wall Penetration into Weathered Rock***

The diaphragm wall was socketed 700mm into weathered rock as this strata was encountered at shallow depth (say 17m) than the proposed termination depth of 18m. Fixity of diaphragm wall would have been achieved considerably due to socketing of wall into weathered rock. The embedment of wall into weathered strata helped in restricting the wall movements.

## CONCLUSIONS

The performance of 6.5m cantilever wall in the mixed soil conditions is analyzed and compared with the actual deflection. The deflection pattern is reasonably matching with the predicted deflection. Since the installed wall is designed to support 12m deep excavation, the influence of deeper wall makes the cantilever portion stiffer. This could be one of the reasons, the actual measured deflections are lower than the predicted deflections. In addition, embedment of wall into weathered rock would have helped in achieving fixity and lead to restrict the deflection of wall. Also, it is concluded that no change in wall movements is observed due to lowering of ground water table, especially in cohesive soils for short term conditions. A slight increment of design soil parameters can be considered for design of retention systems of similar environment.

## ACKNOWLEDGEMENTS

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## REFERENCES

1. CIRIA 580, Embedded Retaining Walls – Guidance for Economic Design.
2. Likitlersuang S, Surarak C, Wanatowski D, Erwin Oh, and A. Balasubramaniam, 2013. Finite element analysis of a deep excavation: A case study from the Bangkok MRT, Journal Soils and Foundations by The Japanese Geotechnical Society. Elsevier B.V., Volume 53, Issue 5, October 2013, Pages 756-773.
3. Stroud, M. A. 1974. The Standard Penetration Test in Insensitive Clays and Soft Rocks, 1st European Conference on Penetration Testing, Vol.1, pp. 372 – 373.
4. Tan Y. C., Liew S.S., Gue S. S., and Taha M.R., 2001, A Numerical Analysis of Anchored Diaphragm Walls for a Deep Basement in Kuala Lumpur, Malaysia.
5. Wanchai Teparaksa and Jirat Teparaks, Jan 2019. Comparison of diaphragm wall movement prediction and field performance for di erent construction techniques, Science Direct.

# DESIGN OF DEEP VERTICAL CIRCULAR DIAPHRAGM WALL USING FINITE ELEMENT ANALYSIS

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## ABSTRACT

Diaphragm walls are often used as earth retention support system for enabling excavations especially when the water table is at shallow depth. Circular diaphragm walls can be used for deep shafts because of their ability to generate hoop forces and thereby eliminating the necessity of struts or anchors. Earth pressure acting on circular retention system is expected to be less when compared with plane strain retention systems because of three-dimensional soil arching effect. This paper presents analysis and design of deep vertical circular diaphragm wall shaft. A two-dimensional axisymmetric finite element analysis is carried out for both serviceability (SLS) and ultimate limit state (ULS) as per Eurocode-7 design approach. The performance of circular diaphragm wall shaft was investigated by simulating various construction stages of excavation and dewatering. The earth pressures obtained from circular axisymmetric finite element analysis are compared with available analytical solutions and are found be in reasonable agreement.

**Keywords: axisymmetric, finite element analysis, circular diaphragm wall.**

## INTRODUCTION

With the change in life style and employment opportunities resulted rapid growth of urbanization throughout world. But, the habitable land is fixed, and humans started using the effective way by either high raised or deep excavated structures. Deep excavations are often carried out for optimum utilization of underground space especially for water and sewerage structures. Diaphragm walls are often used as retention support systems for enabling excavations. Evaluation of earth pressure is one of the important aspects for the design of earth retention systems. Conventional earth pressure theories like Rankine's (1857) or Coulomb's (1773) are useful for plane strain conditions only. Historically, study of the lateral earth pressure was focused on plane strain conditions. Previous studies on earth pressure against non-plane strain sections like circular shaft include Terzaghi (1943), Berezantzev (1958), Prater (1977) etc. which are based on limit equilibrium methods or slip line solutions. Also, most of the design standards or guidelines such as Eurocode or CIRIA 760 guidelines used in engineering practice do not adequately address the earth pressure calculation for retaining systems other than plane strain.

Lateral earth pressure acting on circular retaining walls generate hoop forces as internal reactions. The hoop forces generated all around the wall yield less bending moment and shear forces in wall due to lateral earth pressure when compared with plane strain condition and thereby eliminating the necessity of struts or anchors. In addition, circular retention systems have reduced earth pressures due to three-dimensional soil arching effect (Kim et al. 2013). Arching is a process of rearrangement of stresses around a region of soil mass, which then becomes subject of lower stress (Paik and Salgado, 2003).

This paper discusses the design and analysis method adopted for circular diaphragm wall shaft for one of the projects of Jacobs Engineering. The design generally involves, verification of diaphragm wall for ultimate Limits state (ULS) and Serviceability limit sate (SLS) conditions. Based on schedule of the project, availability of machinery and local experience of contractors, diaphragm wall was selected as suitable earth


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retention support system. The earth pressures based on axisymmetric finite element analysis were compared with available theoretical solution of Kim et al. (2013) to verify and demonstrate the effectiveness of finite element analysis of circular diaphragm walls.

## SITE CONDITIONS AND SOIL PROPERTIES

### Subsurface Stratigraphy

Based on the available site investigation data and laboratory test results, the classification of soil and rock was arrived as per BS 5930 and local practice. The subsurface stratigraphy consists of reclaimed fill layer of approximately 15.0m thick from EGL followed by a soft marine clay layer with thickness of 7.0m. The Marine clay layer is followed by S(VI) grade residual soil formation (stiff silt) of 20.0 m thickness. S(VI) grade soil is underlain by completely weathered S(V) grade rock formation of maximum thickness 46.0 m mainly in the form of sandy silt. The S(V) grade rock is resting on medium strong to strong, moderately weathered S(III) grade formation of Sandstone with interbedded Siltstone with traces of S(IV) grade rock found in few boreholes. Sandstone and siltstone were encountered independently and interbedded with each other at different depths in different boreholes. The completely weathered S(IV) grade rock traces are ignored and are considered as completely weathered S(V) grade rock formation for conservative ground model. Based on the information revealed from all the boreholes and previous experience, a design ground model is established for the design of diaphragm walls of circular shaft. The ground water is observed at a depth of 2.0 m from EGL and design ground water level considered at EGL.

### Soil Properties

Table 1 presents the characteristic design parameters of soils/rock used in the present axisymmetric finite element analysis. Effective shear strength parameters are used for drained analysis. The shear strength parameters are derived from available laboratory and field test data with due consideration being given to local experience as well.

**Table 1 Characteristic Design Parameters of Soil and Rock**

| Soil /Rock type | Thickness (m) | Effective shear strength parameters | | Undrained shear strength (Su, kPa) | Youngs modulus Eu (MPa) (undrained) |
|---|---|---|---|---|---|
| | | Cohesion (kPa) | Friction angle (°) | | |
| Reclaimed Fill | 15.0 | 0 | 30 | - | 10 |
| Marine clay | 7.0 | 1 | 22 | 27 | 5.5 |
| S(VI) | 20.0 | 3 | 30 | - | 47 |
| S(V) | 46.0 | 10 | 30 | - | 88 |
| S(III) | 97.0 | 120 | 35 | - | 200 |

## DESIGN ASSUMPTIONS AND PROJECT REQUIREMENTS

The diaphragm wall is designed for a maximum depth of excavation of 82.5 m from EGL with diameter of 36.0m as per the project requirement. Additional excavation (unplanned) of 10% of retained height, to a maximum of 0.5m has been accounted on the passive side for the ultimate limit state case (ULS) as per Eurocode 7, part 1 (BS EN 1997-1:2004 here after referred as EC-7) design guidelines (Cl. 9.3.2.2). A general surcharge load of 20 kPa is assumed to act at an offset distance of 2.0m from the outer edge of diaphragm wall. Ground water level is assumed to be hydrostatic with initial ground water level at EGL. Dewatering is considered in the analysis and water level during each excavation stage is maintained least 0.5m below the planned excavation level.

## DESIGN APPROACH

Serviceability Limit State (SLS) and Ultimate Limit State (ULS) analyses are carried out in the present axisymmetric finite element modelling in PLAXIS 2D. As per EC-7 design approaches (Cl. 2.4.7.3.4), ULS calculations are performed by adopting Design approach 1, which includes combination 1 and combination 2 to evaluate the maximum forces in the diaphragm wall. Tables 2 and 3 show the partial factors adopted in the analysis for actions and materials.

**Table 2 Partial Factors ($\gamma_F$) on actions or effects of actions**

| Action | | Design Approach | |
|---|---|---|---|
| | | DA1-1 | DA1-2 |
| Permanent | Unfavourable | 1.35 | 1 |
| | Favourable | 1 | 1 |
| Variable | Unfavourable | 1.5 | 1.3 |
| | Favourable | - | - |

As per EC-7-1 Cl. 2.4.7.3.2, the application of partial factors to actions coming from or through the soil (such as earth or water pressures) could lead to design values which are unreasonable or even physically impossible. Hence, factors are applied directly to the effects of actions derived from representative values of the actions. Therefore, in the current analysis for DA1-1, partial factors are factored down by 1.35 (1.5/1.35 =1.1) i.e. the **($\gamma_F$)** for variable – unfavorable actions is 1.1 and rest of partial factors considered as 1.0. Partial factor of 1.35 is applied directly to the effects of actions (computed forces).

**Table 3 Partial Factors for soil parameters**

| Parameter | Design Approach | |
|---|---|---|
| | DA1-1 | DA1-2 |
| Angle of shearing resistance | 1 | 1.25 |
| Effective cohesion | 1 | 1.25 |
| Undrained shear strength | 1 | 1.4 |
| Unconfined strength | 1 | 1.4 |
| Weight density | 1 | 1 |

## FINITE ELEMENT MODEL DESCRIPTION

A two-dimensional axisymmetric finite element model was developed in PLAXIS 2D (version 2017) to assess the performance of the circular diaphragm wall and simulate various construction stages including excavation and dewatering for the construction of diaphragm wall. The proposed construction sequence is

simulated to estimate the probable maximum structural forces and lateral deflections in the circular diaphragm wall shaft. Diaphragm walls are constructed as panels and are discontinuous in circumferential direction. The discontinuity can be accounted in modelling by considering diaphragm wall as orthotropic element Zdravkovic et al. (2005). Orthotropic analysis can be carried out by reducing stiffness in circumferential direction. Previous Studies on stiffness reduction include Cabarkapa et al. (2003), Zdravkovic et al. (2005) and Cui Jian et al. (2017). The value of reduction factor adopted in previous studies varied from 1 (i.e. isotropic) to a maximum of 10. In the current analysis, to estimate maximum forces along the diaphragm wall, the stiffness in circumferential direction is reduced by factor of 10 for orthotropic analysis. The ring beams are modelled as plate element at every 5.0m vertical spacing, which provides an additional stiffness to stabilize the circular diaphragm wall. To generate maximum hoop forces, the diaphragm wall is also modelled as isotropic element for evaluation of hoop forces.

The dimensions of the axisymmetric finite element model used in the analysis are 110.0m (width) x 185.0m (depth). The edges of the model are kept far away from outer edge of shaft to reduce the boundary effects. Figure 1 shows 2D axisymmetric finite element model including diaphragm wall, ring beams and soil stratigraphy. The bottom boundary was fixed in both vertical and horizontal axis directions and the vertical boundaries were fixed only in horizontal direction. The soil layers are modelled using Hardening-soil (HS) model which, best represents soil behavior for deep excavation problems. Several PLAXIS runs are carried out to find the feasible thickness of diaphragm wall from construction and strength and ended up with 1.20 m thick. Hence, uniform thickness of 1.2 m of diaphragm wall is considered for stability of the excavation support system. The dimensions of the ring beam are 0.5m thick and 1.0m deep. The diaphragm wall and ring beam are modelled as plate element with Young's modulus of 34 GPa. The excavation of soil is planned to carry out in 5.0m intervals with installation of ring beams at every 5.0m depth along the depth of diaphragm wall.

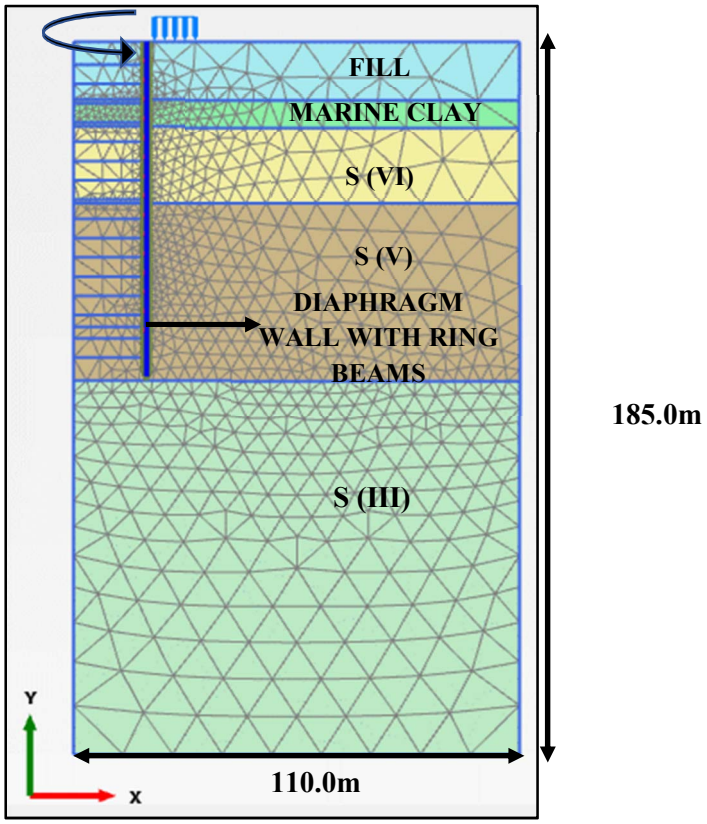



**Fig. 1. Axisymmetric PLAXIS 2D Finite element mesh showing soil stratigraphy and diaphragm wall**

After reaching final excavation level, base raft is activated. Design ground water level is considered at existing ground surface on soil retaining side. On excavation side, it is assumed that dewatering shall be performed during excavation by maintaining at least 0.5m below excavation level.

For Drained analysis, the sub-soils are defined as drained materials and assigned effective stress parameters (C' & φ'). For undrained analysis, two approaches have been carried out in the present analysis; undrained-A and undrained-B. As recommended by PLAXIS manual, modelling of undrained-A analysis is carried out using effective stress parameters (C' & ϕ') and marine clay layer is set as undrained-A. In this case, pore pressure provides right effective stress path that leads to failure at a realistic value of undrained shear strength (Cu or Su). For undrained-B analysis, undrained shear strength parameters and effective stiffness parameters are used for marine clay layer.

## RESULTS AND DISCUSSION

### *Results of finite element analysis*

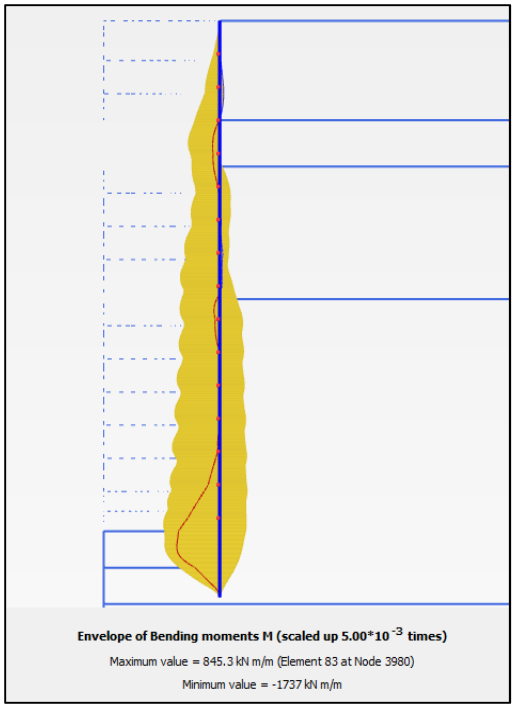
Table 4 summarizes bending moment and shear forces obtained from finite element analysis. Isotropic analysis yielded critical forces for hoop compression whereas, orthotropic analysis yielded critical forces in bending, shear and deflection. Figure 2 shows critical bending moment and shear force envelopes of diaphragm wall for design approaches. For orthotropic analysis, the earth pressure is transferred through bending and shear because of reduced stiffness in circumferential direction due to discontinuities of diaphragm wall during construction. Due to uniform stiffness in horizontal and vertical direction of circular diaphragm wall, isotropic analysis yielded critical values for hoop forces indicating that major force due to earth pressure is transferred by hoop action. Figure 3 show results of displacements for serviceability limit state analysis. The maximum deflection of the diaphragm wall for serviceability limit state is within the acceptable limits of client's project specifications. The maximum hoop forces envelope along the length of the diaphragm wall is shown in Fig. 4.

**Table 4 Summary of Bending moment and Shear Forces for Ultimate Limit State (ULS)**

| | | Forces in Diaphragm wall | |
|---|---|---|---|
| Limit state | Analysis type | Bending Moment (kN.m/m) | Shear force (kN/m) |
| DA1C1 | DRAINED | 2321 | 511 |
| | UNDRAINED A | 2345 | 473 |
| | UNDRAINED B | 2335 | 509 |
| DA1C2 | DRAINED | 1344 | 241 |
| | UNDRAINED A | 1355 | 241 |
| | UNDRAINED B | 1356 | 258 |

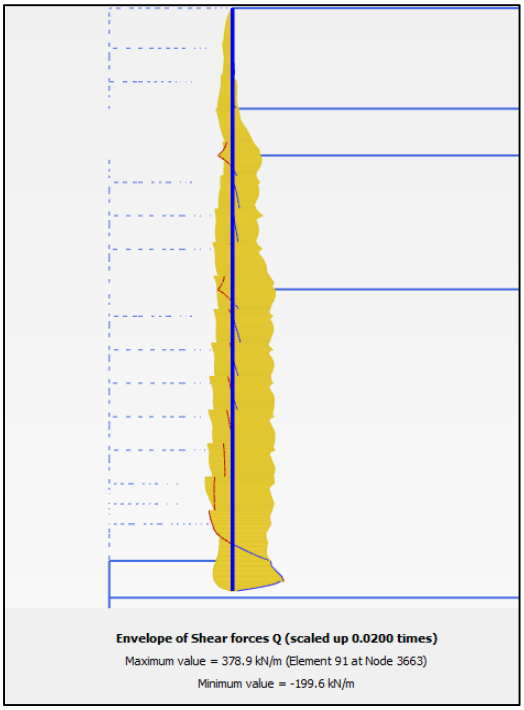
**Table 5 Summary of Deflections for Serviceability Limit State**

| Limit state | Analysis type | Deflection (mm) |
|---|---|---|
| SLS | DRAINED | 93 |
| | UNDRAINED A | 92 |
| | UNDRAINED B | 92 |




**(a)**

**Fig. 2. a) Bending Moment envelope (unfactored) of diaphragm wall for Undrained A analysis (DA1-C1)**




**(b)**

**Fig. 2. b) Shear force envelope (unfactored) of diaphragm wall for Drained analysis (DA1-C1)**

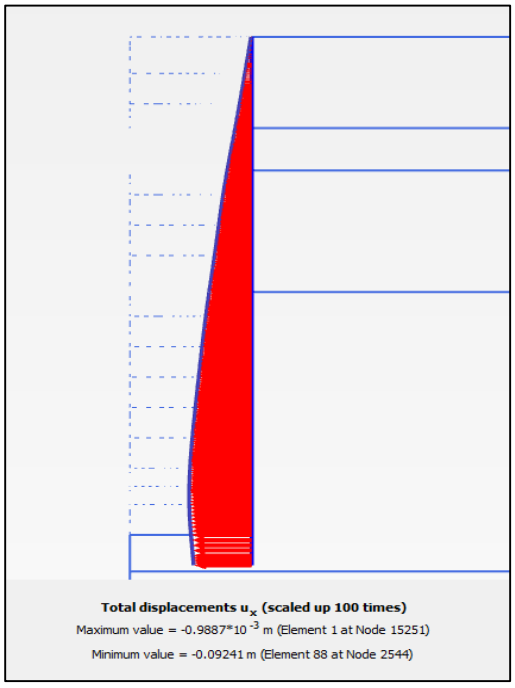



**Fig. 3. Horizontal deflection of diaphragm wall for drained analysis (SLS)**

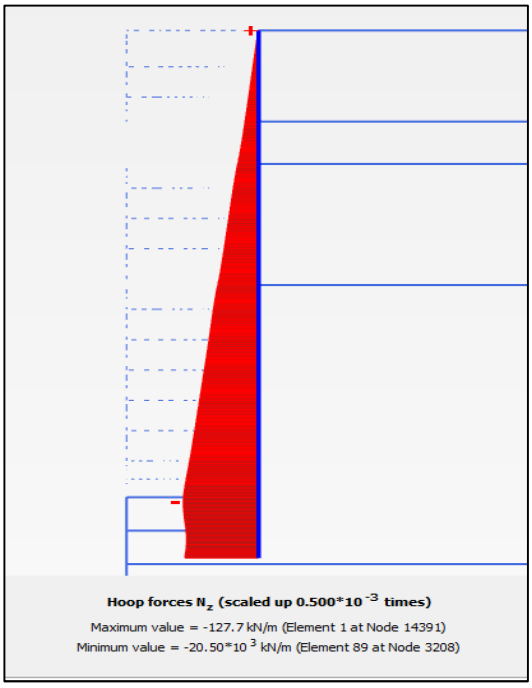



**Fig. 4. Hoop force (unfactored) on diaphragm wall for Drained Analysis (DA1-C1)**

The reduction of stiffness in circumferential direction to simulate orthotropic behavior of has shown considerable effect on bending moment in the diaphragm wall. Table 6 shows bending moment values for both isotropic and orthotropic analysis. It can be observed that due to reduction of stiffness by factor of 10 in circumferential direction, the increase in bending moment is more than 5 times for DA1C1 and more than 2 times for DA1C2. Figure 5 shows bending moment envelopes for undrained A analysis of governing combination DA1C1 for both isotropic and orthotropic analysis. However, orthotropic analysis generated less hoop force in circumferential direction as expected when compared with isotropic analysis. The reduction of hoop forces due to orthotropic behavior from isotropic behavior of diaphragm wall in the present analysis is approximately 25 to 35 percent. Table 7 shows hoop forces calculated for various analysis. This shows that the force transfer behavior is changing from hoop in circumferential direction for isotropic analysis to bending action in the diaphragm wall for orthotropic analysis.

**Table 6 Comparison of bending moments for orthotropic and isotropic analysis**

| Bending Moment (kN.m /m) | | | | |
|---|---|---|---|---|
| Analysis | DA1C1 | | DA1C2 | |
| | ISOTROPIC | ORTHOTROPIC | ISOTROPIC | ORTHOTROPIC |
| DRAINED | 437 | 2321 | 603 | 1344 |
| UNDRAINED A | 440 | 2345 | 546 | 1355 |
| UNDRAINED B | 440 | 2336 | 573 | 1356 |

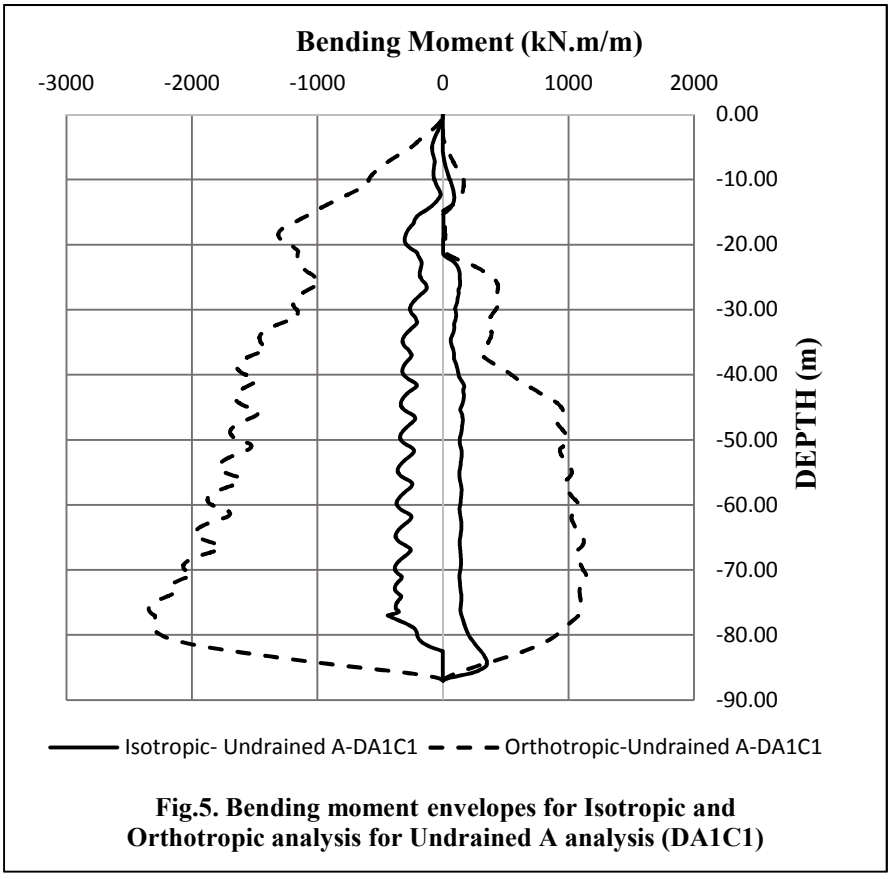



**Fig.5. Bending moment envelopes for Isotropic and Orthotropic analysis for Undrained A analysis (DA1C1)**

**Table 7 Comparison of hoop forces for orthotropic and isotropic analysis**

| Hoop Force (kN/m) | | | | |
|---|---|---|---|---|
| Analysis | DA1C1 | | DA1C2 | |
| | ISOTROPIC | ORTHOTROPIC | ISOTROPIC | ORTHOTROPIC |
| DRAINED | 27680 | 20345 | 24966 | 15824 |
| UNDRAINED A | 27318 | 20380 | 24090 | 15830 |
| UNDRAINED B | 27544 | 20393 | 24511 | 15749 |

***Comparison with analytical solution***

To verify the appropriateness and demonstrate the effectiveness of axisymmetric finite element analysis of circular diaphragm wall, the lateral earth pressures obtained from the analysis are compared with theoretical solution of Kim et al. (2013). Figure 6 shows lateral earth pressures acting on vertical circular shaft calculated by theoretical approach given by Kim et al. (2013) and earth pressures obtained in the present finite element analysis of circular diaphragm wall. The earth pressure from results of finite element analysis correspond well with theoretical solution given by Kim et al. (2013) and a reasonably good agreement between finite element analysis and theoretical method is observed. The maximum lateral earth pressure occurred at approximately 40.0-42.0m below the existing ground level for both finite element analysis and theoretical solution results. Decrease in lateral earth pressure from maximum value is observed indicating soil arching effect from both theoretical approach and finite element analysis.

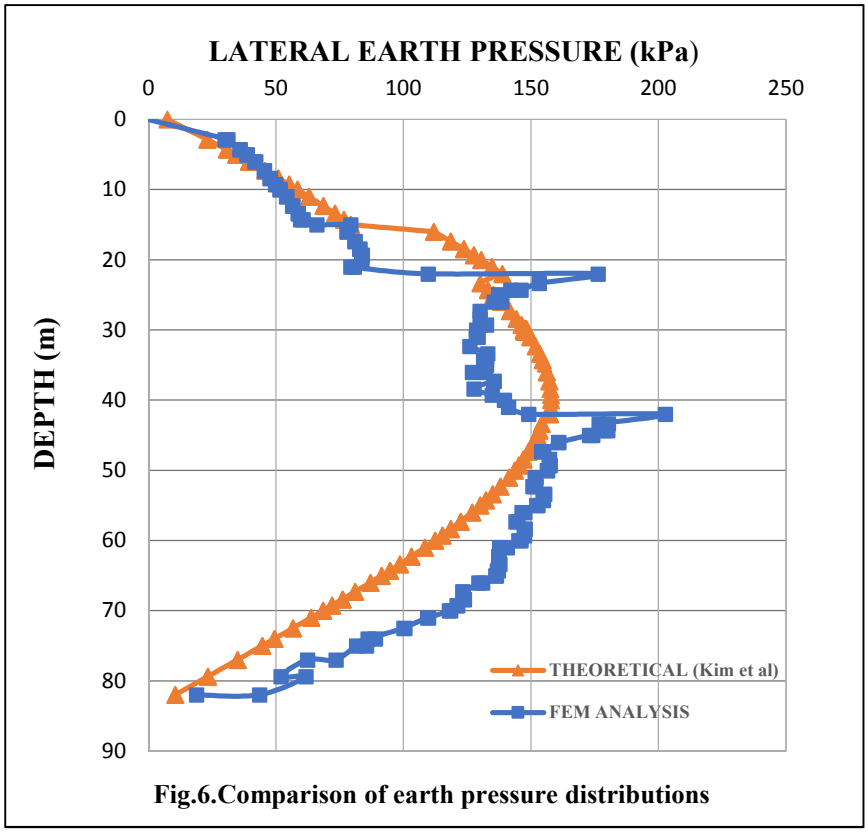



**Fig.6.Comparison of earth pressure distributions**

## SUMMARY AND CONCLUSIONS

This manuscript highlights the use of axisymmetric finite element analysis for the design of deep vertical circular diaphragm wall as per Eurocode-7. Isotropic and orthotropic analysis of circular diaphragm wall is carried out to evaluate maximum bending moment, shear force and hoop forces. Due to uniform stiffness in isotropic analysis of diaphragm wall, maximum hoop forces are generated. To account for discontinuities in diaphragm wall panels during construction, orthotropic analysis is carried out which yielded maximum bending moment, shear forces and deflection. The reduction of stiffness in circumferential direction by factor of 10 has shown to increase bending moment by more than 5 times, which shows the importance of analyzing orthotropic behavior of circular diaphragm walls. Orthotropic analysis has generated less hoop forces in the diaphragm wall by 25 to 35 percent approximately when compared with isotropic analysis as the force transfer due to earth pressure is predominantly through bending action in orthotropic behavior of diaphragm wall. The present study considers reduction factor of 10 for stiffness for orthotropic analysis to evaluate maximum forces in the diaphragm wall, however the realistic reduction factor of stiffness in circumferential direction will be less than 10 and depends on construction of the diaphragm wall panels. The lateral earth pressures thus obtained from axisymmetric finite element analysis are compared with available theoretical solution and a good agreement is observed indicating the efficacy of finite element analysis for deep circular excavations.

## ACKNOWLEDGMENTS

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## REFERENCES

Berezantzev, V. G. (1958). Earth pressure on the cylindrical retaining wall. In: Proceedings Conference on Earth Pressure Problems, volume 2, pages 21–27. Brussels.

BS 5930 BSI (1999) BS 5930:1999+A2:2010: Code of practice for site investigations. London, UK, BS.

BS EN 1997-2004, Eurocode 7, Geotechnical Design- Part 1 General Rules.

Cabarkapa, Z., Milligan, G.W. E., Menkiti, C. O., Murphy, J., and Potts, D. M. (2003). Design and performance of a large diameter shaft in Dublin boulder Clay. In: Newson, T. A. (ed.), Proceedings of the BGA International Conference on Foundations, pages 175–185. Thomas Telford Ltd., London.

Coulomb, C. A. (1776). Essai sur une application des regles des maximis et minimis a quelques problems de statique relatifs a l'architecture. In: Memoires de Mathématique et de Physique, Présentés a l'Académie Royale des sciences, par divers Savants, et lûs dans ses Assemblées, Paris, pages 343–382.

CUI Jian-feng, HU Jian-hua, HE Wei, HE Yao-bei. Hoop stiffness and its reduction factor of diaphragm wall panels for circular excavation Chinese Journal of Geotechnical Engineering Vol.39 No.11, Nov 2017

Gaba, A., Hardy, S., Doughty, L., Powrie, W.,and Selemetas, D. (2017). CIRIA C760 Guidance on embedded retaining wall design. CIRIA, London.

Kim, D.H., Cha, M.H., Lee, D.S., Kim, K.Y., Lee, I.M., 2009. Earth pressure acting on vertical circular shafts considering arching effects in soils. Tunell. Technol. 12 (2), 129–144.

Kim, K.Y., Lee, D.S., Cho, J.Y., Jeong, S.S., 2013. The effect of arching pressure on a vertical circular shaft. Tunneling and Underground Space Technology. 37, 10–21.

Paik, K.H., Salgado, R., 2003. Estimation of active earth pressure against rigid retaining walls considering arching effects. Geotechnique 53 (7), 643–653.

PLAXIS 2D (2017) Plaxis reference manual 2017, Delft, the Netherlands.

Prater, E. G. (1977). Examination of some theories of earth pressure on shaft linings. Canadian Geotechnical Journal, 14(1):91–106.

Rankine, W.J.M., 1857. On the stability of loose earth. Philos. Trans. Royal. Soc., Lond. 147 (1), 9–27.

Terzaghi, K. (1943). Theoretical Soil Mechanics. John Wiley & Sons, New York.

Wong, R. C. K. and Kaiser, P. K. (1988). Behaviour of vertical shafts: Reevaluation of model test results and evaluation of field measurements. Canadian Geotechnical Journal, 25(2):338–352.

Zdravkovic, L., Potts, D. M., and St. John, H. D. (2005). Modelling of a 3D excavation in finite element analysis. Geotechnique, 55(7):497–513.

# EFFECT OF ELASTIC NON-LINEARITY OF SOIL AND PILE ON UPLIFT CAPACITY OF TRANSMISSION TOWER

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## ABSTRACT

The transmission line tower foundations are subjected to significant uplift loads during its design life due to damage forces, wind forces and conductor tensile forces acting on the super structure.. In India the transmission towers are designed and tested as per IS:802:1995(2006).The IS:4091:1979 provides guideline for design of foundation for transmission line structure. Present study is an attempt to determine load-settlement characteristics of model tension transmission tower of 220KV capacity embedded in silty-sand deposits using bored cast-in situ pile of L/d ratio 4,6 and 8( where L= length of pile , d= diameter of pile). The uplift capacity is determined using specialised fabricated tension load frame of 10T capacity. The experimental procedure includes determination of index properties and engineering properties of field soil, casting of concrete piles, fabrication of model tower and load tests on tower-foundation system. The load response of tower-foundation system towards different loads and displacements was analysed. The lateral load-displacement curves were obtained using test data. The vertical displacement and settlements were measured to obtain ultimate load carrying capacity of foundation .The effect of L/d ratio on the ultimate capacity of foundation and differential settlement was analysed. Using Brom's method for short pile in cohesionless soil the ultimate soil resistance per unit length was determined for each slenderness ratio of the pile . For every failure load the pile displacements and rotations were obtained. The shear force and bending moment distribution at the failure were obtained along the length of pile.

**Key words: Uplift capacity, Transmission tower, Pile foundation**

## INTRODUCTION

A transmission tower structural system are generally consists of overhead power transmission lines, steel lattice tower structures as super structure and foundations. The foundations are generally located at the four corners (legs) of the tower structure. The foundations of transmission towers can be classified as axial load foundations and moment load foundations. Axial load foundations indicate the cases where lateral load acting on a tower would be transferred as uplift and compressive loads on the individual foundations at each corner. Pile foundations are considered as axial load foundations, which are effective in resisting lateral tower loads but vulnerable to differential settlements. The transmission tower foundations are subjected to significant uplift loads during its design life due to damage forces, wind forces and conductor tensile forces acting on the super structure. So it is important to determine the uplift capacity of foundation. There are many theories available to determine the uplift capacity of piles such as Vesic 1971; Meyerhof 1973;Awad & Ayoub 1976; Sowa 1970. All of these theories gives estimation of pile uplift capacity under pure uplift loading. Whereas foundation of structures like lattice transmission tower gets subjected to the simultaneous effect of uplift and compression. There is very few research has been done on this aspects. With the growing demand of electricity it is required to assure safety and economy of such structures. The model test can be a useful way to determine the effect of pile embedment length , skin friction and soil properties on the uplift capacity of pile foundation.


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## SCOPE OF INVESTIGATION

In this study the efforts have been made to determine the effect of loads acting on super structure on the pile foundation. The pile located at four legs of model transmission tower has been subjected to the uplift and compressive forces simultaneously. The bored cast in-situ piles were used as a foundation for the model transmission tower. The field tests have been carried out on the model tower foundation system with piles having varying L/d ratio. The uplift displacement and settlements are measured using dial gauges located at each tower legs. The lateral movement of tower head and pile is also measured. The data obtained from test is then compared with the theoretical models. The contribution of skin friction in total capacity is also analysed.

## MATERIALS AND METHODOLOGY

### SOIL PROPERTIES

The testing program was carried out on the pile foundation embedded in the soil of Ahmedabad city of Gujarat state, India. The engineering and index properties of soil is shown in table 1. The soil is classified SP-SM as per IS classification.

**Table -1 Soil Properties**

| Parameter | Value |
|---|---|
| Angle of internal friction Φ | 28° |
| cohesion **(C)** | 35 kPa |
| Soil classification | SP-SM |
| Bulk unit weight | 15.9 kN/m$^3$ |
| Specific Gravity (G) | 2.67 |
| Moisture content(w) | 4.22% |

### PREPARATION OF FIELD

Three pile foundation sets having length to diameter ratio 4, 6 and 8 were made. On which the test series was initiated. The distance between each set was kept at least 1m so there won't be any disturbance occurring to adjoining group during the testing.

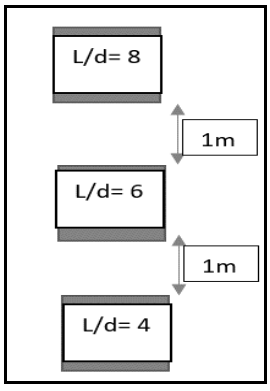



Fig. 1 Arrangement of pile foundation sets of L/d ratio 4,6 and 8 in field

## MODEL TRANSMISSION TOWER

The model transmission tower having height 1.3m was fabricated. The dimension of the tower is shown in fig. 1. The distance between each tower leg was 40 cm at base.

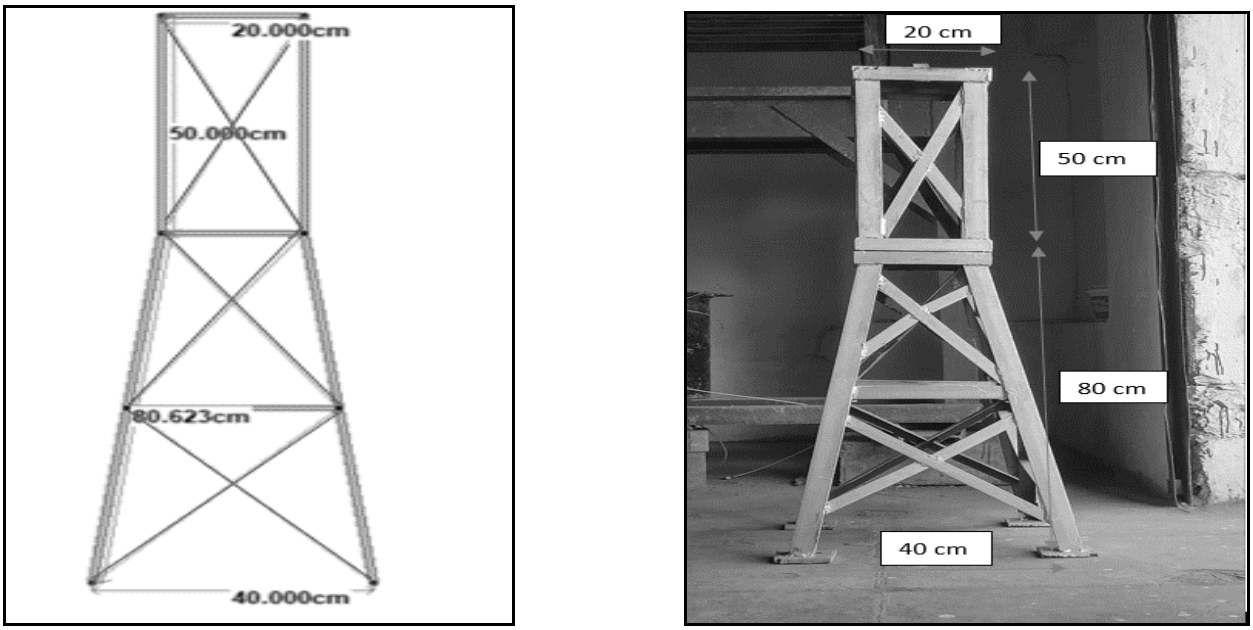
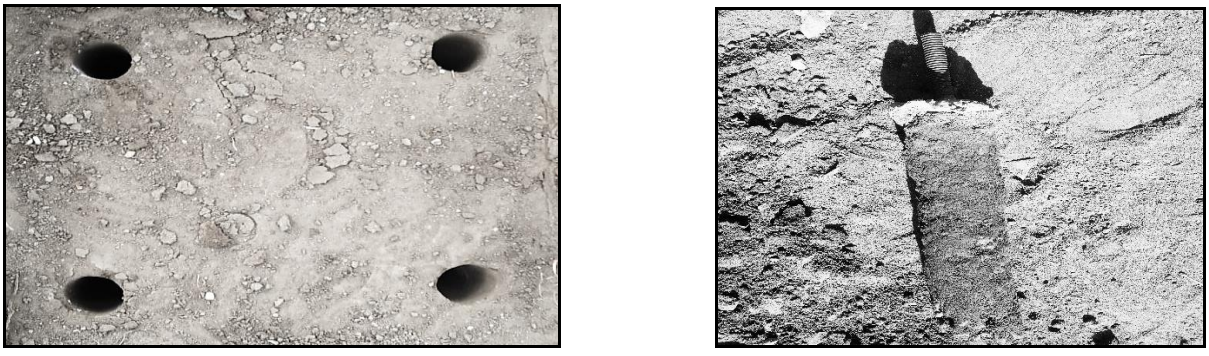



Fig.2 Model transmission tower

## PILE FOUNDATION

The bored cast in-situ piles were installed in the field having diameter of 0.05 m and the selected length were 0.2 m , 0.3 m and 0.4 m. The Piles were installed using specialised boring tool to make the bore holes of the same diameter as that of pile shown in fig.3(a). After casting of piles there was no loading applied on it and was left undisturbed for 28 days. One trial pile was also casted to ensure integrity of pile fig. 3(b)).

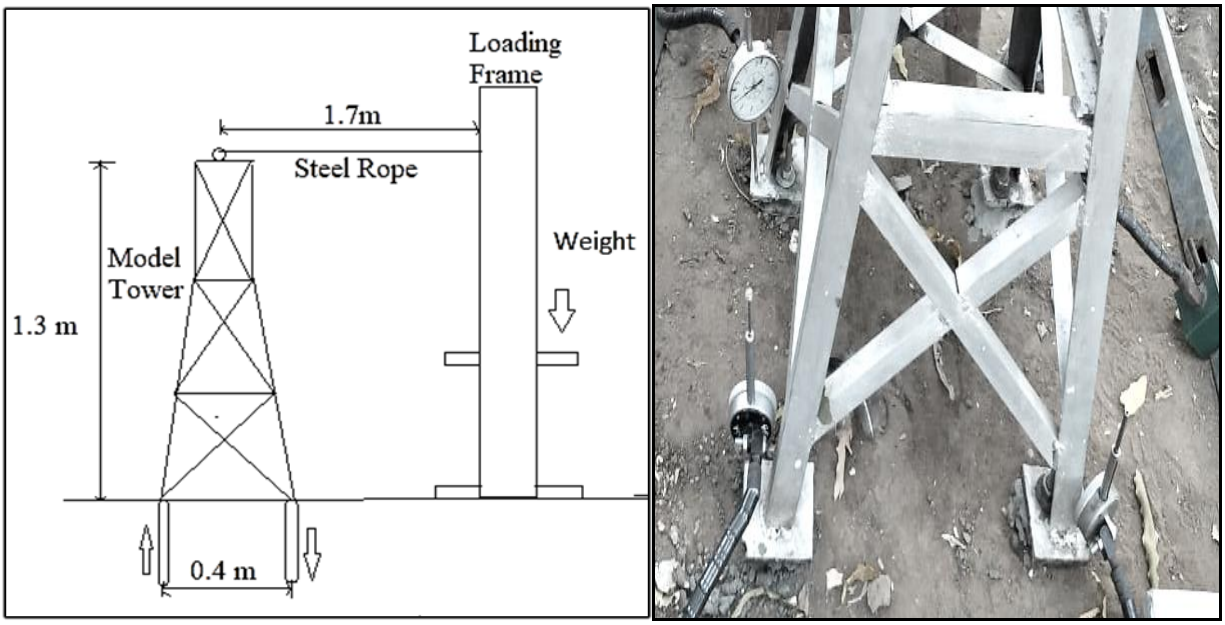


(a) (b)

Fig. 3 (a) Boreholes for pile casting , (b) Trial pile

## EXPERIMENTAL SETUP

The specially fabricated high capacity loading frame was used to apply load on the model tower. The welded hook on the top of the tower was pulled with help of inextensible steel rope of high capacity(7-8 tons)with pully arrangement attached with loading bucket at the end. Which generated uplift in the one side of the legs(tension) and on the other side the legs were subjected to compression. Thus piles were subjected to tension and compression. The Schematic diagram of field setup is shown in figure 4 (a) and figure 4(b) shows the dial gauge arrangement to measure uplift displacement and settlements at the tower legs.




(a) (b)

Fig:4 (a) Schematic Diagram of test setup, (b) Dial gauge arrangement at tower legs

## TESTING PROGRAM

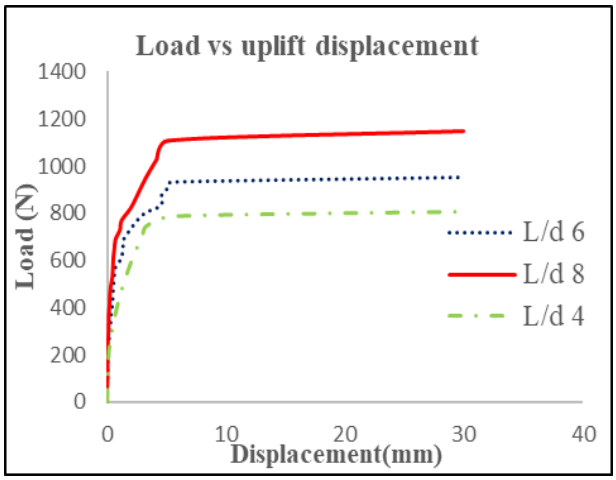
The field testing was carried out on the three cast in-situ pile sets having L/d ratio 4/6 and 8. The detail of testing program is shown in table 2.The uplift displacements, settlements and horizontal displacement were measured during the test. The tests were commenced till the complete failure occurred.

**Table 2 Testing program**

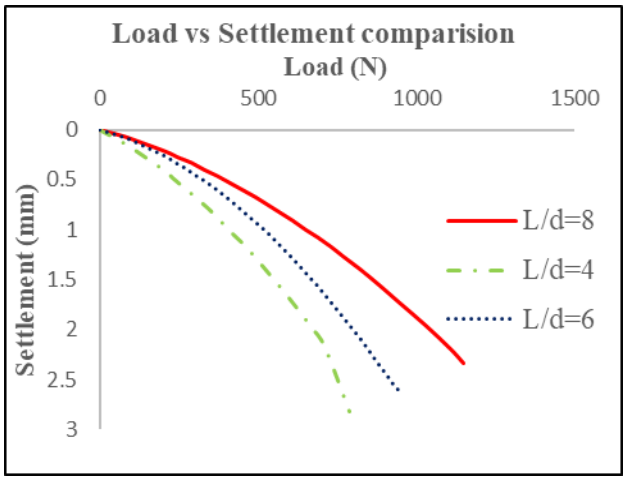
| Sr. No. | L/D Ratio | Symbol |
|---|---|---|
| 1 | 4 | TP-4 |
| 2 | 6 | TP-6 |
| 3 | 8 | TP-8 |

## TEST RESULTS

From the data obtained during test the plots for load vs uplift displacement fig;5(a) , Load vs Settlement comparison fig;5 (b) were obtained.



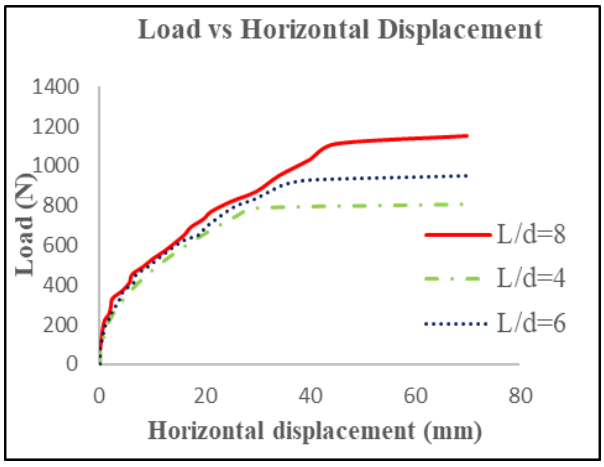

(a)



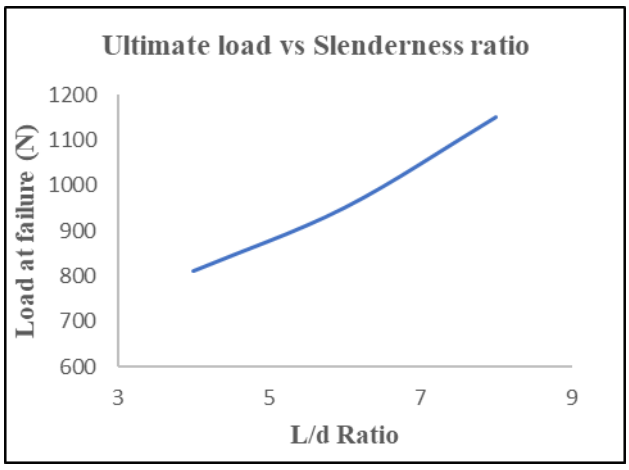

(b)

Fig 5; (a) Load vs uplift displacement plot , (b) Load vs settlement plot

From the load vs settlement plots(fig 5 (a), it can be observed that the after reaching the uplift displacement of 5 mm there is sudden increase in displacement with very less increment in load. Thus it can be said that the uplift capacity is almost fully mobilized till the uplift displacement of 5 mm(10% of pile diameter) occurs. On the other hand from the Load vs settlement plot it can be observed that the settlements are very less compared to the uplift displacement and it was also observed that the limiting settlement criteria of 10% pile diameter was not achieved in any of the pile, indicating that the capacity of compression pile was not fully mobilised and the failure took place way before it due to failure of uplift(tesnion) piles.




(a)




(b)

Fig 6; (a) Load vs horizontal displacement, (b) Ultimate load vs Slenderness ratio

The ultimate load at failure was 810N, 950N and 1150N for piles having slenderness ratio 4, 6 and 8 respectively. The increase in the capacity is due to increased embedment depth and increment in the self -weight of the pile as the length is increased. It can be seen that there is

significant increment in load capacity of foundation with the increment in slenderness ratio 4 to 8.

## THEORETICAL UPLIFT CAPACITY

The estimation of uplift capacity was done using the three theoretical models namely Meyerhof's model(1973), Awad and Ayoub's model(1976) and Standard model which assumes the failure takes place on a cylindrical surface along the shaft.

## MEYERHOF'S MODEL(1973)

The net uplift capacity $P_{nu}$ can be given using the equation 1.Where, $K_u$ = uplift coefficient, d is the diameter of the pile, $\gamma$ is the unit weight of the soil, L is the length of the pile, $\delta$ is the soil– pile friction angle.

$$P_{nu} = \frac{\pi}{2} Ku\ d\gamma L^2 \tan\delta \ldots\ldots\ldots\ldots (1)$$

## AWAD AND AYOUB'S MODEL (1976)

Awad and Ayoub (1976) suggested an equation for estimating the ultimate uplift resistance of a circular pile embedded in sandy soil as shown in equation 2,

$$Pu = \left(\frac{\gamma}{2}\right) \mu d L^2 \tan^2\left(\frac{\phi}{2} + \frac{\pi}{4}\right) \ldots\ldots\ldots\ldots\ldots (2)$$

Where, $\mu$ is the Coefficient of friction (0.33 for cast-in-situ concrete piles and 0.25 for all other piles), d is the diameter of the pile, $\gamma$ is the unit weight of the soil, L is the length of the pile, $\Phi$ is angle of internal friction.

## STANDARD MODEL

Assuming that failure takes place on a cylindrical surface along the shaft the net uplift capacity of a vertical pile can be estimated as per equation 3 given below,

$$Pnu = \frac{\pi}{2} k_s \gamma d L^2 \tan\delta \ldots\ldots\ldots\ldots (3)$$

Where, Ks is the lateral earth pressure coefficient, d is the diameter of the pile, $\gamma$ is the unit weight of the soil, L is the length of the pile, $\delta$ is the soil– pile friction angle.

## SKIN FRICTION

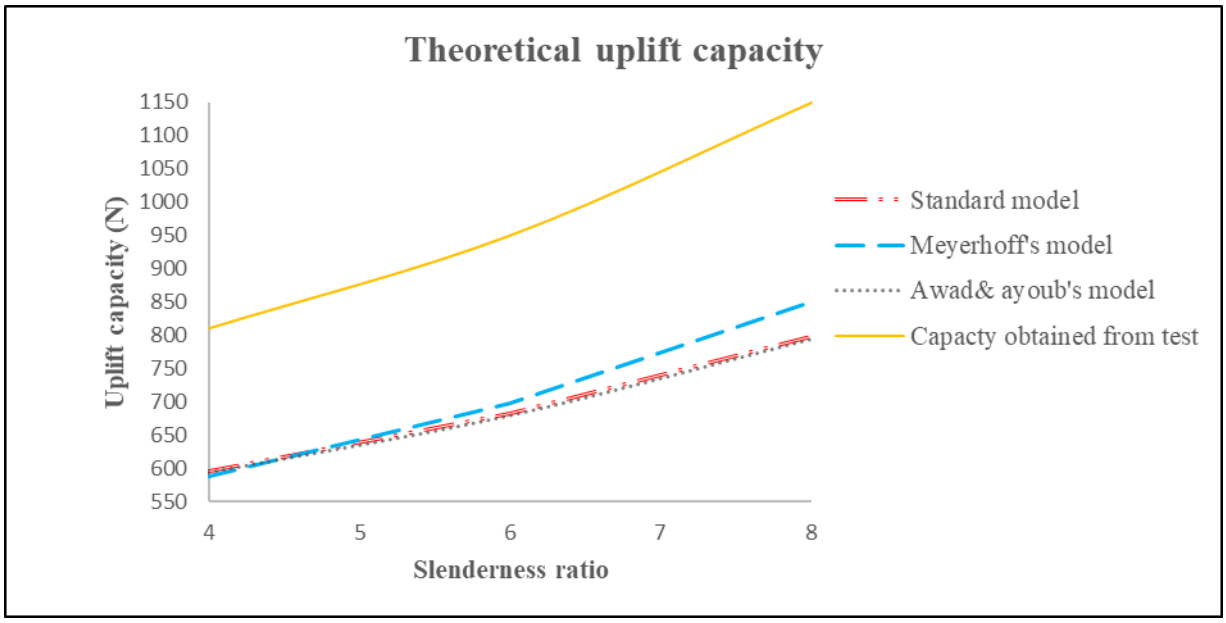
The frictional skin resistance offered by straight shafted pile can be given by using the theoretical estimation model given by Coyle & Castello (1981). The equation to estimate skin friction resistance can be given by equation 4.

$$Q_f = A_s Q_0 K_s \tan\delta \ldots\ldots\ldots\ldots (4)$$

Here, $Q_o$= Average overburden pressure, As= Embedded shaft area, Ks=Coefficient of earth pressure, δ= Angle of internal friction between pile and soil material.

## COMPARISON BETWEEN THEORETICAL AND OBTAINED LOAD CAPACITY

The comparative plot between measured uplift capacity and capacity obtained using theoretical models is shown in figure 7.




Fig;7 Comparison of obtained capacity with theoretical models

The capacity obtained using Meyerhof's model are 587.68 N, 697.28N and 850.72 N for slenderness ratio 4,6 and 8 respectively. The capacity obtained using Awad & Ayoub's model is 593.75N , 680.96N and 795.04N for slenderness ratio 4, 6 and 8 respectively. It was observed that the theoretical capacity obtained from standard model is comparatively lesser than Meyerhof's model but it is quite similar to that of Awad & Ayoub's model. But in every case it is quite smaller than the values obtained from the test. The positive error of 36% , 39.13% and 44.11% was observed for slenderness ratio 4,6 and 8 respectively using standard model. The positive error of 37.83% , 36.24% and 35.17% for slenderness ratio 4, 6 and 8 were observed using Meyerhof's model. And by using Awad & Ayoub's model the positive errors of 36.42% , 39.50% and 44.65% for the slenderness ratio 4 , 6 and 8 respectively.

## COMPARISON OF SKIN FRICTION FOR DIFFERENT L/D RATIO

Using theory given by Coyle and Castello (1981) the value skin friction was obtained for different L/d ratio and is shown in fig 8(a). The percentage contribution of the skin friction in the total uplift capacity of pile is then calculated and shown in the figure 8(b). The value of earth pressure coefficient assumed for L/d ratio 4,6 and 8 was respectively 1.2 , 1.0 and 0.8. The values obtained for skin resistance are 0.198 kN, 0.370 kN and 0.527 kN respectively for L/d ratio 4, 6 and 8. .The contribution of skin friction was found out to be 26% for slenderness ratio 4 in the ultimate load capacity. For the slenderness ratio 6 it is found out to be 40.66% in the ultimate load capacity and it increases up to 47.9% for the slenderness ratio 8. It was observed that the contribution of skin friction to the ultimate load capacity increases with the increment of the embedment depth of pile in the soil

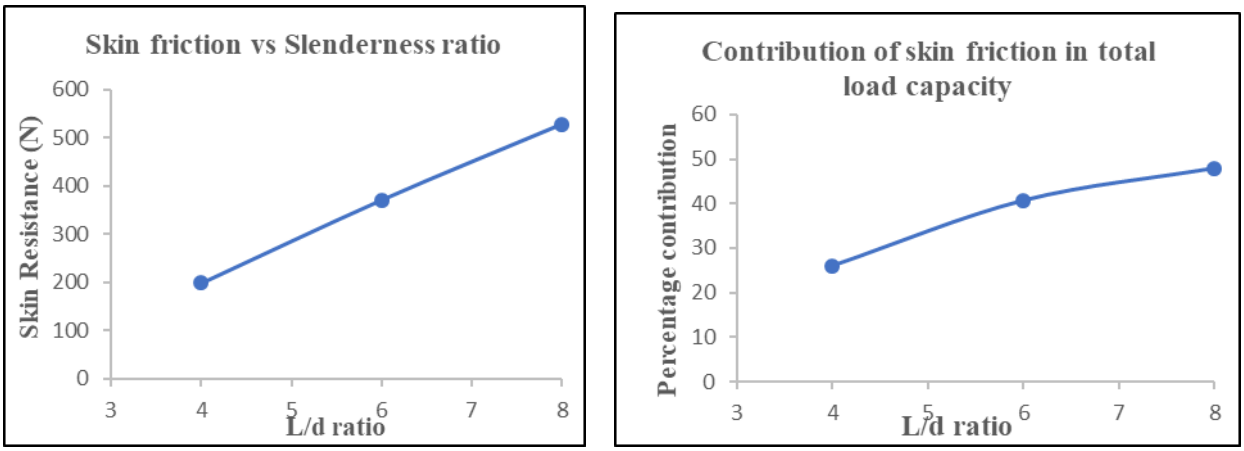



Fig. 8 (a) Plot of Skin friction vs slenderness ratio, (b) Contribution of skin friction in total load capacity

## CONCLUSION

- It is observed that the capacity of piles under tensile forces( forces inducing uplift) is significantly less than the piles under compressive forces. The uplift capacity increase by 17.28% for slenderness ratio 6 and by 41.97% for slenderness ratio 8.
- The displacement equivalent to 48.6% of total displacement required to mobilize complete load carrying capacity of compression pile was achieved at the failure.
- By comparing the test results with the theoretically estimated results it was observed that capacity obtained using theoretical model is on conservative side. It was observed that the capacity obtained using Meyerhof's model was more near to the actual capacity ,The positive error of 37.83% , 36.24% and 35.17% for slenderness ratio 4,6 and 8 were observed using Meyerhof's model.
- It was observed that the complete failure occurs when the very small amount of load is applied in addition to the load which induces the 0.1d(diameter of pile) uplift displacement. The increment of 4.54%, 6.57% and 4.39% of load induces the complete failure of system for slenderness ratio 4,6 and 8 respectively.
- The contribution of skin friction in total uplift capacity was determined as 26%, 40.66% and 47.9% for slenderness ratio 4, 6 and 8 respectively.

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## REFERENCES


1. Awad A. and Ayoub, A. "*Ultimate Uplift Capacity of Vertical and Inclined Piles in Cohesionless Soil*". Proc. 5th Conf. on SMFE, Budapest. pp.221-227.year-1976
2. Coyle H.M. , Castello R.R. "*New design correlations for piles in sand*" Journal of Geotechnical and Geoenvironmental Engineering, Vol.107,p.p.965-986,year-1981
3. Doohyun Kyung, Junhwan Leen "*Load-carrying behaviour of transmission-tower connected foundations subjected to different load directions"* Soils and Foundations, year- 2015
4. Meyerhof, G.G. "*Uplift Resistance of Inclined Anchors and Piles*" Proc.8th Int. Conf on SMFE, Moscow. Vol.2.1, pp.l67-173, year-1973
5. Sowa V.A "*Pulling Capacity of Concrete Cast-In-Situ Bored Piles*" Canadian Geotechnical Journal, Vol.7, pp.482-493, year-1970
6. Vesic, A.S., "*Breakout Resistance of Objects Embedded in Ocean Bottom*", Journal of SMFE Div., ASCE, Vol.97, No.9, pp.ll83-1205 year-1971

# EFFECT OF UNDER-REAM ANGLE AND BULB SPACING ON CAPACITY OF UNDER-REAMED PILES IN MARINE CLAY

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Benny Mathews Abraham, Professor, Cochin University of Science and Technology, bennymabraham@gmail.com

## ABSTRACT

Even though many model studies have been conducted to understand the behavior of under-reamed piles, most of them have been in sand. While in practice, under-reamed piles are provided in clayey soil. Hence, this study was undertaken to understand the behavior of these piles in a scenario closer to the one in practice. It aims at understanding the variation in capacity of single under-ream piles with under-ream angle and the variation of capacity of double under-reamed piles with bulb spacing tested in marine clay. A total of 9 model piles (timber) were tested, out of which four were single under-reamed with under-ream angles of 15°, 30°, 45° and 60°. There were four double under-reamed piles with bulb spacings of $1.0D_u$, $1.25D_u$, $1.5D_u$ and $1.75D_u$, where $D_u$ is the diameter of the bulb. Also, one normal pile with a diameter same as that of the stem diameter of under-reamed was tested to compare the results. The results of tests on under-reamed piles in marine clay are more or less in agreement with the relevant Indian Standard. It is hoped that the results of this study would be of help to geotechnical engineers.

**Keywords: marine clay, under-reamed pile, under-ream angle, bulb spacing.**

## INTRODUCTION

Under-reamed piles are piles in which the stem is enlarged at one or more locations forming bulbs. If there is only one bulb formed, then it is called single under-reamed pile. If there are two bulbs formed, then they are called double under-reamed and if more than two bulbs are formed, then they are called multi-under-reamed piles. Typical details of under-reamed piles are as shown in Figure 1 below. Under-reamed piles are generally cast in situ piles which could be bored compaction piles also. The use of under-reamed piles dates back to the 1940s, when it was first used in Texas, US. Later on, these piles found great use in India where black cotton soil, which was expansive, was abundant. Since the 1950s, under-reamed piles have been extensively used in expansive clays in India. They have found applications in resisting uplift forces as well, as in case of transmission lines, docks etc. CBRI (Central Building Research Institute) has conducted intensive research on under-reamed piles (Chandrasekharan et al. (1978)) (Mohan et al. (1969)).


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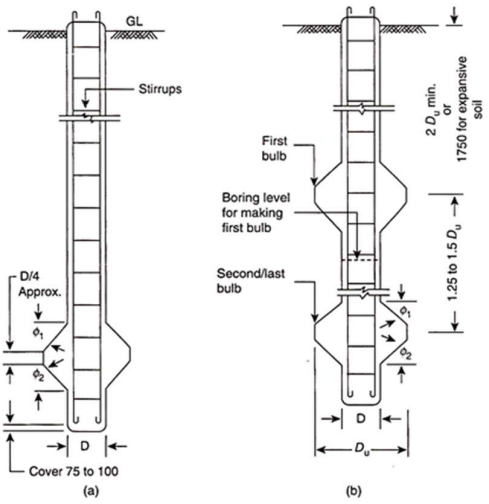



**Fig 1. Typical details of a) single under-reamed and b) double under-reamed pile (IS 2911 Part III)**

Bulbs are regarded as the most significant benefit of these piles compared to other types of piles since a desirable performance can be achieved with lower stem diameters and lengths. Instead of using long and wide piles for carrying heavy loads, under-reamed piles with widened cross-sections or bulbs, can be used. Performance of under-reamed piles strongly depends on soil conditions. They are found to be most effective in expansive clays, where the expanded bulb acts as an anchor and prevents movement of the foundation with soil. In addition, by placement of a convex heave on the piles, bearing capacity of the piles in remoulded and even loose soils can be increased. Under-reamed piles have also been used as pre-cast foundations in some instances and in such cases, they have proven to densify the soil (in the case of sandy soils). The diameter of the bulb is usually fixed between 2-3 times the diameter of stem.

Research on under-reamed piles began as early as 1930s. But most of the experimental research had been done to understand the performance of under-reamed piles in sand. Whereas in actual practice, under-reamed piles constructed as bored cast in situ piles, are generally preferred in clayey soils. Through this study, it is expected to understand the effect of under-ream angle and bulb spacing on the ultimate capacity of single and double under-reamed piles respectively. This study would be more closely comparable to the most common scenario on field where under-reamed piles are preferred. Also, this study would help in determining the most effective under-ream angle to be provided so as to attain the maximum ultimate capacity, for a given diameter of stem and bulb, in the case of single under-reamed piles. This would help in validating the suggestions given in IS 2911-Part 3 regarding under-ream angle and bulb spacing for different soil types.

In a numerical study conducted on under-reamed piles by George and Hari (2016) it was understood that the zone of influence in uplift is predominantly in the immediate vicinity of the pile shaft. The soil mass involved in failure of double under reamed pile is concentrated within the space between the bulbs. This could partly be reason for increase in capacity of double under-reamed piles. The study concluded that double under-reamed pile is a highly cost-effective option in terms of material input. But under-reamed piles can be constructed only in cohesive soils which has the capability of standing on its own without caving in for proper bulb formation. Another observation is that the suggestion by IS 2911 in regard of keeping bulb spacing as $1.25D_u$ to $1.5D_u$ is found to be effective in mobilizing the cylindrical friction action for the range of undrained shear strength considered in the study. A 3D finite element analysis was carried out by Vali et al. (2017) to evaluate the bearing capacity and settlement of embedded tapered and under-reamed piles for different geometrical and geotechnical variables. It was observed that, for under-reamed pile, the bearing capacity tended to converge to a constant value as the

angle of internal friction was increased. There was not much increase in the capacity of under-reamed pile with increase in overall pile length. Number of bulbs had a significant effect on the bearing capacity of the under-reamed pile. The settlement of the pile without bulb was substantially larger than that of the under-reamed piles with bulbs. An investigation using finite element to calculate the ultimate bearing capacity of single under-reamed pile in different soil conditions was done by Shrivastava and Bhatia (2008). It was observed in this study that, in all the cases, ultimate bearing load increased linearly with increase in Young's modulus, stem diameter and length of the under-reamed pile. The author suggests conversion of linear variation into factors K, α and J. The book titled "Analysis and Design of Substructures – Limit state design" by Swami Saran (1998) gives the information that the action of soil cylinder between the bulbs takes place only when the bulb spacing is less than 1.5 times the bulb diameter. The static formula can be applied only in that case. It is also noted that the spacing of the top bulb should be at least at a distance of twice the diameter from the ground level for expansive soils.
A study by Tanuja and Binil (2016) on steel under-reamed pile models of stem diameter 25mm and bulb diameter 75mm, concluded that ultimate capacity in sand increases to nearly 305% when an under-ream bulb is introduced to a uniform diameter pile. Tests were conducted to study the variation in capacity of double under-reamed piles with pile spacing. They also compared the bearing capacities of piles with one, two and three under-ream bulbs and found that the capacity increases with increase in the number of bulbs. A study conducted on several under-reamed piles of varying diameter and length at different sites by Prakashi (2017) show that the safe pile capacity as obtained from subsoil properties, differs from the safe pile capacity shown in the IS 2911 (Part III). In a study conducted to understand the behavior of isolated bored enlarged base pile under sustained load by Chandrasekharan et al. (1978), it was found that isolated bored under-reamed piles failed by punching shear. The skin friction is mobilized with a movement of 2.0 to 3.0 percent of pile diameter and the value of skin friction is about 40%. The ultimate bearing capacity may be worked out using bearing capacity factors for punching mode of failure. A study was conducted on large diameter belled piles (LDBPs) to arrive at a new failure criterion by Zhang et al. (2018). The new semi-empirical method is proposed for interpretation of failure load to account for the effect of enlarged base on load-carrying capacity of belled piles. In addition, elastic shaft shortening is also considered. The proposed method captures a more consistent degree of mobilization of tip resistance and, hence enabling estimation of moderately conservative interpreted failure load for LDBPs of varied shaft diameter, enlarged base and length.

## MATERIALS AND METHODS

*Soil*

The marine clay used for the experiment was collected from a site near Kundannoor in Ernakulam District. The site was one of a commercial building and the foundation planned for the structure was under-reamed piles with a depth of around 12m, stem diameter of 0.4m and a bulb diameter of 1m. The required quantity of marine clay was collected from one bore hole below a depth of 6m from the ground level. The clay brought from site was thoroughly mixed to ensure uniform moisture content and was then placed in plastic bags and kept airtight to ensure no variation in moisture content. The basic properties of marine clay collected from site was determined by conducting various laboratory tests. Tests were conducted to determine liquid limit, plastic limit, particle size distribution and shear strength. The natural moisture content was also determined by collecting samples from five different locations after mixing. The properties of the soil as determined by laboratory tests are as given in Table 1. below.

**Table 1. Properties of marine clay used in the study.**

| Properties | Value |
|---|---|
| Natural moisture content (%) | 117 |
| Liquid limit (%) | 135 |
| Plastic limit (%) | 56 |
| Shrinkage Limit (%) | 30 |
| Plasticity Index | 79 |
| Liquidity Index | 0.94 |
| Grain size distribution | |
| - Clay Size (< 0.002mm) (%) | 52 |

| | |
|---|---|
| - Silt Size (0.002mm – 0.075mm) (%) | 20 |
| - Sand size (>0.075mm) (%) | 28 |
| Shear Strength (kPa) | 11.7 |

@ density of 1.37 gm/cc and moisture content of 117%, determined by Vane shear test

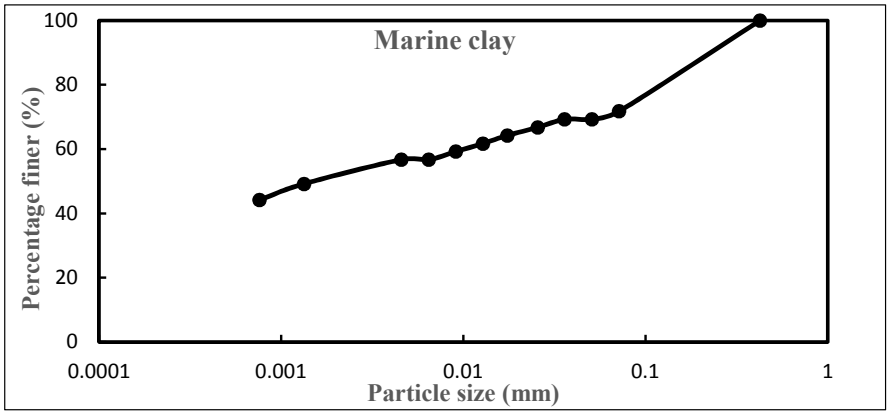



**Fig 2. Particle size distribution of marine clay used in the study.**

*Under-reamed pile models*

The models of under-reamed piles were made out of timber (Mahogany). There were 9 different models of under-reamed piles made in all as shown in Figure 3. Out of them four were single under-reamed with varying under-ream angles of 15°, 30°, 45° and 60° and four were double under-reamed piles with varying spacings between the bulbs viz. $1.0D_u$, $1.25D_u$, $1.5D_u$ and $1.75D_u$ and the last pile made was a uniform diameter pile with a stem diameter of 4cm, common for all the piles. The bulb diameter was kept a constant (10cm) for both single and double under-reamed piles. The length of all the 9 piles was maintained the same as 45cm.



**Figure 3.a). Models of uniform diameter pile and single under-reamed piles with varying under-ream angles (15°, 30°, 45° and 60°); b) Models of double under-reamed piles with varying bulb spacing viz. 1.0 Du, 1.25Du, 1. 5Du and 1.75Du.**

*Metal tank*

A metal tank of dimensions 45x45x60cm was used to provide boundary to the soil in which the pile models were to be tested. The dimension of the tank was fixed based on the diameter of the bulb. The soil surrounding the pile model was to extend at least 2 times the diameter of the bulb in all directions. The pile was placed at 150mm from the bottom of the tank to account for the depth of influence of pile while settlement. The tank was filled with soil upto a height of 150mm from the bottom and then the pile model to be tested was placed. The remaining portion of the tank was filled with the pile model in it. To achieve a uniform density and to avoid air voids, in the case of marine clay, it was thrown into the tank from a small height and the layers were frequently levelled. The weight of clay filled in the tank was kept a constant to ensure same density for all the tests. The verticality and position of the pile model was checked frequently to ensure proper placement. For the purpose of this study, a hydraulic loading mechanism was adopted. Load was applied through pumping oil and the load applied was measured using a proving ring. The proving ring, fitted with a dial gauge of least count 0.002mm, had a capacity of 1000 kg. The tank was placed on the loading frame and load was applied through hydraulic jack. The experimental set up is as shown in Figure 4.



**Fig 4. Experimental set-up**

## EXPERIMENTAL INVESTIGATIONS, RESULTS AND DISCUSSION

*Pile load tests on single under-reamed piles*

Four single under-reamed pile models with under-ream angles 15°, 30°, 45° and 60° were tested. Also, a normal pile of uniform diameter, same as stem diameter of under-reamed piles and of the same length as under-reamed piles, was tested. The load-settlement curves for all the five pile load tests conducted on the model piles are compared in the graph shown in Figure 5.

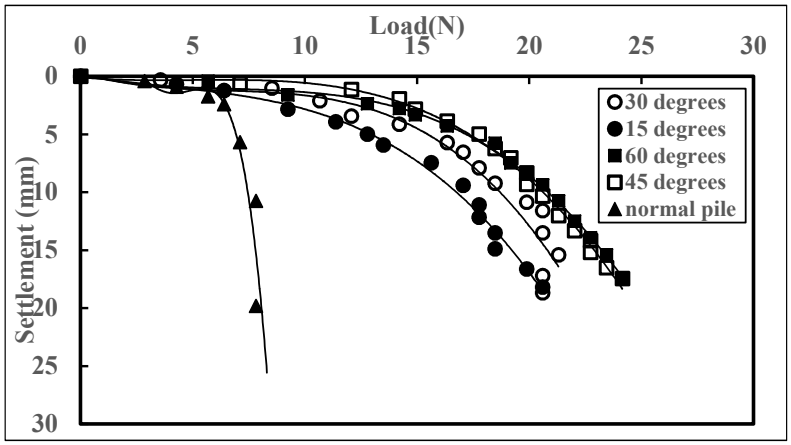



**Fig 5. Load Vs Settlement curve for single under-reamed piles with different under-ream angles.**

From the results of pile load tests, it was observed that normal pile had the least ultimate capacity as compared to single under-reamed piles, as is summarized in Table 2. The maximum ultimate capacity was determined as the capacity corresponding to 3 settlements viz., 12mm, 7.5mm (7.5% of $D_u$) and 4mm (10% of D). The capacity was observed to be maximum for under-reamed pile with under-ream angle of 45° in all the three cases. The under-reamed pile with least capacity is the one with under-ream angle of 30°. Under-reamed pile with under-ream angle 15° was found to have a capacity higher than that with under-ream angle 30°, but lower than that of 60°.

**Table 2. Ultimate capacity of normal pile and single under-reamed pile models**

| Under-ream angle | Capacity (N) (corresponding to 12mm settlement) | Capacity (N) (corresponding to 7.5mm settlement (7.5%$D_u$) | Capacity (N) (corresponding to 4mm settlement (10%D)) |
|---|---|---|---|
| 90° (normal pile) | 74.3 | 69.5 | 64.4 |
| 15° | 174.2 | 147.7 | 115.0 |
| 30° | 192.8 | 170.1 | 142.2 |
| 45° | 212.2 | 188.3 | 161.1 |
| 60° | 212.5 | 186.8 | 155.7 |

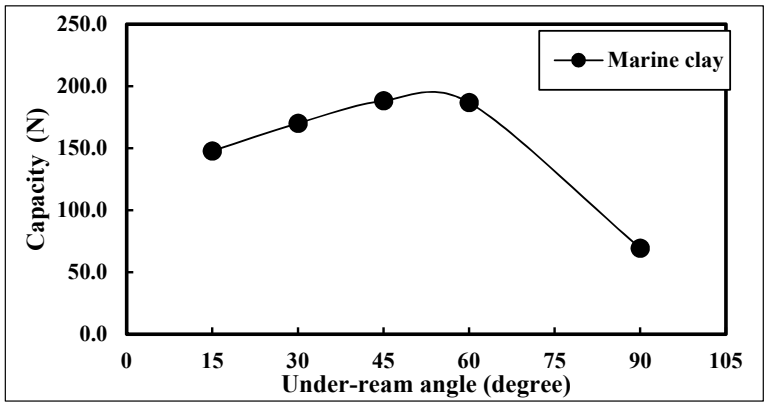



**Fig 6. Variation in capacity (corresponding to 7.5mm (7.5% Du) settlement) of single under-reamed piles.**

*Pile load tests on double under-reamed piles*

In the case of double under-reamed piles, the variation in capacity with the spacing of under-reamed bulbs was compared. Double under-reamed pile models with four different bulb-spacing viz. 1.0 $D_u$, 1.25 $D_u$, 1.5 $D_u$ and 1.75 $D_u$, where $D_u$ is the diameter of the bulb. The length of all the piles were fixed as 50cm having effective depth of pile 45cm and 5cm projecting above the surface of soil to enable proper load application. The load settlement curves for the tested piles are plotted as given in Figure 7. The load settlement curves of the double under-reamed piles with varying bulb spacing are compared with that of a single under-reamed pile with the same under-ream angle of 30°. As can be observed from Table 3, the double under-reamed pile with a bulb spacing clearly has the highest capacity corresponding to all settlements considered, which is in consensus with IS 2911- Part 3 which suggests an optimal spacing of 1.25 – 1.5 times $D_u$.

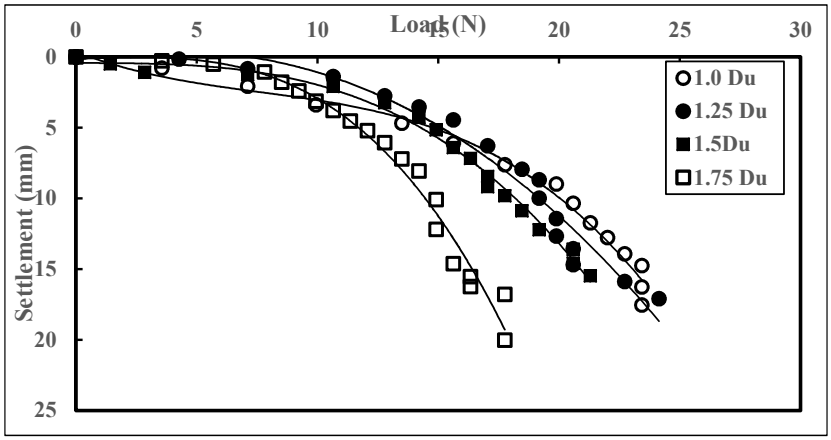



**Fig 7. Load Vs Settlement curve for double under-reamed piles with different bulb spacings.**

From the results of the tests, it was found that the load-settlement curves of the double under-reamed piles did not show much of a variation, at least for smaller settlements with the spacing of bulbs. However, considering the load corresponding to 12mm settlement, it was found that the pile with a bulb spacing of 1.25$D_u$ had the highest capacity. When the spacing was further increased beyond 1.25$D_u$, the capacity showed a decreasing trend, with the pile with bulb spacing of 1.75 $D_u$, having the least capacity. The ultimate capacity considering the load at 12mm, 7.5mm and 4mm (10% of D) settlement are given in Table.4.5. The maximum capacity for a bulb spacing of 1.25$D_u$ may be explained by the formation of soil cylinder, which increases the effective diameter of the pile in between the bulbs.

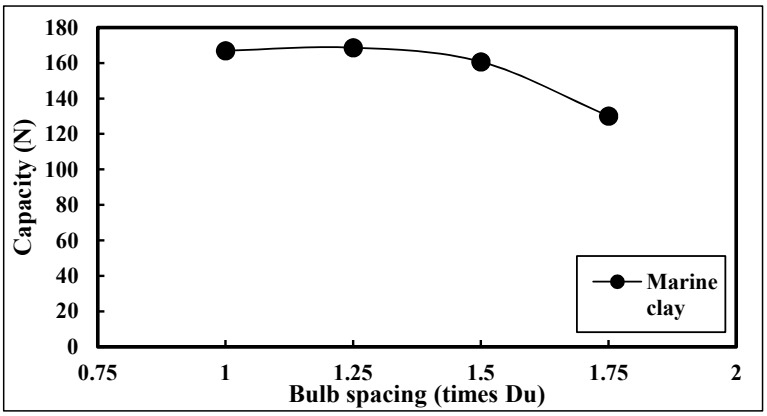



**Fig 8. Variation in capacity (corresponding to 7.5mm (7.5% Du) settlement) of double under-reamed piles.**

**Table 3. Ultimate capacity of double under-reamed pile models**

| Bulb spacing | Capacity (N) (corresponding to 12mm settlement) | Capacity (N) (corresponding to 7.5mm settlement (7.5%$D_u$) | Capacity (N) (corresponding to 4mm settlement (10%D)) |
|---|---|---|---|
| 1.0 $D_u$ | 208.1 | 167.0 | 124.0 |
| 1.25 $D_u$ | 199.2 | 168.6 | 137.2 |
| 1.5 $D_u$ | 188.9 | 160.7 | 128.4 |
| 1.75 $D_u$ | 150.5 | 130.1 | 107.6 |

## SUMMARY AND CONCLUSIONS

The present study was aimed at understanding some parameters that affects the capacity of under-reamed piles, which included the under-ream angle (in case of single under-reamed piles) and bulb spacing (in case of double under-reamed piles) in marine clay. Through a series of model studies conducted, it was understood that the capacity of under-reamed piles is indeed influenced by the above-mentioned factors. The following conclusions can be drawn from the experiments, observations and analyses conducted.

- The capacity increased to at least twice the value when an under-ream bulb was introduced to a normal uniform diameter pile.
- The capacities of double under-reamed piles were not found to be higher than that of single under-reamed pile with under-ream angle of 30° corresponding to settlements of 7.5mm and 4mm. This was not in agreement with the theory of formation of soil cylinder giving a notably higher capacity for double-under-reamed piles.
- In marine clay, for a single under-reamed pile, the maximum capacity was observed to be for a pile with an under-ream angle between 45° and 60° as against the suggestion by IS 2911-Part 3 to provide an under-ream angle between 30° to 45°.
- In the case of double under-reamed piles, the capacity did not have a striking variation with bulb spacing. The double under-reamed pile with a bulb spacing of 1.25 $D_u$ showed maximum capacity.

The results obtained are in more or less in agreement with the suggestion of IS 2911-Part 3 to provide an under-ream angle of 45°, as is also followed on site. The results reinforce the suggestion to provide a spacing of 1.25$D_u$ to 1.5$D_u$ between bulbs in the case of double under-reamed piles.

## REFERENCES


Chandrasekharan V., Garg K.G., Chandra Prakash. 1978. Behaviour of Isolated bored enlarged base pile under sustained vertical loads. J-STAGE Volume 18 Issue 2 Pages 1-15.

George B. E., Hari G. 2016. Numerical investigation of under-reamed piles. Indian Geotechnical Conference. Chennai.

IS2911. 1980. Code of practice for design and construction of pile foundations Part- III - Under-reamed Piles. Bureau of Indian Standards.

Mohan D., Murthy V.N.S , Jain G.S. 1969. Design and construction of multi-under-reamed piles. Roorkee: CBRI.

Prakashi, Somdev. 2017. A comparative study on safe pile capacity as shown in Table 1 of IS 2911 (Part III): 1980. Journal of Institution of Engineers (India): Series A 98 (1-2): 185-199.

Saran, Swami. 1998. Analysis and design of substructures - Limit state design. New Delhi: Oxford & IBH Publishing company.

Shrivastava N., Bhatia N. 2008. Ultimate Bearing Capacity of Underreamed Pile – Finite Element. International Association for Computer Methods and Advances in Geomechanics (IACMAG). Goa.

Tanuja C., Binil G. 2016. "Parametric study of under-reamed piles in sand." International Journal of Engineering Research and Technology Vol 5. Issue 7.

Vali R., Khotbehsara E. M.,Saberian M., Li J., Mehrinejad M. & Jahandari S. 2017. A three-dimensional numerical comparison of bearing capacity and settlement of tapered and under-reamed piles. International Journal of Geotechnical Engineering (June).

Zhang L., Chen Q., Gao G, Nimbalkar S., Chiaro G. 2018. A New Failure Load Criterion for Large-Diameter Under-Reamed Piles:. International Journal of Geosynthetics and Ground Engineering.

# INSTALLATION OF LARGE DIAMETER BORED PILE SOCKETED IN HARD ROCK WITH RCD FOR MTHL, MUMBAI

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**Abstract**

One of the recent developments that has been carried out in Indian construction industry for deep foundation is the use of reverse circulatory drilling popularly known as RCD for large diameter bored piles. RCD is preferred to be used for large-diameter piles, deep depth, hard rock drilling and bored piling in marine condition, because its operation is faster compared to other conventional drilling and easier to clean the bore hole. The fluid and cuttings flow up the drill pipe and down the annulus. In softer formations, this causes less washout and makes sampling faster, the process is known as airlifting. Conversely, for the installation purpose of the machine, it requires tubular casing or commonly known as pile liner of sufficient thickness and the RCD is mounted upon the liner and this liner needs to suffice the additional load of the RCD rig and works as top down drilling. The liner is generally embedded with help of vibro-hammer, oscillator / pre boring up to the rock level and then impact hammer may be used to terminated half to 1 meters in the rock depending upon the qualitative nature of the rocks i.e. weathering, RQD, strength of rock etc.

The present study has been carried out for "Mumbai Trans Harbor link package 01" or popularly known as MTHL Bridge, the longest sea crossing bridge in India once completed. The above said project is being construed by L&T construction for MMRDA across Mumbai bay between Sewri on Mumbai side and Chirle on the Navi Mumbai side in Maharashtra India. For this project, various large dimeter piles are socketed into hard rock with the help of RCD and conventional drilling, and it is the case study on RCD drilling i.e., premature refusal of liner or over driving of liner, sinking of liner, drilling issues faced on using RCD, along with advantages of RCD vs conventional drilling in hard rock in Mumbai region.

**Keywords: washout, vibro-hammer, refusal, RCD**

## INTRODUCTION

Mumbai Trans Harbour Link or MTHL, once completed would be India's longest sea bridge. It was first proposed in 1970s, however the first concrete attempt was done in 2004. But due to some undeclared reasons, its execution started only in 2018.

The total length is 21.8 kilometres comprising 2x3-lane highway supported principally on bridges, with interchanges and access ramps. The bridge construction is divided into three packages for the efficient and timely construction of the project. Package 1 has been awarded to L&T Construction which comprises the inter-tidal as well as marine location. At the completion of the project, the aesthetic view of the bridge would look as shown in the figure 1.


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*Figure 1 Aesthetic View of MTHL Package - 1*

## REGIONAL GEOLOGY AND SUB - SOIL PROFILE

A total of 556 boreholes were drilled in land as well as marine location, based on sub-surface investigation guidelines as per MORTH, which states that the investigation should be done up to four times the diameter of the pile foundation in the hard rock, however as per design requirement 10m in hard rock (RQD > 60%) was decided since as per Employer's requirement RQD > 60 % is mentioned as hard rock. As per the sub-surface investigations, it was found out that the breccia was pre-dominant in land portion, which is having low strength compared to the basalt which in contrast is of very high strength and is in abundance in marine portion.

The extensive sub-surface studies shows that the surficial deposit of the project location comprises mostly very soft to soft marine clay up to a depth of 10 to 15 meters. It is underlain by completely weathered rock to highly weathered basalt in the marine portion with fresh basalt rock at deeper depths. However, in the land portion, the intrusion of breccia is generally found. The tabular summary of the sub-soil profile as follows:-

**Table 1 Sub-soil Stratigraphy (Marine Portion)**

| Strata | Designation | Average Layer Thickness (m) |
|---|---|---|
| 1 | Filled up soil | 3 |
| 2 | Stiff Clay | 6 |
| 3 | Completely Weathered Rock | 4 |
| 4 | Highly Weathered Rock | 5 |
| 5 | Moderately Weathered / Fresh Rock | 9 |

The sub-soil profile can be pictorially represented as below, which depicts that in marine portion, there is a substantial deposition of clay layer (in pink colour), as can be seen from the below figure.

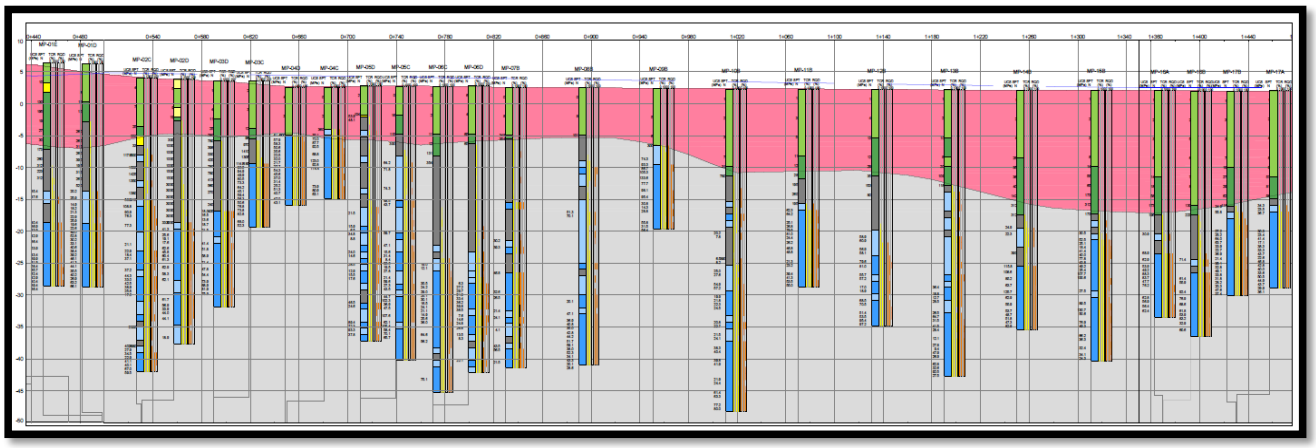


*Figure 2 Inter-tidal Sub-soil Profile*

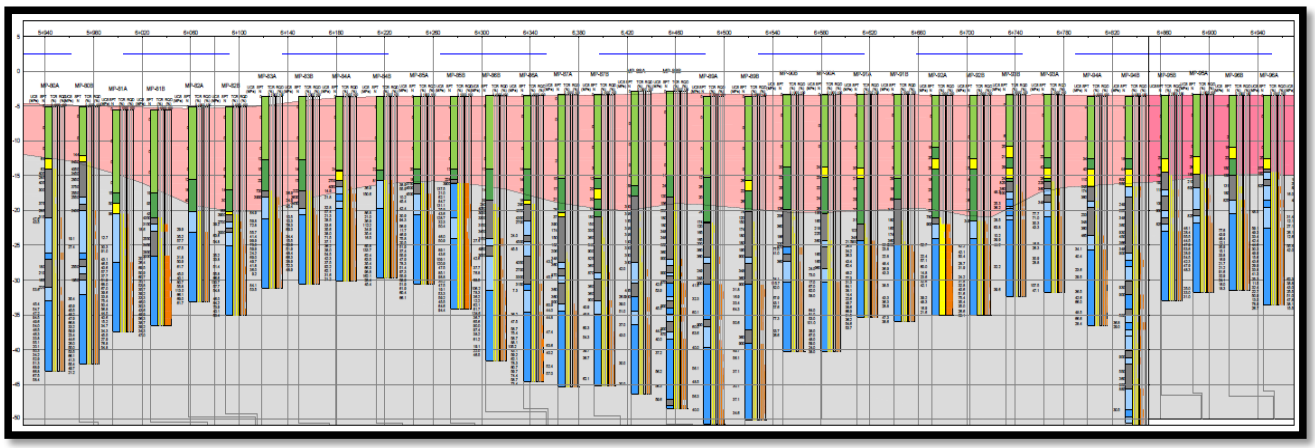


*Figure 3 Marine Sub-soil Profile*

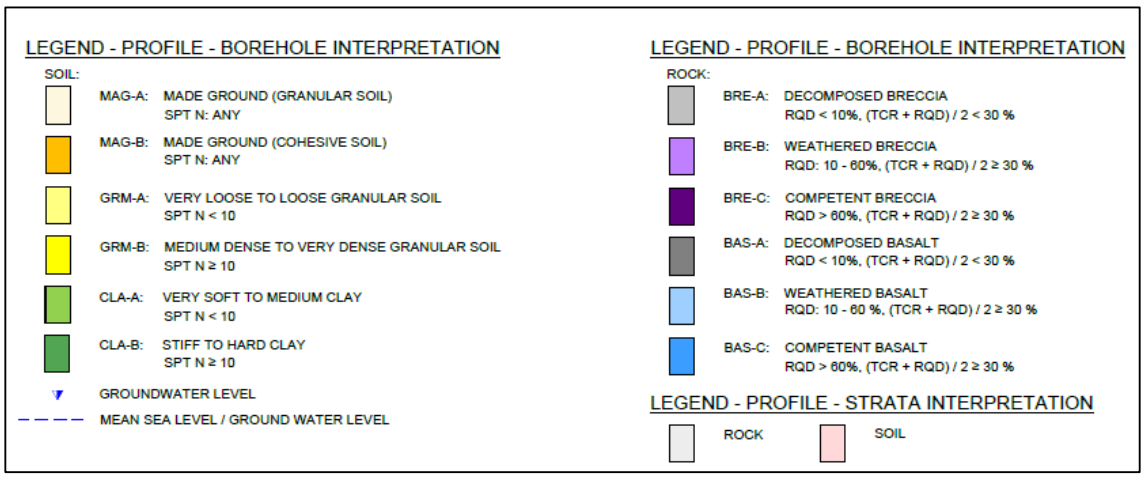



*Figure 4 Marine Sub-soil Profile*

## FOUNDATION DETAILS

As per the Employer's requirement (ER), it is specified that the shallow foundation shall be allowed in land and inter-tidal location only if the bearing stratum is available at shallow depth. However, in marine portion, pile foundations were only specified. The pile foundations were designed as end-bearing and is designed to sufficient depth below RQD > 60%, as is stated under the ER. The pile foundation is designed to bear the considerable loads (axial load of around 2200 MT) as the service life of the bridge is 100 years and following the same, it was decided to go with 2.2m diameter pile foundation in the marine portion for a minimum of 35 to 50 meters. A total of 1251 piles are to be constructed in the marine portion, out of which majority of piles are of 2.2m dia. in the marine location and inter-tidal location.

Considering the sub-surface geological data, in the land portion the site progressed with the hydraulic hammer, however in the marine portion where there is presence of strong basaltic rock, it was difficult to proceed with the hydraulic rig thus RCD rigs were deployed, considering the past experience in Bandra- Worli Sea Link, where RCD rigs were working successfully in marine portion in the similar geological conditions.

## PILING ACTIVITIES

Considering the quantity of piling activities in the marine location and the construction of large diameter piles, it was decided to construct the piles by using Reverse Circulatory Drilling (RCD) machines. For which, L&T construction procured the piling rigs PBA 818 from the German manufacturers MhWirth. It was decided based on the past experience of Bandra - Worli Sea Link where RCD machines were




*Figure 4 RCD Rig in Operation*

successfully used and the foundation activities were completed at much higher rate than the conventional hydraulic rigs.

The Reverse Circulatory Drilling (RCD) rigs unlike the conventional piling rigs works on the principle of compressed air injecting into the drill pipe below the water level and just above the cutting head. As the air rises and expands within the drill pipe, the density in the internal liquid column is greatly reduced, leading to a difference in pressure between the liquid in the drill hole outside and the liquid inside the drill pipe. Due to the higher density in the outer column, the solids pass from the drill hole through the cutting head suction opening, and rise up through the drill pipe.

The major advantages of RCD is that it has higher rate of penetration compared to direct circulation drilling, and also it is cheaper in terms of construction activities due to its higher drilling efficiency.

## PROBLEMS ENCOUNTERED IN PILING ACTIVITIES

The major challenges which the site faced during the piling activities was the mobilization of the piling rigs and heavy equipment in the inter-tidal portion.

Considering that, the Construction Methods and Plan Cell (CMPC team) decided to construct Temporary Access Bridge (TAB) on steel tubular piles of diameter 900mm installed with vibro-hammer. The tubular piles were installed up to the refusal level considering the temporary loads of vehicles and wind.



*Figure 5 Temporary Access Bridge*

Additionally, RCD rigs were installed in the inter-tidal as well as marine portion. The RCD was clamped over the steel tubular piles which are first driven into the soil using vibro- hammer In case of RCD, the top-drive system mounted on the casing must react against the casing during drilling, so the casing must be sufficiently embedded into firm soil to provide a stable platform on which the machine can work. As per the design, the assumed design level for liner was embedment 1m into the rock, however the RCD was getting clogged in the stiff clay portion and it was difficult to proceed further.

It was then decided to proceed with grabber through to remove stiff clay layer and then again the vibro-hammer was installed to seat the liner up to the stiff clay layer and then proceeding further into the firm soil so that the working platform would be stable to proceed for further drilling.

Additionally, it was suggested to proceed with the following construction methodology. Steel liner were installed upto refusal with the help of vibro-hammers:-

- Hammer grab for excavation through hard stratum and boulder layers and then using vibratory hammer again to reach the refusal (or)
- Impact hammer for breaking the hard stratum and to reach the refusal (or)
- Pre-boring up to the design liner toe level and then vibro - hammer again to reach the refusal.

By employing any of these methods, we shall be ensuring the liner toe level in to competent rock stratum and it will help us in sealing the drill hole and avoid liner sinking.

The following pictorial representation can be seen for the sequential progression of methodology of RCD drilling, which is as:-

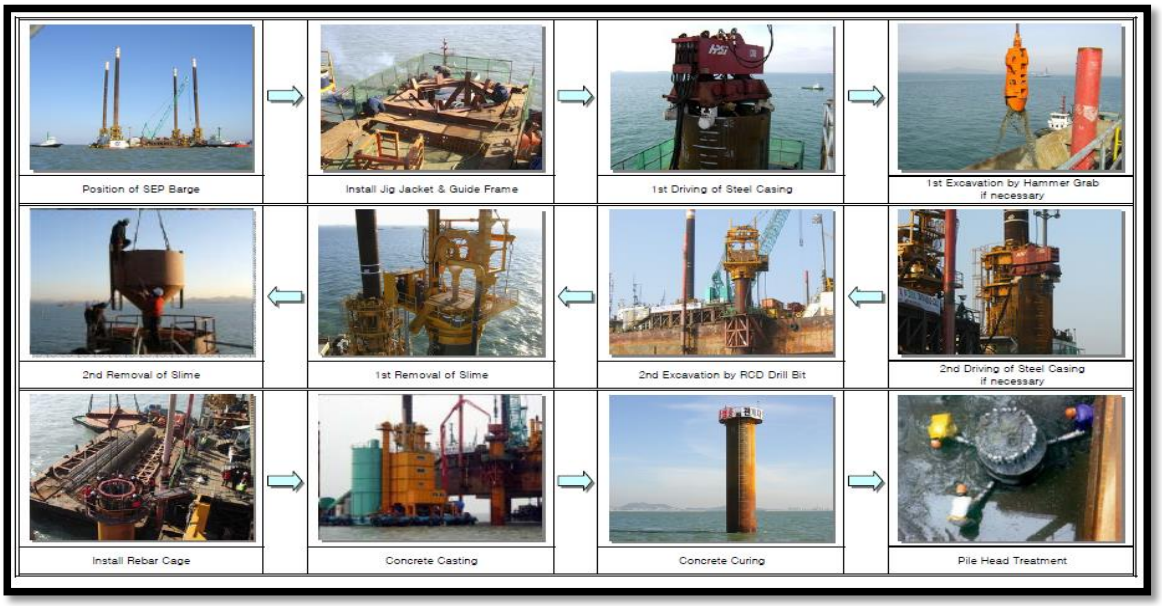



*Figure 6 Proposed Construction Methodology*

## OTHER CHALLENGES

Due to the very high design loads, testing piles using the Kentledge piles would be very difficult.

Hence, it was proposed to perform Bi-directional load test, as the kentledge load could have cause fatal accident due to the stacked concrete blocks arrangement. The test load of 9,800 Metric Tons was performed, which is the heaviest pile load test as of now in the country.




*Figure 7 Bi-directional load test performed at MTHL site*

## BI-DIRECTIONAL LOAD TEST PROCEDURE



*Figure 8 Positioning of jacks in the Pile Cage*

The test utilizes the hydraulically embedded hydraulic jacks inside the piles. The cell positioning is provided based on the ground conditions and the loading details to be carried out. The load cells expansion pushes the upper shaft upwards and down shaft downwards, which would mobilize the shaft friction, and the end bearing of the upper and lower part. Based on the relationship for the load and displacement with respect to time, the bearing capacity of the pile is calculated. Fig. 9 represents the load Vs displacement graph of the pile load test done at pier location in marine.

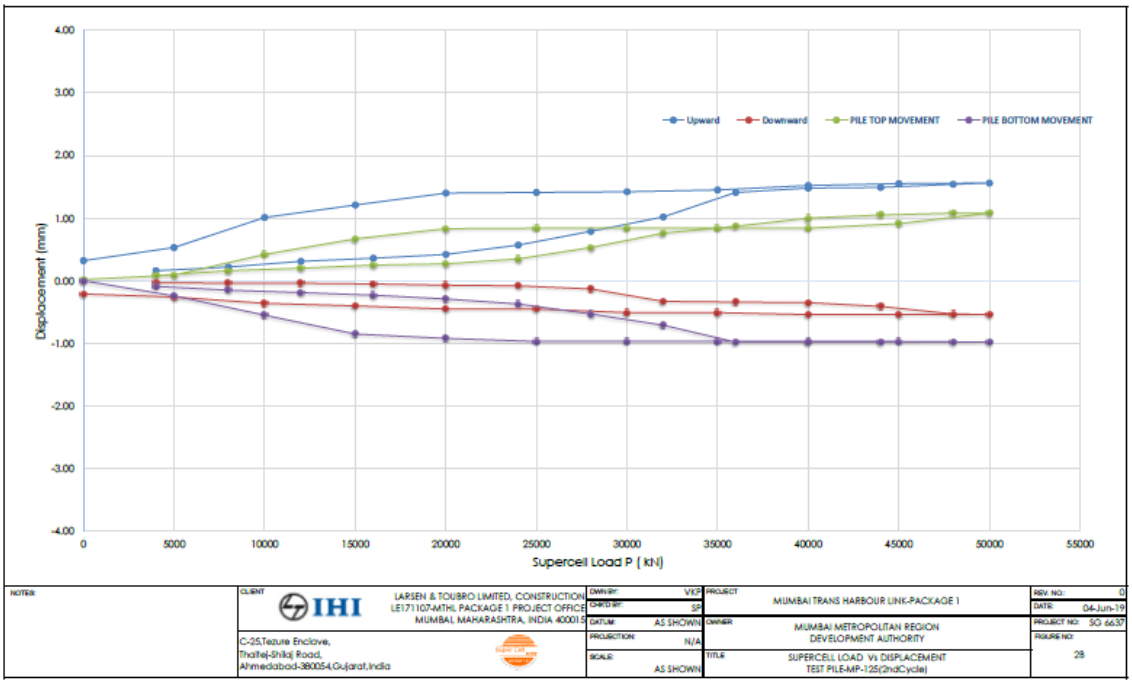



*Figure 9 Load Vs Displacement Curve for Bi-directional Load test*

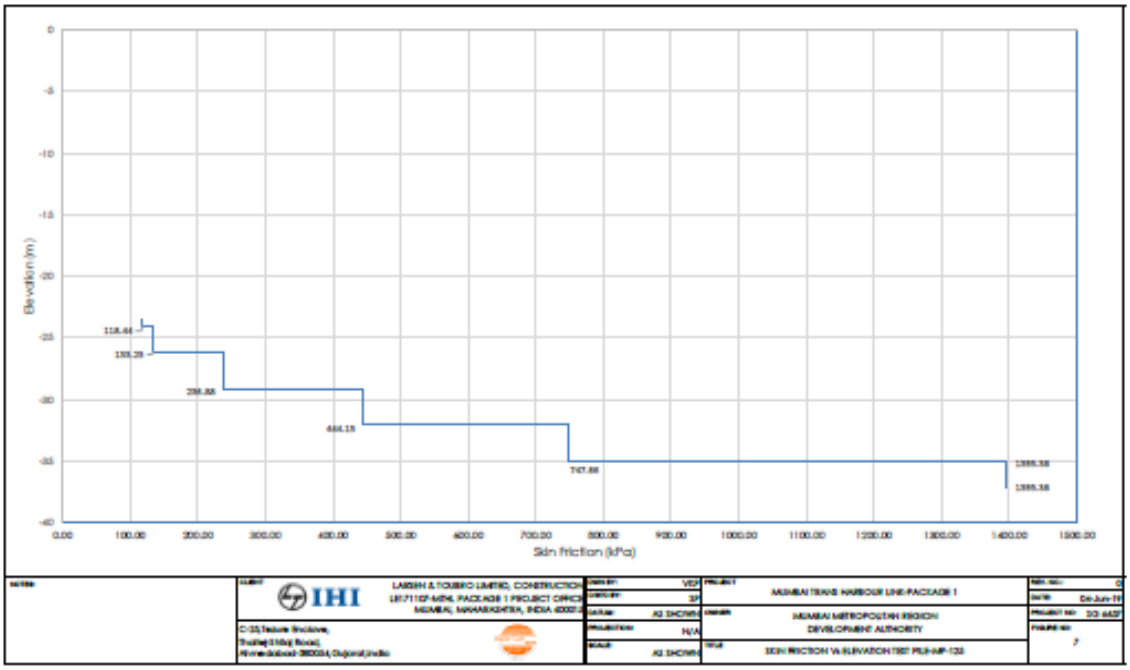



*Figure 10 Skin – friction Vs Elevation from Bi-directional Test Results*

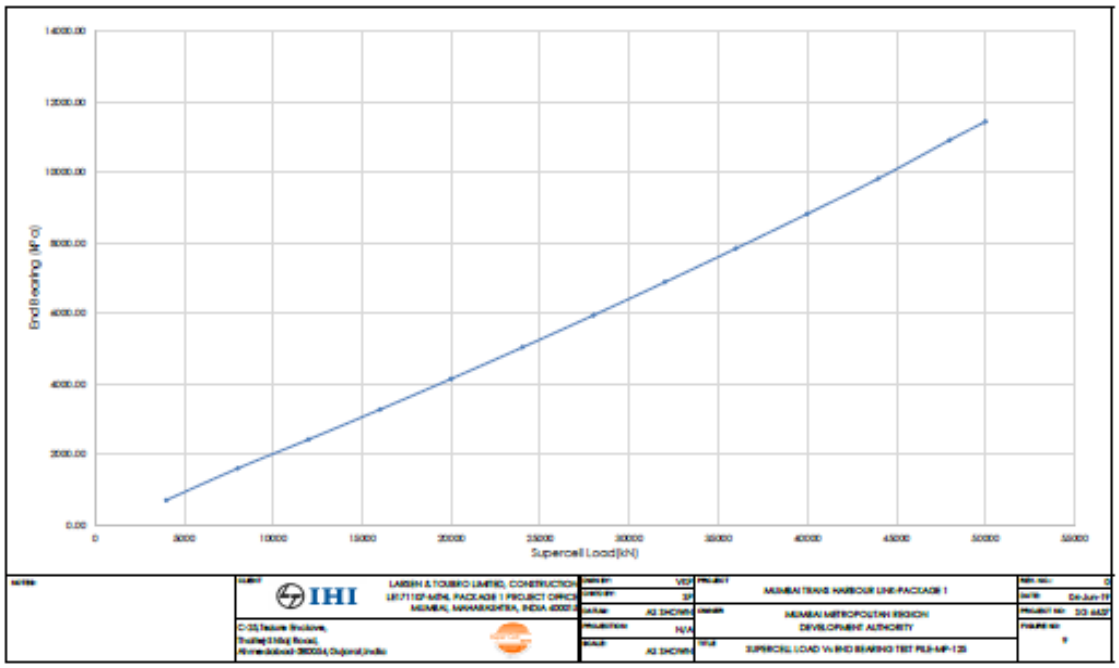



*Figure 11 End - Bearing Vs Elevation from Bi-directional Test Results*

## SUMMARY AND CONCLUSION

The construction of such an enormous project includes the construction of large - diameter piles to support the super-structure loads, which were constructed utilizing the Reverse Circulatory Drilling (RCD) rigs. These rigs are cheaper and efficient in construction due to high efficiency, but the main concern is that its heavy load needs to be seated over the liner and the liner shall be terminated/fixed into the firm soil, otherwise there is a possibility of tilt of the RCD rig, which would be hamper production and could be a safety concern. The situation arose where the steel liner got clogged into the firm strata patch unlike the assumed design consideration of 1m embedment of steel liner into the rock. To deal with this unexpected layer of soil, it was decided to use grabber to terminate the liner up to the presumed design level, which was 1m into the rock. The heavy machinery and cranes were having huge loads for which a TAB (temporary access bridge) was constructed. The TAB was constructed on steel tubular piles of diameter 900mm, terminated into the refusal.

Furthermore, for carrying out the huge load test on these piles, it was difficult to proceed with conventional methods. Therefore, Bi-directional load test was carried out. As per Indian code, there was no provision for carrying out this test, however as per ASTM, the bi-directional load test for test load of 9,800 Metric Tons was performed using jacks of 300 Metric Tons, making it country's heaviest load test till date.

## REFERENCES


- Drilled Shafts: Construction Procedures and LRFD Design Methods FHWA-NHI-10-016.
- Drilled Shaft Construction for the Bandra -Worli Sea Link Project Felice, C. W. / Brenniman, H.
- BS 8004: 1986 – British Standard of Practice for Foundation.
- CP4: 2003 – Singapore Standard Code of Practice for Foundation.
- M.L.Gupta, S R Sharma, A. Sundar and S.B. Singh (1987) Geothermal studies in the hydrated granitic region and the crustal thermal structure of Southern Indian shield, Technophysics, 140, pp 257 – 264.
- IRC 78 March 2018 Amendment to IS2911 Part 4)
- ASTM D8169: Standard Test Methods for Deep Foundations Under Bi-Directional Static Axial Compressive Load
- Meyer, P.L., Holmquist, D. V. and Matlock, H. "Computer Predictions for axially loaded Piles with Non-Linear Support", Proceeding of the 7th Offshore Technology Conference. Paper No. 2186, Houston, Texas 1975.
- Canadian Foundation Engineering Manual 2006 4th edition, Canadian Geotechnical Society. Bi Tech Publishers Limited, BC, Canada.

# LOW-STRAIN PILE INTEGRITY TESTING AND ANALYSIS

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## ABSTRACT

Non-destructive testing of piles has gained fair acceptance for various purposes e.g., quality control/quality assurance, verification of existing conditions, and quantification of dimensions. The correct use of this technique can greatly simplify and expedite investigation, and be economical in addressing concerns or question on pile conditions. Equally, its incorrect use can cause controversies, delays, and/or create adverse reputation for the technology.

Integrity issues for *cast-in-situ* piles have resulted in a serious need for non-destructive test inspection methods to detect the extent and location of serious flaws and prevent failures under service conditions. Integrity inspection performed after installation is often the most reasonable alternative available to assess the pile quality. A most commonly used post construction integrity test is the low-strain pile integrity test. In this type of integrity testing, a low strain dynamic impulse is given to the pile top by a hand held hammer and the velocity of the pile top is monitored using an accelerometer. The low strain impulse will generate a low stress wave that propagates through the pile and reflect at places where there are changes in the properties of concrete, cross-sectional area of the pile or stiffness of the soil surrounding the pile. Such reflections are detected by the accelerometer at the pile top and the Time vs. Velocity plot recorded by the accelerometer is used to identify any defects in the pile shaft. There are two main methods of data analysis associated with the pile integrity test viz. the Pulse Echo Method (PEM) and the Transient Dynamic Response (TDR) method.

This paper presents the testing, analysis and interpretation of pile integrity test data by Pulse Echo Method from various project sites relating the different construction methods in Indian soil conditions.

**Keywords: low-strain integrity testing, stress wave, pulse-echo**

## INTRODUCTION

Deep foundation construction is an inherently ‘blind’ process, i.e. the final product is not readily available for visual inspection. The quality control / quality assurance process for such foundations is almost always through indirect measurement of other parameters, such as performance of the installation equipment, resistance to driving or drilling, examination of drilled cuttings, etc. Therefore, the quality of the final product is often a function of the installer’s know-how and the inspector’s experience. Even the most experienced foundation contractors acknowledge that there is an initial ‘learning period’ for each project, essentially impacted by ground conditions, equipment utilized, and installation processes. A process whereby confidence in the quality of the installed pile is expeditiously attained is essential to the contractor to confirm the adequacy of the deployed construction methods and vital to the engineer to verify the competence of the foundation installed. The integrity test method can be a valuable tool in rapidly making these evaluations as piles are installed.

## LOW STRAIN INTEGRITY TESTING

Pile Integrity Testing, is a non-destructive testing technique, sometimes referred to as the sonic pulse echo method. It involves applying low strains to a foundation element (especially pile) using light hammer impacts and evaluating the collected force (optional) and velocity records to deduce qualitative information for the foundation system. Standards covering pile integrity test include IS: 14893 and ASTM D5882.


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***Basic Theory***

Pile Integrity Testing is based on the theory of wave propagation in a linear direction. For a linear elastic pile having a length an order of magnitude greater than its width, stress waves travel in the pile at a wave speed (c), such that

$$c = \sqrt{E/\rho} \quad [1]$$

Where E = pile material elastic modulus and ρ = mass density. The applied force F, imparted by hammer impact and the particle velocity v, at any point are related such that

$$F = Zv \quad [2]$$

Where Z = proportionality constant, also known as impedance; it is a measure of pile resisting change in velocity. Pile impedance for various size piles can be defined as

$$Z = \frac{EA}{c} \quad [3]$$

Change in impedance is related to change in pile cross-sectional area A, as well as pile material quality. Increase in pile impedance or soil resistance forces results in a decrease in measured pile top velocity. Conversely, decrease in pile impedance, results in increased velocity. By observing changes in impedance, pile quality can be assessed and dimensions estimated.

***Testing Equipment – Pile Integrity Tester (PIT)***

The equipment that is in common use for pile foundation evaluation is manufactured by Pile Dynamics, Inc. (PDI) USA. The PIT equipment is very compact and readily portable, consisting of a hammer, a motion sensor (accelerometer), and a processing cum display unit. The hammer size varies from about 500g to 4kg. The processor stores and analyzes the recorded signals. Components of the PIT are shown in Fig. 1.

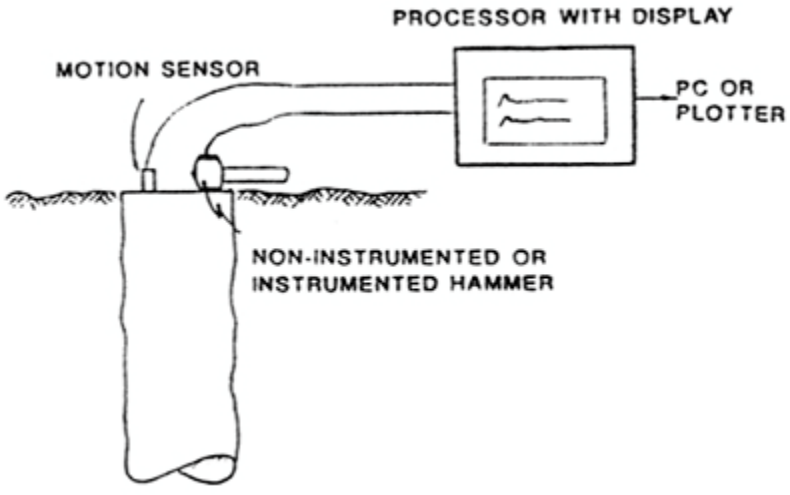



**Fig. 1 Components of PIT (Rausche et al. 1992)**

In utilizing PIT equipment, one must recognize the capabilities and limitations of the method. The quality of the integrity test results is a direct function of the operator's familiarity with the system and experience with pile foundations, e.g. factors such as pile surface preparation for attachment of the sensors, use of certain hammer weight for certain pile size (diameter & depth), amount of effort used to impact the pile top surface, data processing while collecting the data, etc., can readily influence the results if their contributions are not recognized. Foundations such as bored piles or *cast-in-situ* driven piles with multiple or large variation in cross-section can result in complex records that are difficult or impossible to analyze. Also, generally piles with L/D ratio not exceeding 30 can produce the necessary signals, without excessive damping due to soil resistance or pile material properties, although this rule can sometimes be deviated and piles with greater ratio reasonable tested under special circumstances. Fig. 2 shows integrity test graph of a pile with L/D ratio of 50. Integrity test does not produce information on pile capacity or pile load transfer mechanisms. Pile integrity testing is, however, capable of producing information on pile quality, e.g. the presence of defects such as voids or breaks, and pile length. Even these capabilities are impacted by assumptions that will need to be made during signal processing, e.g., assumptions on the propagation of wave speed based on judging the pile materials. In addition, even under ideal conditions, it is prudent to allow a level of uncertainty in the results although the level of uncertainty is affected by the confidence in available information. It is not uncommon to assume integrity test results on pile length to vary by as much as 10%, especially that wave speed variations of ±5% are known to be quite possible due to varying pile material quality.

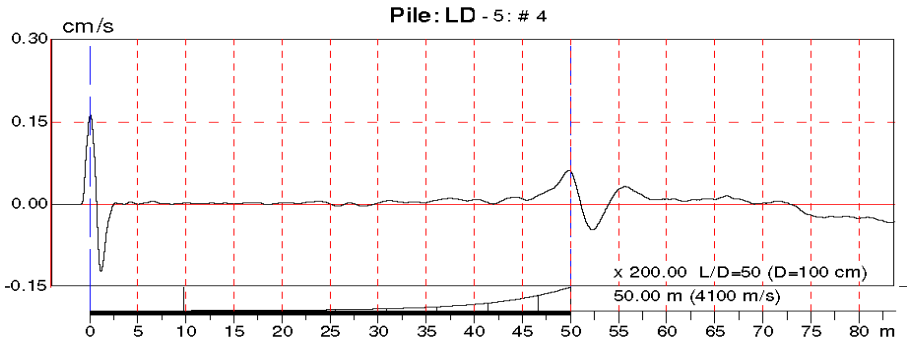



**Fig. 2 Integrity Test Graph of pile with L/D = 50**

## DATA COLLECTION

The data collection effort is extremely important if data evaluation is to be meaningful. The accelerometer must be firmly attached. Probably re-attach accelerometer every time between acquiring sets of records to make sure. The top concrete can, and always should, be evaluated visually for hardness and for contaminated concrete. If contaminated concrete is observed, it should first be removed prior to test. Fig. 3 shows photo of data collection at site.

While collecting integrity test data on site, random noise is most easily minimized by averaging several records. Even small defects within a short distance of the surface may distort the sensor signal with high frequencies; sensing and impacting at several pile top locations may eliminate such difficulty. For large diameter shafts, it is recommended practice to test at several locations around the top of the pile to inspect for local defects near the pile top. For any pile it is helpful to have more than one record to compare consistency of results. *Consistent records are the result of uniform impacts on sound concrete, transducer*

*systems that are properly functioning, motion sensors that are firmly attached, and the apparatus for recording, reducing and displaying data properly functioning (ASTM D 5882).*



**Fig. 3 Low-Strain Integrity Testing**

## DATA PROCESSING

Soil friction and poor concrete quality are often the greatest obstacles for successful integrity testing (latter the most) since they dissipate the stress wave energy. Exponential amplification with time may help overcome these problems, but only if the records contain discernible details reflected by the lower pile portion (in case of high skin friction). Late-occurring vibrations imposed at the pile top are also amplified and distorted so interpretation after amplification should be performed carefully. The magnification function (MA) is best kept constant on similar pile lengths on the same jobsite and is often chosen to make the toe reflection visible. MA begins shortly after the input pulse with a value of unity and increases exponentially to some factor at the pile bottom. The amplification function should be shown graphically. The result of the application of MA is shown in Fig. 4. Exceptions would be piles of different size and/or in different soil conditions. If the signal exhibits ringing, try a softer blow or try to minimize other potential vibration sources (any ongoing activity in the vicinity of test area such as pile chipping).

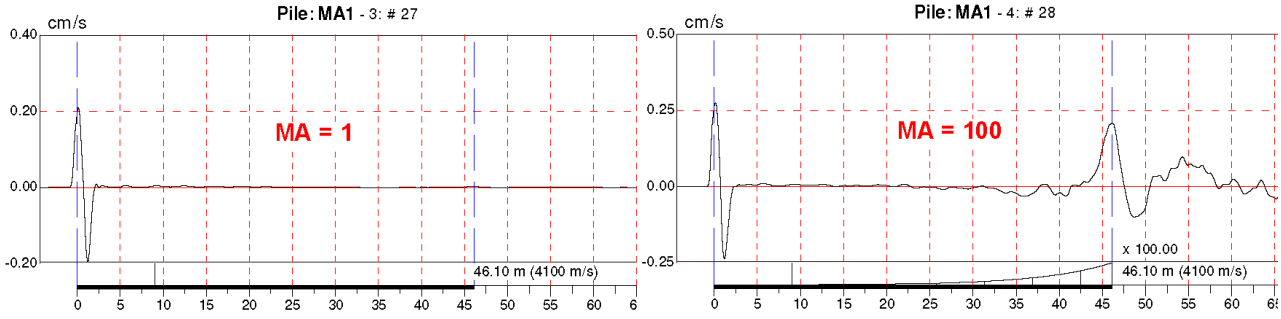



**Fig. 4 Integrity test graphs with different magnification factors**

Magnification Delay (MD) identifies the beginning of the magnification function. It should start early enough so that effects of soil and internal pile damping are properly compensated for. Making the MD value so large that amplification only starts shortly before 2L/c produces a false impression of a smooth pile with a clear toe signal and is totally improper. Graphs with different Magnification Delay factors are shown in Fig. 5. It is best to use the 20% of the pile length as default (or a fixed value slightly larger than the pulse width) and increase MD only where there is greater than 20% free pile length above grade.

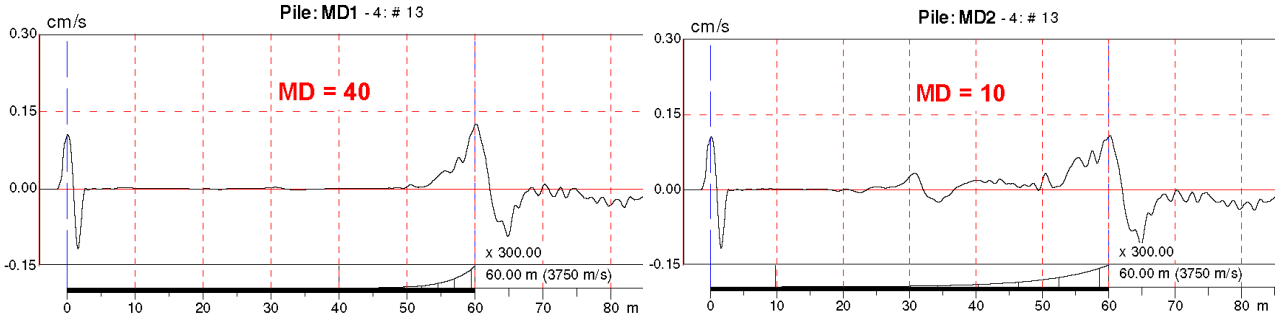



**Fig. 5 Integrity test graphs with different Magnification Delay factors**

High Pass Filter (HI) helps remove slowly changing record portions usually caused by soil resistance. If HI is too low (corresponding to a high cut-off frequency), then the record may lose features that are important for damage or length identification. The lowest suggested value is about ten times the signal pulse width or 50m whichever is higher.

'Wavelet' (WL) and 'Low Pass' (LO) are filters generally used to smooth the velocity trace. Hence both filters should not be used for the same record. The Wavelet filter is preferred over 'LO'. If Wavelet is used, LO pass filter should be set to zero. If chosen with high value (corresponding to a low frequency limit), these filters can remove significant high frequency components of the record and cause an improper interpretation. A 'WL' value of about equal to the impact pulse width is preferred (typically 1m). Figure 6 shows processed integrity test graph with above discussed parameters necessary for analyzing the data.

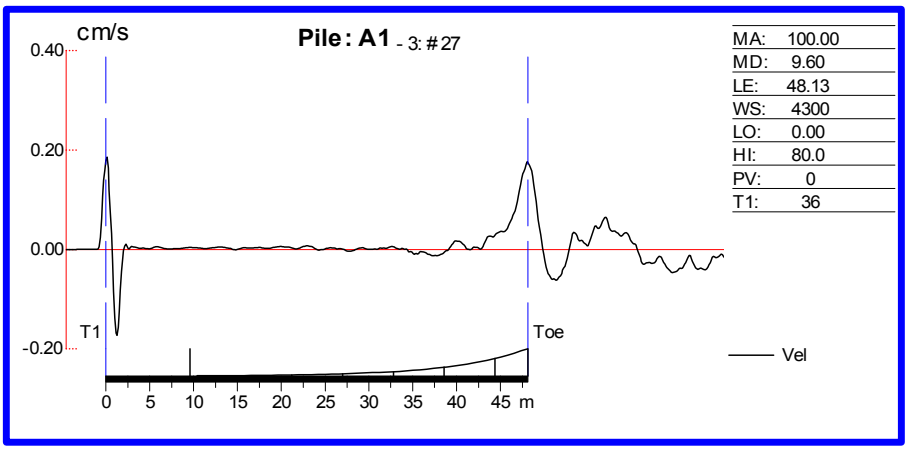



**Fig. 6 Typical processed integrity test graph**

## DATA EVALUATION

Although the acceleration curve could be interpreted, integration to velocity generally enhances the record by bringing out details otherwise overlooked. Both force and velocity have a positive value at the beginning of the record (PIT equipment assigns a positive value to the initial input). The force returns to zero as the hammer rebounds from the pile top. A record from a perfect shaft would have the initial impact (positive sign) followed by a flat (zero) response until a reflection (usually positive sign) from the bottom is observed. In practice there is usually shaft friction which, by absorbing energy, shifts the velocity record negative after the initial impact. For a stiff concrete pile in relatively weak soil (strength compared to the concrete), the toe reflection will have the same sign (positive) as the velocity input. Fig. 7 shows velocity plot of a sound pile in soil. For a pile with a fixed end (such as a rock socket), the toe reflection may be of the opposite sign (negative) as the velocity input. Fig. 8 shows velocity plot of a pile well socketed in hard rock.

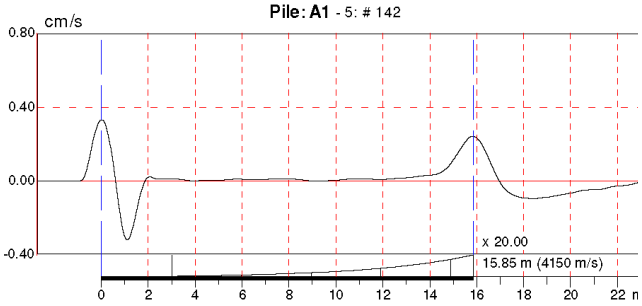



**Fig. 7 Typical integrity test graph for a uniform pile shaft in soil**

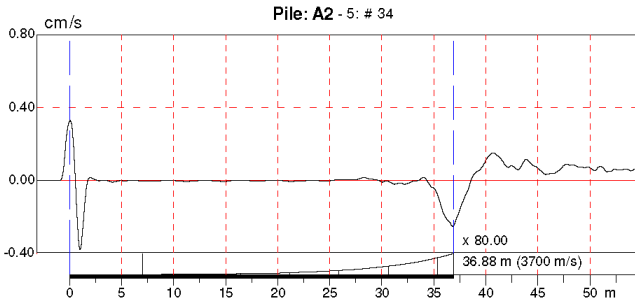



**Fig. 8 Typical integrity test graph for a pile in hard rock**

Other reflections, observed only in the velocity record, are caused by changes in the pile impedance ($Z=\rho.A.c$). Such reflections can be due to changes in either the cross-section or the pile material (density). A region of impedance reduction exhibits a positive reflection (same sign as initial input, which for this discussion is assumed as a positive signal); an impedance increase causes a negative reflection (opposite sign of initial input). A local decrease (necking) would have a positive reflection followed by a negative reflection (positive – negative cycle). Example is shown in Fig. 9. A local increase (bulb/bulging) would have a negative reflection followed by a positive reflection (negative – positive cycle). Sample integrity test graph is shown in Fig. 10. The reflections must be interpreted to determine whether the associated changes are acceptable or of serious concern to the integrity of the pile. The magnitude of the reflection is related to the size of the impedance change. The resistance level R (low, mid or high) affects the magnitude of the response; the size of the change also influences the reflection.

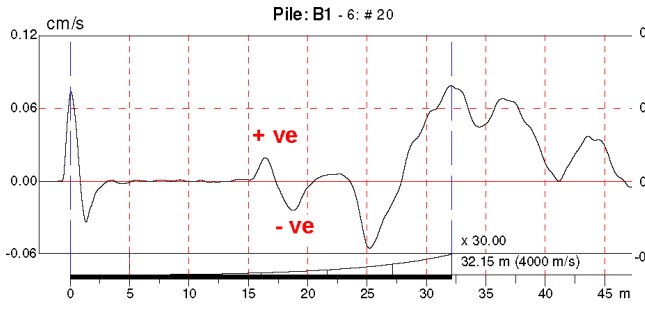



**Fig. 9 Typical integrity test graph showing impedance reduction**

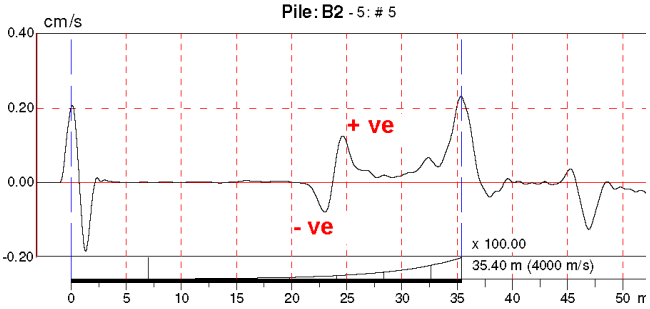



**Fig. 10 Typical integrity test graph showing increase in impedance**

## *Interpretation of Complex Integrity Test Data*

Most of the times we don't get test data as seen before in previous sections hence ASTM D 5882 states *'Engineers with specialized experience in this field are to make final integrity evaluation as interpretation of data is a matter of engineering judgment and experience'*. Below are examples of few complex integrity test data.

Figure 11 shows integrity test graphs from two different piles (C1 and C2). An impedance increase (negative – positive cycle) can be clearly seen at 25m in both the piles. But pile C1 shows yet another reflection (positive – negative cycle) at 37m. How this should be classified? Is this a matter of size and extent of impedance change at 25m?

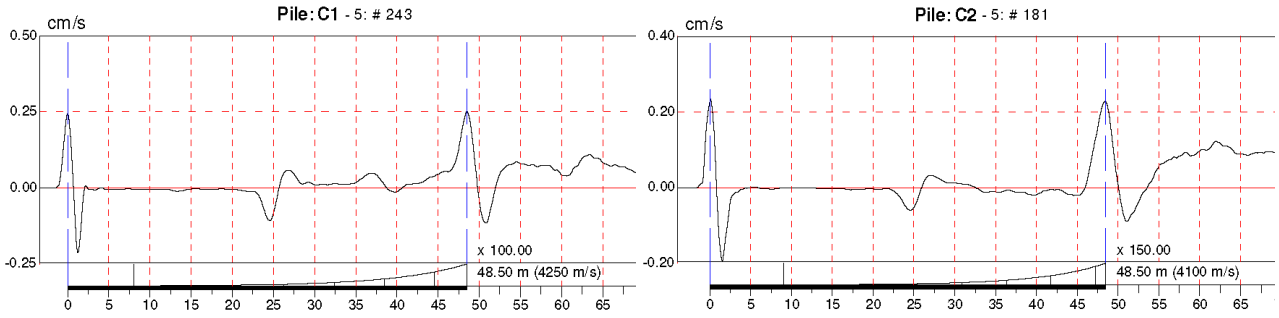



**Fig. 11 Typical integrity test graph showing impedance change**

Figure 12 shows integrity test graphs from two different piles D1 and D3 showing impedance decrease (positive – negative cycle). The graph of Pile D1 shows impedance decrease at 16m, 32m and 48m. Toe response is also possibly clear at 53.21m (i.e. pile length as per site records). Subsequent reflections at 32m and 48m are secondary from the impedance decrease at 16m. Pile D3 shows impedance decrease at 12m, 24m, 36m, 48.43m (pile length as per site records). Again these are most likely the secondary reflections from the same impedance decrease at 12m. Pile toe response (if any) might have got super-imposed with these reflections. Secondary reflections are also observed beyond 48m (60m, 72m and 84m) all in multiples of the original defect at 12m.

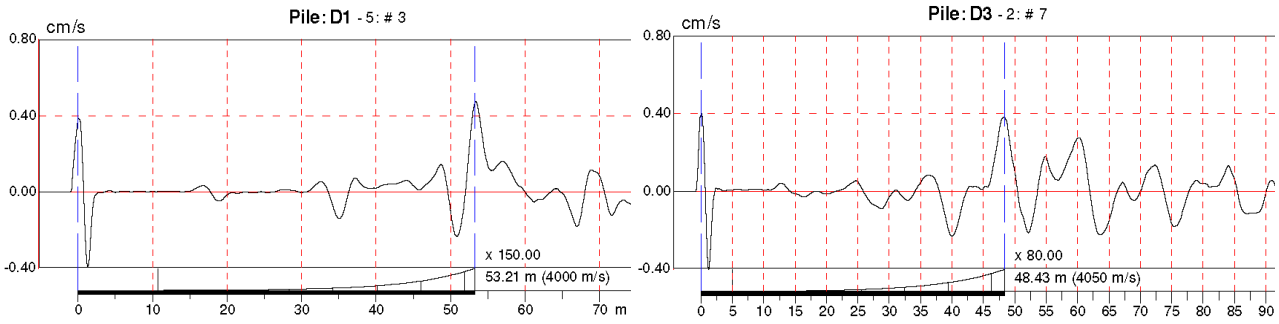



**Fig. 12 Typical integrity test graph showing impedance change**

Figure 13 shows integrity test graphs from two different piles (H1 and H2) with possible impedance changes of same nature and at multiple levels within the pile shaft. Pile H1 shows impedance decrease at 11m and again at 16m. Further reflections at 26m and 32m cannot be guaranteed and so is the reflection from the pile toe. Pile H2 shows possible increase in impedance at 19m and also at 32m. Toe reflection in this case may be considered as clear at 56.36m (pile length as per site records).

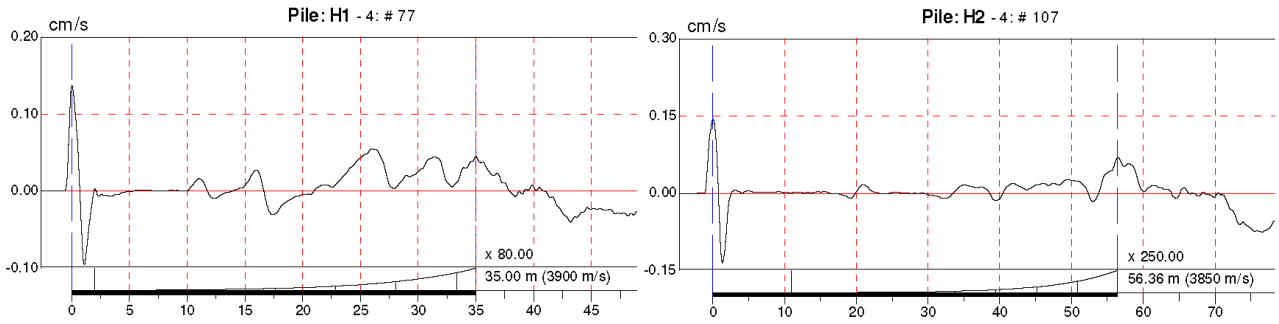



**Fig. 13 Integrity test graphs showing change in impedance at multiple levels**

Figure 14 shows some interesting data which in fact is from two different sites. Pile M1 shows increase in impedance at 21m and its mirror (reverse) reflection can be seen at 42m. As per site information the pile had a permanent casing till 21m and the soil investigation report does not indicate any soft soil layer around 42m. Similar observations were in case of Pile M2 with increase in impedance around 18m and its mirror reflection at 36m. It should also be noted that these were one of those many other piles tested that showed such mirror reflections.

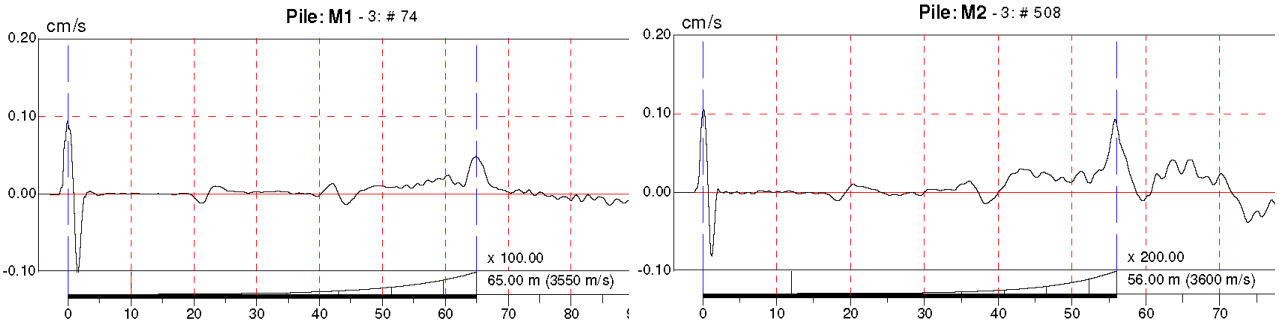



**Fig. 14 Integrity test graphs showing change in impedance with mirror reflection**

Figure 15 shows integrity test graphs from two different piles with either improper pile top surface or improper sensor attachment. Such reflections (wavy pattern) are commonly obtained by the inexperienced operators. Also no attempt should be made to conduct the integrity test on a weak or loose concrete surface. Such integrity test data are good for nothing and contain no meaningful information which can be utilized for interpretation and for concluding the pile integrity.

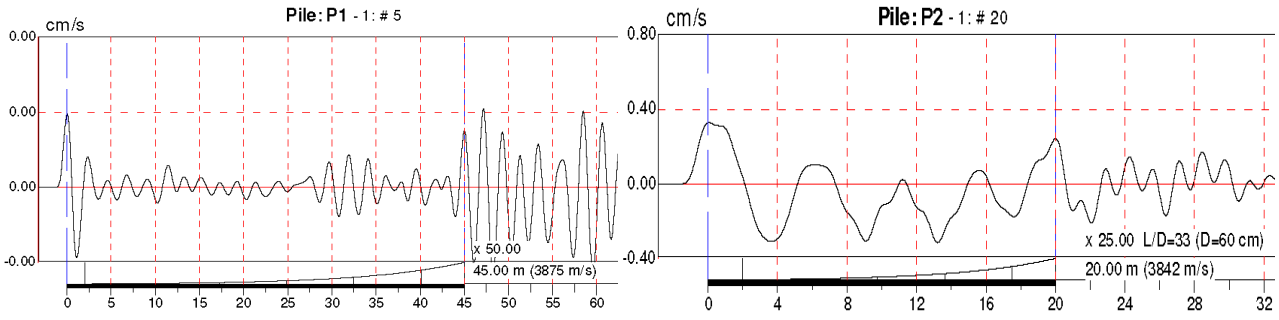



**Fig. 15 Integrity test graphs from piles with improper pile top surface**

## ACCEPTANCE CRITERIA FOR PILE INTEGRITY TESTING

As per ASTM D 5882, *the low strain impact integrity test evaluation should not be used as the sole factor in establishing pile acceptance or rejection. A contingency plan should be formed that allows the engineer to possibly perform further tests or dictate pile repair or replacement prior to the low strain impact integrity testing, in case a serious defect is indicated*. Hence the test results can generally be placed into the following four categories and each one of them may require a different course of action.

A – Clear toe response. No apparent defect.
B – Clear indication of serious defect. Toe reflection is usually not apparent.
C – Indication of possible defective shaft, although toe response is apparent.
D – Inconclusive data

**Category A** piles are fairly uniform piles within the accuracy limits of the method. It is generally agreed that a defect that affects less than 20% of the pile cross section cannot be easily detected with certainty (actually this is strength of the method since it does not create questions where only minor problems exist) (G. Likins et al. 2000). In this category pile integrity is satisfactory and unless there is some other reasons to suspect the shafts, they are generally acceptable. Of course, this assumes that the pile length indication is also satisfactory.

**Category B** piles are somehow defective and doubtful piles and some contingency plan must be used. Extra tests could be made to prove the acceptability of the pile. If the defect is near the ground surface, excavation to expose and repair the defective pile portion is generally possible.

**Category C** piles may be assigned a reduced capacity. Also other pile tests may be considered or excavate and repair if the defect is located near the ground surface. If the pile is a friction pile and the defect is located far down the pile, the upper soil resistance above the defect may render the force in the pile at the defect as acceptable structurally and the defect may in some cases be not serious.

**Category D** piles may have poor data due to poor concrete quality at the pile top. Trimming the pile to a lower elevation where good quality concrete is assured and retesting the pile is an alternative. Other reasons for inconclusive data are long or irregular pile shapes (with multiple anomalies) producing complex records. Extra tests or expert advice for data interpretation may help solving the problem.

## LIMITATIONS

The successful application of the technology, however, requires understanding its limitations as well, including operator's familiarity with the system and experience with pile foundations, applications to bored piles or driven cast-in-place piles with potentially multiple or large variation in cross-sections, L/D ratio limits, etc.

The method works particularly well on solid pile cross sections such as concrete piles. It has only limited applicability to steel piles but concrete filled pipes can be tested.

In cases where there are mechanical joints or full section cracks, the wave cannot usually cross the resulting gap. Therefore only the portion above the gap is really tested.

The pile length effectively investigated by integrity test depends not only on soil damping but also on the non-uniformity of the pile, because any change in pile quality or shape produces reflections. The first non-uniformity detected will be more reliably analyzed than additional sources of reflection farther down the shaft. Highly non-uniform piles produce complex records which are difficult to analyze. In such cases

the records collected under comparable circumstances can be compared. Records that differ significantly from the 'normal signature' should be further investigated.

Pile integrity test gives no information about load carrying capacity. To confirm pile capacity a static load test (as per IS: 2911) or a high-strain dynamic load test (as per ASTM D4945) are required to be performed.

## SUMMARY AND CONCLUSIONS

Since the late 1990's, low strain integrity testing has gained wide acceptability in India in the foundation engineering and construction community. It has become an important tool for verifying the pile integrity. It is relatively inexpensive and with only minimal pile preparation, can reasonably be performed on all working piles. It can be successfully used during initial stages of a project to assist engineers and contractors with quality control / quality assurance needs.

The method offers several advantages over other testing methods, including other NDTs, for its rapid deployment, mobility, speed and cost. A large number of piles can be tested in a short time, probably 50 – 60 piles in a day. It is capable of quickly producing information on the possible presence of defects such as voids, breaks, discontinuities, or inclusions, and provides estimates on pile length.

The analysis and data interpretation process also has to be done carefully, particularly when choosing magnification and filter parameters. Fully automated data processing is usually not advisable; each step of the data processing procedure should be reviewed as to its effect on the records.

The processed data should be plotted and the records classified. Depending on the data interpretation certain additional actions may be required on the construction site. It is important that such possible actions are clearly established prior to conducting pile integrity test.

## REFERENCES

ASTM Standard D 5882 (2007) "Standard Test Method for Low Strain Pile Integrity Testing of Deep Foundations" ASTM International, www.astm.org

Likins, G. and Rausche, F., 2000. Recent advances and proper use of PDI low strain pile integrity testing. Proceedings of the 6th International Conference on the Application of Stress-Wave Theory to Piles, Sao Paulo, Brazil, September 11-13, pp 211-218.

Massoudi, Nasser and Teferra, Wondem, 2004. Non-Destructive testing of piles using the low strain integrity method. Proceedings of the 5th International Conference on Case Histories in Geotechnical Engineering, New York, NY, April 13-17, pp 1-6.

Rausche, F. and Goble, G.G., 1979. Determination of pile damage by top measurements. American Society of Testing and Materials, Symposium on Behavior of Deep Foundations, Raymond Lundgren, Editor, Special Technical Publication, ASTM STP 670, pp. 500-506.

Rausche, F., Likins, G. and Shen, R.K., 1992. Pile Integrity Testing and Analysis. Proceedings of the 4th International Conference on the Application of Stress-Wave Theory to Piles, Hague, The Netherlands, pp. 613-617.

# NEW TESTING APPROACH FOR PILE LOAD TESTS IN INDIA

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## ABSTRACT

Static Load testing is conventional technique for testing of pile in which loads are applied vertically to the top of the pile and generally known as 'top down testing', has been in practice from many years all over the world and has become a standard test for the foundation engineering. This method cannot be used under certain circumstances like in limited space, maximum magnitude of load. There is a finite limit to the load that can be applied with either kentledge, the use of reaction piles or anchors, and as the load increases the cost of testing also increases.

A new test method seeks to overcome all the large-scale problems associated with this top down testing technique. This method will divide the foundation elements into two or more sections and these can be loaded axially as required using a portion of the foundation element as a reaction. Bi-Directional Static pile load test is economical and less time-consuming test as compared to conventional static load test. Therefore, Bi-Directional static pile load test can be considered as alternative test for the upcoming heavy load testing of pile foundations.

**Key words: Load test, Bi-Directional static load test, Super cell, S-Cell, Foundation engineering, Equivalent Curve.**

## 1. INTRODUCTION

There are several reasons why engineers specify full scale loading tests out of which the optimization of the foundation design is the most interesting from a geotechnical point of view. From structural significance satisfactory performance of the element under load is of similar importance. Also, from a foundation engineering or contractor's perspective it is important to achieve adequate or sufficient capacity from the surrounding ground with the approved design.

Static load testing, where loads are applied vertically to the top of the pile and often referred to as 'top down testing', has been used for many years and has become a 'standard' test for the world of foundation engineering. Bi-Directional pile load test method attempts to overcome all the associated problems of conventional top-down static load testing techniques. The Super cell (S-Cell) load test is a bi-directional, axial, compressive, static load test conducted on deep foundations to evaluate the soil-pile resistance. This paper attempts to highlight some of the advantages of S-Cell over the conventional methods in Indian scenario.

## 2. BI-DIRECTIONAL PILE TESTING

### 2.1 INSTALLATION

The S-Cell assembly is installed into the pile cage or carrying frame, either at or close to the pile toe, or along the shaft at a level where approximately equal capacity will be available above and below known as balancing point (Fig. 1).

The pile is constructed as usual and the cage is placed in the pile bore with the S-Cell assembly attached. Strain gauges installation and hydraulic connections are made and cables and hoses are brought up to the pile head. A guide arrangement is constructed for insertion of the tremie into position wherever needed. Sediments and cuttings should be removed from the pile bottom before concreting. Concrete is then pumped at slow and steady rate into the pile shaft and around the S-Cell assembly with proper


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supervision to avoid damaging the cell or altering the size and shape of pile (IRC:78-2014, Amended in 2018).

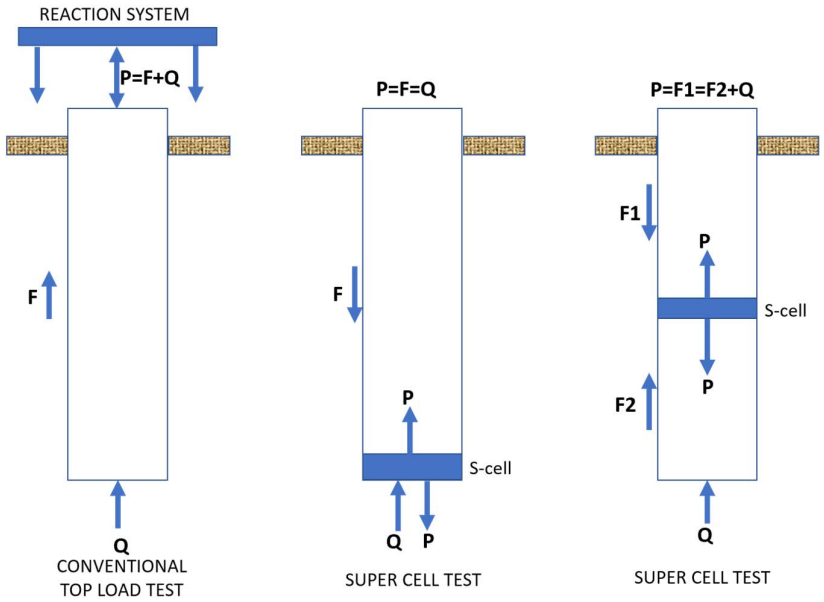



**Fig. 1 Test Method Comparison**

The area required for work in both overhead and lateral direction is reduced drastically as compared to any other static load testing method. Testing has been performed with very less space like inside buildings, under overpasses, and at offshore locations.



**Fig. 2 Assembly of S-Cell, 1.2 m above from bottom of pile attached in cage of 1.5m dia.**

## 2.2 SAFETY

In conventional method of 'top-down' testing the safety considerations are very challenging, especially at high loads as very tall kentledge assemblies need to be constructed or reaction beams need to be assembled high off the ground. In contrast, bidirectional testing has all reactions generated from within the pile itself.

As shown in Fig. 3(a) and (b), a horizontal beam used for reference only, all the things which are visible at ground level is the pile head and the top of pile instrumentation. The test area is little bit more than the perimeter of the pile shaft.



**(a)** **(b)**

**Fig. 3 Assembled set up for Bi-Directional Static Load Test.**

**(a) LVDT's with Reference Beam**

**(b) Data Logger, Data taker, Control Box and Pressure Pump are connected together**

## 2.3 METHOD OF OPERATION

In top-down testing, when load is applied at the pile head via a reaction system, an equal force, P, is applied downward to the pile and upward to the reaction system. All of the load measured at the pile head is applied to the pile which is equivalent to the sum of mobilising skin friction and the end bearing, P=F+Q.

With an S-cell placed at the toe of the pile, the load is applied directly to the end bearing. The base resistance is used to mobilise the skin friction and vice versa. Therefore, the skin friction and the base resistance mobilised are equal until one or the other reaches ultimate capacity or the S-Cell system exceeds its capacity, P=F=Q.

Where the skin friction (F1+F2) is expected to be higher than the base resistance, the S-Cell can be placed at some balance point along the pile shaft where P=F1=F2+Q. The pile element above the S-Cell uses the friction and end bearing below as a reaction (Fig. 1).

Detailed analysis of the soil properties along the pile shaft can be made by embedded strain gauges or other devices. The main application of using S-Cell for deep foundation load testing method has been elevated, from costly, time overwhelming, little scale field tests, to progressive, value effective complete static load testing of dedicated preliminary or working piles.

## 2.4 TESTING PROCEDURE

Embedded jack assembly is used to apply load to the test pile following the specified procedures recommended by engineer. The resulting test load applied to the pile is twice the load measured in the jack assembly. Do not exceed the structural capacity of the pile or the jack assembly if at all possible.

Although there is no safety concern, the pile properties may change near the load and the test may be shortened or the data may be compromised in part.

To avoid excessive creep and possible structural failure of cast-in-place concrete piles, the concrete should generally achieve approximately uniform strength throughout the pile and at least 85 % of the mix design compressive strength before the pile is tested.

Prior to performing the test, circulate hydraulic fluid through each jack to verify connectivity, saturate the system, and flush any blockages. Setup a reference beam with tent setup such that it protects reference beam and LVDTs against wind and heat.

Apply load as per the loading sequence confirmed by engineer, the loading may also be halted or continued at the direction of the engineer. The engineer may specify a longer time interval for the final load increment to assess long-term creep and for the final zero load to assess rebound behaviour.

Displacements shall be measured with the load still in place at the end of the relevant period of maintained loading, and are relative to a datum established before the load has been applied to the pile. The readings can be displayed graphically as the test progresses. Thus, load/displacement data recorded above and below the S-Cell level is available for immediate assessment (as that illustrated in Fig. 4).

All the stored data will be analysed after end of the test to calculate load distribution at strain gauge levels, Ultimate Skin friction and end bearing at failure load or at maximum test load, to draw equivalent top load movement curve.

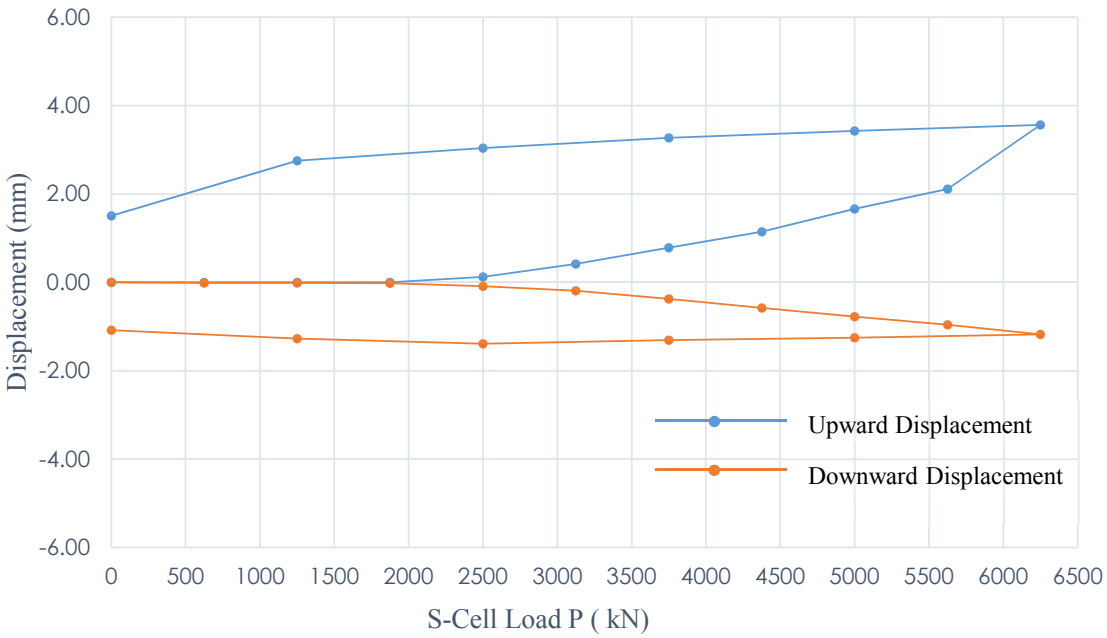



**Fig. 4 S-Cell Load Vs Displacement curve**

The total load mobilised in the pile is twice that applied by the S-Cell system, because the load is applied in two opposite directions (bi- directional) simultaneously, Self-weight (including buoyant force) of the pile is considered during calculation of balancing point. Therefore, the stresses within the concrete are half of the required stress by an equivalent top-down load test.

Comparisons of static loading results between “top-down” loading and bi-directional testing have provided excellent correlation, (England 2008).

## 3. IMPORTANCE OF HIGH-QUALITY DATA

Recognition of good quality static load testing is very important. The Least count of LVDT and Pressure transducer are 0.01 mm and 0.01 MPa respectively. However, the significance of maintaining the load truly constant and the potential for analysis of the displacement–time behaviour to facilitate the long-term load settlement analysis of the pile behaviour is not always obvious.

## 4. COMPARISONS

### 4.1 HIGH LOAD

There are lots of practical problems which are experienced when trying to assemble a top down load test, when load is more than 4 MN i.e. approx. 400 ton. Lots of space and machinery required to make proper arrangement. Although multiple jacks are arranged in order to reach the required loading, the design and assembly of a suitable reaction system becomes an engineering challenge.

By using S-Cell testing method there is saving in terms of cost, transport, installation and erection of kentledge, anchors or anchor piles as well as the associated reaction system required above ground level. There is improvement in terms of safety, at the head of the pile the assembly of a loading system is not required and the loads applied are buried. These features can make it preferable over top down static load testing.

The installation/welding of an S-Cell on an existing steel reinforcement cage can typically take one or two days with proper alignment. But for a reaction system for loads up to 4 MN may take over 20 days to make proper platform and reaction arrangement with help of loading blocks.

There is advantage of using of multiple cells in a Bi-Directional test, whenever high load is required the arrangement will be accordingly made with proper geometry such that it will not block the concreting arrangement.

Dr. Jorj Osterberg developed this testing technique in 1987, and subsequently helped to start the company Loadtest, founded in 1991. Since then, more than 3,000 O-cell tests have been conducted around the world including the 206 MN load world record test (A. Ayithi et. al 2013). Recently with single level S-Cell, 80 MN load test is done on a 2.2 m diameter pile, 31.7 m long, in Project MTHL at Mumbai India.



**Fig. 5 Bi-Directional test in progress (80 MN) at Project MTHL, India**

### 4.2 ROCK SOCKETED

It is difficult to evaluate skin friction in rock sockets because it is not possible to ensure that the load applied is reached the area of interest. In top down tests, it is found that the skin friction is often indistinguishable from the stiff end bearing and extra sensors within the pile body are required to evaluate the friction distribution. Bi-Directional tests can be arranged to apply the load directly into the rock sockets (or other zone of interest) and thereby the resultant behaviour is more readily interpreted. There are many applications of Bi-Directional tests in rock sockets but in few cases the frictional behaviour has been significantly underestimated.

### 4.3 USE OF TEST PILE

A misconception often applied for this test is that any pile tested to over 1.5 times its working/service load is not suitable for integration into the structure. Total capacity of pile is nominally unchanged in case of a top down test because locked in stresses at the base will reduce the pile settlement upon reload. Similarly, in bi-directional tests, after fully grouting in and around the cell generally the original stiffness is reverted with increase in base stiffness behaviour up to loads more than service load.

### 4.4 SELECTION OF TEST PILE

When it comes to expendable preliminary pile tests, it is obvious that these need to be preselected. For proof tests, it can be perceived a disadvantage to have to select the pile to be tested using the bidirectional approach. In practice, preselection is also common place when choosing a pile for proof loading in top down tests as practical reasons, such as space restrictions, often impede the installation of anchors or erection of reaction systems. Of particular advantage with bi-directional tests, is that the working area around the pile required for the test is minimal and significantly less than that needed for a top-down test.

### 4.5 TESTING

Bi-Directional statics load test will take less time as compared to conventional top down test, hence also known as Quick static load test (ASTM D8169/D8169M-18).

### 4.6 TOP LOAD MOVEMENT CURVE MUST BE CALCULATED

It is useful to see the results of a bi-directional load test in the form of a curve showing the load versus settlement of the pile if it were top-loaded. There are some methods to find the equivalent top movement curve but generally the test results can be combined by adding the loads mobilized at equal displacements. Some additional parameters like elastic shortening, buoyant force, modulus of elasticity of Pile along the length of pile etc are needed to keep in consideration for calculating equivalent top load movement curve. A smaller number of comparisons have been made between the equivalent pile head movement and top loading test results around the world. So, there is need to find out appropriate method to draw equivalent top load movement curve and verify the results of Bi-Directional test.

### 4.7 APPLICATIONS

While in several countries up to 1% of the piles installed are static load tested, it still appears more routine to apply loads from top to down at the pile head. However, in many circumstances the full-scale testing using top-down testing method is obstructed and bidirectional testing is becoming conventional. To date about 15 tests using S-Cells have been performed at project VIH, FOB Sabarmati river front, MTHL, Signature Bridge Dwarka etc in India and the interest in using the method and the number of different applications is continues to grow. S-Cell has been installed in precast piles and some of the tests have been carried out over water (off shore). Also notable is the application of Bi-Directional testing to bridge foundations, driven principally by cost and practical aspects.

### 4.8 COST

The current comparison between cost of top down and bi-directional load tests in the India shows that up to 7.5MN loading costs per MN are similar in both conventional and Bi-Directional load test. At higher loads, Bi-Directional tests become more cost effective because only the cell cost increases, rest of the instrumentation remains same. As the number of Bi-Directional tests increases, transportation cost and mobilization of instruments and personnel are also reduced. Hence the cost of test will reduce with increase in test load and number of tests (Schmertmann and Hayes, 1997).

## 5. CONCLUSIONS

Although the execution of top-down load tests has been optimized and made as cost effective as possible, there is scope for yet a further development in the practice of full-scale foundation testing using Bi-Directional tests.

The S-Cell is being used routinely in many parts of the world and provides:

• Unlimited loading capability, as compared to limited capacity in top-down testing.

• Considerable cost savings at loads above approximately 7.5 MN.

• Use of Bi-Directional S-Cell testing technique enables collection of full-scale data even under the most extreme and difficult conditions by providing significant advantages in terms of space and safety.

• No reaction system is required with its concomitant transportation and erection at ground level and the test energy is safely buried well below ground.

• The Bi-Directional test is a static maintained load test and has benefited by the use of automatic data acquisition techniques and load maintenance for accurate, efficient data processing and analysis to calculate skin friction and end bearing separately.

• The S-Cell test method particularly excels in offshore testing environments and the top of the foundation element does not need to be cast above water.

**ACKNOWLEDGMENTS**

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**REFERENCES**

1. England, M., 2008. Review of methods of analysis of test results from bi-directional static load tests. Deep Foundations on Bored and Auger Piles, BAP V, Ghent, pp.235-239.
2. Fellenius, B.H., 2015. Analysis of results of an instrumented bidirectional-cell test. Geotechnical Engineering Journal of the SEAGS & AGSSEA, 46(2), pp.64-67.
3. Kim, S.R. and Chung, S.G., 2012. Equivalent head-down load vs. Movement relationships evaluated from bi-directional pile load tests. KSCE Journal of Civil Engineering, 16(7), pp.1170-1177. doi: 10.1007/s12205-012-1700-8.
4. Osterberg, J.O., 1998, June. The Osterberg load test method for bored and driven piles the first ten years. In Proceedings of the Seventh International Conference & Exhibition on Piling and Deep Foundations (pp. 1-17).
5. Osterberg, J., 2000. Side shear and end bearing in drilled shafts. ASCE Geotechnical Special Publication, (100).
6. Osterberg, J.O. and Hayes, J.A., 2002. The Osterberg load cell as a research tool. In proceedings of the international conference on soil mechanics and geotechnical engineering (Vol. 2, pp. 977-980). AA BALKEMA PUBLISHERS.
7. Schmertmann, J.H. and Hayes, J.A., 1997. The Osterberg cell and bored pile testing – A symbiosis, The Third International Geotechnical Engineering Conference – Cairo University-Egypt.

# OPTIMIZATION OF PROJECT SCHEDULE AND COST USING AVAILABLE STEEL SECTIONS AS FOUNDATIONS FOR A DEEP SEATED STRUCTURE

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## ABSTRACT

In a bauxite mining project, located near a sea shore, foundations for a 24 m deep Car Dumper Vault (CDV) was designed as partially compensated raft. Excavation and dewatering schemes were designed and constructed by another agency and it included installation of three levels of sheet piles. The scope of author's organisation was limited to design and construction of CDV buildings only. However, when excavation reached about 23 m depth below FGL, east side of the pit was failed initiating more than 2 m of lateral displacement of the sheet piles. The excavation system was re-designed with open cut. The CDV foundation was re-designed as 800 mm dia. bored cast-in-situ piles socketted in underlying bed rock. It was estimated that two month time would be required to install required 136 nos. of piles and estimated cost would be about USD 6,00,000. During discussions, it was learn that 15,000 MT of structural wide flange bearing pile section HP 360 x 133 was purchased by the Owner in 2008 for a project conceived at that time which was later abandoned. To utilize the available HP-sections as CDV foundation, corrosion tests were carried out. Installation of designed 330 nos. of H-piles was completed within one month including load tests. Total cost of installation of H-piles was about USD 2,00,000. Thus, the H-Pile sections helped the Owner to shorten the project schedule and reduce the project cost considerably.

**Keywords: Deep excavation, compensated raft, bored piles, steel H-piles**

## INTRODUCTION

A crushing and washing plant was designed for a bauxite exploration and mining project located in sea coast in West Africa. The Project includes the mining and crushing of high quality bauxite at the mine face and the transport of bauxite to the port area where it is stored before being loaded into ships for export. The bauxite ores from mines would be transported from mines by railway. Bauxite would be offloaded from trains through a dual rotary car dumper. The car dumper was required to unload the rail cars where the car dumper should tip two ore cars, with a maximum gross weight of 120t each, simultaneously into the ore receiving bin. Tippler hoppers received the material and transfered it onto two apron feeders. From the apron feeders, bauxite would be fed to the downstream stacker feed conveyor which would exit through a tunnel to the surface and to a transfer station. The conveyor would be housed in a concrete inclined tunnel.

The train unloading facility including all auxiliary equipment and services would be contained within an enclosed building including a car dumper control room. The car dumper was located at ground level therefore the dump hopper, apron feeders and stacker feed conveyor were all located below ground level.


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Therefore, this equipment is housed in a concrete vault and tunnel that extends approximately 24m below ground level.

The bottom of the vault was located at about 24 m below the finished ground level. Detailed geotechnical investigations were carried out to design various substructure and superstructures. Design and construction of excavation system and dewatering schemes was carried out by another agency. The scope of author's organisation was limited to design and construction of CDV buildings only. Foundations for the vault were designed as partially compensated raft utilizing the advantage of deep excavation required for the construction. The side slopes of the excavation were supported by three levels of sheet piles. Unfortunately, when the excavation reached about 23 m, at one side of the excavation, sheet pile supports were failed causing internal movement of the sheet piles and the slopes. Due to disturbance occurred in the founding soils the foundation system was redegined and CDV raft was designed to be supported on bored cast-in-place piles socketted in the underlying bedrock without considering the pressure relief due to excavation. However, the project schedule was badly impacted and there was an urge from the Owner to reduce the project schedule.

It was learnt that at the Owner's stock rooms at the site, huge quantity of structural wide flange bearing pile section HP 360 x 133 was stocked which the Owner had purchased for another work at the same location long back. The project was later abandoned and the H-sections remained unused. After health checking of the H-sections, the CDV foundation was redesigned to be supported on steel H-piles driven up to the bedrock. It ensured considerable savings in time and cost which was the requirement of the project.

This paper briefly describes the sequence of actions taken during the design stages and emphasizes on smart thinking and utilization of easily available materials for construction which may substantially reduce the cost and project schedule.

## GEOTECHICAL CHARACTERISTICS AT THE SITE

The project site was located in a large sedimentary basin infilled by Ordovician to Devonian strata and invaded by mafic sills and dykes. These strata rest unconformably on the older Archean-Proterozoic crystalline complex.

Organic soils with organic debris were encountered near the surface. The thickness of the organic soil layer ranged from 1.3 to 4.8 meters. The organic soils near the surface were underlain by lateritic soils and subsequently different layers of clay matrix as well as sandy to very sandy clays. The bedrock was encountered at a depth of about 37 m from the finished ground level (FGL). The various sub-surface strata encountered at site is summarized below.

Table 1. General soil profile at site

| Layer | Description |
|---|---|
| Layer-I | Hydraulically placed SAND FILL, from 0.8 m to 5.1 m thick and an average thickness of 3.3 m. |
| Layer-II | Very soft to medium stiff Upper CLAY, with organics, from 0.1 m to 5.8 m thick and an average thickness of 2.6 m. |
| Layer-III | Medium stiff to hard or medium dense to dense TRANSITION sands, silts, clays, and gravels, from 0.2 m to 9.9 m thick and an average thickness of 2.9 m. |
| Layer-IV | Medium stiff to very stiff Lower CLAY, contains SAND interbeds of varying thickness, from 1.9 to 22.3 m thick and an average thickness of 13.6 m. |
| Layer-V | Medium dense to dense Lower SAND, contains CLAY interbeds of varying thickness, from 5.3 m to 20.9 m thick and an average thickness of 13.9 m. |
| Layer-VI | BEDROCK, weathered SANDSTONE. It encountered at a depth of about 37 m below EGL. |

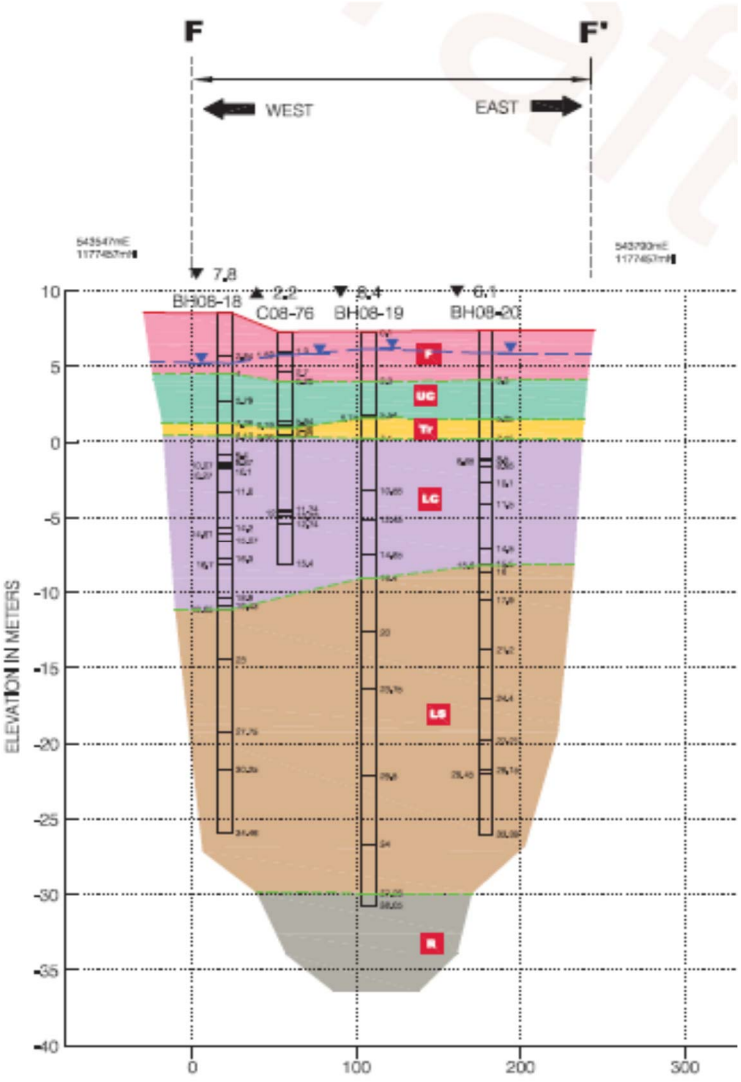



Figure 1. Subsurface conditions (see Table 1 for descriptions)

Table 2. Average soil properties of different soil layers

| Layer | N_SPT | γ (kN/m³) | φ' (°) | c' (kPa) | $s_u$ (kPa) | $E_s$ (MPa) |
|---|---|---|---|---|---|---|
| LAYER I | 30 | 17.5 | 40 | 0 | - | 50 |
| LAYER II | 3 | 16.5 | 20 | 10 | 18 | 7 |
| LAYER III | 25 | 20.5 | 28 | 10 | 150 | 50 |
| LAYER IV | 7 | 17.5 | 20 | 15 | 120 | 35 |
| LAYER V | 20 | 20 | 36 | 0 | - | 34 |

## BACKGROUND AND PROBLEM STATEMENT

The CDV bottom raft was about 24 m depth from the FGL. Considering ground water table (GWT) at FGL, the effective overburden pressure at that depth was about 24 x 0.8 = 19.2 T/m$^2$. The total pressure on the raft from the CDV structure was calculated as about 26.7 T/m$^2$. Therefore, considering the pressure relief due to excavation, a net pressure of 7.5 T/m$^2$ was generated at the bottom of foundation raft. The bearing capacity and settlement of the founding soils was calculated and it satisfied the requirement both from shear criteria as well as settlement criteria. Accordingly, the foundation was designed as partially compensated raft and also excavation scheme was finalised. The side slopes was supported by driving three levels of steel sheet piles at different levels at three sides of the excavation excepting one side where access ramp and tunnel was constructed. Pump out test was not available for designing the dewatering system. However, based on the laboratory test results of soil samples, coefficient of permeability was computed based on established empirical correlations. Accordingly, dewatering scheme was developed and few deep wells were installed at site for dewatering ground water during excavation.

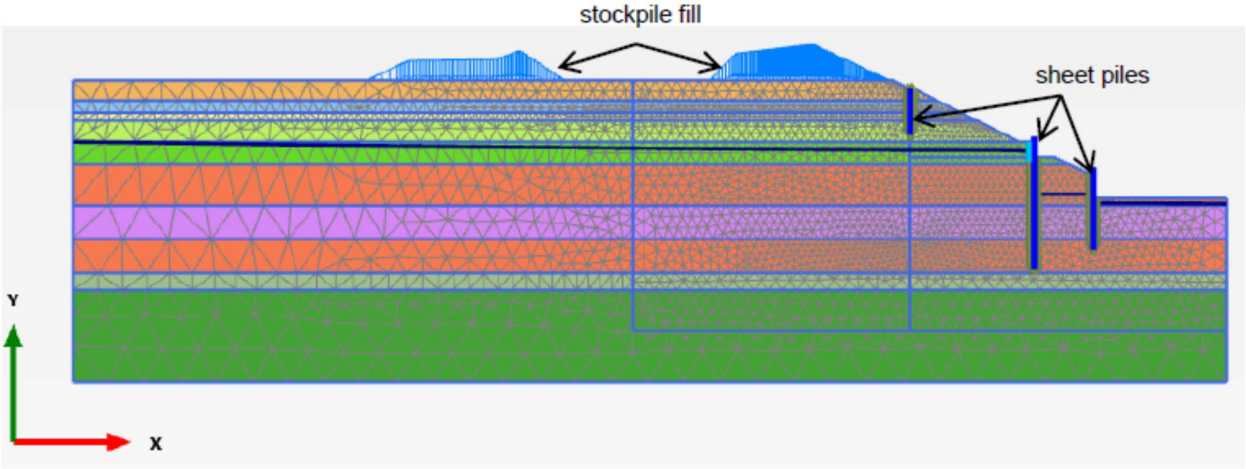



Figure 2. Finite element model showing sheet pile locations

Excavation was started and sequentially the sheet piles were driven at the designed levels. However, when excavation reached at about 23 m below the FGL, the east side slope had failed causing inward movement of all the three levels of sheet piles. The failure slope passed below the excavation level which was evident from the heaving of soils at the base at some distance away from the east side.

Extensive geotechnical investigation was carried out after the failure of the east slopes which included boreholes, piezocone penetration tests, pressure meter tests and pump out tests. Detailed analysis was carried out to find out the reason of the slope failure and it was concluded that the failure happened due to a combination of (i) saturation of the soils due to heavy rainfall and (ii) inadequate dewatering arrangement which introduced additional hydrostatic pressure. The east side slope was re-designed with open excavation in a flatter slope whereas the other two sides were supported by sheet piles. Technical reports related to the failure of the slope and subsequent remediations are captured elsewhere and are not included in this paper in detail rather the challenges in the design and execution of the foundations system within a very short time is described in the subsequent paragraphs.

Due to failure of the east slope and subsequent structural disturbances of the underlying soils due to soil movement, advantage of foundation design utilizing the pressure relief at foundation soils due to excavation was lost. Therefore, foundation system was redesigned with bored cast-in-situ concrete piles socketted in underlying sandstones. Based on the critical load combinations, the CDV raft was designed to be rested on 136 numbers of 800 mm bored piles. Approximate length of piles was 15.0 m below cut-off-level with a minimum embedment of three times of pile diameter within the bedrock. From the crushing strength of rocks and available drilling tools, it was assessed that progress of drilling through the bedrock would be very slow. It was estimated that on an average one pilling rig, working in two shifts, could install maximum three numbers of piles per day. Getting suitable piling contractor at that region was difficult and after lot of search, one prospective bidder could be identified who could mobilize maximum two numbers of piling rigs capable of socketting piles in bedrock.




Figure 3. View of the east slope prior to failure



Figure 4. View of the east slope after failure

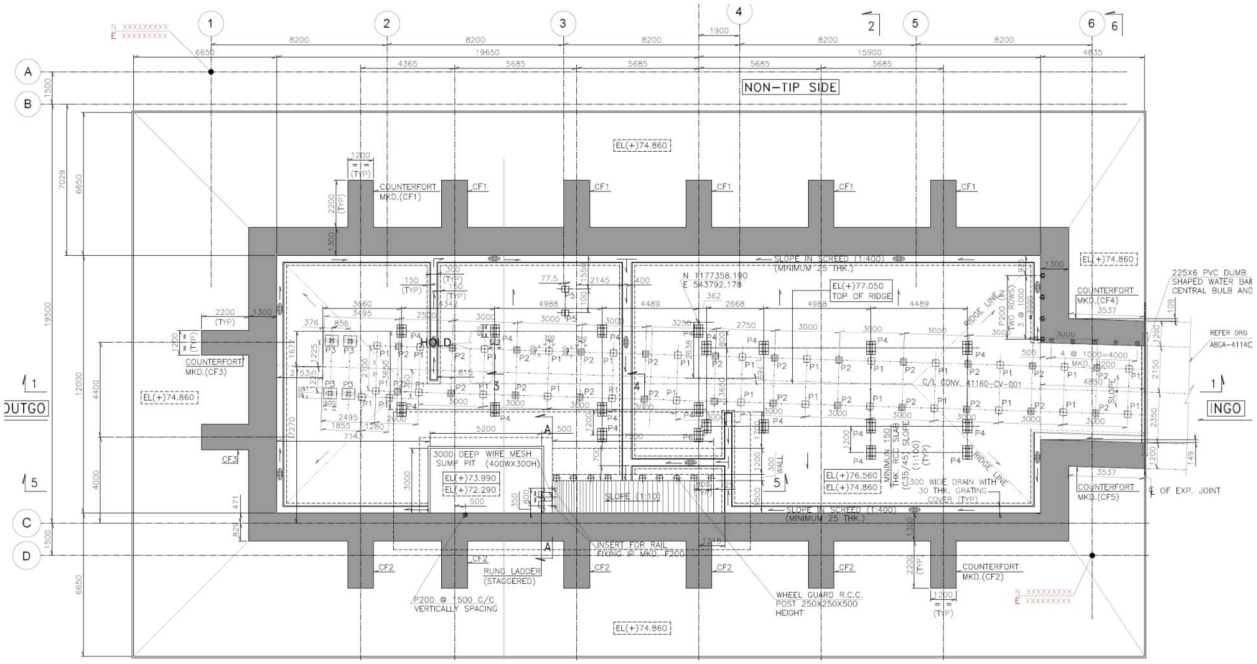



Figure 5. Layout of redesigned solution using 136 numbers of 800-mm diameter bored piles

Therefore, considering two hydraulic pilling rigs would operate simultaneously, maximum 6 nos. of bored piles could be installed in a day. Hence, to complete construction of 136 nos. of piles, 23 day time was required. Prior to installation of working piles, test piles required to be installed and tested. It was estimated that two month time was required for execution of piling works including test piles installations, carrying out pile load tests and construction of working piles. Cost of installation of 136 nos of 800 mm dia bored piles including test piles and load testing was estimated at about USD 6,00,000. The project was conceived as a fast track project from the very beginning. However, due to the slope failure and subsequent re-design of the deep excavation and foundation system, valuable project time was

already lost. Therefore, due to change in foundation system, both the project cost and the project schedule had badly affected. In particular, delay in project schedule in such a first track project was a big concern to the Owner. Therefore, savings of construction time was on top priority and alternate foundation system with lesser construction time was highly desirable.

## SEARCH FOR ALTERNATE FOUNDATION SYSTEM

Different options of alternate foundation system were investigated including construction of driven cast-in-situ piles. However, it was estimated that construction of driven cast-in-place concrete piles would not save considerable time partly because of longer time of mobilisation of such rigs and subsequently construction of test piles, load tests and construction of the working piles.

At this stage of design, it was learn that 15,000 MT of structural wide flange bearing pile section HP 360 x 133 was purchased by the Owner in 2008 for a project conceived at that time. However, later on, the project was withdrawn and the structural sections were kept on the Owner's store. The steel grade and dimensions of the section was as below:

**Pile type**
Section: HP 360 x 133 @ 133 kg/m
**Steel**
Steel Grade: S355JR per EN 10025.
Minimum Yield Strength: 355 MPa
**Dimensions**
h: 352 mm
b: 373.8 mm
Flange Thickness: 15.7 mm
Web Thickness: 15.6 mm
Length: 18 m

It was then explored whether the available HP-sections could be used as steel piles for the CDV foundation. One key concern was the probable corrosion of the structural members during the storage period. After discussions with the Owner, it was decided to carry out corrosion tests of the structural steel sections. After adequate number of testing, it was concluded that negligible corrosion of the steel members had taken place during the storage period and the structural members were suitable for use as steel piles. However, for additional safety, it was decided to consider reduction of 1 mm of thickness all around the section to determine structural capacity of the section. The geotechnical load carrying capacity of the steel piles was also evaluated by carrying out fresh geotechnical investigation after failure and the design capacity was finalized. To check the drivability of the steel piles and to carry out load tests, few dummy piles were installed. After obtaining satisfactory results, static load tests by maintained load method was carried out on one of the selected dummy pile and design capacity was verified. Design pile capacity was also evaluated based on the pile driving records using dynamic formula and cross-checked with the results of static load test. All the results corroborated each other nicely and verified correctness of the estimated theoretical pile capacity. Design of CDV foundation with H-pile was carried out. Based on the design, all total, 330 nos. of steel piles was required for the CDV foundation. From the driving records of test piles, it was concluded that about 55 min. to 60 min. time is required for complete installation of each working pile. Therefore, it was estimated that each piling rig, operating two shifts in a day, would be able to install 14–15 nos. of steel piles per day of work. Hence, considering two hydraulic pile driving machines would operate simultaneously, 28-30 no. of steel piles might be installed per day and total time required for installation of 330 nos. of steel piles was estimated at 11 -12 days. Finally,

execution of the installation of steel piles including driving of dummy piles, carrying out load tests on test piles, and installation of working piles was completed within 30 days. Thus, almost 30 days of project time was shortened which had a huge positive impact on the project.

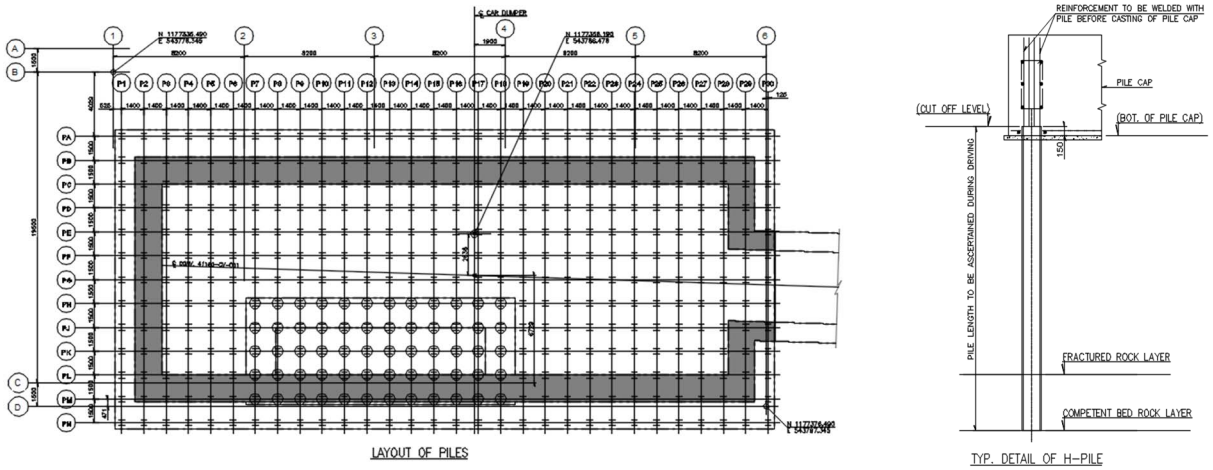



Figure 6. Layout of redesigned solution using 330 numbers of driven H-Piles

Further to the shortening of the project schedule, adaptation of the HP-sections as steel piles also enabled considerable savings in the project cost. Since the steel pile sections were available at site, free of cost, it required only the cost of corrosion protection measures and installation. It was estimated that total cost of installation of 330 nos. of steel piles would be about USD 2,00,000. Thus, about USD 4,00,000 was saved by changing the foundation system from rock socketted bored cast-in-place piles to driven H-piles.



Figure 7. View of H-piles being installed at site

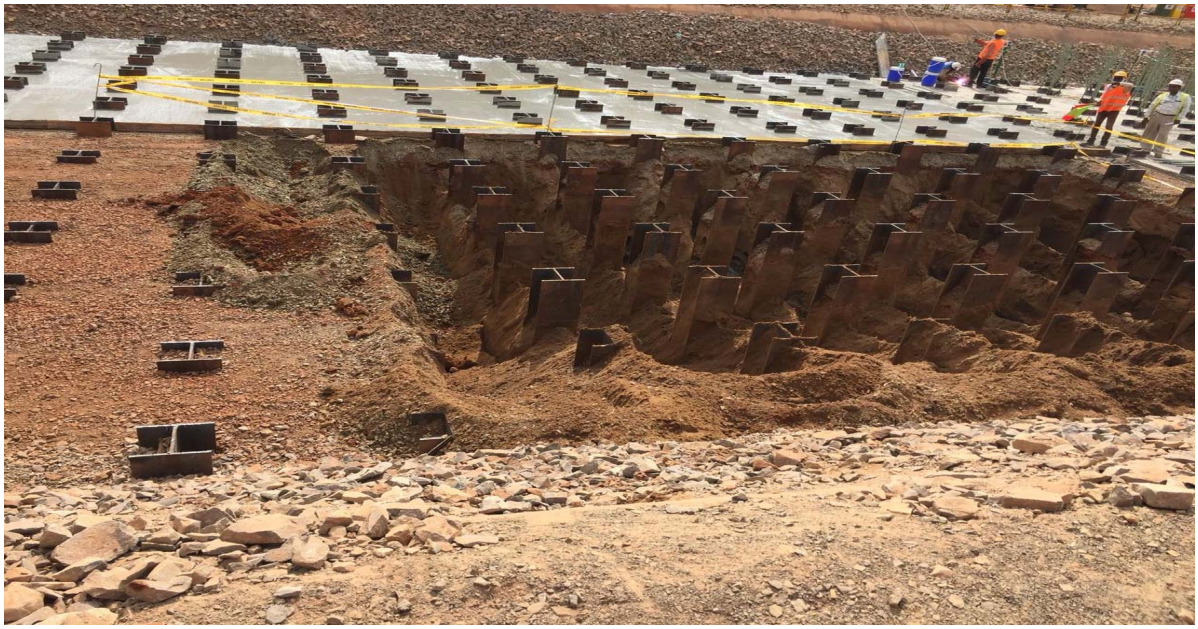


Figure 8. H-piles installed at site

## CONCLUSION

Any important project consist a project implementation schedule which is very important to complete the project within stipulated time and estimated cost. Delay in commissioning of the plant also results delay in production from the plant which has a great negative impact on the profitability of the project.

Due to failure of the sheet pile support structures, considerable project time was lost in carrying out the root cause analysis, new geotechnical investigation, re-design of excavation scheme and foundation design etc. Therefore, it was necessary to adopt a smart foundation system which could be executed efficiently within a short time period. Adaptation of driven steel H-piles reduced the foundation construction time by almost one month compared to construction of rock socketted bored cast-in-place piles. Further, the H-pile sections being readily available at site with free of cost, the foundation cost could be reduced by USD 4,00,000. Therefore, the design of CDV foundation with unused H-Pile sections helped the Owner to shorten the project schedule and reduce the project cost considerably.

## ACKNOWLEDGEMENTS

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## REFERENCES

1. ANSI/API, "Geotechnical and Foundation Design Considerations - recommended practice" First Edition, April 2011
2. BANNERJEE, P.K. and DAVIES, T.G., "The behaviour of axially and laterally loaded single piles embedded in nonhomogenous soils", *Geotechnique,* Vol. 28, No. 3, Sept. 1978, pp. 309–26.

3. Coyle, H. M. and R. Ungaro (1991), "Improved Design Procedures for Vertically Loaded H-Piles in Sand" *JGED, ASCE*, vol. 47 GT 3, March, pp 507-528.
4. Department of Highways, State of Ohio, December 1947, "Investigation of the strength of the connection between a concrete cap and the embedded end of a steel H-pile", *Research Report No. 1,*
5. Eurocode 7: BS EN 1997-1:2004 Geotechnical design; Part 1: General rules
6. M. J. Tomlinson, "*Foundation design and Constructions*", Seventh Edition.
7. M. J. Tomlinson, "*Pile Design and Construction Practice*", Fourth Edition.
8. Pike, C.W. and Saurin, B. F., Buoyant Foundations in soft clay for oil refinery structures at Grangemouth, *Procedings of the Institute of Civil Engineers,* part 3, 301-334, 1952.
9. REESE, L.C., COX, W.R. and KOOP, F.B., "Analysis of laterally loaded piles in sand". *Proceedings of the Offshore Technology Conference,* Houston, Texas, 1974, paper No. OTC 2080.
10. TERZAGHI, K. and PECK, R.B., "*Soil Mechanics in Engineering Practice",* John Wiley, New York, 2nd edition, 1967, pp. 546–7.

# RESPONSE OF A SINGLE PILE LOCATED ON SLOPING GROUND UNDER DYNAMIC LOADING

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ABSTRACT: Pile foundations are extensively used to support the laterally loaded structure. The structures like high-rise building, transmission tower, power station and offshore structures are subjected to lateral load in addition to the vertical load. The behavior of the piles on sloping ground under earthquake loading is different from the behavior of the pile located on the horizontal ground surface because of the large permanent displacement. In the present study, the numerical analysis has been carried out by using PLAXIS 2D. The analyses are carried out on single model pile under dynamic loading to determine the behavior of the soil-pile system. A single pile of embedment length to diameter ratio (L/D) of 16, 25 and 33 were located on the horizontal ground surface and slope angle of 1V:1.5H, 1V:2H and 1V:3H. The pile was subjected to dynamic loading (Bhuj earthquake, Gujarat) at its base. From the analysis, the acceleration, displacement and bending moment of the pile under each case was studied. Based on the results, it is concluded that as L/D ratio increases, the acceleration and maximum displacement and bending moment of the pile decreases.

***Keywords:*** *Dynamic loading; Numerical analysis; Sloping ground; Single pile, PLAXIS 2D.*

## 1. INTRODUCTION

A study of laterally loaded piles under dynamic loading has received considerable attention from the designers and researchers for more than three decades. The dynamic response of a pile subjected to external excitation is a complex phenomenon resulting from the interactions between the pile and the surrounding soil. The structures like high rise buildings, transmission towers and offshore structures are subjected to large lateral loads due to strong winds or earthquakes. The behavior of pile foundations under earthquake loading is an important issue that widely affects the performance of the structure. And also, many big cities in the world are built on sloped ground. Most of the structures which are constructed on soft clay with sloped ground surface are subjected to large displacements due to high lateral loads. Most of the coastline have surface which are sloping towards the waterfront. The design of pile foundation in coastal region to resist lateral load is primarily based on the limiting deflection criterion, considering the safe operation of the superstructure. Lateral loading on pile foundations due to wave and earthquake loadings are cyclic in nature, which bring additional complexity to the soil-structure interaction problem. In such cases, studying the interaction between the soil and the pile foundation due to lateral load is essential. In this paper, the numerical analysis are carried out to determine the effect of acceleration, displacement and bending moment for piles of different embedment length to diameter ratio (L/D) installed on various sloping ground conditions.

## 2. LITERATURE REVIEW

The dynamic response of pile subjected to horizontal excitation is a complex matter involving the non-linearity at large displacements, pile separation and slippage.Gaul (1958) conducted static and dynamic model tests on instrumented aluminium pipes embedded in bentonite soil medium. The dynamic load is applied by means of a mechanical oscillator and highly amplified strain variations during the vibrations are recorded. Their study shows that at relatively low frequency, magnitude and location of maximum bending moment does not vary significantly as compared with static case. Hayashi and Miyajima (1962) performed tests on vertical steel H-piles installed in sand of different relative density. Forced vibration tests are conducted and bending moment, lateral deflection of pile head and acceleration response is measured. Frequency response curves


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of single piles are plotted and natural frequency of single pile is obtained from these tests. However, the results of such dynamic 1-g model tests are affected by the 'box effect' and inappropriate stress levels. Prakash and Aggarwal (1965) reported that the zone of influence of a dynamically loaded pile can extend to greater distances such as 30 times the width of pile in the direction of loading and 15 times the pile width in the orthogonal direction. Tajimi (1969) proposed an analytical model considering the pile to support a concentrated mass and the underlying soil is assumed as viscoelastic (linearly elastic material with frequency dependent viscous damping). Nogami and Novak (1977) obtained closed form solutions for the same problem neglecting the effect of superstructure on the pile head response but considering hysteretic type material damping of the soil layer. Nogami and Konagai (1988) worked on the analytical expression in frequency-domain for the dynamic response of a single pile assumed as a massless cylinder embedded in an elastic infinite medium. El Naggar and Novak (1996) developed a computationally efficient model that accounts for non-linear behavior of soil, discontinuity conditions like slippage and gapping at soil-pile interface and energy dissipation through different types of damping. Dou and Byrne (1996) applied dynamic loading through shake table test and preserved the stress levels of the prototype for the model through hydraulic gradient similitude technique. It is observed that the bending moment in model pile reached its peak value when the frequency applied is close to the resonant frequency. It is also found that the location of maximum bending moment shifted downwards as the pile head load is increased. In addition to this, dynamic p-y curves in dense sand are also presented which exhibited highly non-linear characteristics up to a depth of four times the pile diameter for a strong level shaking. Dou and Byrne (1997) demonstrated that wave reflections at the boundary walls are negligible when the loading frequencies are far below the fundamental frequency of the soil layer. Bentley and El Naggar (2000) used 3D FE program ANSYS to develop a model that can account for kinematic soil-structure interaction of piles and used it to evaluate the dynamic pile response with respect to input ground motion. It is shown that for an elastoplastic soil with separation allowed, the lateral response of pile head is very close to the free field response at very low frequency of seismic excitation. Abdoun and Dobry (2002) discuss the critical locations in shear and bending response of deep foundations in liquefied soil based on the reported case history. Maheshwari et al. (2003) studied the effects of plasticity and work hardening of soil on the free field response and also the kinematic response of single piles using the advanced plasticity-based model in a finite element formulation. Maheshwari et al. (2004) confirmed that the material non-linearity significantly affects seismic response of pile foundation which in turn is dependent on the frequency of excitation. Boominathan and Ayothiraman (2007) carried out a comprehensive investigation on single piles installed in soft clay under static and dynamic lateral loads. It is noted that the dynamic magnification factor reduces with an increase in magnitude of load due to large hysteretic damping originating from the extreme non-linear behaviour of soft clay. The active length under dynamic lateral load is increased by about 1.5 times compared to that under static lateral load. Maiorano et al. (2009) performed 3D finite element analysis on a pile subjected to dynamic lateral load to examine the kinematic bending moments in different subsoil conditions. The piles are represented as elastic beams while the soil is modelled using linear elastic constitutive model. The results are used to predict the transient peak bending moments at the soil layer interface. Yang et al. (2011) through his shaking table test demonstrates that the dynamic p-y curves are highly affected by the input acceleration amplitude. Di Laora and Rovithis (2015) carried out a numerical treatment using the rigorous finite element technique to evaluate the kinematic bending moment of long piles embedded in a continuously non-homogenous soil resting over a rigid base. During the analysis, soil stiffness is allowed to vary with depth in a parabolic fashion with its initial value at the ground surface to be zero or a finite quantity. The active length of pile in non-homogenous soil layer is finally proposed considering the effect of kinematic interaction. Deendayal et al. (2016) investigated the effect of the earthquake on a single pile located on the sloping ground. A single pile was located on varying slope with different length to diameter (L/D) ratios. It is observed that the acceleration, displacement and bending moment decreases with the increase in L/D ratio. And also, the displacement and bending moment increases significantly as slope increases from horizontal ground surface to the slope under consideration.

## 3. NUMERICAL MODELING

### *3.1 Governing Factor*

Finite element method requires several approximations to obtain the solution during numerical modeling. The suitability of the approximation used in the modeling approach is based on the conditions in the field. Plane strain conditions are assumed to hold in the plastic deformation regime developed during the dynamic loading. The problem being modeled is complex and has to be simplified to obtain a solution. Two major factors affecting the lateral response of piles are: (1) the properties of the soil (2) soil-pile interaction at the interface.

### *3.2 Constitutive Model*

A constitutive model to be satisfactory it must be able to define the material behavior for the concerned range of stress and strain, identify the model parameter by standard material test and physically represent the material response to change in applied stress and strain. In the present study, a two dimensional finite element program PLAXIS 2D is used for the analysis. PLAXIS automatically imposes a set of general boundary conditions (standard fixities) to the actual geometry model. The Mohr-Coulomb model is used for depicting the soil behavior. The Mohr-Coulomb model is straightforward and quick and accounts for the elastoplastic behavior of the soil with utmost simplicity.

### *3.3 Geometry Model*

A geometry model is a composition of points, lines and cluster. Points and lines are created in the draw area by entering the coordinates in the command line. Then the clusters are generated by program. Each cluster gives the properties to simulate the behavior of soil. The clusters are divided into soil element during mesh generation. In addition to this, structural object such as the beam representing a pile can be assigned to the geometry. Once the geometry model has been created, the pile parameters are assigned to the corresponding geometry component. When the full geometry model is defined and all geometry components have their initial properties, then the finite element mesh is generated. Fig. 1 shows the typical models in case of L/D 33 for sloping ground.

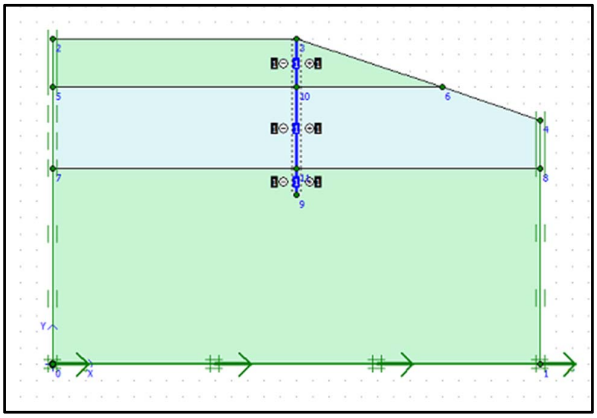


Figure 1. Geometric model for L/D = 33, 1V:3H.

### *3.4 Soil-Pile Interaction*

The Mohr-coulomb model describes the behavior of interfaces for the modeling of soil-pile interaction. The pile consists of the beam element with special interface elements providing the interaction between the pile and the surrounding soil. The pile is considered as linear elastic and its behavior is represented by using stiffness properties. The interface strength reduction factor accounting for the effect of soil-structure interaction at the interface is chosen on a conservative basis in the current study.

*3.5 Boundary Condition*

An absorbent boundary is aimed to absorb the increasing stress in the boundaries caused by the dynamic loading; otherwise, it will be reflected inside the soil body. In this model, the absorbent boundaries are generated at the left, right and the bottom boundary.

*3.6 Material Properties*

The parameter values for soil and pile are adopted from *(*Abdoun, T and Dobry R, 2002*)*. A single concrete pile of diameter 0.6 m is used for the FEM analysis in the present study. An axial modulus (EA) is taken as 3.56 x $10^5$ (kN/m) and Rigidity modulus (EI) is taken as 8000 (kN/$m^2$).An embedment pile consists of a beam element and it provides the interface between the beam and the soil enclosed by the pile. The angle of internal friction ($\phi$) and dilatancy angle ($\psi$) for sand-1 is 32 and 2, and for slightly cemented sand-2 is 35 and 5 respectively. The detailed properties of soil and pile used for modeling are listed in Table 1 and 2.

Table 1. Values of soil and interface properties.

| Description | Sand - 1 | Slightly cemented sand -2 |
|---|---|---|
| Unit weight ($\gamma$) (kN/$m^3$) | 25 | 28 |
| Material model | Mohr-coulomb | Mohr-coulomb |
| Young's modulus($E_s$) (kN/$m^2$) | 38000 | 49000 |
| Poisson's ratio ($\mu$) | 0.25 | 0.3 |
| Angle of internal friction ($\phi$) | 32° | 35° |
| Dilation angle ($\psi$) | 2° | 5° |
| Interface strength reduction factor ($R_{inter}$) | 0.67 | 0.8 |

Table 2. Values of pile properties.

| Description | Pile |
|---|---|
| Axial modulus (EA) (kN/m) | 3.56 x $10^5$ |
| Rigidity modulus (EI) (kN-$m^2$) | 8000 |
| Equivalent thickness (t) (m) | 0.519 |
| Diameter (D) (m) | 0.6 |

*3.7 Finite Element Mesh Generation and Calculation*

The geometry of the soil-pile model is divided into finite elements forming the mesh. After generating the finite element mesh, the actual finite element calculations are executed. The calculation involves two phases. First, the prescribed displacement is activated while defining the calculation phases. Second, the earthquakes have been simulated in the dynamic analysis. The displacements are reset to zero in order to analyze the effect of the earthquake in detail. Earthquake loads are usually applied by means of prescribed horizontal displacements. The earthquake loading is applied at the bottom of the pile. In the present study, aftershock of the Bhuj Earthquake (Gujarat) data presented in Fig. 2 is used. The analysis has been carried out for all cases and variation of bending moment and maximum displacement is observed. Fig. 3 shows the deformed mesh for L/D ratio 33 on 1V:3H slope.

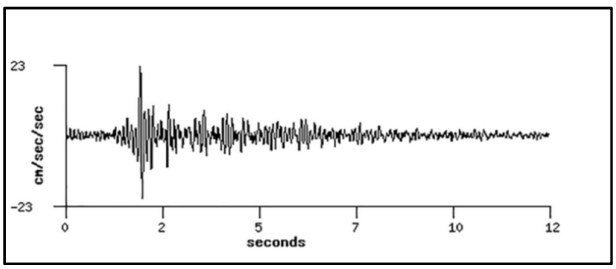



Figure 2. Acceleration-time data for the Bhuj/Kachchh aftershock recorded at the Bhuj station.

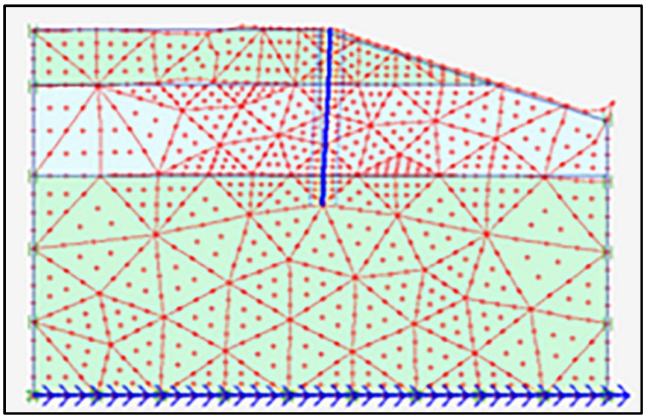


Figure 3. Deformed Mesh for L/D = 33, 1V:3H.

## 4. RESULTS AND DISCUSSIONS

The lateral load behavior of a single pile due to dynamic loading is studied by using lateral load – deflection curves. The variations of acceleration are determined for different L/D ratio in case of horizontal and sloping ground. The effect of slope and L/D ratio on maximum bending moment and maximum displacement is also analyzed.

### *4.1 Effect of L/D Ratio on Acceleration Curves for horizontal and sloping ground.*

The pile head/cap is more vulnerable to failure due to reduction in soil resistance along its shaft and also due to high slenderness of pile. In the present study, maximum acceleration is measured at pile head. Fig. 4 shows the variation of acceleration versus time graphs of a pile (at pile head) for different L/D ratios (16, 25 and 33) for horizontal ground surface. From the results, it is observed that as L/D ratio increases, the maximum acceleration decreases. When L/D ratio changes from 16-25, the maximum acceleration is observed to decrease by 59.82%. This behavior is observed due to increase in embedded length of pile or increase in the flexibility of soil-pile system. Fig. 5 shows the variation of acceleration with time for a pile (at pile head) with different L/D ratios (16, 25 and 33) installed at the crest of 1V:1.5H slope.The percentage decrease in maximum acceleration with change in L/D ratio 16-25, 25-33 and 16-33 for the horizontal ground and slope of 1V:3H and 1V:1.5H are shown in Table 3, Table 4 and Table 5 respectively. From the results, it is observed that as L/D ratio increases, the maximum acceleration decreases irrespective of slopes. When L/D ratio changes from 16-33, the maximum acceleration is observed to decrease by 61.50% and 43.25% on slopes of 1V:3H and 1V:1.5H respectively. The magnitude of maximum acceleration recorded on relatively steeper slope of 1V:1.5H is much higher for L/D ratio of 33. This is because of the reduced passive wedge of soil in front of the pile which may lead to lower inertial resistance from the soil present in the region of active pile length. This can cause the net earthquake force to predominate over the weaker resistance forces resulting in higher acceleration at the pile head.

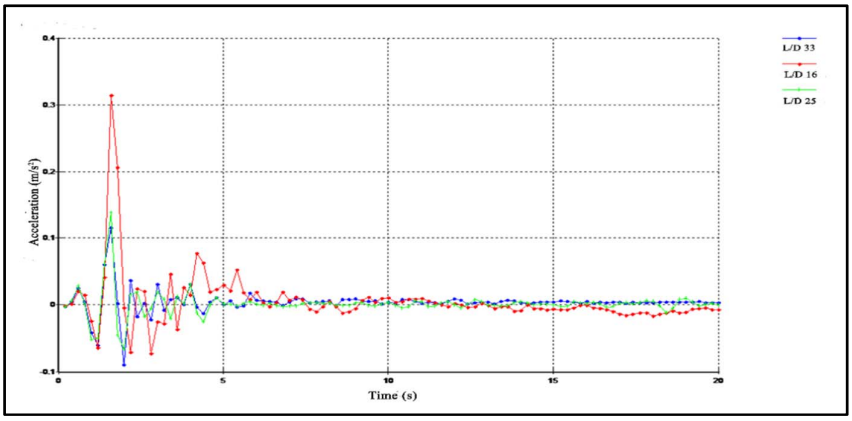



Figure 4. Acceleration-time response for different L/D ratio's on horizontal ground

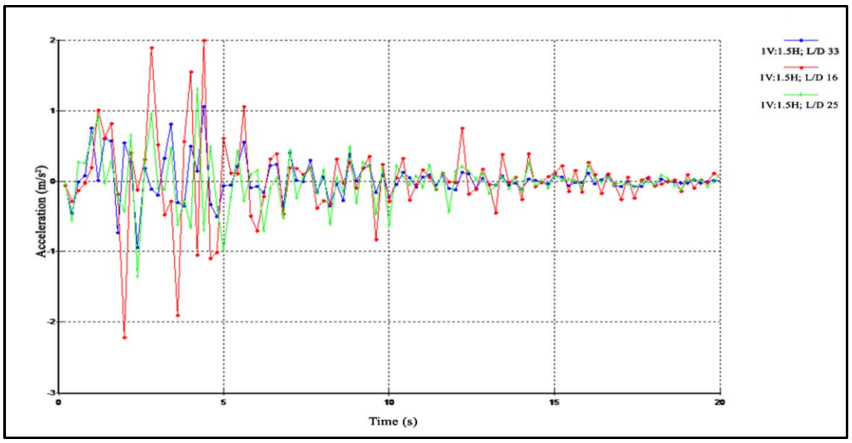



Figure 5. Acceleration-time response for different L/D ratio's on 1V:1.5H slope

Table 3. Percentage decrease in maximum acceleration due to change in L/D ratio for horizontal ground.

| | Horizontal ground Surface | | |
|---|---|---|---|
| L/D ratios | 16 | 25 | 33 |
| Max. Acceleration ($m/s^2$) | 0.326 | 0.131 | 0.110 |
| Change in L/D ratios | 16-25 | 25-33 | 16-33 |
| % Decrease | 59.82% | 16.03% | 66.25% |

Table 4. Percentage decrease in maximum acceleration due to change in L/D ratio for 1V:3H slope

| | Slope 1V:3H | | |
|---|---|---|---|
| L/D Ratio | 16 | 25 | 33 |
| Max. Acceleration ($m/s^2$) | 1.46 | 0.985 | 0.562 |
| Change in L/D ratio | 16-25 | 25-33 | 16-33 |
| % Decrease | 32.53% | 42.94% | 61.50% |

Table 5. Percentage decrease in maximum acceleration due to change in L/D ratio for 1V:1.5H slope

| | Slope 1V:1.5H | | |
|---|---|---|---|
| L/D Ratio | 16 | 25 | 33 |
| Max. Acceleration ($m/s^2$) | 2.00 | 1.459 | 1.135 |
| Change in L/D ratio | 16-25 | 25-33 | 16-33 |
| % Decrease | 27.05% | 22.20% | 43.25% |

*4.2 Effects of L/D Ratio and Slope on Displacement*

Fig. 6 shows the variation of maximum displacement with slope for different L/D ratios. From the graph, it is observed that as L/D ratio increases, the maximum displacement decreases. It is also noted that as slope increases from horizontal ground surface to steeper 1V:3H, 1V:2H and 1V:1.5H slope, the maximum displacement increases significantly. The maximum displacement observed on a slope of 1V:1.5H for L/D=16 increased considerably for the present dynamic loading considered at the pile base. This is mainly due to the heavy reduction in inertial resistance from the soil in the wedge. The present result is found to be good agreement with the literature results *(Deendayal R and Sitharam T G, 2016)*.

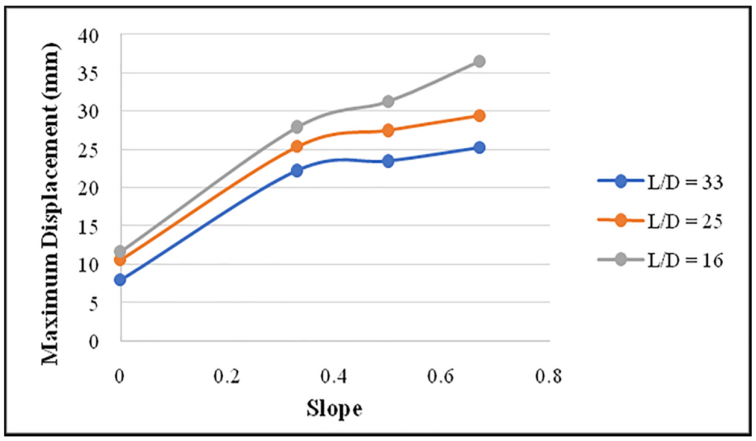



Figure 6. Maximum Displacement vs Depth curve for L/D = 16 with different slope.

4.3 *Effects of L/D Ratio and Slope on Bending Moment*

The effect of slope on the maximum bending moment is shown in Fig.7 for different L/D ratio. From the graph, it is observed that the increase in embedment length to diameter ratio, the maximum bending moment of the pile decreases. This behavior is due to the increased depth of fixity arising from the larger relative stiffness of the soil-pile system. And also, if the slope increases from ground surface (zero slope) and becomes steeper (1V:1.5H), the bending moment also increases. This is due to the reduction in the passive resistance of the soil in front of the pile. It can be noted that the bending moment increases by 50-80% as the slope approaches 1V:1.5H for the range of L/D ratio's (16-33) considered in the study.

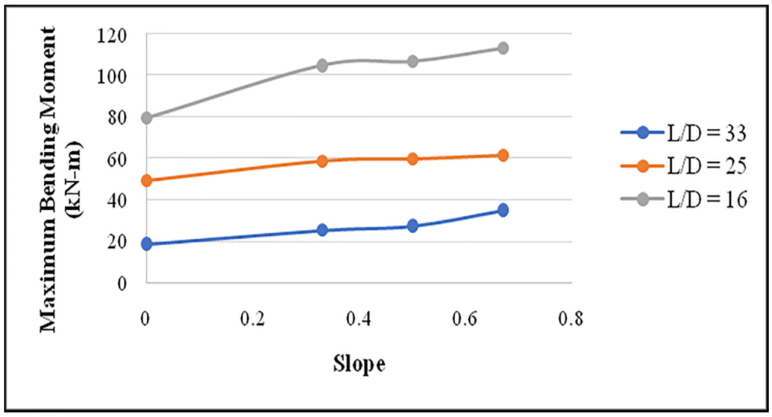



Figure 7. Variation of maximum bending moment with slope for different L/D ratio.

## 5. CONCLUSIONS

A finite element analysis has been performed to evaluate the seismic response of a single pile installed on slopes. The developed analysis has been validated with the similar study carried out by Deendayal et al. (2016). The major conclusions from the study are presented below.

1. As L/D ratio increases, the maximum acceleration decreases irrespective of slopes. When L/D ratio increases from 16-25, 25-33 and 16-33 the maximum acceleration observed a percentage reduction of 16.03-66.25%, 32.53-61.50% and 27.05-43.25% for horizontal ground surface, 1V:3H slope and 1V:1.5H slope respectively. This behavior was due to the increase in embedded length that enhanced the flexibility of soil-pile system.
2. From the study, it is very clear that as L/D ratio increases, the maximum displacement has been reducing. This is due to the decrease in active pile length that contributes to deflection as L/D ratio increases. And also, when ground surface changes from horizontal to steeper slopes, the maximum displacement increases tremendously especially for the case of L/D = 16.
3. It is noted that the bending moment decreases significantly with increase in L/D ratios. It is also observed that steeper slopes like 1V:1.5H has a considerable effect on increasing the bending moment.

## 6. REFERENCES


Abdoun, T. & Dobry, R. 2002. Evaluation of pile foundation response to lateral spreading. *Soil Dynamics and Earthquake Engineering*, pp. 1051-1058.

Bentley, K. J. & Naggar, M. H. E. 2000. Numerical analysis of kinematic response of single piles. *Canadian Geotechnical Journal*, 37(6), pp. 1368-1382.

Boominathan, A. & Ayothiraman, R. 2007. An experimental study on static and dynamic bending behaviour of piles in soft clay. *Geotechnical and Geological Engineering*, Vol. 25, pp. 177–189

Di Laora, R. & Rovithis, E. 2015. Kinematic Bending of Fixed-Head Piles in Nonhomogeneous Soil. *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 141, No. 4, 04014126, pp. 1-9

Dou, H., & Byrne, P. M. 1996. Dynamic response of single piles and soil-pile interaction. *Canadian Geotechnical Journal*, 33(1), 80–96.

Dou, H., & Byrne, P. M. 1997. Model studies of boundary effect on dynamic soil response. *Canadian Geotechnical Journal*, 34(3), 460–465.

Deendayal, R. & Sitharam, T. G. 2016. Effect of Earthquake on a single pile located in sloping ground. *International Journal of Geotechnical Earthquake Engineering,*Vol. 7, pp 57–72.

El Naggar, M. H. & Novak, M. 1996. Non-linear analysis for dynamic lateral pile response. *Soil Dynamics and Earthquake Engineering*, 15(4), pp. 233–244.

Gaul, R. D. 1958. Model study of a dynamically laterally loaded pile. *Journal of the Soil Mechanics and Foundations Division,* 84(1), pp. 1–33.

Hayashi, S. &Miyajima, N. 1962. Dynamic lateral load tests on steel H-piles. *Proceedings, Japan National Symposium on Earthquake Engineering,* Tokyo, Japan.

Maheshwari, B. K., Truman, K. Z., Gould, P. L. & El Naggar, M. H. 2003. Three-dimensional nonlinear seismic analysis of single piles using FEM: effects of plasticity of soil. *International Journal of Geomechanics,* ASCE, 5(1), pp. 35-44.

Maheshwari, B. K., Truman, K. Z., El Naggar, M. H., & Gould, P. L. 2004. Three-dimensional nonlinear

Nogami, T., & Novak, M. 1977.Resistance of soil to a horizontally vibrating pile. *Earthquake Engineering & Structural Dynamics,* 5(3), 249–261.

Nogami, T., &Konagai, K. 1988. Time Domain Flexural Response of Dynamically Loaded Single Piles. *Journal of Engineering Mechanics*, ASCE, 114(9), 1512–1525.

Prakash, S & Aggarwal S. L. 1965. Study of a vertical pile under dynamic lateral load. *Proceedings of Third World Conference on Earthquake Engineering,* Vol. 1, pp. 215-229.

Tajimi, H. 1969. Dynamic analysis of a structure embedded in an elastic stratum, *Proceedings of 4th World Conference on Earthquake Engineering,* Santiago, Chile, Vol. 3, pp. 54-69

Yang, E. K., Choi, J. I., Kwon, S. Y., & Kim, M. M. 2011. Development of dynamic p-y backbone curves for a single pile in dense sand by 1g shaking table tests. *KSCE Journal of Civil Engineering*, 15(5), 813–821.

# SOIL-STRUCTURE INTERACTION FOR A TENSION PILE PULLED WITH STRAND

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## ABSTRACT

This is a unique tension pile where the pull is at a point, a certain distance below the top of the pile. The pull is transferred by a cable consisting of high tensile strands. The HDPE duct of the cable prevents bond between the strands and pile concrete in upper portion. While soil friction on the pile will act downwards on the whole length, there is tension in the bottom portion of pile, there will be compression at the top portion. This fact came to light while test loading this pile. Also, micro-strains in the pile concrete was measured as a routine. The resulting stress pattern in the pile is analysed in the paper.

**Key words: Tension, compression, annular space, high tensile strands, side friction**

## GENERAL

Tension piles are subjected to pull out forces. Use of tension piles is common in civil engineering practice. Pull out forces come about in several ways. Common occurrences are: 1) when a structure is subjected to overturning moment, 2) A-frame anchorage of a tie back of berthing structure, 3) uplift on an underground water tank in empty condition etc. The pull out force will be transferred to the pile from the superstructure direct or from the pile cap. Tension is normally transferred through steel rods to the frame. Usually, tension piles are reinforced to cater to the tensile force. This is done by taking the reinforcement rods into the pile cap or superstructure directly. Tensile force in the pile is counteracted by the side friction between the soil and the pile. Side friction depends on the lateral soil pressure acting from all around on the pile surface. Lateral pressure will go on increasing towards the bottom. This is given by the expression:

$$P_D = K\gamma h \tag{1}$$

Where,

K – coefficient of earth pressure at rest

$\gamma$ – Effective unit weight of the soil

h – depth below ground level

Please see **Fig 1.1**.

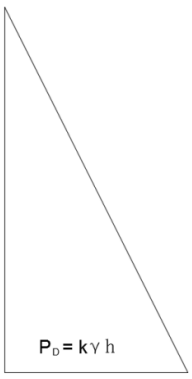



**Fig. 1.1. Lateral pressure**




**Fig. 1.2. Side friction**

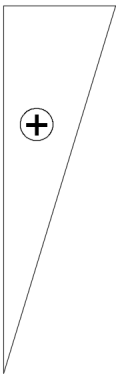


**Fig. 1.3. Tension in pile**

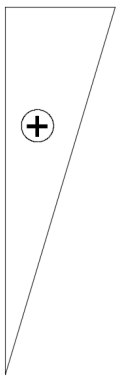


**Fig. 1.4. Tensile stress**

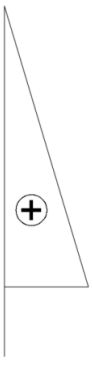


**Fig. 1.5. Applied load < soil resistance**


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Side friction on the pile is the product of this pressure and the friction coefficient. This coefficient is taken as $tan\delta$ where $\delta$ is angle of wall friction between the pile and the soil. This is illustrated in **Fig 1.2.**

**Fig 1.3** shows the tension in pile when the pull on the pile and resistance from the soil is balanced. This is very simple. Stress in the pile is the tensile force divided by the cross sectional area. This is given in **Fig 1.4.** There is also an intermediate case. When the applied load is less than the resistance of the soil, resistance of the full depth of soil is not required to resist the applied force. The extra length of the pile will be redundant. This is illustrated in **Fig 1.5**.

**UNIQUE PILE**

The above are all simple usual cases, where the pull is applied at the top of the pile by extended reinforcements going into the cap. The stress in concrete is maximum at the top. Towards the bottom, the downward friction of soil will go on reducing the pull out force and so also stress in the pile. But there is a different type of pile where the pull is transferred not at the top of the pile. Please see **Fig 2.**

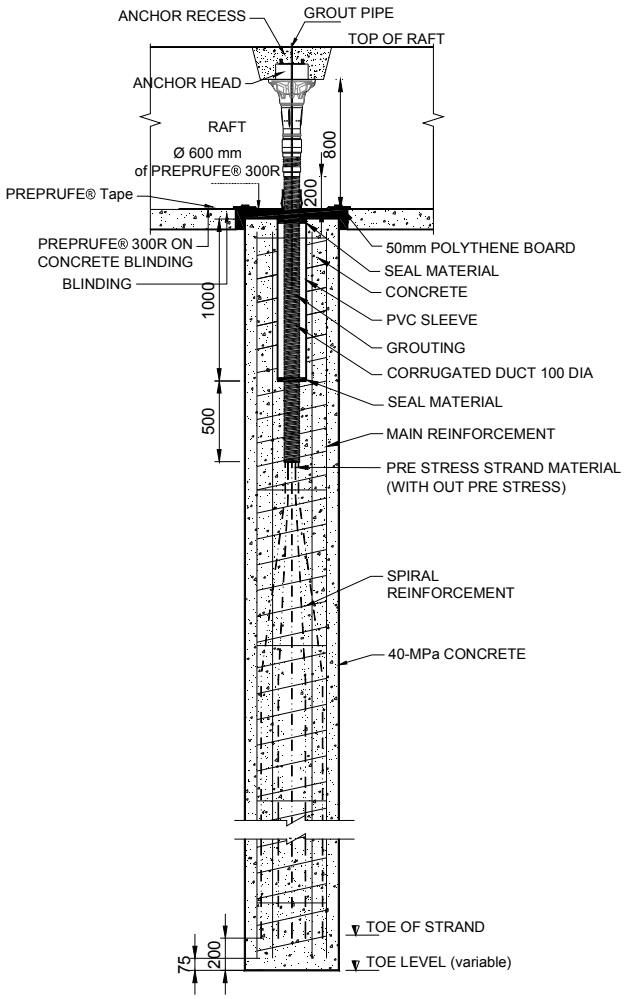



**Fig. 2. Arrangement of strands and cable**

At the top 1000mm of the pile, there is an annular space. Purpose of the annular space is for the pile to accommodate horizontal movement of the load. The tensile force is applied to the pile through a cable consisting of high tensile strands. These strands are flared out from the cable and embedded into the pile concrete. The strands are tied to the reinforcement cage of the pile before lowering the cage into the pile bore. The pile is 600mm in diameter. The duct of the cable is made up of high density poly ethylene. Please see that the HDPE duct extends 500 mm downwards below the annular space. This 500mm length is to act as a buffer against progress of corrosion. The cable is taken to the top of the bottom slab of the superstructure and anchored there. The pile is to cater to a tensile load. It can be seen from **Fig 2** that the pull out force will be transferred to the pile from the tip of the HDPE duct. This is almost 1500mm from the actual top of the pile. Above this level, HDPE duct will prevent any bond developing between the pile concrete and the cable. Friction of soil on the pile will act on the entire length of the pile. Under the action of the pull, the whole pile will tend to move up. Side friction on the periphery will act in the opposite direction – downwards.

**TRANSFER OF FORCE**

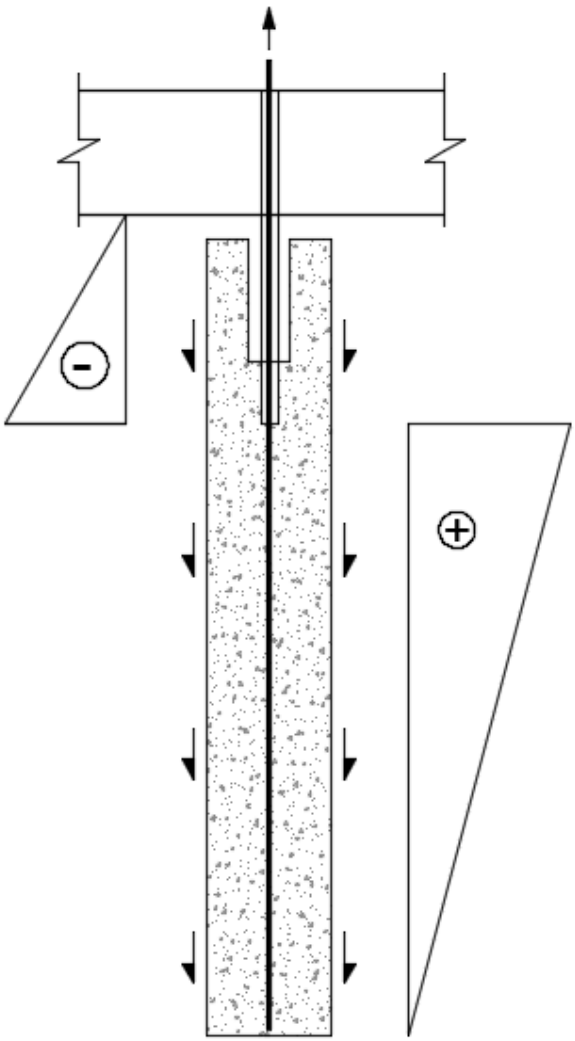



**Fig. 3.1. Schematic force diagram**

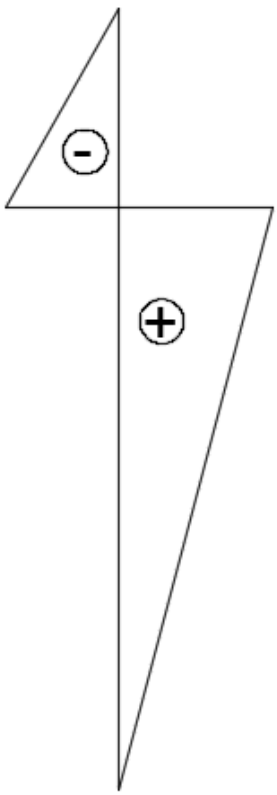



**Fig. 3.2. Combined force diagram**

A schematic force/ pressure diagram on the pile is given in **Fig 3.1** and Fig **3.2**. The pull is applied at the bottom level of the HDPE duct. The portion of the pile below this level will be in tension as the side friction on the pile will act opposite to the pull. At the same time, the portion above this level will be under compression. Being, an integral part of the pile, the portion above also will tend to move up.

In other words, this portion will be pushed up by the portion below it. This will be resisted by the side friction which is downwards. Since the pushing force is at the bottom, this portion will be under compression. This pile is a part of a group of piles to support an underground reservoir in Qatar. Uplift on the piles come from the uplift on the empty reservoir. A number of piles were load tested. Interesting data was obtained as the pile was instrumented. Levels of strain gauges are shown in **Fig 4.**

**STRAIN GAUGES**

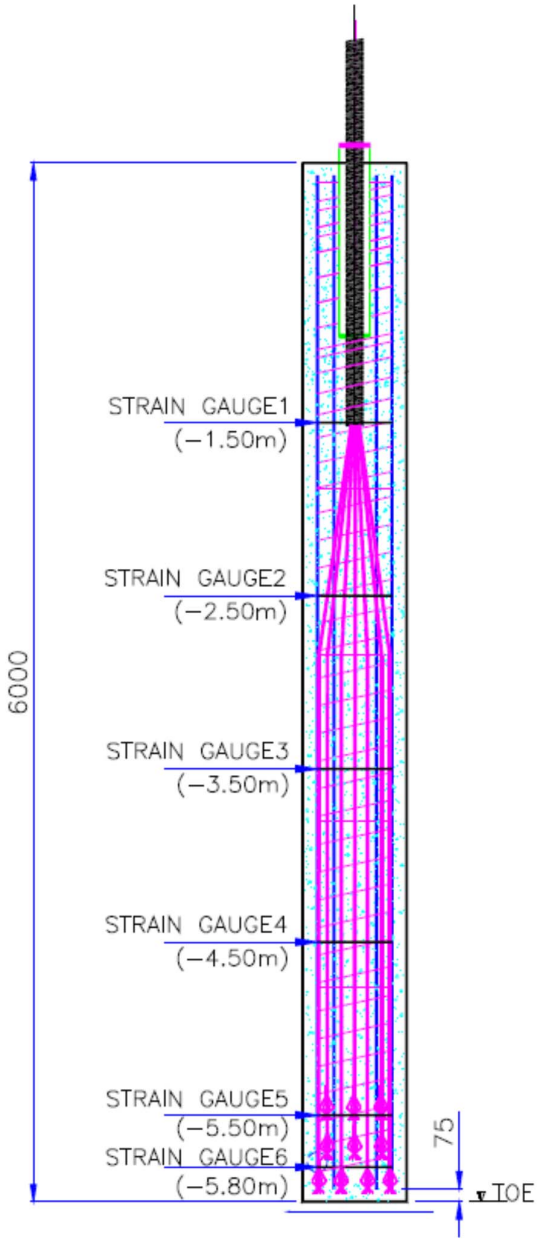



**Fig. 4. Location of the strain gauges**

Please see that strain gauges were fixed at six levels in the pile from -1.5 to -5.8. The strain gauges were monitored at various stages of the load test. 4 numbers displacement transducers were installed between a reference beam and the top of the pile at 90 degrees apart to monitor the displacement of the pile top.

## GEOTECHNICAL DATA

The subsoil consists of simsima lime stone, about 18m thick. There are layers of Midra shale and Rus formation underneath this. Standard penetration tool could not penetrate into simsima lime stone and could be termed as 'refusal'. The rock had a RQD range of 0 to 95, with an average of 50. The underlying layer of Midra Shale had the similar RQD values. The underlying layers of Fahahil velates and Rus formation also had the same properties. Safe bearing pressure for spread foundation on rock was arrived at as 450kPa. Please see **Fig 5** for the samples from investigation.




**Fig.5. Sample collected from simsima drill**

## STRUCTURAL DATA

Structural Data of the pile are given below:

Diameter of the pile =600 mm

Concrete strength (C40) =40 MPa $E_c$ =32400 MPa

Mild Steel ($E_s$) =210000 MPa

Area of cross section of pile =282600 $mm^2$

Area of steel =4396 $mm^2$

Ultimate strength of strands =1860MPa

Force at a cross section =Strain*((Area of concrete*$E_c$)+(Area of steel*$E_s$)) (2)

Latter part of the equation works out to 1.0079E+10 N

Micro strains are obtained from the strain gauges and are tabulated in **Table 1**below:

**Table 1. Strain gauge readings**

| | Micro strain at different levels*10E+9 | | | | |
|---|---|---|---|---|---|
| Level | 750 kN | 1500 kN | 2250 kN | 3000 kN | 3750 kN |
| | 50% | 100% | 150% | 200% | 250% |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| -1.5 | -7.27 | -23.67 | -33.85 | -49.44 | -67.2 |
| -2.5 | 14.94 | 36.69 | 69.19 | 103.02 | 113.4 |
| -3.5 | 3.32 | 7.00 | 9.71 | 11.71 | 17.3 |
| -4.5 | 2.86 | 5.96 | 8.06 | 8.57 | 12.1 |
| -5.5 | 0.81 | 1.57 | 1.82 | 2.21 | 3.0 |
| -5.8 | 1.15 | 2.14 | 3.11 | 3.70 | -4.9 |

Micro strain readings are plotted along the axis of the pile. The graph is given in **Fig 6** below.

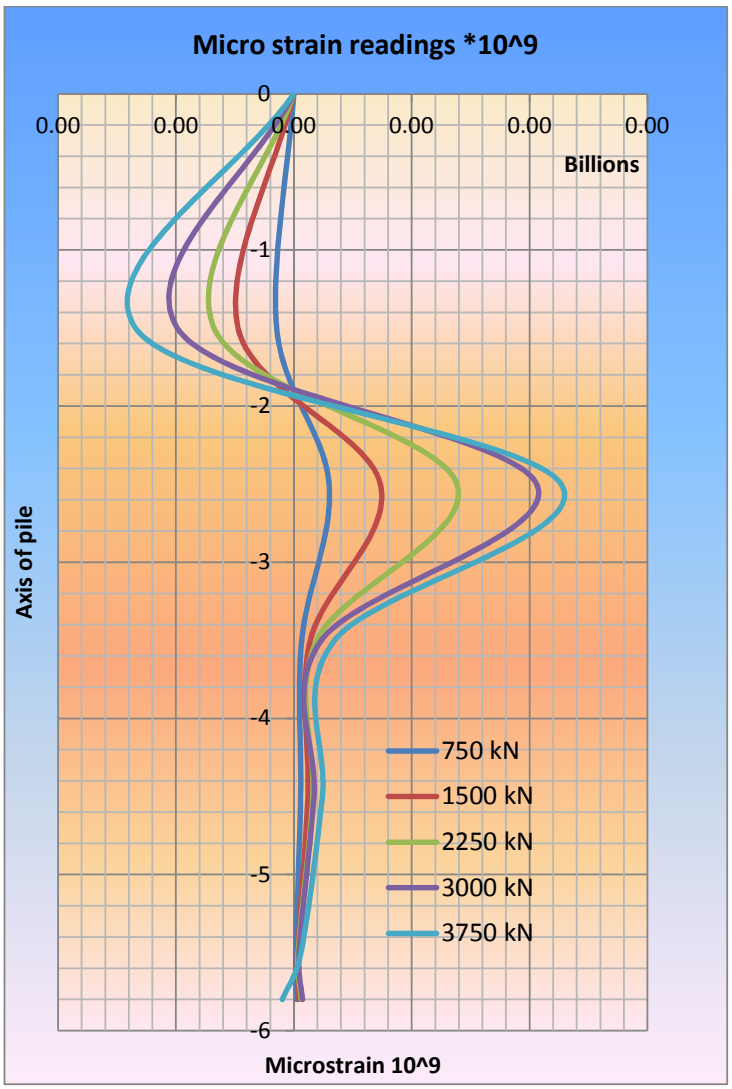



**Fig. 6. Micro strain readings along the pile**

The above readings were converted into Loads using the **Eq (2)** and are given in **Table 2** below:

**Table 2. Load calculations**

| | Load calculation (kN) | | | | |
|---|---|---|---|---|---|
| Level | 750 kN | 1500 kN | 2250 kN | 3000 kN | 3750 kN |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| -1.5 | -73.326 | -238.611 | -341.234 | -498.306 | -677.136 |
| -2.5 | 150.593 | 369.336 | 697.359 | 1038.372 | 1143.197 |
| -3.5 | 33.510 | 70.519 | 97.840 | 118.074 | 174.372 |
| -4.5 | 28.833 | 60.120 | 81.289 | 86.428 | 122.249 |
| -5.2 | 8.159 | 15.785 | 18.336 | 22.232 | 30.6047 |
| -5.5 | 11.641 | 21.550 | 31.305 | 37.296 | -49.58078 |

Computed values of force in a cross section are given in **Fig 7** below.

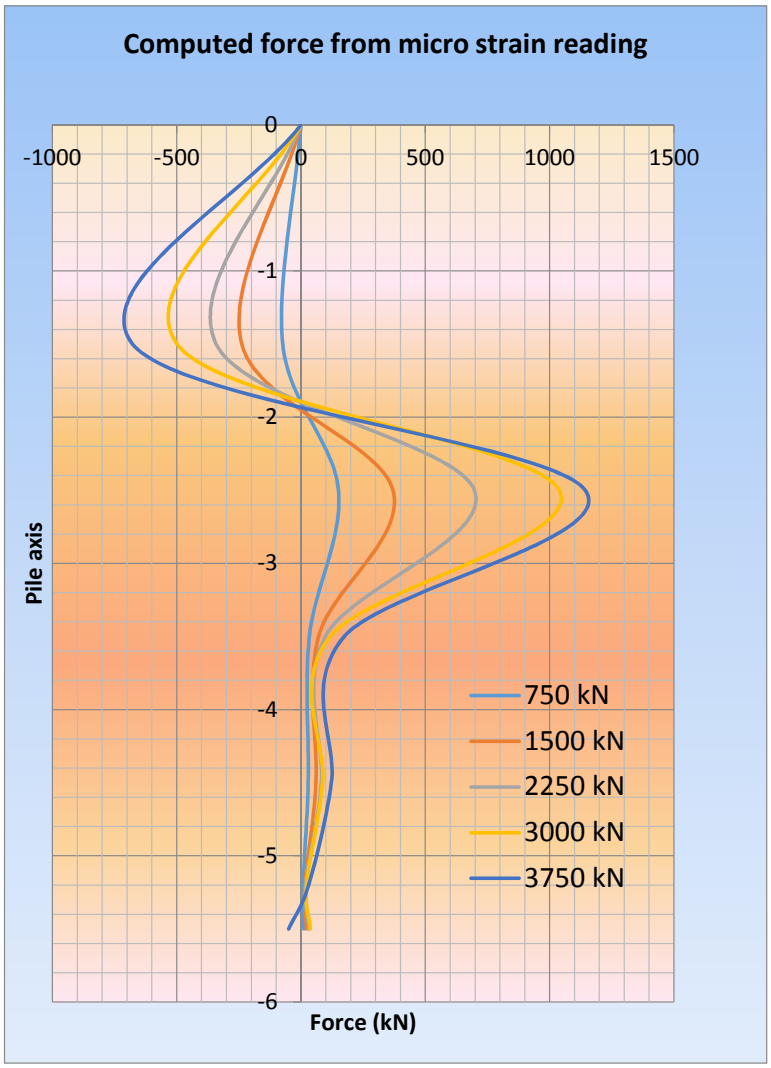



**Fig.7. Force along pile axis**

Fig 3.1 shows the theoretical stresses in the pile. The theoretical profile shows a sudden reversal of stresses at level -1.5. The stresses are compressive above and tensile below this level. Forces in **Fig 7** have been computed from strain gauge readings. They are discrete points along the axis of the pile. For example, there is one strain gauge at -1.5. The next one is at -2.5. The strain gauge at -1.5 will read the maximum compression even if maximum tension also occurs there. The strain gauge at -2.5 is certainly in the compression zone. Values in between are not reflected in the graph correctly. This is understandable. It was found that the movement of the top was 0.06mm for the design load and 0.68mm, 1.1mm and 1.42mm for 150%, 200% and 250% of the design load. Residual movement was found to be 0.22m. This was against a structural requirement of 3.29mm.

## CONCLUSION

This is a unique case of a tension pile where the pull is transferred at a finite distance below the pile top. The pull is effected by flexible high tensile strands. There is an annular space of one metre length at the top through which the cable passes. This is to facilitate horizontal movement of the load. The HDPE duct is present for another 500mm below the annular space cutting off any kind of bond between the strands and pile concrete. The pile was load tested to two and a half times the design load. The strain gauges installed in the pile provided an interesting picture of tension and compression along the pile axis.

## REFERENCES

Chandramohan P.V. and Ravi Kishore, 2017, 7th Conference on Deep foundation technologies for infrastructure development in India, 'Foundation in simsima lime stone for an underground reservoir'

Arab Centre for Engineering Studies 2015., Geotechnical site Investigation for proposed GTC/518/2012 Kahramaa Al Muntazah, Qtar.

# GEOHAZARD PROTECTION OF INFRASTRUCTURE

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## ABSTRACT

In alpine environment various geohazards affect infrastructure causing not only financial loss but even more fatalities of their users.

As the transport infrastructure - rail and road - is becoming increasingly important for economic development and strategic connectivity, the identification of natural hazards and the resulting hazards is an increasingly important planning field for geotechnical engineers.

An attempt will be made to focus attention on the various types of natural hazards - rockfall, mass movements, mudslides, avalanches etc. - not only after an infrastructure facility has been put into operation, but already in advance - when selecting routes and planning.

The different possibilities - temporary security measures, permanent security for the protection of the infrastructure - will be examined and discussed based on examples and experiences made in Europe. All phases are considered, starting with the construction of the infrastructure through to operation.

## GENERAL TECHNICAL PRINCIPLES

In Europe design of geohazard protection measures follows depending on kind of protection device Eurocode 0 – Basis of structural design, Eurocode 1 – Action on structures, Eurocode 2 – Design of concrete structures, Eurocode 3 – Design of steel structures, Eurocode 4 – Design of composite steel and concrete structures, Eurocode 5 – Design of timber structures, Eurocode 6 – Design of masonry structures, Eurocode 7 – Geotechnical design, Eurocode 8 – Design of structures for earthquake resistance, which contain the basics of structural design, execution supervision, control and maintenance.

In addition to these European codes there are several national technical rules for natural hazard protection devices – like in Switzerland or Austria.

Execution and quality control and maintenance are covered in additional European standards and technical rules. All these standards are based on the semi probabilistic principle using partial safety factors.

Necessary conditions are the identification of risks and data by qualified personnel, design by qualified and experienced designers, adequate supervision of execution and quality control measures on site, execution by trained and experienced staff, use of building materials according to approvals, maintenance of the structures in an adequate extent and duration to provide safety and durability and use of the structures according to design.


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## TYPES OF NATURAL HAZARDS

In Alpine there are several different types of natural hazards, some of them will occur only in either high alpine or cold climate, whereas the other can be found all over the world.

In general hazards are either creeping or mass movements – snow, rock, soil, water or debris – and are either man made or caused by evolution – permanent erosion process. They can be caused by nature, but also by manmade influences.

*Avalanches*

Risk of avalanche can be identified on slopes steeper than 28° with a lack of high obstacles – boulders or trees, so that the snow layer can start slipping on the surface. Different types of avalanches occur – dry avalanches with a very high speed up to 280 km/h, whereas avalanches of wet snow move slowly but their weight might be up to 800 kg/m³. Snow creeping also occurs on steeper slopes and might harm building infrastructure.

*Debris flow*

As a result of sudden heavy precipitation or snowmelt, debris flows may occur. Debris flow is not only a phenomenon in high alpine but can also occur in arid zones after heavy rainfall.

*Slope slide*

One can find slopes of different inclination all over the word – they are result of a permanent evolution of the world's surface caused by erosion of various reasons – frost heap, thermal heap, wind, water. All embankments have developed in accordance with the material properties of the values assigned to them. Their stability is round 1.0 depending on vegetation cover.

*Rock fall*

For rockfall hazards the above mentioned applies as well besides the rock structures and geological formation.

## TEMPORARY OR PERMANENT MEASURES?

Europe has a long experience in dealing with natural hazards as many alpine crossing railway lines were built already in second half of the 19$^{th}$ century, which had to be protected. Also, roads crossing alpine areas – at the beginning there was not much traffic – had to be protected. Even at the beginning it became evident, that the impact of singular events – avalanches, rockfall, debris flow – had to be minimized on infrastructure. This aim got more and more important, as traffic increased, and more infrastructure was built in previously undeveloped areas. Another reason for protecting infrastructure of natural hazard impact was the increasing question of liability. As a result, the cost of protective measures rose exorbitantly and, in addition to permanent measures, temporary measures were also considered.

*Temporary measures*

Depending on the type of hazard there are different methods as temporary protection against those – shut down of infrastructure, artificial avalanche release, cleaning of rock slopes in regular intervals.

Whereas a shutdown is not accepted any more different measure have been invented to release avalanches – man made disruption of snowpack by different means – blasting by artillery, blasting by igniting a gas-air mixture, blasting by detonating an explosive charge. These release measures must be executed regularly to prevent accumulation of snowpack causing devastating avalanches.

To reduce the risk of rockfall on infrastructure the earliest measure was regular cleaning of the rock slopes by climbers during and after the freeze-thaw cycle in spring. This very effective measure was introduced for the commissioning of the railway lines.

The investment for these temporary measures is comparably low, but the maintenance cost can raise rather quickly.

*Permanent measures*

In contrast, permanent measures cause high investment costs with low maintenance costs.

To protect railway infrastructure against natural hazards – rockfall, debris flow, avalanches - tunnels and protection galleries were built to protect against natural hazards – avalanches, debris flow.

As snow was a regular impact on infrastructure man tried to improve the stability of the snowpack by permanent retaining structures made of wood, steel or a combination of both, earthwork or reinforced concrete structures and combining it with engineering biological measures – planting trees and protection woods.

To reduce the impact of debris flow collecting basins and protective dams were designed and used where possible besides tunnels and protection galleries.

Protection fences, dams against rockfall started also early and were improved.

To prevent the formation, od mudslide or reduce creeping of slopes drainage or other stabilizing measures can be carried out.

**DESIGN**

*General technical specifics*

The measurement objectives pursued by the principles and rules of application are achieved if the assumptions mentioned in EN 1990 to EN 1999 apply.

- The general assumptions for EN 1990 are the selection of the load-bearing system and the structural design are carried out by appropriately qualified and experienced persons.
- The construction is carried out by trained and experienced personnel.
- Appropriate supervision and quality control during design and construction, i.e. in factories, production facilities and on the construction site are ensured.
- Building materials and products shall be used in accordance with the provisions of EN 1990 or EN 1991 to EN 1999 or the relevant design standards, material or product standards.
- The supporting structure is properly maintained.
- The supporting structure is used in accordance with the planning assumptions.

Additional assumptions may be made for special construction projects. In Austria these assumptions can be found in ONR 24800, 24801, 24802, 24803, 24805, 24806, 24810 for the

design, erection and maintenance of protection structures against avalanches, debris flow, rockfall. Here you can also find information on the return period and size of such an event.

Based on this the decision whether a temporary method or permanent method should be applied can be made.

In Austria exists mapping of risk zones of natural hazards at least for infrastructure and housing zones in valleys and surrounding areas, where the kind of hazard and risk zones may be found.

*Design procedure*

Based on risk analyses or experimental trials and simulations, regarding the assumptions concerning return period and size (loads) design of the structure and the foundation can be done according to the above-mentioned codes.

The most important item is the appropriate identification of the active zones, which demands a highly qualified engineering by experienced specialists. Based on this nowadays simulations help determining the impacts in the passive zones, which is needed for the design of dams, tunnels, galleries.

For temporary measures such as fixed mounted blasting devices – GASEX, Wyssen Avalanche Tower and others – which are erected in the avalanche opening zone foundation engineering is carried out according to EN 1997-1 Geotechnical design. Impacts on these structures and foundations are snow creeping and the impacts due to blasting.



Fig 1 – GASEX avalanche blaster

Permanent measures can be designed either as active working – in the opening zone of the hazard or as passive working close by the infrastructure to be protected. Depending on that the different impacts and load cases must be considered.

Active working devices are retaining structures in steel, wood, concrete or sometimes earthwork, stabilizing of slopes by technical means or drainage measures, rock walls. Active measures are located mostly in areas with difficult access and are therefore a big challenge for designer and contractor.

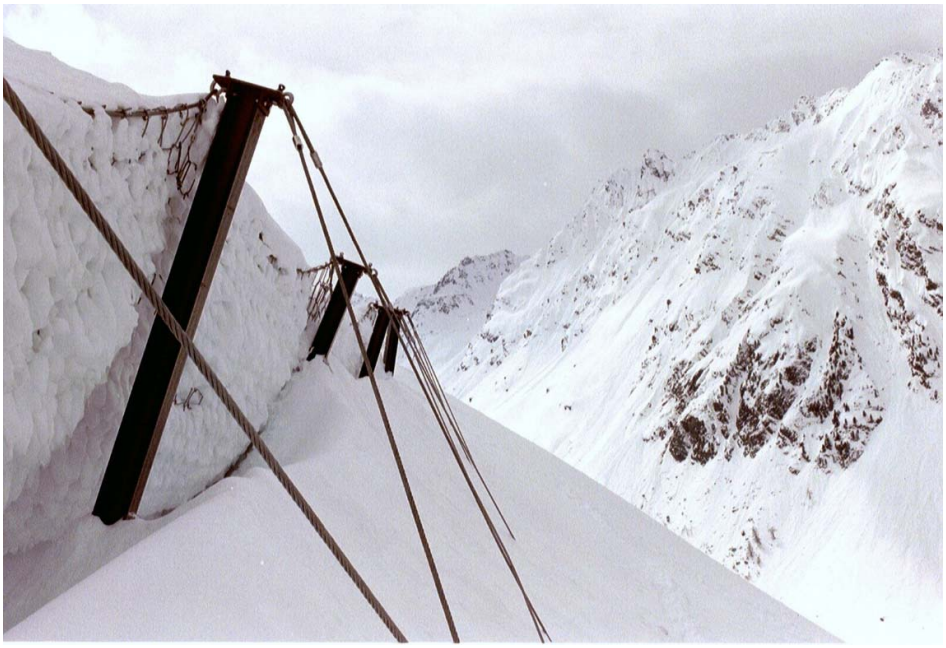


Fig 2 – retaining structure



Fig 3 – flexible avalanche retaining fence

Passive working devices such as protection dams, protection galleries must be designed for a high impact and are massive structures. They are usually permanent measures designed for a 100-year lifespan. To optimize protection of infrastructure it might be advisable to combine measures in active and passive zones. Challenges are foundations and structures as well due to the high static and dynamic loads. Due to restricted place deep foundations are the right choice for these structures.



Fig. 4 – protection gallery – rockfall hazard

Another challenge is the correct assumption of earth pressure on galleries situated in slopes – experience showed that the regular earth pressure model does not comply to the situation on site. In most cases sophisticated finite element modelling does not lead to a correct solution, when soil parameters are incorrect or missing or their interpretation is wrong. My advice would be to keep to conventional earth pressure calculations and then in a second step try to compare the result to results found by finite element or comparable methods.

## EXECUTION

As mentioned before there can be measures in the active zone or /and in the passive zone of natural hazards.

Active zone is often up in the mountains in steep environment, which is hard to access. Access roads do not exist or if there are several, they can used only b lightweight equipment for erection. An alternative is supply of equipment or building material by helicopter transport. To reduce material volumes deep foundations, micro piles, anchors or nails are the appropriate choice.

As foundation in mountain areas is in many cases the most challenging part some advice to execution.

Heavy machinery for execution cannot be brought to the construction site due to access problems, therefore the selection of flexible and lightweight equipment is advisable.

With hollow bar systems injection is the main item to keep eye on it, especially in a hot but also cold climate. Depending on available cement grade combined with compact soil or porous rock, it might be tricky to get them grouted properly. It might cause problems under cold conditions – temperatures below freezing, where the use of heated water and injection grout may be considered. Hot climate and high temperatures accelerate the setting process of injection grout and must be considered when deciding the appropriate cement grade.

Depending on soil conditions different methods of casting the drill cuttings may be used for hollow bars systems, - e.g. air, water – but the best outcome can be achieved by flushing with cement grout and continuous injection during drilling.

To improve bearing capacity by increasing friction it might help to use a special injection head and higher pressure, so that the diameter of the injection body may be increased.

With cased, bored micro piles it is easier to provide the correct diameter but depending on soil type – e.g. clay or silt - casing may limit side friction. Handling the casing might be difficult under extreme site conditions – e.g. steep terrain –and is time consuming and increases cost.

To improve side friction in clay or silt it might help to postgrout through a system of additional injection pipes. This works well also in cracked or porous rock.

If cracks are wide or in karst the use of "stockings" prevents loss of injection grout.

During drilling and injection in permafrost soil the induction of heat must be limited and on the other hand freezing of injection grout must be prevented. This can be achieved by use of specific mixtures and antifreeze agents.

Depending on site conditions and accessibility drill rigs can be mounted on different carriers as shown in the following pictures.




Fig. 5 – excavator mounted rig



Fig. 6 – crawler type vehicle mounted rig

In steep environment sledge mounted drilling rigs, which are secured by ropes or walking excavators equipped with hydraulic hammer, excavator shovel, drilling rig and appropriate air supply might make work easier in such places.

Fig. 5 and 6 show drilling rigs used with hollow bar system micro piles, Fig. 7 and 8 show the cased boring system for GEWI piles.



Fig. 7 – drilling rig on sledge unit



Fig. 8 – lightweight drilling rig on walking excavator

The passive zone and the planned protection measures are mostly situated in easier to access environment close by road or rail infrastructure therefore material supply and transport is not a big problem.

## EXAMPLES – CASE STUDIES

### 1. Rockfall protection of highway by protection fences

In western Austria a highway had to be protected from rockfall events and the decision for the solution was to erect protection fences and secure the slope beneath by stabilizing with nails and high-performance mesh.

Following a thorough rockfall simulation the appropriate height and energy class were determined, keeping in mind, that such a fence must be kept away from obstacles it should protect or infrastructure - road or rail – at least 3-5 meters, depending on energy class.

In the shown project the fence was situated 15 meter above the highway on top of a steep and weathered rock slope. To stabilize the slope a combination of nails and high-performance mesh was used.



Fig 9 – project area



Fig. 10 – rock slope cleaning before starting nails and mesh



Fig. 11 – finished protection – fence+mesh



Fig. 12 – rockfall protection works in active zone



Fig. 13 – drilling and mounting works

**2. reinforcement of protection gallery due to earth pressure problems**

Regular maintenance and survey of infrastructure constructions showed an increase of possible rockfall and besides that a deformation of a rockfall protection gallery along a highway. According to the damages and the deformation a n inappropriate earth pressure modelling was the reason for this damage and not an event by rockfall. As a new risk analysis showed a higher impact on the protection gallery as well the Austrian Highway Concessionaire decided not to dismantle the structure but to reinforce it keeping up traffic on the highway

The first step was to remove a part of the soil cover, which lead to repositioning to the initial position. The second step was to reinforce the tunnel profile by building an additional tunnel lining.



Fig. 14 – lining scaffold preparation

The old tunnel lining was cleaned, bonding anchors were set and afterwards the new lining was installed in two steps – reinforcing cage and then concrete works. Every five days another section was finished, so works could be terminated bay end of summer.



Fig. 15 – reinforcing the new lining

**3. avalanche protection by temporary and permanent measures in active zone**

To protect a highway passing a mountain zone with high average snowfall a project of combined temporary measures and permanent measures in the active zone – that is the zone, where avalanches are to start – was worked out and had to be finished within 6 months of possible time where the site was free of snow.

As temporary measure the erection of GASEX avalanche blasters was chosen, where a mixture of propane and air is ignited to disrupt the snowpack and cause smaller avalanches to release.

The system can be operated by remote control and each ignition can be verified.

The big advantage of this system is the reliable effectiveness so that closures of road and rail infrastructure can be limited to very short periods. It has been installed alongside several main roads and railway lines as well as in ski resort, to provide avalanche safety at a fair investment. In some cases, the system is run by a PPP-model covering the design, erection and operation as well.

In addition to the point by point protection for greater areas a permanent system of avalanche barriers can be installed in the active zone and may be combined with bioengineering measures – growing trees to form protective forests.



Fig. 16 – GASEX blaster in active avalanche zone



Fig. 17 – avalanche barriers in active zone



Fig. 18 – avalanche barriers and bioengineering – protective forest

In the passive zone the same measures as protection from rockfall are used – dams, reservoirs or tunnels and protective galleries.

## 5. Slope stabilization

At existing infrastructure often instable slopes caused by erosion problems of different reasons are a risk for operation and existence.

In many cases cuttings are done to steep without securing them – both rock slopes and soil slopes. If there are no severe impacts by heavy rain or other frost or heat, they seem to be stable, but long term they are a risk.

When slope stabilization is part of the design simpler solutions are effective and mostly cheaper than restoration afterwards. Stabilization by using nails and high-performance mesh combined with micropile systems or anchored slabs is state of the art and can be done nearly in any environment – steep slopes etc.



Fig. 19 – securing an instable slope at a railway line – nail and mesh



Fig. 20 – slope stabilization by anchored slabs along a highway




Fig. 21+22 – slope stabilization by micropile A-frame - drilling and concrete works



Fig. 23 – finished stabilization works

Stabilization also included drainage, discharge of surface water and bioengineering measures.

## CONCLUSIONS

The safety of infrastructure is not only important for operation without accidents and costly closure of infrastructure but as import for durability of all infrastructure.

When considered at the beginning of design in the correct manner and by experts dealing with natural hazards it is possible to implement a durable and safe infrastructure at reasonable costs. Experience showed that neglecting this fact leads to problems in durability, in operation and safety and long-term leads to higher investments. It must be clear that maintenance is an important part of a working infrastructure and must not be neglected.

References:


EN 1990 Eurocode – *Basis of structural design*

EN 1991-1 *Eurocode 1: Actions on structures*

EN 1992-2 *Eurocode 2 – Design of concrete structures*

EN 1993-1 *Eurocode 3 — Design of steel structures*

EN 1994-1-1 *Eurocode 4 – Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings*

EN 1995-1-1 *Eurocode 5 – Design of timber structures - .Part 1-1: Common rules and rules for buildings*

EN 1997-1 *Eurocode 7: Geotechnical design — Part 1: General rules (consolidated version)*

EN 1998-1 *Eurocode 8 – Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings*

EN 14199 *Execution of special geotechnical works — Micropiles*

GRUNDBAU TASCHENBUCH Teil 1, 2, 3 – Ernst&Sohn 2009 and 2018

DYWIDAG DSI – Approval documents GEWI+ and Hollow Bar IBO

ISCHEBECK – Approval documents hollow bar TITAN

ONR 24800 *Protection works for torrent control – Terms and their definitions as well as classifications*

ONR 24801 *Protection works for torrent control – Static and dynamic actions on structures*

ONR 24802 *Protection works for torrent control – Design of structures*

ONR 24803 *Protection works for torrent control – Operation, monitoring, maintenance*

ONR 24805 *Permanent technical avalanche protection – Terms, definitions, statical and dynamic load assumptions*

ONR 24806 *Permanent technical avalanche protection – Design of structures*

ONR 24810 *Technical protection against rockfall – Terms and definitions, effects of actions, design, monitoring and maintenance*

# INNOVATIVE TUNNELLING PRACTICES ADOPTED TO MITIGATE VARIOUS EXECUTION CHALLENGES IN A RAIL TUNNEL CONSTRUCTION IN INDIA

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## ABSTRACT

As a part of 115 Km long new railway line project between Obulavaripalle and Krishnapatnam Port stations, two tunnels of 0.96 Km and 6.62 Km long were constructed using NATM method. The tunnelling posed various execution challenges interms of geology and terrain. Preliminary geotechnical and geophysical investigations were carried out and modern construction management techniques were used. The biggest challenge was to carry out tunnelling for a length of about 500m through low overburden locations of 4m to 6m in a forest and hilly terrain. Designers recommended cut and cover method and alternative methods were explored to avoid delay of Project completion. An innovative method of tunnelling was conceptualised and successfully carried out by ground improvement externally and controlled blasting with additional support system internally. Extensive monitoring was done with the help of instrumentation and total station survey both inside of tunnel and on ground surface during tunnelling. Faster execution through low cover zones resulted in savings in cost and time. Consequently, both the tunnels could be completed in record time of 25 months duly sustaining forest ecology and environment. The successful execution of these tunnels despite various challenges is expected to motivate all Tunnel Engineers and agencies to execute tunnel projects faster.

**Keywords: tunnel, geology, ground improvement, instrumentation**

## INTRODUCTION

Tunneling is a highly specialized field requiring a blend of technical knowledge, field experience and engineering judgement (Indian Railways, 2012). Rail Vikas Nigam Limited, a company under Ministry of Railways was assigned the job of construction of a new 115 km long rail road at a cost of INR 1900 cr in the southern part of India in the state of Andhra Pradesh. The Railroad was envisaged to provide connectivity of Krishnapatnam Sea Port with hinterland. Part of the project was construction of two Tunnels of 0.96 Km and 6.62 Km long through a hilly terrain situated in a forest area at a cost INR 500 cr. A horseshoe shaped Tunnel for providing a single line track with 8m in width and 8m in height with an excavated face area of 56 $m^2$ and finished area 44 $m^2$ was planned. The Tunnel was also designed for Electric Loco Traction. New Austrian Tunneling Method (NATM) was adopted for Tunneling.

One of the major execution challenge was three low cover zones of 60m long in Tunnel 1, 180m and 240m long in Tunnel 2 which were having an overburden of about 4 to 6m. In addition, The tunnels were situated in a hilly and forest terrain. The designer recommended carrying out tunneling through these low cover zones by conventional cut and cover method. Few natural streams were flowing across tunnel alignment which would pose flooding threat in case of cut and cover excavation. The restricted availability of forest land, extra time required for obtaining additional forest land to dispose of spoil while carrying out open excavation for tunnel, problems associated like flooding during cut and cover etc., forced the construction team to explore alternative tunneling options through the low cover zones.

Previous projects of transportation tunnel projects of Konkan Railway, Pir Panjal Tunnel in Jammu and Kashmir etc took many years to complete the tunnelling due to various challenges like terrain, geological, hydrological challenges besides law and order issues. It is very essential to complete projects on time to reduce cost and time over runs and these two tunnels could be completed faster in 25 months by adopting innovative approaches and out of box thinking which would motivate tunnel engineers.


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## GEOLOGY

The Tunnels were situated on Eastern Ghats Province that consists of an intensely deformed and metamorphosed assemblage of phyllite and shales (Cumbum Formation), quartzite and shale (Bairenkonda Formation) as per the Detailed Design and Investigation report of Ayesa-WilburSmith (2012). The weak rock mass predominantly resulted in wedge failures. The joints at many locations were filled with clay making the face susceptible for failure due to slickenside. The rock masses were classified in to 5 groups depending on the Geology. Various support systems like rock bolting, lattice girders at suitable spacing and shotcrete with different thickness were recommended for each class of rock mass. At number of locations, seepage was observed to an extant of about 50L per minute. Plugging of leakage was resorted to during the tunneling besides carrying out pumping of water.

*Seepage control measures*

Seepage through tunnels occurs due to the pore water of soil or rock above the top of tunnel. The seepage problem was predominant where the height of over burden was more. However, it is very difficult to identify the source where seepage was emerging from. Geologists identified possible locations of aquifers. When plugging was done at one location, fresh seepage was noticed at other locations. Chemical compounds were tried to mitigate the seepage. However, it was noticed that it was very costlier proposition. As such, a novel method was tried and found to be successful and economical. The locations where seepage was noticed was pin pointed first. The holes or cavities were stuffed with cotton waste first which reduce the seepage and then further plugged with cement capsules. The cement capsule once set was found to be mitigating seepage quite effectively. The seepage was however slowly spreading to adjacent areas. During this time, shotcrete was applied on the top and side surfaces of the tunnel. This activity was continued at identified locations and mitigated seepage. After completion of tunneling, the entire tunnel was lined using impermeable membrane along with geotextile layer and side drain pipe mechanism making the entire length a dry tunnel.

## LOW COVER ZONES (C&C)

The height of tunnel is about 8m and the designer recommended a minimum overburden of two times the maximum dimension of Tunnel as suitable for normal tunneling ie., 16m. As such, wherever the over burden was less than 16m, it was designated as Cut and Cover zone (C&C) (see Fig. 1). There were three such low cover zones of 60m long in Tunnel 1, 180m and 240m long in Tunnel 2 with an overburden of about 4 to 6m.Open excavation was planned in these reaches, Tunnel is constructed with reinforce cement concrete for the horseshoe shape and back filling is done. Usually, cut and cover locations were tackled by excavating the ground up to the bottom of Tunnel with proper side slopes and later by completing concreting of tunnel profile followed by backfilling. Various challenges associated with cut and cover zones are discussed below.



**Fig. 1. Alignment showing low cover zone between hills**

*Forest Land Issues*

The Tunnels were aligned in the forest area and acquisition of forest land was a time consuming activity with an average lead time of about three to four years. The width of forest land available at the C&C locations was 32m. The available land was found to be inadequate to stack the excavated spoil and for stacking of materials during concreting and other operations. Additional forest land was found required to adopt cut and cover activity and the acquisition process would be delaying the Project completion besides increasing time overruns and cost overruns.

*Natural Streams across Tunnel Alignment*

Across the Tunnel alignment at C&C-2, one major stream was flowing during rainy season. Cut and cover operation during rains was expected to hamper the progress as flood water can enter tunnel and cause havoc besides delaying completion.

## INNOVATIVE METHODOLOGY

In view of the constraints and challenges mentioned above, it was decided to carry out tunneling carefully even at the C&C zones in lieu of open cut and cover. It was contemplated to improve the strength properties of overburden ground mass inorder to mitigate the possibility of either caving in or settlement of overburden mass. Various methodologies were explored so that the properties of overburden rock mass were modified which would also help increase standup time by improving its tensile and shear strength properties. Besides the methodology was also expected to take care of any possible squeezing of rock mass since squeezing behavior in phyllite and clay-shales is common due to deformability and poor strength properties (Barla et al., 2008). It was judiciously decided to improve the overburden mass strength properties by ground improvement techniques externally and by providing additional support system to tunnel internally.

### GROUTING AND MIRCO PILING

*Grouting*

Grouting was effectively used at various places to stabilize landslides and improve ground properties. The effect of grouting is to displace water from fissures and pores in the ground and which were filled with grout under pressure. It was opined that grouting would suit the situation to fill the fissures and joints in rock mass. Chemical grouting was not recommended due to cost constraints and Cement grouting was selected as ideal to suit the site conditions. The Grout used on this project consisted of 1 portion of Ordinary Portland Cement 53 grade, 1 portion of Micro fine Cement and 2 portions of water by weight. The grout was mixed and pumped in to the 100mm dia. holes at a pressure of 3 bar.

*Micro Piles*

Abramson et al., (2002) demonstrated after conducting laboratory and field tests that group and network effect of micro pile system provided higher load bearing and shearing capacities. It was opined that Micro Piles were best suiting the site conditions to improve tensile and shear strength properties of overburden rock mass. Micro piles were also expected to pass through various layers of rock mass and would knit them as one soil mass similar to the rock bolt. However, mostly rock bolts were used in the arch portion of tunnel and for stabilizing side slopes. Micro piles perfectly suited for this top down knitting of overburden mass. A steel 'L' shape rebar of 25mm dia, 6m to 9m long having yield strength of 500 MPa was used as micro pile by inserting the rebar in the grout holes alternatively to strengthen the overburden mass.

*Methodology*

The top 2m of overburden contained Colluvial mass due to running of streams over years underlain by weathered rock mass. The spacing of grout hole/micro pile was maintained at 1.5m c/c above the tunnel portion and 2m c/c beyond up to available right of way of about 12m. (see Fig. 2). The grout was pumped in to the 100mm dia. holes at a pressure of 3 bar. With the use of Packers (see Fig. 3), the permeation improved by 200%. On alternate grout holes, a steel 'L' shape rebar of 25mm dia and 6m to 9m long was inserted to act as a micro pile (see Fig. 3). A total of 3000 nos. of grout holes and 3000 nos. of micro piles were carried out to stabilize the overburden mass from the surface before commencing tunneling.

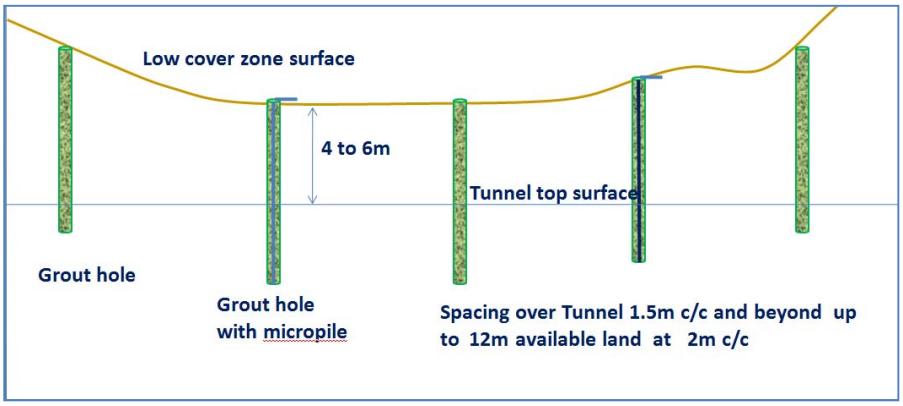



**Fig. 2. Layout of grout holes and micro piles**



**Fig. 3. Grouting with packer and mircro pile in the grout hole**

## PROBLEM OF NATRUAL STREAMS ACROSS ALIGNMENT

There were four natural streams of minor in nature and one major stream at the C&C zones. The major stream was susceptible for caving-in due to seepage, presence of boulders and colluvial soils, less overburden. With a view to preventing seepage, the bed of the minor natural streams was cement grouted which helped reduce seepage. The major stream was anticipated to be posing serious challenge during rains causing flooding of tunnel. It was apparent that the entire tunnel could be flooded in case of any eventuality of caving-in failure during tunneling. A Reinforced Concrete (RCC) apron was planned at the major stream passing in C&C 2 (see Fig. 4). This ensured a smooth flow of stream over the apron besides mitigating seepage. The apron was designed as a self supporting structure so that it would remain intact even if there was any caving in failure of soil or rock below the apron during tunneling. All these activities were completed well ahead of tunneling. During tunneling through cut and cover and particularly under the major stream area there was no kind of failure or seepage.



**Fig. 4. A major stream in C&C 2 area before and after apron**

## INTERNAL SUPPORT SYSTEM WHILE TUNNELLING

The internal support system consisted of face stabilization, provision of lattice girders, spiling and shotcrete with mesh, rock bolts (see Fig. 5) as explained below:

*Face Stabilization*

It was decided to carry out face stabilization by carrying out face sealing with shotcrete, and cement grouting of face whenever the face was found to be weak and containing many foliations. The RMR and Q values of rock mass were continuously monitored and the face was stabilized as required.

*Lattice Girders*

Lattice girders at a spacing of 1m were planned to be installed immediately after the blasting and de-mucking to support the roof.

*Spiling*

Self Drilling (SD) bolts of 32mm diameter and 9m long with an overlap of 3m were planned to be inserted to act like pipe roofing besides grouting the periphery with cement grout through SD bolts. This was expected to stabilize the rock mass near top surface and also would help in preventing collapse or over break.

*Shotcrete with wire mesh fixing*

As a temporary support system, one first shotcrete layer was applied followed by fixing wire mesh along the ceiling and further shotcrete layers up to 200mm total thickness were applied.

*Rock bolts*

Additionally rock bolts of 25mm dia. of 3m long were planned inserted at ceiling to stabilize the Tunnel. Resin capsules and cement capsules were used in the hole driven to anchor the rock bolts.

*Face Blasting Operation*

The pull of normal Tunnel face was about 3m by charging 110 nos of holes with 3200 N of emulsion explosives triggered by Non Electric Detonators (NED). The C&C Tunnel locations were meticulously handled by reducing the pull to 1m and by charging 80 holes with 800 N of explosives. 25 no. of dummy holes along periphery were ensured to reduce over break and impact of blast above the roof.

Further, for C&C 1 for all cycles of blasts, face sealing was done since the unconfined compressive strength of rock mass was 10 Mpa vis-à-vis 20 Mpa of C&C 2 & 3. Holes were drilled on the face and then grouted and it was ensured that grout emits from adjacent holes. The internal support system and careful blasting operations helped the tunneling safely and successfully through the low cover zone.



**Fig. 5. Internal support system with lattice girder, wire mesh and shotcrete**

## INSTRUMENTATION

As a part of NATM, extensive instrumentation was done in the entire Tunnel with an extra care in the C&C locations. The following Table 1 gives the details of various instrumentation done at C&C Zone with a comparison with normal tunnel. The surface levels were monitored with total station and fixed targets.

**Table 1. Comparison of Instrumentation**

| Type of Monitoring | Tunnel Normal Location | C&C Zone |
|---|---|---|
| Convergence Anchors | @50m | @ 20m |
| Shotcrete Stress Cells | @ 150m | @ 30m |
| Bi-Reflex Targets | Nil | @ 10m |

*Analysis of instrumentation results*

The results of the instrumentation are shown below (see Fig. 6 and 7) for a particular period so as to give the concept of how monitoring was carried out:

The shotcrete stress cells recorded a variation in stress level of less than 0.02 MPa. Similarly, the results of all stress cells was analyzed periodically and cross verified with other monitoring tools like surface levels etc. In the main tunnel portion, Multi Point Borehole Extenso Meters (MPBX) were used to monitor the movements at a distance of 5m, 10m and 15m.

The Convergence Anchors showed a maximum movement of about 0.17mm which was nominal. The convergence anchors were observed to be quick and tangible way of monitoring movement of surface. As shown in inset of Fig. 7, the special tape was used to measure the length of LS to RS, RS to Crown and Crown to LS periodically and compared.

The Surface levels were daily monitored and a maximum settlement of 9mm was observed. Reflective targets inside tunnel on the roof were fixed and monitored continually, which helped safe tunneling.

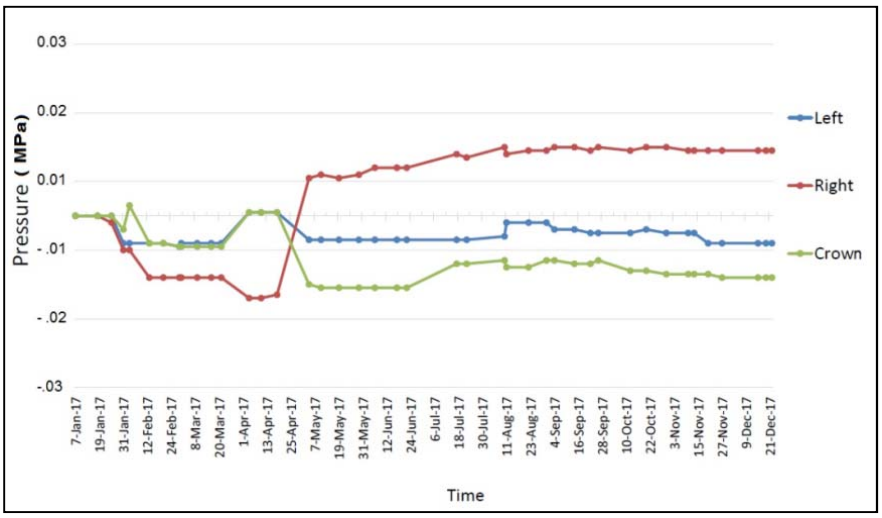



**Fig. 6. Observed values of shotcrete cells**

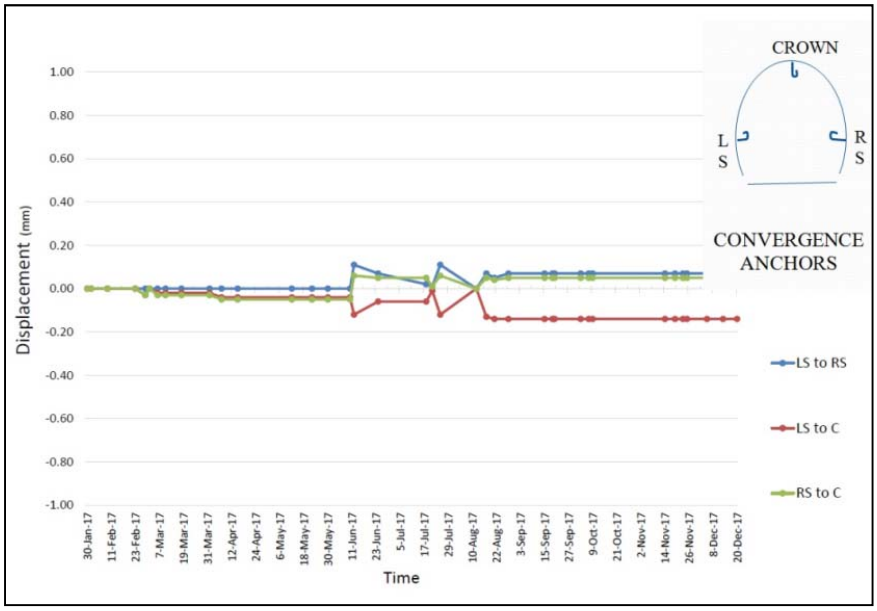



**Fig. 7. Observed values of convergence anchors**

## COST BENEFITS

The cost of grout hole was INR 1800/m and cost of micro pile was INR 2000/m. The entire cost of completing Tunneling in all the C&C zones was INR 10 cr. The assessed cost by carrying out traditional cut and cover in these 3 locations was INR 25 cr plus cost of obtaining additional forest land. The time of completion of the said locations was 5 months. The time that would have taken for cut and cover method was assessed to be 17 months thus there was saving of about one year. Besides, cut and cover operations could not be commenced until additional forest land was acquired.

## CONCLUSIONS

Usually, cut and cover method was being adopted in low cover zones of any tunneling project throughout the globe though it carries risks of slope failures during construction or flooding during rains. The case study highlighted an alternative method to cut and cover. The methodology adopted in this project envisaged tunneling in lieu of cut and cover by strengthening the over burden mass with cement grout holes and micro piling. The ground improvement technique adopted not only improved shear strength of the overburden mass but also helped mitigate the seepage. The streams present on the surface were well contained by suitable grouting or apron thus ruling out the possibility of caving-in failure during tunneling or seepage.

Tunneling was carried out by reducing pull to 1m per each blast cycle duly providing required internal supports like rock bolts, shotcrete, lattice girders, splicing and face grouting. All these precautions made successful tunneling possible inspite of various challenges without resorting to cut and cover method. Extensive monitoring of tunnel site from surface and inside tunnel through instrumentation helped safe execution of tunneling. Finally, the C&C 1, 2 and 3 were completed successfully in a period of 5 months including surface treatment and tunneling. Final RCC lining was carried out in C&C locations as a secondary support system.

The innovative method resulted in considerable savings in construction costs to a tune of approximately INR 15 cr besides huge savings in time. The success instilled confidence in the tunnel engineers at site and expected to motivate tunnel engineers to help complete tunnels faster in the times to come. This method can be certainly adopted in similar challenging locations with proper design and care.

## REFERENCES


1. Indian Railways., Guide Lines for Design and Construction of Tunnels, RDSO, Lucknow, India (2012).
2. Ayesa-WilburSmith., Detailed Design and Investigation Report of 2 Tunnels on Obulavaripalle and Krishnapatnam New Railway Line Project, India (2012).
3. Barla,G. and Barla, M., Innovative Tunnelling Construction Methods in Squeezing Rock, Italy (2008).
4. Abramson, W. L., Lee, S. T., Sharma, S., and Boyce, M.G.. Slope Stability and Stabilization Methods. Second Edition, John Wiley & Sons, Inc., New York (2002).

# LATERAL RESPONSE OF ROCK SOCKETED SINGLE PILE DUE TO THE EFFECT OF OVERLYING SAND LAYER

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## ABSTRACT

This paper examines the effect of top sandy soil on the response of piles to investigate the adequate socket length needed for rock socketed piles subjected to a lateral load. The performance of rock socketed piles has been investigated by finite element based numerical analysis using PLAXIS 3D software. In this analysis, the concrete pile behaviour is assumed as linear elastic. The sand layer is treated as an elastic-perfectly plastic material based on hardening soil model. The behaviour of rock is simulated by assuming an elastic-perfectly plastic material defined by Hoek-Brown model. The lateral responses of single pile are analyzed by varying the rock socket lengths ($L_r$) from 0 to 4 times the pile diameter ($D_p$). Parametric studies are performed by varying the ratio of soil embedment length ($L_s$) to relative stiffness (T) values ($L_s$/T). The limiting rock socket length ($L_r$) is proposed with respect to pile displacement criteria for various $L_s$/T ratios corresponds to the ground with top loose sandy soil underlying a moderately weathered rock with weak strength. The maximum beneficial rock socket length ($L_r$) for the ground conditions used in this paper is found to twice the pile diameter ($D_p$).

**Keywords: pile, lateral load, soil embedment length ($L_s$), rock socket length ($L_r$) and relative stiffness (T)**

## INTRODUCTION

Pile foundations are normally socketed in rock layers mainly to increase the resistance against lateral loads. A pile subjected to lateral load in a homogeneous sandy soil itself is a complex phenomenon. It gets further complicated in case if the pile embedded in the ground with top layers of sand and bottom of the pile is socketed into the rock mass. The response of laterally loaded pile socketed into the rock primarily depends on the soil embedment length ($L_s$) and relative stiffness between pile and soil (T) as mentioned in IS 2911. As per IS 14593: 1998 the rock socket length of at least one to four times the pile diameter is to be considered based on the broad type of rock mass. Sometimes, it is great difficult to socket the pile into the rock to such a longer depth which may involve considerable time and expenses. Also, the code does not speak anything about the rock socketing length with respect to the type of pile loadings.

Very few researchers have reported on the techniques developed for laterally loaded rock socketed piles. Among them, Zhou et al (2018) evaluated the effect of Geological Strength Index (GSI), axial compressive strength, Hoek-Brown constant on the bedrock properties and p-y curves. The pile behaviour with respect to GSI shows that the maximum bending moment, shearing force and lateral deformation of pile decreases with increasing GSI. Singh et al (2017) reported the lateral load-deflection behaviour of rock-socketed pile under the combined loading has affected by both socket length and soil cover depth. Muthukumaran and Prakash (2017) conducted experimental studies on rock socketed piles in non-homogeneous sand overlying rock. It was reported that the effect of socketing is less significant in case of the pile with minimum free standing height. Karthigeyan and Rajagopal (2012) studied the response of rock socketed piles by varying the rock socket length and by keeping the top sand layer depths constant. It was observed that the bending moment on the pile section becomes unaltered for a rock socketing depth beyond two times the width of the pile. Reese (1997) establishes the non-linear relation between ultimate soil resistances with deflection and includes reduction factors for secondary structures in terms of RQD.


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The reduction in bending stiffness (EI) and increase in deflection because of load increment have less influence over bending moment. Deflection may also occur to secondary structures in rock with weak fillings. Carter and Kulhawy (1992) proposed a method to predict ultimate lateral load carrying capacity of rock socketed shaft from p-y curves for allowing non-linearity along the length of the shaft. Radhakrishnan and Leung (1989) investigated the load transfer behaviour of rock socketed piles in highly weathered sedimentary rock found very close to ground surface having numerous closely spaced joints and no major discontinuities. The behaviour of pile was elastic under normal working loads and less shaft friction was developed along the upper part of the shaft due to the presence of rigid pile cap. The design engineers dealing with larger lateral loads on piles are interested to know the optimum lengths up to which the pile can be embedded into the rock to achieve the desired lateral resistance. However, a very few researchers have addressed on the effect of rock socket length by considering the top soil depth ($L_s$) on the lateral response of rock socketed piles.

In view of the above, this paper presents and discusses the numerical results obtained from a 3D finite element analysis using PLAXIS 3D software on rock socketed piles under lateral load. The numerical analyses have been carried out to investigate the behaviour of piles socketed into a moderately weathered rock with various $L_s/T$ ratios by considering the influence of top homogeneous loose sandy soil. The limiting rock socketing length within which maximum beneficial effect can be achieved for piles under lateral loads are discussed.

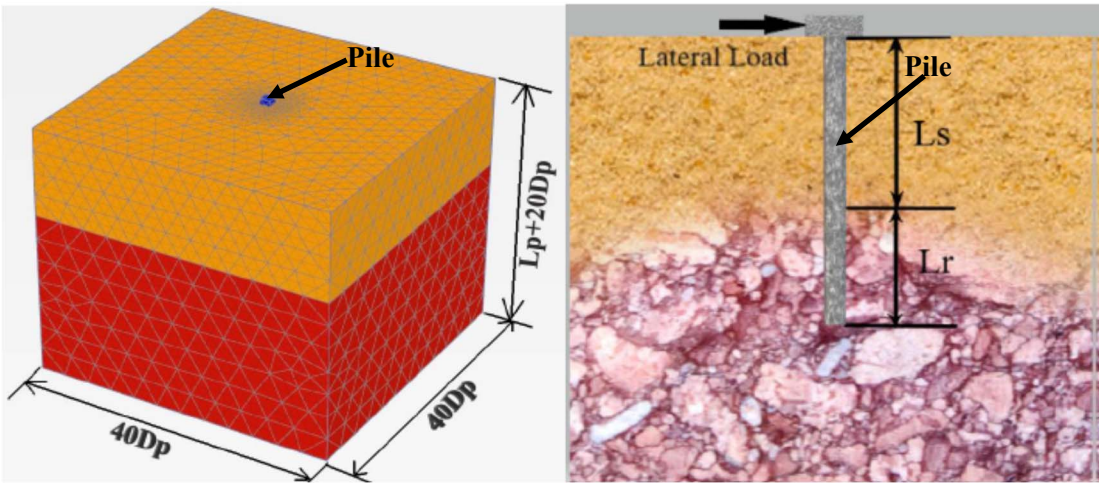



**Fig. 1. 3D Finite element mesh with pile – soil rock profile**

## NUMERICAL MODELLING DETAILS

The present analyses have been carried out using the finite element software PLAXIS 3D to predict the response of piles installed through the top uniform homogenous loose sandy soil layer followed by a moderately weathered rock with weak strength under lateral loading as shown in Figure 1. In the analysis, the soil cover depth ($L_s$) is varied in terms of the ratio $L_s/T$ = 0.75, 1.5, 2.25 and 3. The soil rock profile with pile is modelled without any boundary effect as shown in Figure 1. The rock socket length of pile ($L_r$) is varied from 0 to 4 times the pile diameter ($D_p$). The quadratic tetrahedral 10-node elements are used to simulate the continuum of soil and pile elements in the model. The interface between the pile-soil as well as pile-rock has been modelled using 12-node element of least thickness. Figure 1 shows a typical finite element mesh consisting of approximately 22,450 elements and 99,000 nodes used for the pile analysis. The mesh has been discretized finely around the pile to account for the stress gradient near the pile-soil and pile-rock interface. The pile has been considered as linear elastic material, the homogenous

soil is considered as elastic perfectly plastic material based on Hardening soil model and the rock have been treated as elastic perfectly plastic material based on Hoek-Brown model.

## PARAMETRIC STUDIES

Series of three-dimensional analyses have been carried out on rock socketed piles under lateral load using PLAXIS 3D software. The top soil layer is chosen as loose sand and the soil depth has been varied in the analysis. Similarly, the moderately weathered rock with weak strength is modelled by Hoek-Brown model parameters. The circular pile of diameter 600 mm with pile cap is modelled as concrete pile of $M_{25}$ grade. The parameters of pile, soil and rock considered for analyses are listed in Table 1. The analyses are carried out in three phases. In the initial phase, the stresses in the soil and rock are calculated by performing gravity analysis. In phase I, the pile installation and interfaces are simulated. The displacements are reset to zero at the end of the initial phase and Phase I. In the last phase i.e., Phase II, Lateral load is applied on the pile head.

**Table 1. Physical properties of pile, sand and rock**

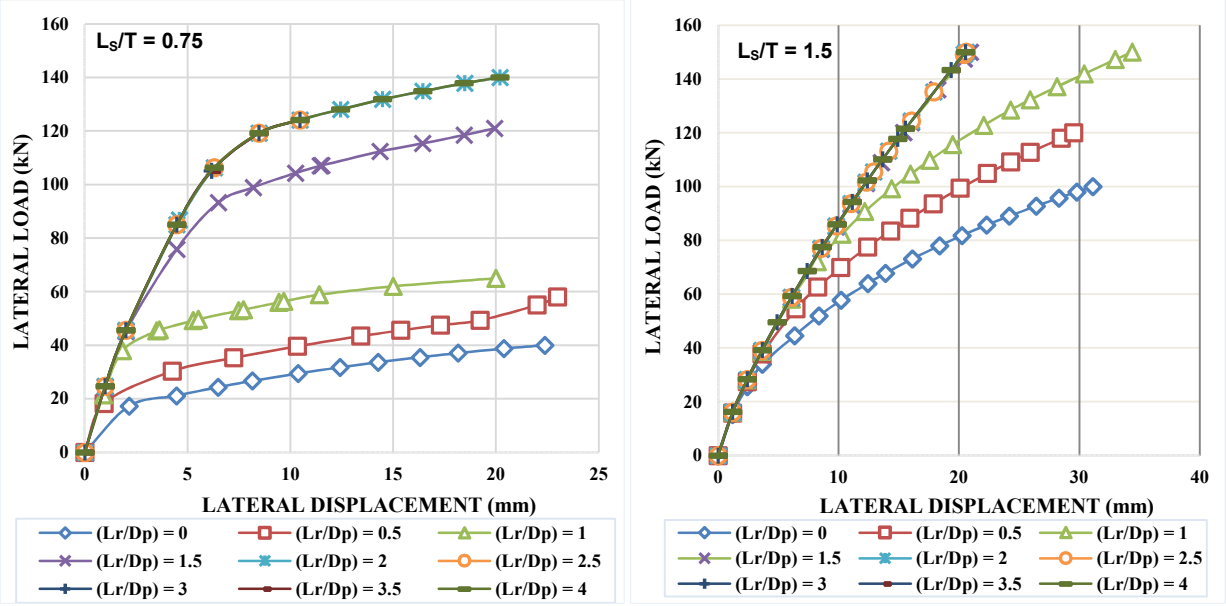
| Parameter | Name with Unit | Loose Sand | Circular concrete pile | Moderately weathered weak rock |
|---|---|---|---|---|
| Relative density | % | 15 – 35 | - | - |
| Model | - | Hardening soil model | Linear Elastic model | Hoek-Brown model |
| Behaviour | Type | Drained | Non-porous | Non-porous |
| Unit weight | $\gamma$ (kN/m$^3$) | 16 | 24 | 21 |
| Friction angle | $\phi$ (°) | 30 | - | 35 |
| Young's modulus | E (kN/m$^2$)<br>Eoed<br>$E_{50}$<br>$E_{ur}$ | <br>20e3<br>30e3<br>90e3 | 25e6<br><br>-<br> | 70e5<br><br>-<br> |
| Uniaxial Compressive Strength | $\sigma_{ci}$ (kN/m$^2$) | - | - | 15e3 |
| Poisson's ratio | $\mu$ | 0.3 | 0.15 | 0.24 |
| Dilatancy angle | $\Psi$ (°) | - | - | - |
| Geological Strength Index | GSI | - | - | 35 |
| Hoek-Brown constant | $m_i$ | - | - | 0.7 |

## RESULTS AND DISCISSION

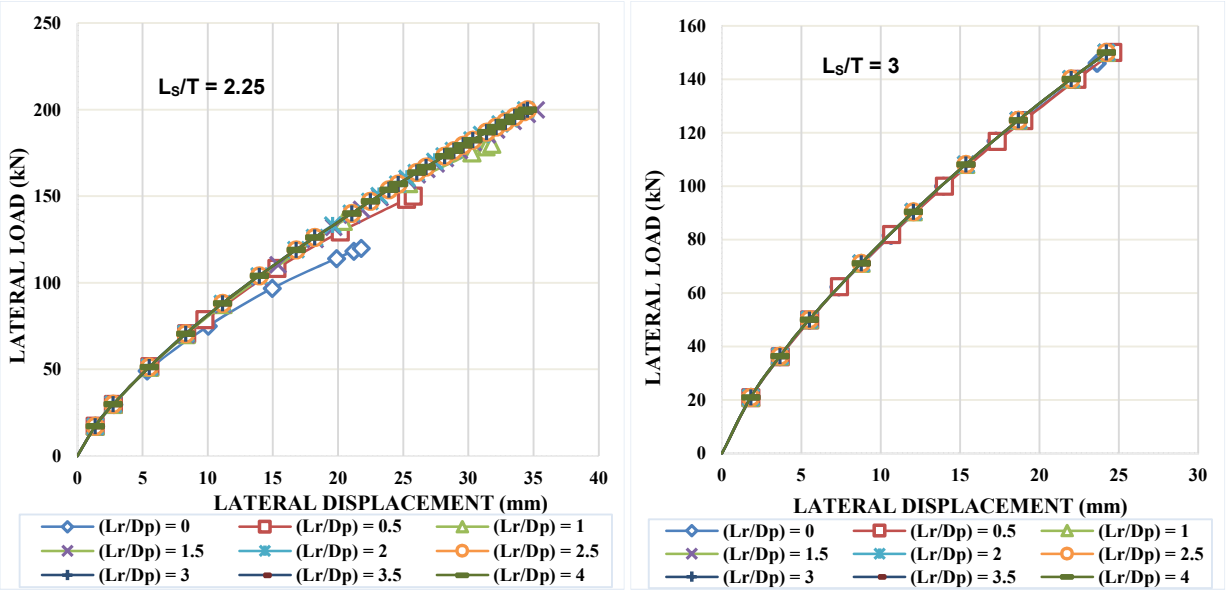
### *Lateral load – lateral displacement response of rock socketed pile*

Figures 2(a) – 2(d) shows the lateral load – lateral displacement curves of rock socketed piles with respect to varying $L_r/D_p$ ratios for $L_s/T$ values of 0.75, 1.5, 2.25 and 3 respectively. It can be seen from the figures that the lateral capacity of rock socketed piles for a given lateral displacement is increased with increase

$L_r/D_p$ ratios. However, this trend is more significant for piles with $L_s/T$ ratios of 0.75 and 1.5 and shows that the effect of $L_r/D_p$ ratio is less significant for $L_s/T$ ratios more than 1.5. Further, it can be seen that the lateral capacity of piles for a given lateral displacement is lesser for $L_s/T$ ratio of 3 as compared with $L_s/T$ ratios of 0.75, 1.5 and 2.25. This indicates that the effects of $L_s/T$ ratios are dominating beyond the $L_s/T$ ratio of 2.25. It is mainly because of an increase in thickness of overlying soil layer. Figure 2(d) shows the lateral capacity of rock socketed piles for $L_s/T$ = 3 corresponding to any given lateral displacement by varying $L_r/D_p$ ratios is not influenced with an increase in socket length. It is further observed that the non-linear nature of lateral load – lateral displacement curves becomes linear as an increasing the rock socket length.



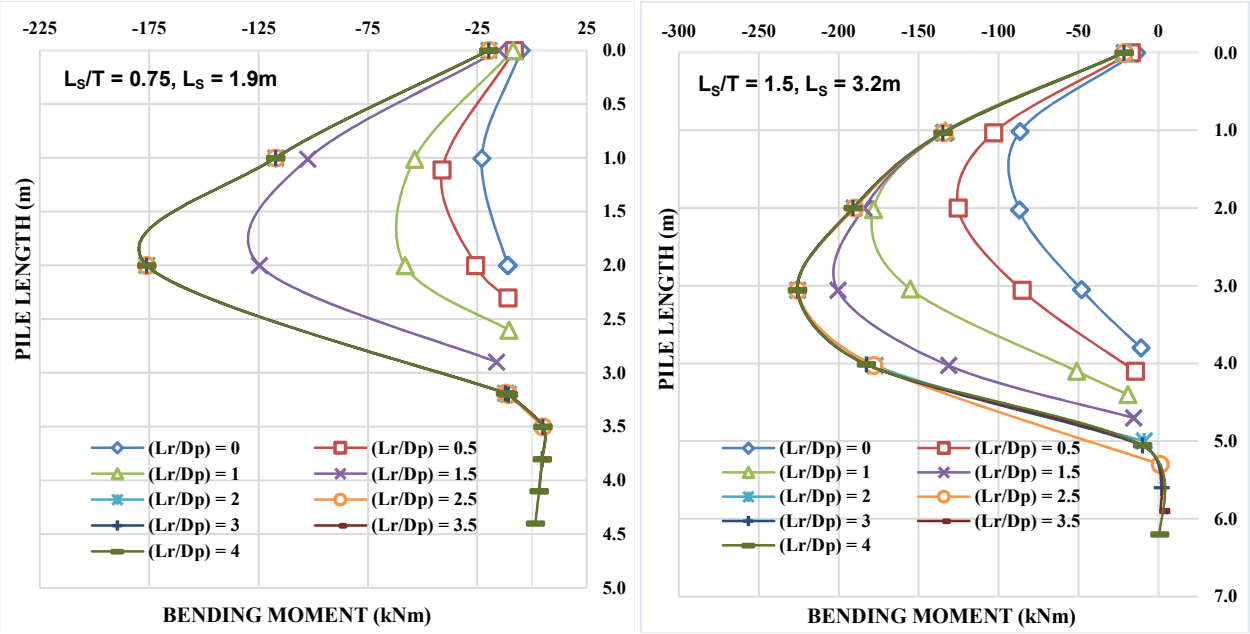

**Figs. 2(a) and 2(b) Lateral load vs lateral displacement curve of piles for $L_s/T$ = 0.75 and 1.5**




**Figs. 2(c) and 2(d) Lateral load vs lateral displacement curves of pile for $L_s/T$ = 2.25 and 3**

***Bending moment variation along the length of rock socketed pile***

Bending moment along the length of rock socketed piles is plotted with respect to varying $L_r/D_p$ ratios for $L_s/T$ varies from 0.75 to 3. Figures 3(a) – 3(d) show the bending moment along the pile length with respect to varying $L_r/D_p$ ratios pertaining to $L_s/T$ ratios of 0.75, 1.5, 2.25 and 3 respectively. In general, the maximum bending moment on the pile section is increasing with increase in $L_r/D_p$ ratios up to 2 and the effect of $L_r/D_p$ ratios are almost nil in case of the $L_r/D_p$ ratios more than this limit. These indicate that the effect of rock socketing length is not significant for $L_r/D_p$ ratio more than 2. Karthigeyan and Rajagopal (2012) also stated that the bending moment in the square pile section are more or less the same for pile socket length beyond 2B.




**Figs. 3(a) and 3(b) Bending moment along the pile length for $L_s/T$ = 0.75 and 1.5**

The depth at which maximum bending moment occurred on the pile is closer to the interface between soil and rock layer with increasing the rock socket length for the ground profile having $L_s/T$ corresponds to 0.75 and 1.5 as shown in Figs 3(a) and 3(b). This indicates that the resistance offered by the rock layer is higher against the lateral load. The depth at which a maximum bending moment occurred on the pile section is within the soil layer for the ground profile having $L_s/T$ ratios corresponds to 2.25 and 3 as shown in Figs 3(c) and 3(d). This shows that the bending moment pattern of such piles is almost similar to the bending behaviour of a long pile in uniform soil. Figure 3(d) shows the bending moment on the pile section is not influenced by increasing the socket length for $L_s/T$ = 3, which indicates that the lateral resistance is contributed only by soil layer and lateral resistance offered by the rock is nil.

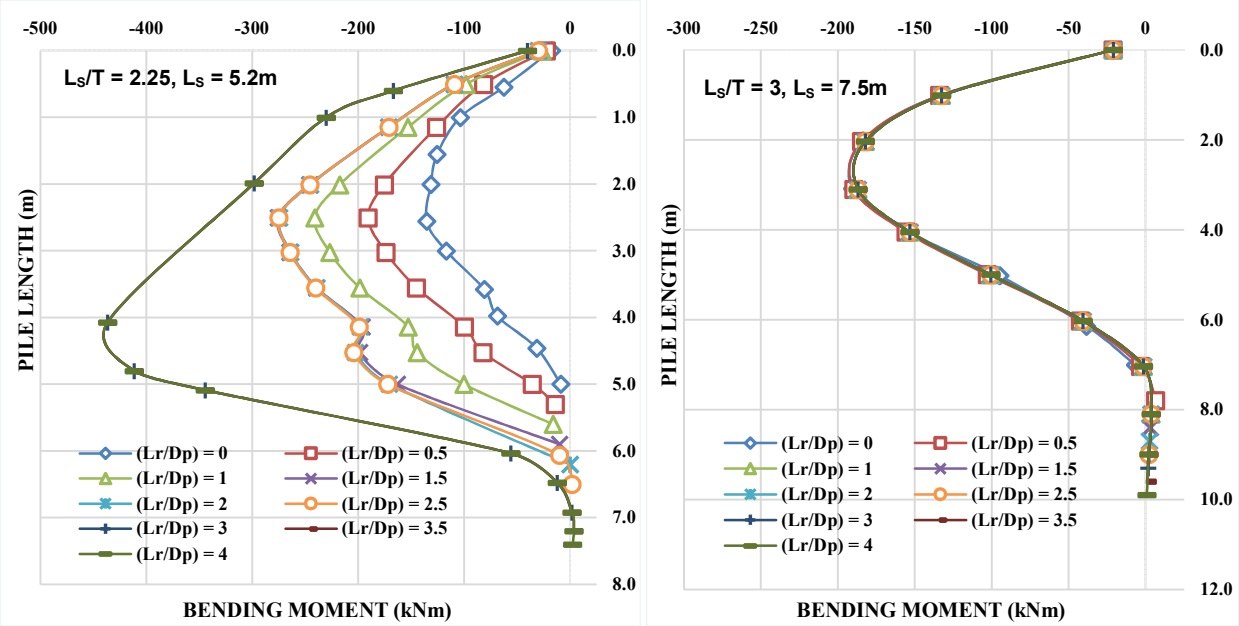



**Fig. 3(c) and 3(d) Bending moment along the pile length for $L_s/T$ = 2.25 and 3**

***Rock socket length***

In order to predict the effect of rock socketing length on the lateral load capacity of piles are calculated corresponds to a lateral displacement of 5 mm, 10 mm and 15 mm with respect to different $L_r/D_p$ ratios by varying the $L_s/T$ ratios from 0.75 to 3. Figures 4(a) – 4(d) show the effect of rock socket length on the lateral load capacities of rock socketed piles pertaining to $L_s/T$ ratios of 0.75, 1.5, 2.25 and 3 respectively. It can be seen from the figures that the lateral capacity of piles are increasing with an increase in rock socket length up to the limiting values and beyond which it become constant. This trend is common for all lateral displacement levels of 5 mm, 10 mm and 15 mm and also in case of the $L_s/T$ ratios of 0.75, 1.5 and 2.25. It is further observed from the figures that the influence of rock socket length starts decreasing with increasing $L_s/T$ ratio and the effect is nil for the $L_s/T$ ratio of 3.

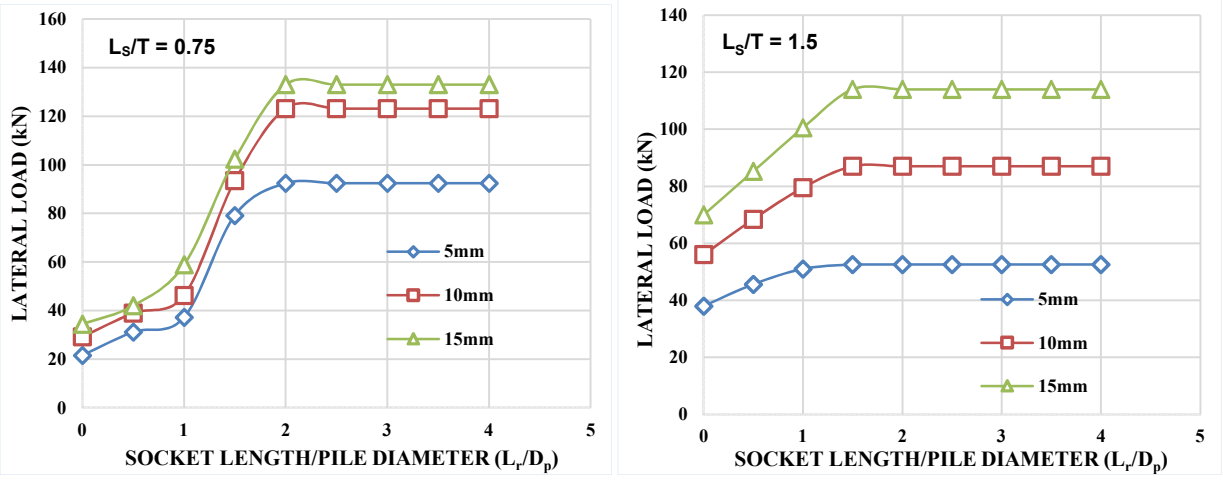



**Fig. 4(a) and 4(b) Lateral load vs rock socket length of piles $L_s/T$ = 0.75 and 1.5**

The limiting rock socket length tends to reduce with increasing $L_s/T$ ratio and the effect of rock socketing become insignificant for the ground profile having $L_s/T$ ratio greater than 2.25 as shown in Figure 4(d). The effective rock socket length for the piles installed in top loose sand followed by a moderately weathered rock is found to ranging from $1.5D_p$ to $2D_p$ in case of the $L_s/T$ ratios between 0.75 and 1.5. It is found to vary from $0D_p$ to $1D_p$ in case of the $L_s/T$ ratios between 2.25 and 3.0.

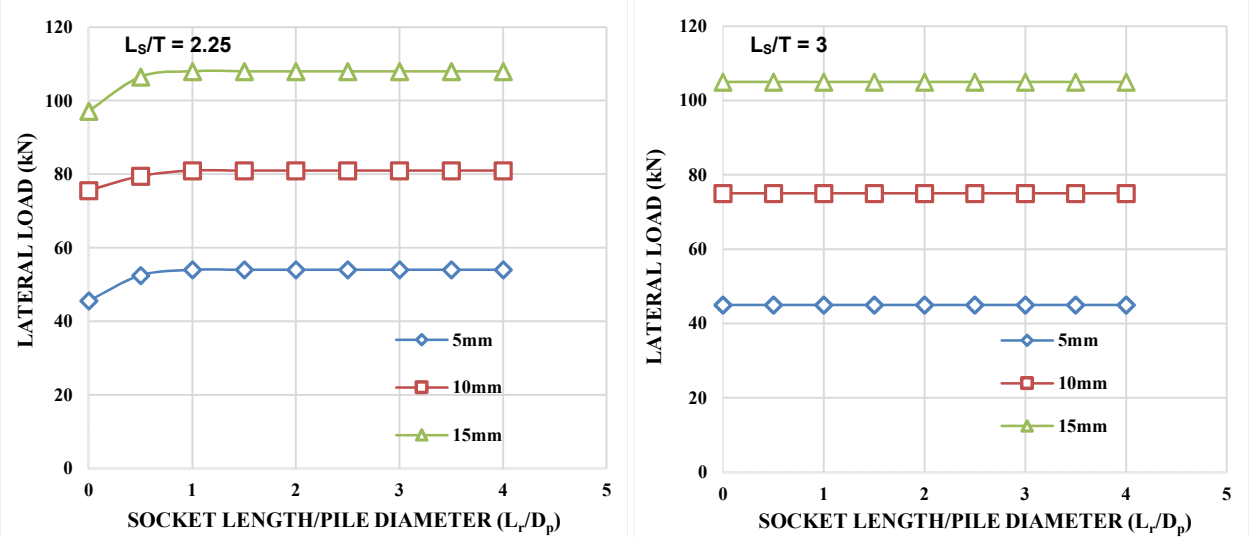



**Fig. 4(c) and 4(d) Lateral load vs rock socket length of piles for $L_s/T$ = 2.25 and 3**

## CONCLUSIONS

The following conculsions are drawn from the results obtained based on a detailed numerical studies carried out for rock socketed piles embedded in the ground profile with top sandy soil followed by a moderately weathered rock under lateral load.

The lateral displacement of rock socketed pile is decreasing with increase in rock socket length. The lateral displacement of pile increases with increase in $L_s/T$ ratio. The bending moment along the pile length is influenced by soil cover depth ($L_s$). The depth of fixity is likely to occur almost near the soil-rock interface, and it tends to move above the interface as $L_s/T$ ratio increases. The influence of rock socket length on the lateral capacity of rock socketed piles is decreasing with increase in $L_s/T$ ratio. The limiting rock socket length is mainly governed by $L_s/T$ ratio and the effect of rock socketing is significant only for $L_s/T$ ratios less than 3. This gives an indication that it is not essential all the time to embed the pile into rock for a standard length and even a small length of socketing of around $0D_p$ to $2D_p$ is sufficient to cater the maximum lateral resistance.

The results presented in the paper are based on full 3D numerical analyses with a set of parameters used in the study. Therefore, it needs further verification using different soil-rock profiles with varying pile diameters for making generalized conclusions.

## ACKNOWLEDGEMENT

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## REFERENCES


Carter, J.P., and Kulhawy, F.H., 1992. Analysis of laterally loaded shafts in rock. Journal of Geotechnical Engineering, Vol. 118, No. 6, pp. 839-855.

Hoek, E., 1983. Strength of jointed rock masses. 23rd Rankine Lecture, Geotechnique, vol. 33, pp. 187-223.

Hoek, E., and Brown, E.T., 1997. Practical estimates of rock mass strength. International Journal of Rock Mechanics and Mining Science, vol. 34(8), pp. 1165-1186.

Hoek, E., Carter, T.G., and Diederichs, M.S., 2013. Quantification of the Geological Strength Index Chart. The 47th US Rock Mechanics/Geomechanics Symposium held in San Francisco, CA, USA.

IS 2911, (Part 1/Sec 2): 2010. Indian standard code of practice on design and construction of pile foundations, part 1: concrete piles, section 2: bored cast in-situ concrete piles, Bureau of Indian Standards, New Delhi, India.

IS 14593, 1998. Indian standard code of practice on design and construction of bored cast-in situ piles founded on rocks-guidelines. Bureau of Indian Standards, New Delhi, India.

Karthigeyan, S., and Rajagopal, K., 2012. Influence of rock socketing on the lateral response of single pile. Indian Geotechnical Journal, Vol. 42, N0. 1, pp. 49-55.

Muthukkumaran, K., and Prakash, A.R., 2016. Behaviour of laterally loaded rock socketed pile in multi-layered soil-rock profile. International Workshop on Geotechnics for Japanese Geotechnical Society Special Publication, Vol. 3, No. 2, pp. 51-55.

Radhakrishnan, R., and Leung, C.F., 1989. Load Transfer Behaviour of Rock-Socketed Piles. Journal of Geotechnical Engineering, Vol. 115, No. 6, pp. 755-768.

Reese, L.C., 1997. Analysis of laterally loaded piles in weak rock. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, No. 11, pp. 1010–1017.

Singh, A.P., Bhandari, T., Ayothiraman, R., and Rao, K.S., 2017 Numerical Analysis of Rock-Socketed Piles under Combined Vertical-Lateral Loading. Symposium of the International Society for Rock Mechanics, Procedia Engineering, Vol. 191, pp. 776-784.

Zhou, C.M., Shao, W., Yin, K.L., and Yang, Z.J., 2018. Estimating the properties of weathered bedrock and pile-rock interaction from the geological strength index. Journal of Mountain Science, Vol. 15, No. 8, pp. 1757-1776.

# RELIABILITY ON QUALITY CONTROL & QUALITY ASSURANCE OF DEEP FOUNDATIONS

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Bhubaneswar

**ABSTRACT**

A safe Design of foundation generally based on two considerations such as, the pressure that is safe from shear failure criteria and the pressure that is safe from settlement criteria. Similarly the load carrying capacity of a pile can be obtained from Vertical load capacity, uplift load capacity and lateral load capacity test on the piles in co-relation with theoretical approach of design. The load test may be of Contentious load test or cyclic load test. The test results can be obtained through soil resistance (P) vs. Deflection (y). The result varies from one type of soil to another type of soil. Similarly the Low Strain, High Strain Pile Integrity Test and Cross Hole Sonic Logging test is also considered to check the soundness of the pile. But it is difficult to carry out test on all the piles. Now, a question of reliability comes on the quality assurance of the structure. Here, an attempt has been made to find out the reliability on the behaviour of foundations in respect to quality assurance of the foundations structure. With the help of statistical analysis, frequency of tests and other data from field & laboratory an approach for reliability can be achieved. This article presents a brief about the process. Since the test over a structure is 'reliable' if it can be counted upon to perform in a dependable manner.

## 1. INTRODUCTION

Reliability can be defined as the probability that an item will perform a required function, under specified conditions without failure, for a specified amount of time."Reliability can also be thought as dependability or stability. A system or component is reliable if it can be counted upon to perform in a dependable manner**.**

From planning, design and execution of deep foundation construction a lot of care is to be taken up. Since the built-up area is to be covered with a wide spread area. Geotechnical investigations and testing of soil are having more important role. As soil is a natural material , we cannot find the homogeneous soil at every point. There is a probability of spatial variation in the soil parameter. But Steel and concrete are manufactured materials, these can be of uniform character.

So it is essential to have soil data in the hands of designer for preparing a reliable design of foundation with the help of geotechnical data.

## 2. SOIL TESTS FOR DEEP FOUNDATIONS TO MEET THE QC STANDARDS.

While the composition and depth of the bearing layer for shallow foundations may vary from one site to another, most pile foundations in a locally encounter similar deposits.


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Since pile capacity based on soil parameters is not as reliable from load tests, as a first step it is essential to obtain full information on the type, size, length and capacity of piles (including details of load – settlement graph) generally adopted in the locality.

Correlation of soil characteristics (from soil investigation reports) and corresponding load tests (from actual projects constructed) is essential to decide the type of soil tests to be performed and to make a reasonable recommendation for the type, size, length and capacity of piles since most formulae are empirical. If information about piles in the locality is not available or reliable, it may be necessary to drive a test pile and correlate with soil data.

Generally, the following tests may be required to obtain certain required geotechnical data:

**1. Direct shear test**

It is conducted to estimated effective friction angle of cohesionless soil. Nonetheless, there are correlations which are developed to evaluate effective friction angle of cohesionless soil.

**2. Standard penetration test (SPT)**

It is performed to determine the cohesion (and consequently the adhesion) to determine the angle of friction (and consequently the angle of friction between soil and the pile and also the point of resistance) for each soil stratum of cohesion less soil of $c-\emptyset$ soil.

**3. Static cone penetration test (CPT)**

This test conducted to determine the cohesion (and subsequently the adhesion) for soft cohesive soils and to check with SPT result for fine to medium sands. Hence for strata encountering both cohesive and cohesion less soils, both SPT and CPT tests are required.

**4. Vane shear test**

It is used to estimate undrained in-situ shear strength of impervious clayey soils.

**5. Undrained triaxial shear strength**

It is carried out on undisturbed soil samples (obtained with thin walled tube samplers) to determine cohesion ( $c$ ) and angle of internal friction ($\emptyset$) for clayey soils. In case of driven piles proposed for stiff clays, it is necessary to check with the ( $c$ ) and ($\emptyset$) from re-moulded samples. Drained shear strength parameters are also determined to represent in-situ condition of soil at end of construction phase.

**6. Pressure meter test**

It is performed to determine the stress- strain curve of horizontal loading. The equipment is either inserted into pre-drilled borehole or a self-boring pressure meter is used. Unless the soil is isotropic, the same value cannot be adopted for the vertical direction. This test is performed specifically for piles that subjected to lateral loading.

**7. Ground water condition and permeability of soil**

Ground water condition and soil permeability influence the choice of pile type to be recommended. Hence the level at which water in the bore hole remains are noted in the bore logs. Since permeability of clay is very low, it takes several days for water in the drill hole to rise up to ground water table.

**8. Possibility of chemical effects on concrete**

Ground water samples need to be tested to consider the possible chemical effects on concrete and the reinforcement. Result of the cone penetration test for the same soil show substantial scatter. Hence, they need to be checked with supplementary information from other exploration methods.

**9. Spatial Soil behaviours** :

Soil is the natural material, which has formed through complex process. The complicated material behaviour which is non linear and stress dependent, numerous widely accepted transformation models are differs spatially. Its property and behaviour changes from place to place. From the in-situ property of soil, It has found from studies that the unit weight of soil varies from 3-7%,effective Stress friction angle (Phi') varies from 2-13%. Undrained shear strength (Su) varies from 13-40% , Compression index Cc varies from10-37%.

Whereas, Coefficient of consolidation ($C_v$) varies from 33-68%, standard penetration test varies from 15-45%, Mechanical cone Penetration test($q_c$) varies fr 15-37% , Dilatometer tip resistance ($q_{DMT}$) varies from 5-15%,Vane shear undrained strength ($S_v$) varies from 10-20%. Interestingly a large variation of 130% to 240% variation occurred in the coefficient of permeability of partly saturated clay (k). And that of coefficient of permeability of saturated clay(k) is from68% to 90 %.

## 3. TESTS FOR DEEP FOUNDATIONS TO ASSURE THE QUALITY

Steel and concrete material is manufactured in controlled conditions, the material behaviour are mostly uniform and the variation is much less. So the quality assurance test is essential for confirmation on reliability of the structures.

Following important test are also required to go ahead with Quality Assurance test. Though these tests are for Quality assurance purpose, there are also some limitations for the purpose. So, care should be taken for performing as per the requirements.

- Pile Driving Analyzer :-High strain dynamic load testing and pile driving monitoring system.
- Thermal Integrity Profiler (TIP)
- Static Load Tester (SLT)

- Pile Integrity Tester (PIT)
- Cross-Hole Analyzer (CHAMP-Q)
- Acoustic Concrete Tester (ACT)
- Length Inductive Test Equipment (LITE)
- Strains and Accelerometers

**Pile Integrity Test**

The PIT performs a low strain dynamic test, based on the sonic pulse echo or transient response theory. It is mostly applicable to determine the integrity of relatively long foundation elements with buried, hard to visually inspect lower portions.

**Evaluation of concrete thickness using Ultrasonic Echo Technology**

The Acoustic Concrete Tester helps assess if a concrete structure is free of defects such as spalls, honeycombing, and horizontal flaws.

**Limitations of integrity test methods**

| Sl. No. | Method | Reference Standard | Limitations |
|---|---|---|---|
| 1 | Pulse Echo (PEM) | **ASTM** D5882 | Depth often limited to 30-40shaft diameter |
| 2 | Transient Response(TRM) | **ASTM** D5882 | Depth often limited to 30-40shaft diameter |
| 3 | Crosshole Sonic Logging (CRL) | D6760 | Sensitive to across tube/concrete bond<br>Fine Horizontal cracks unlikely to be detected<br>Depth limited only by cable length |
| 4 | Gamagama Logging (GGL) | None | Storage and transport of gamma Source<br>Depth limited only by probe cable length |
| 5 | Thermal Integrity Profiling (TIP) | **ASTM** D7949 | Fine Horizontal Cracks unlikely to be detected<br>Depth limited only by thermal wire or probe cable length |
| 6 | Non-destructive integrity test of Piles | **IS 14893** | Does not provide load carrying capacity of Pile, Verticality of displacement in position of piles |

## 4. STANDARD ERROR OF MEASUREMENT

To get correct value in the quality assurance numbers of tests should be conducted. Sometimes the test results also varies from each other . The closest values of test results are to be considered as corrected value. The method of NDT pile testing involves high skill and use of computerized equipments. Therefore, the tests should be performed and interpreted by trained and experienced personnel.

Standard deviation of error of measurement in a test or experiment is closely associated with the error variance, which indicates the amount of variability in a test administered to a group that is caused by measurement error.

The standard error of measurement is a function of both the standard deviation of observed scores and the reliability of the test. When the test is perfectly reliable, the standard error of measurement equals 0. When the test is completely unreliable, the standard error of measurement is at its maximum, equal to the standard deviation of the observed scores.

The standard error of measurement serves in a complementary role to the reliability coefficient. Reliability can be understood as the degree to which a test is consistent, repeatable, and dependable. The reliability coefficient ranges from 0 to 1: When a test is perfectly reliable, all observed score variance is caused by true score variance, whereas when a test is completely unreliable, all observed score variance is a result of error.

The Pearson product-moment coefficient measure of reliability is commonly used for the calculation of the standard error of measurement, and the interclass correlation coefficient is also appropriate to use in many situations.

## 5. RELIABILITY IN PILE DESIGN APPROACH

### *4.i PRESENT DESIGN APPROACH OF VERTICALLY LOADED PILE:-*

Ultimate axial load carrying capacity

$Qu = p \times L \times q_{us} + A_b \times q_{ub} - W$

Where, p= pile perimeter = $p_1 \times D$, D= pile diameter, L= length of pile, $A_b$ = Area of Pile base, W= weight of pile, $q_{us}$ = Ultimate skin frictional resistance, $q_{ub}$ = Ultimate unit end bearing resistance.

Design load capacity: Qu/ FoS( FoS varies from 2-3)

***Inadequacies in the present approach***

Over a wide range of area and load the formula cannot be unique . In situ behaviour of pile foundation is being influenced by variability in soil properties. Serviceability states and Ultimate state are separate for each approach.

### *4.ii RELIABILITY DESIGN APPROACH*

This can be achieved in the following steps:-

Step 1: Cone tip resistance profile and determination of soil parameter

Step 2: Interface parameters

Step 3: Evaluation of Variability

Step 4 : Reliability Analysis

Basic random variables: Undrained shear strength near pile shaft, near pile tip, constants and allowable settlement /serviceable settlement.

Standard deviation, scale of fluctuation and variance reduction of undrained shear strength is obtained from CP data.

-COV of measurement error: COV m= 15% (Phoon & Kulhaway1999)

-COV of transformation variability: $COV_{tr}$= 29% (Phoon & Kulhaway1999)

-COV of serviceability settlement : 58.3% (Zhang et al. 2005)

**Design load**

***Reliability analysis of laterally loaded Pile:***

For an applied lateral load pile foundation is considered to be satisfactory:

Lateral displacement at pile head doed not exceed allowable displacement (1% diameter of Pile)

Maximum bending moment does not exceed moment capacity of pile section. The design approach can be made out of the parameters obtained from CPT Profile. The Mean ($M_{max}$) & Variance of Maximum moment for the design load is to be obtained to get the Reliable based design. (Here the Coefficient of Variation $COV_{kh}$ =39%)

In this reliability study following observations have found

- Probabilistic analysis of soil profile provides a format for quantifying the information about the subsurface condition of the site and it also provides the basis for predicting the reliability of the pile foundation.
- Depending upon the uncertainty level and spatial variability of soil, allowable load can be suggested.
- It is useful to choose a suitable value for serviceability limit, so that the combined reliability index is ensured from both the considerations of Ultimate Limit State and Serviceability Limit State.

## 6. CONCLUSION

Reliability of QC & QA in deep foundation can be obtained, when the testing instruments are used of properly calibrated one; the technical personals conducting tests are of highly skilled & experienced and at the same time for caste in situ concrete piles, the age of concrete should not be less than 15 days. Some complementary tests are also to be added to get the reliable result. The data required for QC and QA are to be perfect so that the proper reliable design results can be obtained. Proper documentation for the purpose is highly essential to analyse the data for obtaining a reliable result.

## Reference:

1. A. Fenton1 and D.V. Griffiths ,Reliability-Based Deep Foundation Design Gordon 2 in Probabilistic Applications in Geotechnical Engineering, Geotechnical Specialty Publication No. 170, ASCE, 2007

2.https://www.statisticshowto.datasciencecentral.com/standard-error-of-measurement/

3. Dr. G.L.S. Babu, Reliability analysis of geotechnical systems

4. IS: 14893:2001 Non-Destructive Integrity Testing of Piles(NDT)-Guidelines

5. Harry A. Kinnison, Aviation maintenance Management, Mc Graw Hill(P)Ltd.New Delhi,2010

# A CASE STUDY ON PRESSUREMETER TEST IN KATNI, MADHYA PRADESH, INDIA.

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## ABSTRACT

The longest railway elevated corridor measuring around 18KM including 12KM at grade is proposed to be constructed at Katni, Madhya Pradesh, India which comes under Western Central Railway. As a part of detailed design work geotechnical investigation work was carried out. Pressuremeter test (PMT) is adopted as high quality in-situ testing of soil strata. The geological profile of Katni is characterized by a top alluvial deposit followed by limestone formation either in solid rock form or as a residual soil having high plastic clay characteristics. At deeper depths due to higher SPT 'N' values, it is difficult to penetrate thin tube sampler for the collecting undisturbed samples. As a common geotechnical practice, undrained shear strength ($S_u$) of soil is evaluated based on the available correlation with SPT N for computation of pile capacity. In this project an attempt was made to evaluate the in-situ undrained shear strength of soil through pressure meter test. This paper covers the investigation process of PMT, the challenges faced during the investigations, and along with the analysis of the PMT results. The analysis of PMT results includes the interpretation of Pressure meter modulus ($E_{PMT}$), limit pressure ($P_L$) and undrained shear strength of soil.

***Keywords: in-situ, limit pressure, Pressuremeter test, undrained shear strength, value engineering.***

## INTRODUCTION

The longest elevated railway corridor in India is proposed to be constructed in Katni, Madhya Pradesh. Katni being a major junction of West Central railways, heavy railway traffic occurrence exists. The freight and passenger traffic is thereby planned to be decongested through the implementation of Katni Grade Separator project.

As a part of geotechnical investigation, Pressure meter test (PMT) which is a specialized in-situ testing is carried out at various locations across the project stretch. The aim of this paper is to illustrate the planning of PMT locations, the practical problems faced during the execution of testing at site with respect to soil profile, the solutions that can be followed to overcome those, followed up by the analysis of raw data to evaluate limit pressure, PMT deformation modulus and undrained shear strength of soil strata.

In geotechnical engineering practice in India the most commonly used field test is Standard penetration test, which is further used to correlate the values of undrained shear strength through established correlations. The problems of using the correlations are that, they are developed for geological strata not inherent to the local geology. Thereby in this project an effort was made to capture the in-situ shear strength properties and do away with correlations which always give an uncertainty to the designer on


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assessing the in-situ shear strength of soil. In this paper we will observe that cohesion values upto 500kPa were achieved in hard clay through analysis of PMT data which otherwise would be difficult to consider for design as the shear strength range given by Stroud (1974) is limited 400kPa. However at ground, there are a lot of challenges in execution of PMT successfully at multiple locations. The challenges faced are different for varying geological profile conditions. But through proper knowledge of geological profile, expertise of PMT instrument and good experience in various projects those challenges can be overcome.

## PROJECT DESCRIPTION:

KATNI is located in the central part of India, and is the 2nd busiest railway junction in Madhya Pradesh. To address the easy moving of traffic a grade separator in UP and DOWN line having a total length of 32KM of which 12KM is on viaduct, 4KM on retaining wall and remaining on the earthen embankment has been proposed. The grade separator is divided in to two lines, UP line for passing the loaded goods trains and DOWN line for returning back. Based on the rail formation level and ground level difference the grade separator is divided in to three parts; Viaduct, retaining wall and embankment. In viaduct portion obligatory spans on major crossings like river & railway lines and composite & open web girder spans in other locations according to RDSO standard were proposed. There are in total 645 supports for the viaduct portion along the alignment in both the lines. For a smooth grade change from embankment to viaduct a U-trough type retaining wall is proposed.

## PHYSIOGRAPHY OF THE SITE:

### *Geology:*

In general Katni is a mining area having limestone and latertic bauxite formations. The geological profile consists of residual soil of limestone and bauxitic origin overlain by a top layer of alluvium soil. The residual soil is mostly high to medium plastic clay in nature. Limestone rock was encountered in highly weathered state in and around the Katni river area.

## GEOTECHNICAL INVESTIGATION:

### *Planning:*

For detailed design the geotechnical investigation is proposed in three stages. Stage-1 investigation comprising of 32 no's of boreholes at a 450m distance apart along the alignment in a staggered fashion has been executed for the preliminary assessment of the strata. In stage-2 investigation, for detailed assessment in total 168 boreholes at 75m distance apart in staggered fashion has been conducted. In both the stages of investigation standard penetration test and undisturbed sampling for laboratory testing has been done. In stage-3 investigation in-situ pre-bored pressuremeter in 8 boreholes, at 4 test depths in each; total 32 pressuremeter tests has been conducted. The test locations and depths were decided by considering the soil\lithological profile from stage-1 & 2 investigations. As collecting undisturbed sample at depth more than 10m is found critical, the pressuremeter test is adopted for strength characterization of the deeper strata.

### *Soil profile:*

Field exploration and laboratory tests from stage – 1 and stage – 2 investigations up to 40m shows the presence of brownish to grey colour silty clay in medium stiff state at surface to hard at deeper depths. In Figure 1 generalised soil profile and Atterberg limits are shown. The soil at deeper depth mostly after 10m depth is found to be intermediate plastic clay in hard state, occasionally with gravel content varying from 12 to 20%. The clay with lime content (in residual soil form) is also located at depths greater than 15m. Coarse sand with gravel content is also found as pockets at some depths.

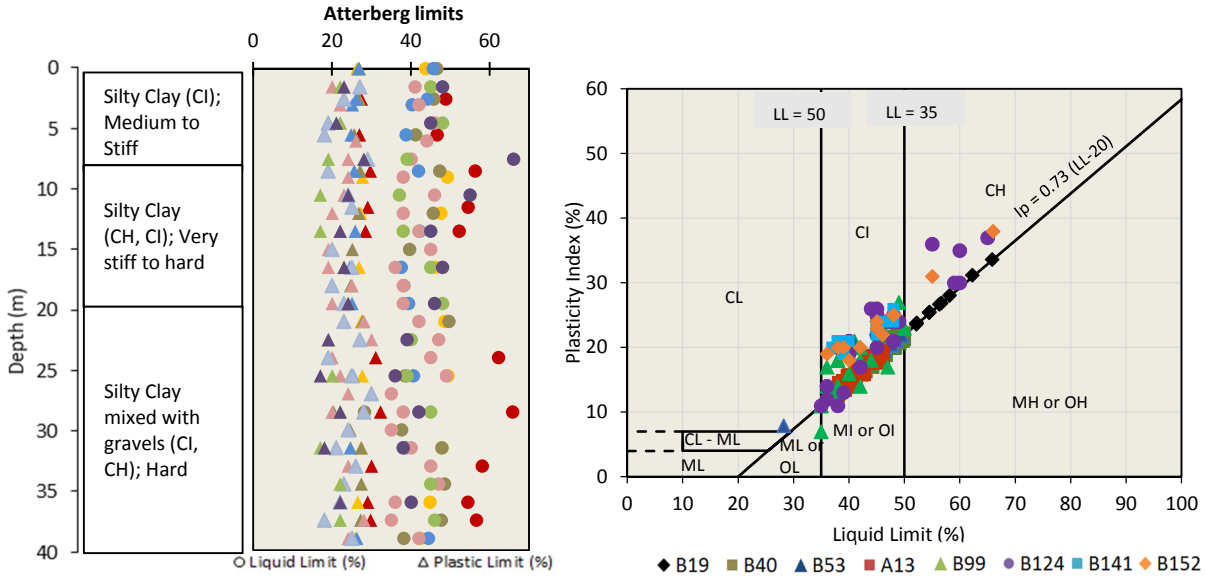



**Figure 1 Soil profile and Atterberg limits at PMT locations**

### *Pressuremeter Test:*

Site investigation and determination of characteristics of soil, soft rock and the transition zone between both; are critical parts of the geotechnical design process, which is commonly exercised by either laboratory or in-situ testing. It is quite apprehensible that the limitations associated with collecting undisturbed soil samples and the reliability of laboratory test results; increase the demand of in-situ testing. As in laboratory testing simulating the boundary conditions mostly affects the results. However, more complex boundary conditions were associated with in-situ tests than those imposed by laboratory test devices. In PMT the boundary conditions and stress-strain relationships were well defined and controlled (Mair and Wood, 1987).

### *Equipment:*

Menard pressure meter system consists of two units: a radially expandable probe which is to be lowered to a slightly large diameter bore and a pressure-volume measuring unit to be rest on the ground surface called as control unit. The probe consists of a hollow tube having rubber membrane outside. There are three cells, one measuring cell and two guard cells. The water is injected to the measuring cell by the application of compressed nitrogen gas and the expansion of probe is measured with volume-change corresponding to the pressure applied. The guard cells were expanded by the application of compressed gas. The function of the guard cells is to remove the end effects of the measuring cell; hence the assumption of plane strain and axisymmetric condition for analysis of expansion of a cylindrical cavity in soil will be maintained during the test.

### *Test procedure:*

Menard pressuremeter test is a pre bored pressuremeter test. A NX sized drilling bit has been used and water is used as drilling fluid. The NX sized casing of 90mm outer dia. and 76mm inner dia. is also driven up to some depth above the test location according to the site requirements. The probe is having outer dia. of 74mm. During drilling a tolerance of 4-5mm for increase in borehole dia. is allowed. At test depth location the borehole has been drilled extra 0.6m deep from the test location. The stress relaxation time, a function of vertical stress and time varies between 90 and 1300 minutes, affects the results significantly Lacasse and Lunne (1982). The probe is lowered to the test depth and test has been initiated within 20

mins of drilling. The lowering of probe and testing has been completed within 1 hr. of insertion of probe in borehole.

After the insertion of probe the water was allowed to flow to the measuring cell from volumeter until the membrane came in to contact to the borehole wall. To counter the end effects, the guard cells were also inflated by compressed gas at a differential pressure of 0.1bar. The size of the pressure increments was chosen based on the strata encountered. At shallower depth the pressure increments were given at 0.5bar intervals till 5bar and 1bar afterwards. At deeper depths the increments were given at 1bar throughout the test. The test was stopped when the volume meter reached its maximum capacity 800cm$^3$or three pressure holds in plastic phase. Around 30s is required to give the each pressure increment. As the pressure applied the readings noted at 30s and 60s, a short-term creep is recorded. The volume change recorded with in two time intervals was plotted with respect to the pressure applied. A typical graph showing pressure-volume change is shown in Figure 2. From graph it is clearly observed that the creep volume at linear portion of the curve is significantly less or zero. As the soil annulus resisting the pressure reaches the plastic stage, the creep volume starts increasing indicating the phase of strain softening.

The field data observed is corrected for the following:

a) The hydraulic head acting on the membrane due to height of the water column above the test depth
b) Volume changes due to the flexibility and compliance of the equipment (at higher pressures the compression of membrane and expansion of tubes)
c) Pressure losses due to the resistance from the membrane.

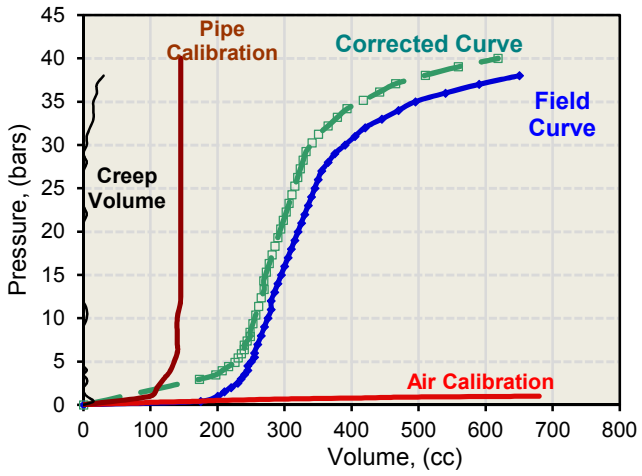



**Figure 2 A typical pressuremeter curves in Borehole B53 at 30m depth**

## *Problems/challenges encountered:*

PMT being a specialized field test has plethora of advantages in terms of assessing the in-situ properties of soil correctly. But execution of the test at site is still an uphill task. The most common problem faced with PMT is bursting of the probe while testing is in progress. The examples of probe blast in this present case study are shown in Figure 3 below. The various reasons are outlined as

- Presence of gravels in clay layer which are sharp edged, thereby when very high pressure is applied to the probe; the gravels puncture the rubber sheath outside.
- Probes which have been used previously many times often burst at high pressures.

The bursting of probes is a serious yet a very common hazard in PMT field work. In the present case study, there were a few probe blasts which resulted in drilling a new borehole as the depths are predefined based on soil profile. In case the soil strata were similar for considerable thickness the PMT was carried out at other depth by maintaining a gap of 2metres from previous test depth. The bursting of probes was suggested to be reduced by using a steel sheath instead of a rubber sheath because of the presence of gravels. After incorporation of the same the bursting of probes was reduced.

The other very common problem associated with field work of PMT is leakage of pipes connecting the Menard Pressuremeter measuring apparatus and the probe. The leakage of the probe is also a challenge faced during the testing in present case study. The leakages in the pipes can be effectively spotted in the Measuring apparatus by checking the volume measuring jar which will show constant rapid reduction of water without stopping. This was overcome by stopping the test at that point and bringing up the pipes and probe to check for leakages. After thorough checking of pipes by lowering in water and ensuring no air bubbles the problems arising due to leakages can be arrested and the same was done in the present case.

The Menard Pressuremeter instrument should be checked for air bubbles, especially the pipes behind the measuring instrument which connects the water storage tank with the measuring jar. In the present case study, a problem was encountered wherein a constant rate of rapid volume reduction was seen with application of 1bar pressure. The pipes were checked for leakages but everything was found to be working perfectly fine. Later through discussion with experts the measuring instrument was checked for air bubbles and after removal of the air bubbles the issue of rapid volume loss was solved. As a precautionary measure, a test was carried out in casing to check for volume loss happening due to leakages in pipes or air bubbles in measuring instrument so that the test section is not disturbed.




**Figure 3 Extracted damaged probe (a) due to sticking of clay to the metal strips (b) burst outer membrane (c) metal strips came out from the probe assembly (d) membrane burst inside the metal strips**

As stated earlier in section, the geological profile consisted of high plastic residual soil which proved to be quite a challenge for carrying out PMT. The soil being of high plastic nature, the probe was getting stuck after completion of test. Thereby it was getting very difficult to extract the probe from the borehole and in many cases, in this project there was considerable damage incurred to the probe while extraction.

This problem occurred multiple times in this case study and the same was somewhat overcome by carrying out dry drilling while doing the test section and by extracting the probe immediately after the pressure has been released in probe after completion of test.

## IMPLEMENTATION:

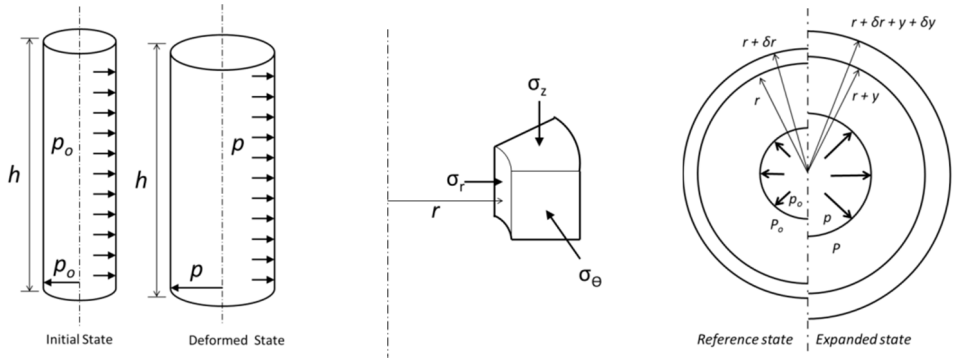
### *Pressuremeter test*

An ideal cavity expansion is illustrated in Figure 4. In cohesive soil, the test is assumed as the undrained deformation of saturated soil, because the time of test duration is very small compared to the drainage time, having a drainage path in order of borehole radius (Palmer, 1972). There is a reduction of radial stresses at the borehole wall when the borehole is drilled. If the stress relaxation and disturbances due to drilling is less, the soil will behave elastically. The pressure is applied after inserting the probe to the test depth. Once the membrane touches the borehole wall the soil responds elastically at each pressure increment. The pressure-volume curve shows a linear behaviour until the soil annulus reaches strain-softening. The volume ($V_i$) at which the curve became linear is equal to the difference between the volume of the hole and the initial volume of the probe. The reference pressure ($P_i$) and volume ($V_i$) is taken from the starting point of the linear portion. The expression for shear modulus for the expansion of a cylindrical cavity is given by Gibson and Anderson (1961):

$$G = (V_o + V_m)\frac{\Delta p}{\Delta v} \qquad [1]$$

Where, G is the shear modulus of the soil, $V_o$ is the initial volume of the probe, $V_m$ is the mean volume of the probe in the elastic region, Δp is the corrected pressure increase in elastic region, Δv is the corrected volume increase in the elastic region for the increment of Δp pressure. The test results commonly presented as an equivalent Young's modulus ($E_{PMT}$) assuming an isotropic elastic soil. According to Menard (1957) the Poisson's ratio (μ) is 0.33.

$$E_{PMT} = 2\,(1 + \mu)(V_o + V_m)\frac{\Delta p}{\Delta v} \qquad [2]$$




(a) (b) (c)

**Figure 4 Ideal expansion of cavity (a) expansion of cylindrical cavity (b) Element of soil in cylindrical symmetry (c) reference and expanded stage of cavity at a radius r**

In the analysis of expansion of cylindrical cavity in elastic- perfectly plastic soil the reference state is taken as the applied pressure in pressuremeter is equal to the in-situ horizontal pressure before the borehole had been done (Palmer, 1972; Ladanyi, 1972). So, the estimation of in-situ horizontal pressure at the borehole wall is necessary for the further analysis. The initial point from the linear portion of the pressuremeter curve is considered as the horizontal in-situ stress. From the kinematics of expansion this state is taken as reference state for all the stress and strains during the test. All volume is normalised with this reference volume ($V_i$) and volumetric strains were calculated from this volume. According to Palmer (1972) the ultimate and apparent peak shear strength can be obtained from the strain softening portion and elastic portion at point of inflections of P: ln (ΔV/V) curve respectively. As, the strength derived from

post peak portion is less sensitive to the disturbances caused by the initial installation by drilling (having high uncertainty in considering the reference condition); the slope at this large deformations give more reasonable average undrained shear strength of soil. In Figure 5, the undrained shear strength ($S_u$) based on Palmer (1972) theory post point of inflection is presented.

## DISCUSSION:

### *Limit Pressure ($P_L$):*

This is defined as the pressure at which the probe volume reaches twice the original soil cavity volume. In present case study $P_L$ was evaluated by calculating the slope of pressure and inverse of corrected volume. The values of limit pressure are presented in Figure 5 and it can be observed that the values are ranging from 20-45bar in general. The limit pressure of hard clay is given as >16bar by Mair and Wood (1987).

### *Undrained shear strength (Su):*

The measured undrained shear strength depends on the stress paths followed during the testing and initial stress states in the equipment/method adopted. In current study the ultimate strengths derived from slope of P: ln (ΔV/V) curve based on the points in the plastic phase of the test. The undrained shear strength values evaluated from the curve are presented in Figure 5. In the present case study, the cohesion values interpreted from PMT in the depth 12-15metres below ground level vary from 250-500kPa for SPT N values varying for 18-80. The cohesion interpreted in the depth range of 15-20metres vary from 400-500kPa for SPT N values from 38-Refusal, whereas from 20-30m depth range from 500-700kPa for SPT N values ranging from 33-Refusal.

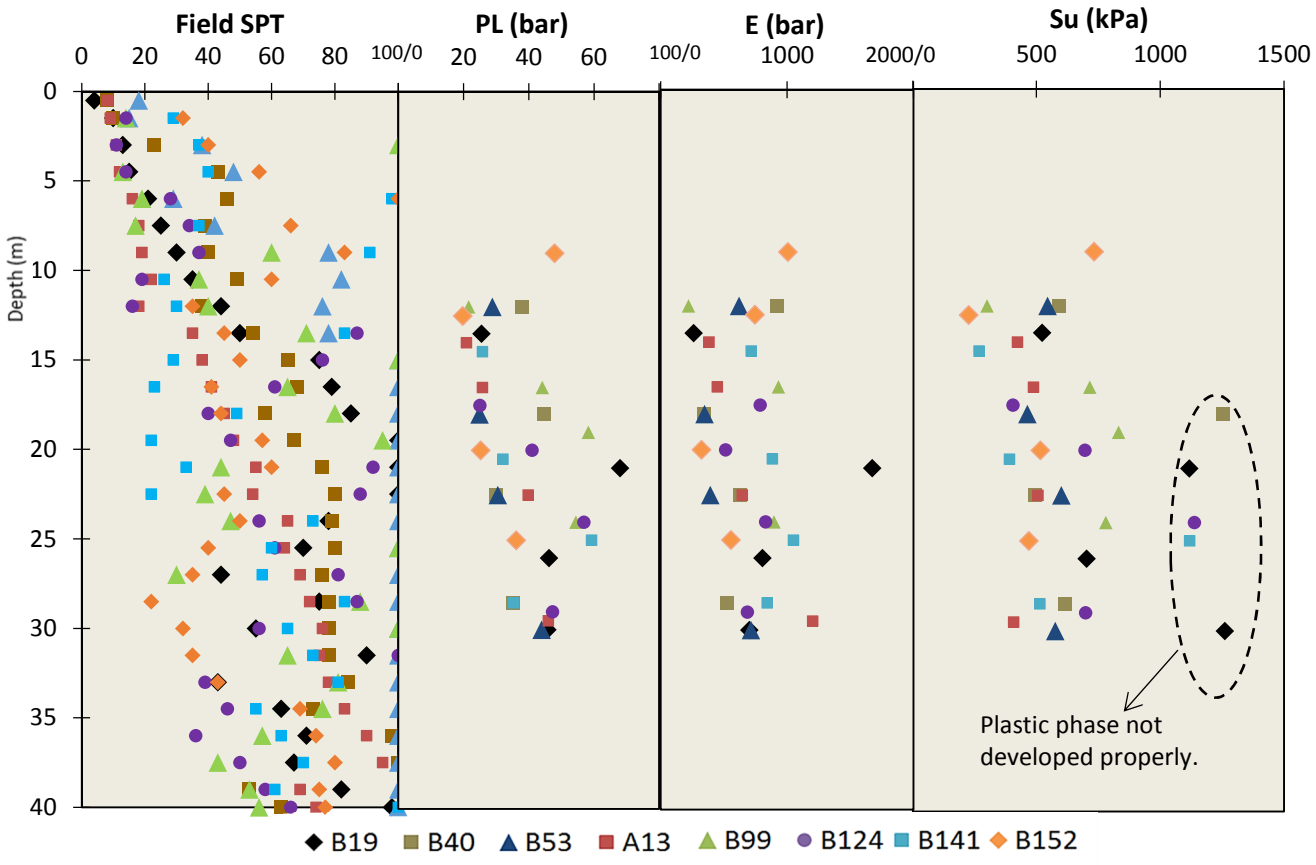



**Figure 5 Profile showing the variation of Field SPT and pressuremeter test results ($P_L$, $E_{PMT}$ and $S_u$)**

In Figure 7 a comparison is drawn between the ranges of cohesion values obtained through correlation from SPT given by Stroud (1974) and the undrained shear strength (cohesion) of soil obtained from interpretation of PMT data. It can be clearly observed that there is a marked difference in cohesion values especially at deeper depths (>15m). The cohesion values obtained from PMT data is much higher (as observed from graph) and reliable as the test is in-situ in nature. However some test results as affected by

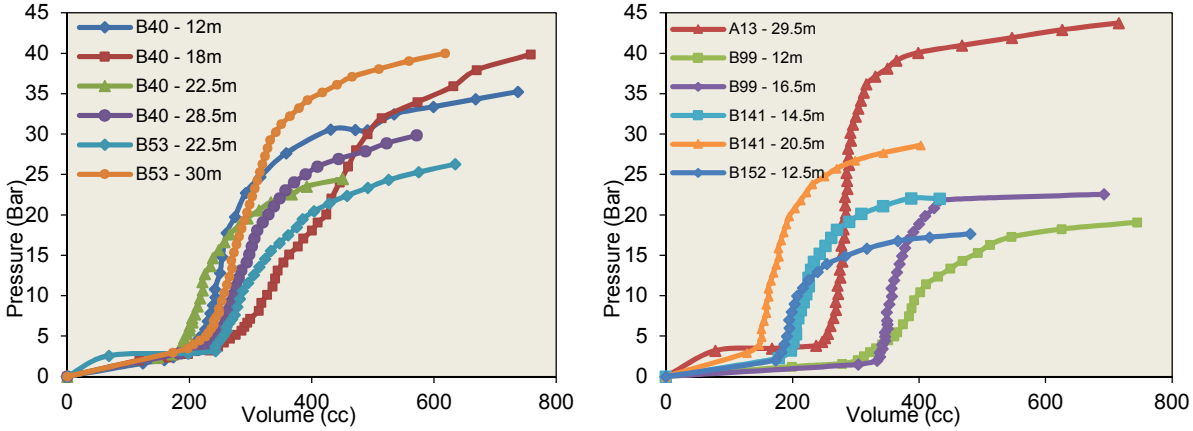



**Figure 6 Pressuremeter curves in test locations**

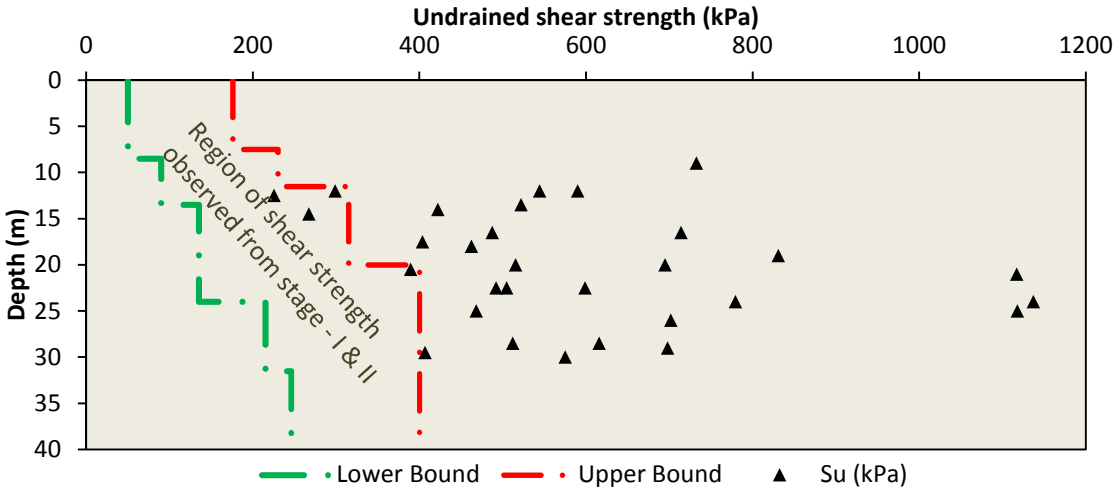



**Figure 7 Generalised undrained shear strength profile observed from Investigation stage – I and II and PMT test.**

the drilling methods (explained later section) also presented to show the risk of using the undeveloped plastic stage test results. Some authors have even reported that the authenticity and correctness of undrained cohesion obtained from PMT better than that obtained from Triaxial UU test. The reason for being higher undrained shear strength is explained by many previous researchers Hansen and Gibson (1949); Wood and Worth (1977). According to Hansen and Gibson (1949) the shear strength mobilized on a vertical plane as in pressuremeter test is much greater than; the shear strength mobilized a plane inclined 45 deg. to the horizontal in triaxial compression test; because the lateral earth pressure applied in pressuremeter test in-situ condition is very high than the confined pressure applied in triaxial compression test. Wood and Worth (1977) stated that on the expansion of cavity the radial stress $\sigma_r$ becomes the major principal stress (more than existing vertical stress $\sigma_z$) and $\sigma_\theta$ becomes minor principal stress. Hence, these two principal stresses causing the failure on a vertical plane. So the vertical stress $\sigma_z$ becomes an intermediate principal stress causing no effect on the soil element.

However, $S_u$ for design is considered from Triaxial UU tests, Pressuremeter test up to a depth of 20m and beyond 20m the values had been limited to 550 kPa. The limitation of $S_u$ had been considered from the limits given by Stroud (1974).

### *Pressuremeter Curves:*

The quality of the pressuremeter test is reflected from the pressure – volume (P-V) curves. The graphs showing distinct elastic, elasto-plastic and plastic zones give the indication of a correct PMT.

In current study the graph at BH-B53 at 30m depth is considered as a standard perfect graph for the comparison of quality of the pressuremeter curves at few other locations. From Figure 8a, it can be observed that the graphs of PMT locations at BH-B19(30m), BH-A13(16.5m), BH-B53(18m),BH-B124(20m) have shifted more towards the right compared to BH-B53(30m) due to oversizing of borehole. As the borehole is oversized more radial expansion (i.e. more volume) is required for the probe to touch the borehole walls hence the test has to be terminated much before the plastic stage is completed. Thereby due to fewer points in the plastic phase correct determination of undrained cohesion becomes an issue.

The Figure 8b shows that the PMT graphs at BH-A13 (22.5m), BH-B141 (28.5m), BH-B152 (28.5m) are towards the left side of BH-B53 (30m). This has reference to the soil profile described earlier which consists of residual soil having clay like soil with gravel content. At locations where the casing was not being able to be driven up to the top of test section, the gravel particles fill up the annular space between the probe and the borehole wall. Thereby the gravel particles are first compressed and the pressure readings can be seen from the start of the test instead of starting after some volume loss required by the probe to touch the borehole wall.

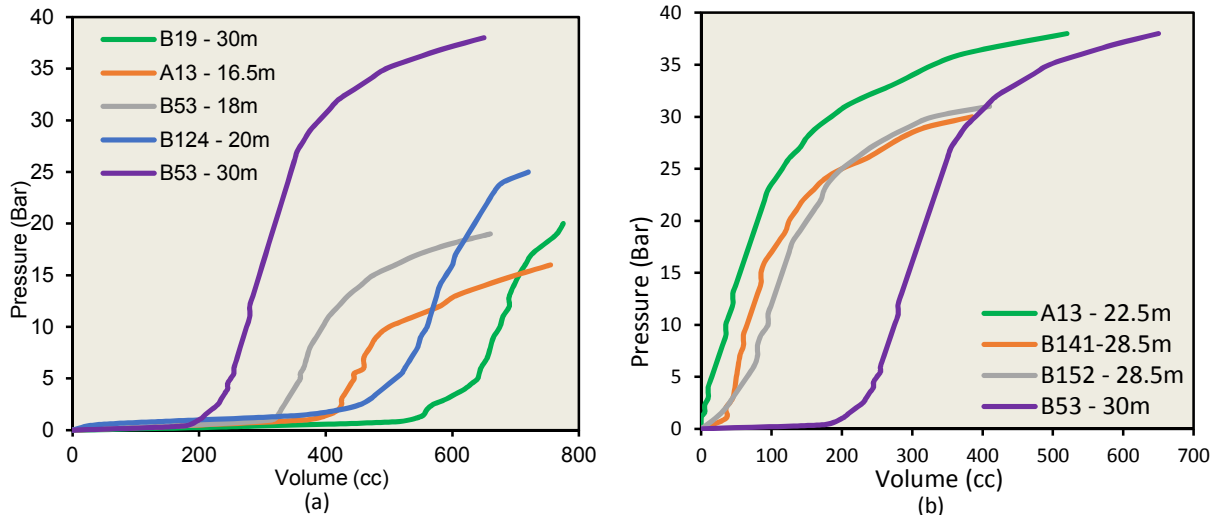



**Figure 8 Pressuremeter curves obtained from different locations. (a) Oversized borehole (b) Undersized borehole**

## CONCLUSION:

From this case study, the implementation of PMT in the field, its planning, the challenges faced, the analysis of PMT raw data and the effect of quality of borehole drilled on PMT is illustrated in detail.

The following points can be concluded from the present case study:

1. The planning of PMT should be based with respect to existing borehole investigation data. Proper understanding of geological profile is required to select the test depths, which enables the optimum use of PMT results.
2. The challenges faced in the field regarding execution of PMT can be overcome by thorough checking of PMT equipment before carrying out the test. Also through proper observation of volumeter readings, discrepancies in the testing can be noticed.
3. The puncturing rubber sheath by the presence of gravels posed a serious issue for the testing. Thereafter steel sheath was used, but at later stage that posed an issue as the high plastic clay was sticking to the probe and it was difficult to extract the probe after the test.
4. The evaluation of PMT raw data to calculate the undrained shear strength ($S_u$) was carried out as per established methodology by Palmer (1972). However the correct value of undrained shear strength can only be established by having more points on the plastic phase of the test i.e. carrying on the test to get at least 3 to 4 readings in the plastic phase.
5. The size of the borehole diameter should be within the tolerance limit for getting the correct results from PMT. The undrained shear strength evaluation ($S_u$) won't be possible if the borehole will be over-sized.
6. The in-situ undrained shear strength of soil evaluated from PMT is much higher and reliable than those obtained from correlations from SPT.

## REFERENCES:


Gibson, R.E. and Anderson, W.F. 1961. In-situ measurements of soil properties with the pressuremeter Civ. Engng. Public Wks. Rev. May 1961, 56(658), 615-618.

Hansen, J.B. and Gibson, R.E. 1949. Undrained Shear Strengths Of Anisotropically Consolidated Clays. Géotechnique, Volume 1 Issue 3, June 1949, pp. 189-200

Ladanyi, B. 1972. In-situ determination of undrained stress-strain behaviour of sensitive clays with the pressuremeter. Canadian Geotech. J. Aug. 1972 9(3), 313-319.

Lacasse, S. and Lunne, T. 1982. In-situ horizontal stress from pressuremeter tests. Proc. Int. Symp. On The Pressuremeter and its Marine Applications, Paris, April 1982. International Association for Engineering Geology, 1982, 187-208.

Mair, R.J. and Wood, D.M. 1978. Pressuremeter Testing: Methods and Interpretation, CIRIA Report, Butterworths, UK, 160.

Marsland, A. and Randolph, M. F. 1977. Comparisons of the results from pressuremeter tests and large in-situ plate tests in London Clay. Geotechnique June 1977 27(2), 217-243.

Palmer, A.C. 1972. Undrained plane-strain expansion of a cylindrical cavity in clay: a simple interpretation of the pressuremeter test. Geotechnique. Sept. 1972, Volume 22 Issue 3, pp. 451-457.

Stroud, M.A. 1974. The standard penetration test in insensitive clays and soft rocks. ESOPT, Stockholm 1974. Stockholm, National Swedish Building Research, pp367-375.

Wood, D.M. and Wroth, C.P. 1977. Some laboratory experiments related to the results of pressuremeter tests. Geotechnique June 1977 27(2), 181-201.

# COMPARISON OF GEOPHYSICAL AND GEOTECHNICAL INVESTIGATIONS OF A DEEP SOIL SITE IN INDO GANGETIC PLAINS

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## ABSTRACT

There has been a renewed interest for sub surface characterization of soil and shear wave velocity measurement using various geophysical methods involving different techniques, testing configurations and inversion methods. The most widely- used geophysical method in earthquake geotechnical engineering is MASW (Multichannel Analysis of Surface Waves) and Horizontal-to-Vertical Curve (H/V) from Microtremor studies. Roorkee city falls in seismic zone IV in the state of Uttarakhand (India) and is located in the vicinity of Himalayan Frontal Fault (HFT). The presence of thick alluvium makes it more vulnerable to seismogenic sources and hence, its characterization is required. Joint fit inversion technique was used to obtain the shear wave velocity profiles. Moreover, the results of this geophysical methods are compared with the available geotechnical investigation of the site. It was observed, that the shear wave velocity obtained from MASW and Microtremor is in good agreement with those obtained from the bore logs data. Hence, both the geophysical and geotechnical investigations are complementary to each other and results of one method can serve as a guideline for the other method.

**Keywords: MASW, Microtremor, Shear wave velocity**

## INTRODUCTION

Many seismically vulnerable regions in India and worldwide are located on deep soil deposits which extend to several hundred meters of depth. The whole North-East, Himachal Pradesh, Jammu and Kashmir and northern –parts of Uttarakhand falls in seismic zone V. It has been well recognized that the earthquake shaking is altered by geological conditions at the location of building. As seismic waves propagates through uppermost layers of soil and rock, these layers serve as filter and they can increase the duration and amplitude of earthquake motion within narrow frequency bands. Moreover, the damage caused by an earthquake is more over soft sediments of soil as compared to firm bedrock outcrops. If an area is situated on deep sedimentary deposit then, it is more prone to earthquake hazard. Hence, there is a need to carefully characterize the deeper soil using the available techniques developed in geotechnical earthquake engineering.

Site characterization, in general, can be defined as the process of identifying the subsurface material properties of soil. There are many parameters adopted for recognizing these material properties, such as, namely, density, shear modulus of soil, shear wave velocity etc. The shear wave velocity of the subsurface materials using the geophysical methods has gained importance in the past years of development in geotechnical earthquake engineering. The most widely used technique in the geophysical methods for the determination of shear wave velocity is Multi Channel Analysis of Surface Waves (MASW) technique with a combination of horizontal-to-vertical curve (H/V) from microtremor measurements.

In this paper an attempt has been made for the seismic site characterization the study region using the geophysical methods MASW and microtremor measurements. Further, geotechnical investigations results are compared with the geophysical data to analyze the difference in estimation of shear wave velocity from these methods.


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## STUDY REGION: ROORKEE (UTTARAKHAND)

In the present study, the region selected for site characterization is Roorkee attributing to its high seismicity and having local site characteristics of soil. Roorkee is situated at latitude 29° 51' and longitude 77° 54' is a city in the state of Uttarakhand. It is located around 30 km from Shiwaliks range which can lead the city to experience strong ground motion if a moderate earthquake occurs on Main Boundary Thrust or on Main Central Thrust. Moreover, Himalayan Frontal Thrust lies very close to Roorkee and to the south of Main Boundary Thrust and Main Central Thrust. The geology of the area is represented by alluvial soil and forms a part of Gangetic alluvial plains.

As per the seismic zoning map of India as incorporated in Indian Standard Criteria for Earthquake Resistant Design of Structures: General Provisions and Buildings, Roorkee lies in seismic Zone IV (IS: 1893-Part I; 2002). Indian Meteorological Department has reported that numerous earthquakes of medium to large size have occurred in the study region as per the data obtained from historical and instrumental records. The major earthquakes reported are (i) The Kangra earthquake (1905) of magnitude 8.0 (ii) The Kinnaur earthquake (1975) of magnitude 7.0 (iii) Bihar-Nepal border earthquake (1980) of magnitude 6.1 (iv)The Uttarkashi earthquake (1991) of magnitude 6.6 and (v) The Chamoli earthquake of (1999) of magnitude 6.8.

In the present study, six sites of Roorkee are selected for the evaluation of shear wave velocity profile of the city using MASW and Microtremor tests. These sites were located at a distance of about 2-3 km from each other. Figure 1 shows the division of Roorkee city namely, as, Roorkee East, Solani River and Roorkee West which is based on the professed existing soil conditions in each area. In Roorkee East, it is expected to be consisting of less hard sub soil while, on the other hand, Roorkee West was expected to have the hardest sub soil. The Solani River was expected to have the soft sub soil conditions. In Roorkee East, three sites are taken within the IIT Roorkee campus which includes Department of Earthquake Engineering (S1), Lal Bahadur Shastri Stadium (S2) and IIT Roorkee Hospital Ground (S3). In Roorkee West, two sites are chosen are the Roorkee Railway Station (S4) and Nehru Stadium Ground (S5). The sixth site that was selected for the study was Solani River (S6).

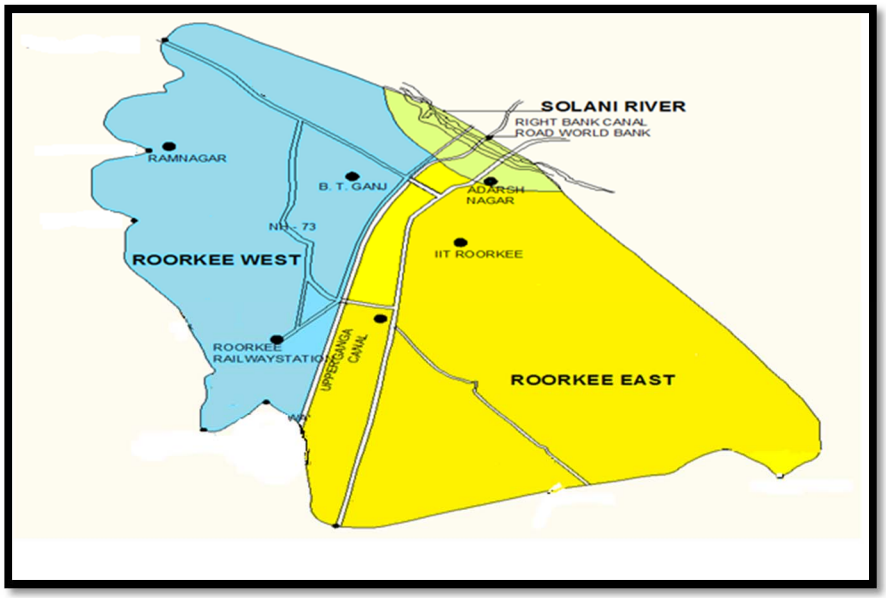



**Fig. 1 Roorkee city division into three main areas i) Roorkee East ii) Solani River and iii) Roorkee West**

## METHODOLOGY

### Multi Channel Analysis of Surface Waves (MASW)

The set up used for investigating the subsurface materials is 9-channel MASW setup. For many applications, such as, microzonation and dynamic response analysis of sites, Multichannel Analysis of Surface wave (MASW) is increasingly being used in earthquake geotechnical engineering. It is generally used for the estimation of shear wave velocity and dynamic properties. Moreover, subsurface material boundaries identification and spatial variations of shear wave velocity can also be measured. In this technique, the recorded Rayleigh waves are transformed into the frequency domain from the time domain using the Fast Fourier Transform (FFT) techniques. The corresponding phase difference $\Delta\Phi(f)$ between each receiver pair of channels is computed for every component of frequency measured. The travel time $\Delta(t)$ between receivers is computed using the following relation

$$\Delta t(f) = \Delta\Phi(f)/2\pi f \quad [1]$$

The distance between each pair of receivers $\Delta d=d_2-d_1$ is known which can be used in estimation of the Rayleigh wave phase velocity [$V_R(f)$] as follows:

$$V_R(f) = \Delta d/\Delta t(f) \quad [2]$$

After the estimation of Rayleigh velocity, the wavelengths ($\lambda_R$) is determined and dispersion curve can be plotted between $V_R$-$\lambda$ or $V_R$-depth.

### Micro Tremor Method

Nakamura (1989) proposed for the use of microtremors for microzonation and in the studies of local site effects, and, is therefore also known as Nakamura technique. In the Horizontal-to-Vertical Spectral ratio (HVSR) technique, a sharp peak of the H/V spectrum at a particular site corresponds to the fundamental frequency of the sediments. In the present scenario, it is widely accepted that this frequency of the HVSR peak reflects the fundamental frequency of the underlying sediments and the amplitude is mainly dependent on its impedance contrast with the bedrock. The major advantages of HVSR method included its low-cost and simple measurement technique and providing the direct estimates of the resonance frequency of sediments. The shear-wave velocity structure cannot be obtained directly from the microtremor HVSR method, but can be derived by synthetic modelling of spectral ratio curve.

It can be inferred from the various researches carried out in the field of H/V spectrum that by applying the inversion technique of H/V spectrum, shear wave velocity structure can be obtained by estimating the thickness of every soil layer. H/V can be efficiently used to estimate the shear wave velocities up to bedrock if there is some methodology adopted to constraint the shear wave velocity of shallow layers (Castellaro and Mulargia, 2009). Parolai et al. (2005) used the surface wave technique for constraining the estimation of shear wave velocity in shallow layers. Hence, in the present study, the same methodology has been adopted, where for constraining the shear wave velocity profile in shallow layers MASW technique is used and for deeper depths HVSR technique is used for the estimation of shear wave velocity.

## DATA ANALYSIS

Grilla (Micromed S. p. A., 2008) software is used for the analysis of the data recorded in the field using Soil Spy Rosina and Tromino as data acquisitions systems for MASW and Microtremor respectively. This software is used to plot the dispersion curves from the records obtained from Soil Spy Rosina which are quite useful for the estimation of shear wave velocity profiles of shallow soil. In the same way, Tromino

records are used to obtain the HVSR curves based on ambient noise records. The HVSR curves obtained are used for the determination of shear wave velocity profiles in deeper layers of soil. After obtaining both the curves, i.e., dispersion curves and HVSR curves, Join Fit module in the Grilla software is utilized where both the curves are placed together simultaneously for the estimation of shear wave velocity profile of deeper layers of soil. In this Joint Fit Module, the basic steps performed are as follows:

i. The dispersion curve is best fitted by setting up an initial synthetic model from estimated shear wave velocity profile having high impedance contrast of half space.
ii. The depth is estimated from the lowest frequency that is in the dispersion curve.
iii. Further analysis is carried out by HVSR curve. It is assumed that each peak corresponds to soil strata having impedance contrast is related to the frequency at which it is observed.
iv. The depth of every soil layer is estimated from the velocity and resultant frequency of HVSR curve. Thus, the estimated parameters generate a synthetic curve and HVSR curve. If there is sound matching between both the curves, then, final profile of the soil strata is obtained.

## SHEAR WAVE VELOCITY PROFILES

The dispersion curve obtained from the record is used to estimate shear wave velocity of the soil by the inversion technique. In the inversion technique, least square approach is employed (Xia et al. 1999) and the shear wave velocity is updated after every iteration. Figure 2 shows the shear wave velocity profiles of all the six sites obtained from the inversion technique. It can be seen from the Figure that at Site 1, the estimated shear wave velocity up to 9m is 190m/s and up to 20 m, it is found to be 250m/s. At Site 2, the topmost 9m has shear wave velocity of 180m/s which increases to 240m/s up to 20m. Similarly at Site 3, the top 9m of soil is having shear wave velocity of 192m/s, which then increases to 260m/s up to 20m depth. At Site 4, the top 2m has shear wave velocity of 186m/s and 300m/s up to 30m. At Site 5, it is seen that the top 2m soil layer is having shear wave velocity of 145m/s which increases to 280m/s up to 26m depth. Finally at Site 6, it is seen that the top 2m soil layer is having shear wave velocity of 186m/s which increases to 300m/s up to 26m depth. It is observed that for Site, Site 2 and Site 3, shear wave velocity profiles are very close to each other as the site are located at a distance of about 1.5 km from each other and with a hard subsoil conditions. On the other hand, Site 4 and Site 5 (Roorkee West) are located on less hard sub soil and also at such a place, where, heavy traffic passes and caused hindrance in the data collection. Site 6 is located on soft soil very near to Solani River and thus a variation is seen in the shear wave velocity profiles among other sites. Table 1 represents the $V_{S30}$ values for the divisions of the study region and also the bedrock depth.

**Table 1 $V_{S30}$ and Bedrock Depth**

| Serial Number | Site Name | $V_{S30}$ (m/s) | Bedrock Depth (m) |
|---|---|---|---|
| 1 | Roorkee East | 357 | 132 |
| 2 | Roorkee West | 273 | 137 |
| 3 | Solani River | 274 | 285 |

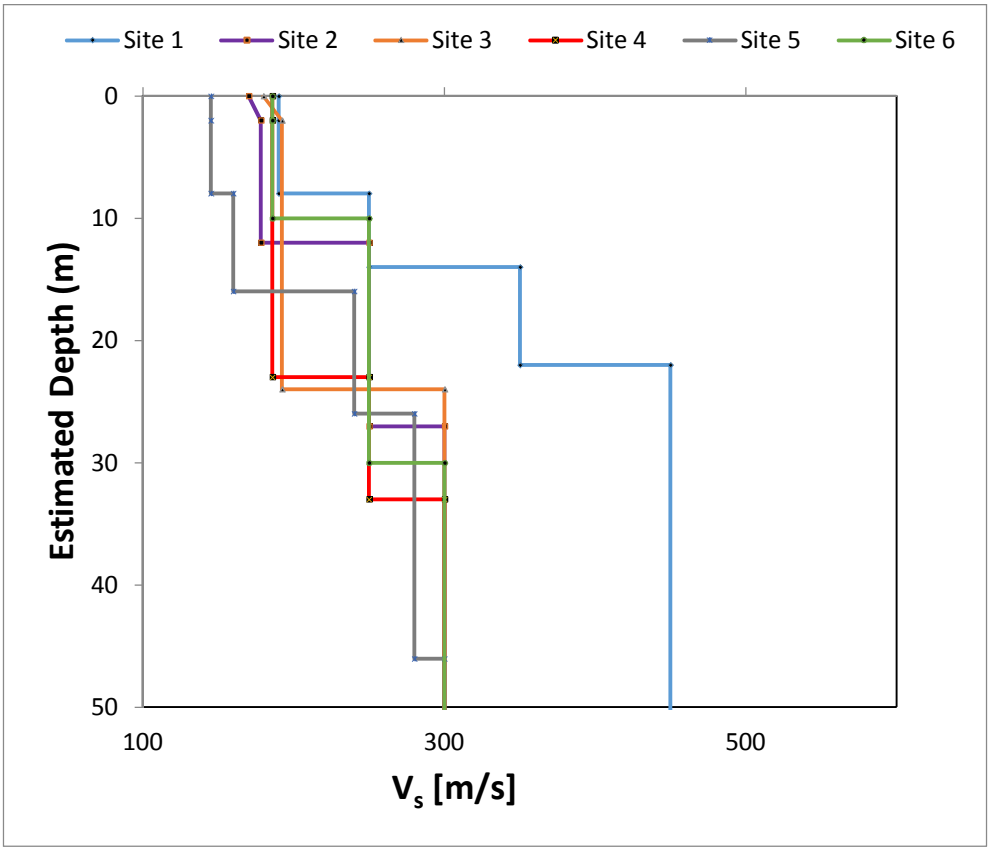



**Fig. 2 Shear wave velocity profiles of all the sites**

## GEOTECHNICAL INVESTIGATIONS

The information for the soil characterization from bore holes data has been obtained from the construction division of IIT Roorkee in respect of those bore holes that were drilled for installation of tube wells in the campus. The bore hole information is available up to a depth of about 150 m. The bore holes indicate that initially there is a layer of poorly graded sands, gravely sands with little or no fines up to a depth of about 45 m, which is over laid on a layer of inorganic clays, gravelly clays, sand clays, silty clays and lean clays of low plasticity of 10 to 15 m thickness. The N values were used for estimating the shear wave velocities for various types of soils using empirical relationships developed by various authors [Shibata (1970), Seed and Idriss (1981), Lee (1990), Hasancebi and Ulusay (2006) etc.]. Table 2 shows the average shear wave velocity obtained by using the Shibata (1970) and Seed and Idriss (1981) empirical relationships between the N-$V_s$.

**Table2 Average Shear Wave Velocity from empirical relations**

| S. No. | Division | SPT (N) Range with depth | Average Shear Wave Velocity ($V_{S30}$) | | | |
|---|---|---|---|---|---|---|
| | | | All Soils | Sand | Silt | Clay |
| 1. | Roorkee East | 9-15 | 350 | 330 | 300 | 320 |
| 2. | Roorkee West | 6-14 | 260 | 230 | 250 | 255 |
| 3. | Solani River | 5-13 | 280 | 230 | 290 | 250 |

## COMPARISON OF GEOPHYSICAL AND GEOTECHNICAL METHODS

It is a common practice in a geotechnical investigations to perform the SPT tests to determine the strength of soil. But, this method is highly expensive and requires boreholes drilling at the sites which makes it more difficult. To overcome its disadvantages, geophysical methods proves to be a better alternative to geotechnical engineers. They are non-invasive and can be performed in a lesser time to estimate the shear wave velocity profiles. From Table 1 and Table 2, it can be observed that the estimated shear wave velocity for the sites is in good agreement from the geophysical and geotechnical tests. Fig. 3 shows the comparison of both the methods with depth. It can be observed from the Fig. that the shear wave velocity obtained from the empirical relations is on lower side as compared to the geophysical methods. However, both the methods show a good agreement for the estimation of shear wave velocity and hence geophysical methods can be used for the site characterization of a site consuming less time.

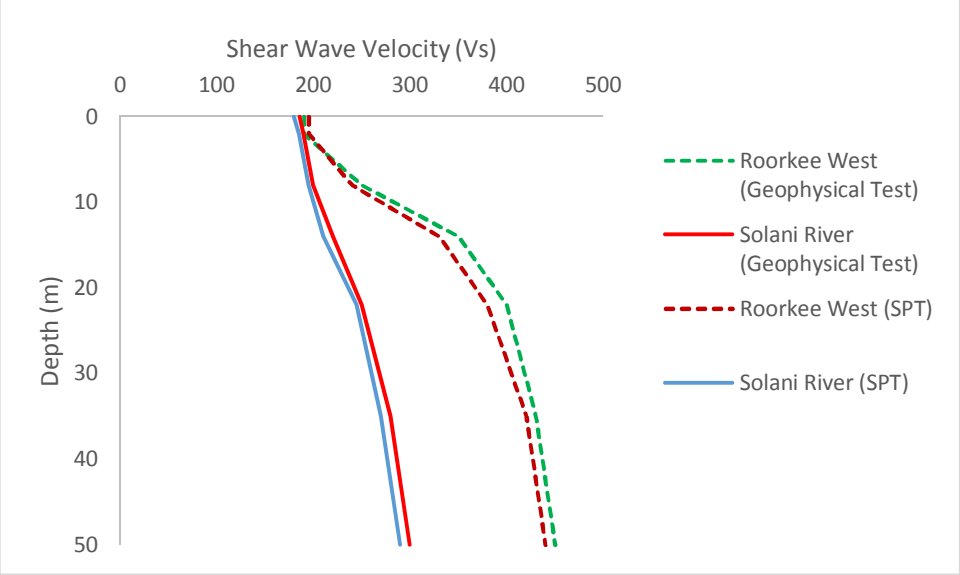



**Fig. 3 Comparison of geophysical and geotechnical methods**

## CONCLUSIONS

Shear wave velocity is an important parameter for understanding the dynamic behavior of soil. Moreover, it can be used for the determination of shear modulus (G) of soil as well as for the site characterization applications of geotechnical earthquake engineering. In this study, shear wave velocity profiles were determined at different sites using the geophysical methods, namely, MASW and Microtremor measurements. A comparison of these values is made with the geotechnical investigations in the region carried out by SPT tests. A good agreement is seen in these methods and thus, non-invasive and less expensive geophysical methods can be used at sites for its characterization. Hence, both the geophysical and geotechnical investigations are complementary to each other and results of one method can serve as a guideline for the other method.

## REFERENCES

Bureau of Indian Standards, 2002. IS 1893 (Part I): Indian Standard Criteria for Earthquake Resistant Design of Structures, Bureau of Indian Standards, New Delhi, India.

Micromed S. p. A., 2008. An Introduction to the Phase Velocity Spectra Module in Grilla, *Micromed S.P.A.,* Treviso, Italy.

Parolai, S., Picozzi M., Richwalski S. M., and Milkereit C., 2005. Joint inversion of phase velocity dispersion and H/V ratio curves from seismic noise recordings using a genetic algorithm, considering higher modes, Geophys. Res. Lett. 32, L01303, doi: 10.1029/2004GL021115.

Castellaro S., and Mulargia F., 2009. $V_{S30}$ estimates using constrained H/V measurements, Bull. Seismol. Soc. Am. 99, 761–773.

Xia, J., Miller, R. D., Park, C. B., 1999. Estimation of near surface shear wave velocity by inversion of Rayleigh waves, Geophysics, 64(3), 691-700.

Seed H. B. and Idriss I. M., 1981. Evaluation of liquefaction potential sand deposits based on observation of performance in previous earthquakes. ASCE National Convention, Missouri,81–544

Lee S. H. H., 1990. Regression models of shear wave velocities in Taipei basin. J Chinese Inst Eng 13:519-532

Shibata, T., 1970. Analysis of liquefaction of saturated sand during cyclic loading, Disaster Prevention Res. Inst. Bull., 13, 563–70.

Hasancebi, N. and Ulusay, R., 2006. Empirical correlations between shear wave velocity and penetration resistance for ground shaking assessments, Bull Eng Geol Environ, 66, 203-213.

# EFFECTS OF MIXING OF BENTONITE SUSPENSION IN GEOTECHNICAL APPLICATIONS

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## ABSTRACT

Bentonite suspension is used as stabilizing fluid to support pile borings, excavation trenches and for cement grouting in many geotechnical applications. Bentonite suspension can also be used with cementitious material as a binder to create self-hardening mixtures such as grouting mortars for construction of cut-off walls. Effective usage of bentonite suspension and grout is strongly influenced by the fluid properties such as thixotropy and viscosity. These are dependent on various factors such as the proportion of bentonite to water, its hydration time, temperature, bleed, mixing methodology, additives etc. When bentonite suspension is being prepared, the objective is to achieve maximum hydration of the bentonite with suitable gel formation which can be achieved by effective mixing. Greater agitation, longer mixing and more hydration results in good quality bentonite suspension. This paper discusses the effects of mixing for various proportions of bentonite to water and bentonite to cement suspensions. From the results of laboratory and field tests conducted in large infrastructure projects, it is understood that with the % of bentonite usage and addition of admixture, properties such as viscosity and bleed can be reduced in the cement- bentonite suspensions. This in turn is primarily dependent on the use of high shear mixing and proper hydration of bentonite.

**KEYWORDS:** bentonite, mixing methodology, viscosity, density, hydration time, bentonite-cement grout

## INTRODUCTION

The key functions of a support fluid are to maintain stability of excavation, flow easily as a liquid with appropriate properties, readily displace by concrete, retain properties over time and not react with concrete. Selection of good quality of support fluid is chosen based on the project site environment, type of equipment used, excavation method, soil type, ground water level, quality of water available etc. Bentonite suspension is commonly used as support fluid to stabilize excavations as it does not flow much into the soil beneath the excavation, suspends waste and prevents sludgy layers building up at the base of the excavation. Good quality bentonite creates low permeable barrier and retards the flow of ground water. Also, bentonite is relatively eco-friendly compared to other support fluids such as polymers (EFCC/DFI, 2019).

Preparation of good quality bentonite and its successful requires understanding and management of its properties. Though bentonite suspension is commonly used for stabilisation of pile bore, establishment of its properties and proper use of bentonite is to be ensured to obtain right quality of pile as well as to save time and achieve economy (Bhattacharya, 2007). In the process of preparation of bentonite suspension, effective mixing and appropriate mixing equipment activates the bentonite resulting in cost effectiveness of the raw material and mixing time (Jefferies, 2013; EFCC/DFI, 2019). Also, hydration time depends on the level of shear a mixer induces. If enough hydration time is not provided for the bentonite, higher concentrations of bentonite (by weight of water) is necessary or additive is to be used to prevent bleeding. This paper details the effect of mixing and hydration of bentonite on various proportions of the bentonite suspension and grout for its usage in heavy foundations and grouting projects.


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## BENTONITE FOR STABILISATION

Bentonite is a very fine-grained clayey material having high percentage (>85%) of montmorillonite and is available as sodium and calcium based. Calcium based bentonite is altered to sodium based for the specific purposes such as stabilization of vertical cut soil surface, like bored piles, diaphragm walls, oil wells, walls of trenches etc. Sodium based bentonites have high swelling, since the valence of sodium cation is less than that of calcium therefore has high liquid limit than other clay materials even quite more than calcium based bentonite. Usually, bentonite forms a seal on the walls of the soil by either of the three different mechanisms namely surface filtration, deep filtration and rheological blocking.

Surface filtration occurs when a filter cake is formed by the bridging of hydrated bentonite particles at the entrance to the pores in the soil, with negligible penetration of the bentonite into the soil (EFCC/DFI, 2019). Deep filtration occurs when slurry penetrates the soil, slowly clogging the pores and building up a filter cake within them. In this case, the seal may penetrate the soil about 40 to 50 mm. where the concentration of bentonite in the filter cake is greater than in the slurry (typically 15% for a slurry containing 5% bentonite). Rheological blocking occurs when slurry flows into the soil until it is restrained by its shear strength. In this case the slurry may flow several metres into the soil. Of these three mechanisms, surface filtration is preferred, since the seal is formed very rapidly with little or no penetration of bentonite into the soil.

## MIXING OF BENTONITE

For stabilisation of trenches and borings using bentonite by surface filtration, good quality bentonite suspension is required. When preparing bentonite fluid, important parameters such as water quality, mixing time, shearing energy, fluid circulation in storage tank shall be taken care. As discussed in the earlier section, suitable bentonite suspension (also referred as slurry/fluid) can be obtained by effective mixing and adequate hydration. Mixing involves initial wetting, hydration and homogenisation. Bentonite is to be prepared using a high shear mixer in hydration tanks or ponds with fluid re-circulation system. During hydration, bentonite swells, and this helps to achieve the required properties of fluid.

For mixing, it is generally preferred to use water with low to medium electrolyte contents as it affects the dispersion and swelling properties of bentonite. The process of delamination of clay particles to form interparticle bonds with water improves the viscosity and gel strength of bentonite. Remixing after 24hrs of original mixing of bentonite fluid is also required for early development of gel strength (i.e. viscosity). However, apparent viscosity is dependent on the original mixing speed (Jefferies, 1982).




**Fig. 1. High shear bentonite mixer and storage tanks for large infrastructure project**

Usually, the bentonite powder is added through a simple venturi-hopper or directly into a high shear mixer (see Fig. 1). The mixing equipment generates enough shear to ensure that all the individual bentonite particles are dispersed in the mixing water. Besides this method bentonite is also mixed by passing it through a funnel which connects at the bottom to a high pressure water jet (using water motor) which leads to the bentonite tank. Fig. 2 illustrates schematic of bentonite mixing process.

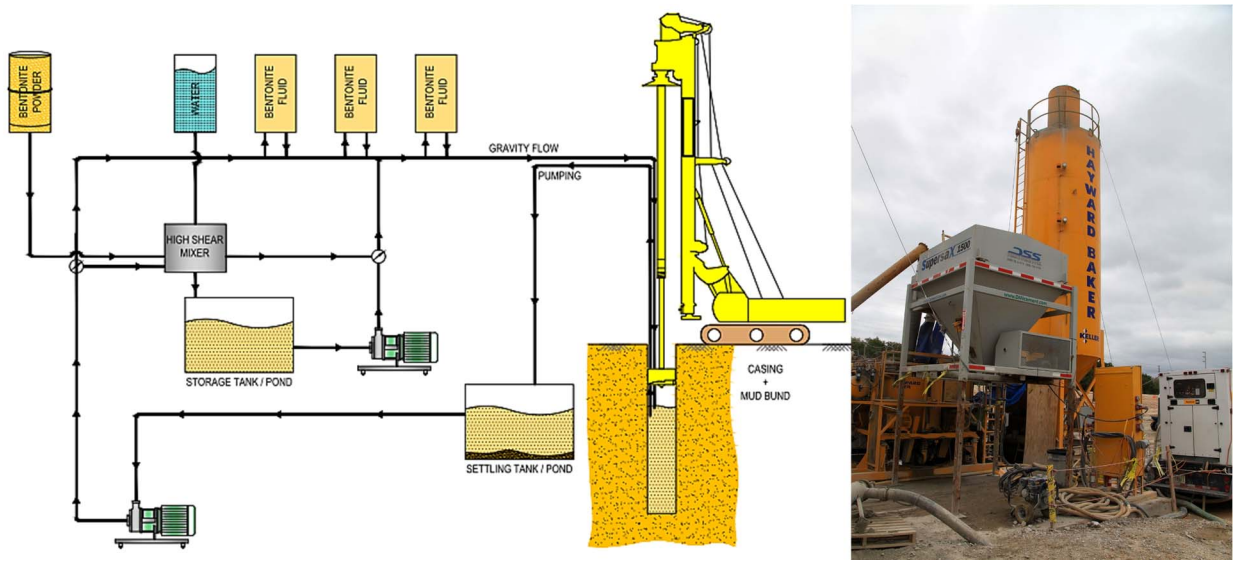



**Fig. 2. Schematic of bentonite mixing process**

The proportion of bentonite powder to water to be added during the mixing depends on the quality of the bentonite and the required viscosity of the slurry. For most applications, concentrations between 3% and 5% by weight are usually practiced.

### *Agitation tanks*

Agitation tanks are used for continuous mixing of bentonite cement grout to prevent setting of cement. Typical cross section of agitation tank is shown in Fig. 3.

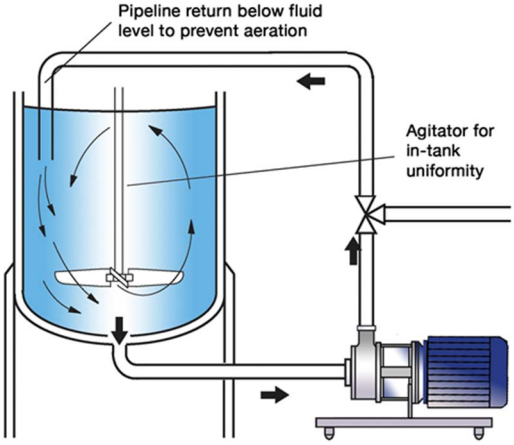



**Fig. 3. Schematic of agitator used for bentonite-cement mixing process**

The main function of agitation tank is:

1. to release any air bubbles trapped during the mixing process
2. to keep the mix in suspension prior to pumping
3. to allow bentonite particles to be separated with water and cement

## HYDRATION AND STORAGE OF BENTONITE

After mixing, the slurry is normally stored in tanks or lagoons where it is kept agitated by circulating through pumps or by injecting compressed air while further hydration takes place. Hydration is time dependent and will continue for many days, but the rheological properties of the slurry will approach limiting values within a few days after mixing. The slurry is normally stored for at least 12 hours before its usage. Typical arrangement of storage of bentonite for the hydration process is shown in Fig. 4.

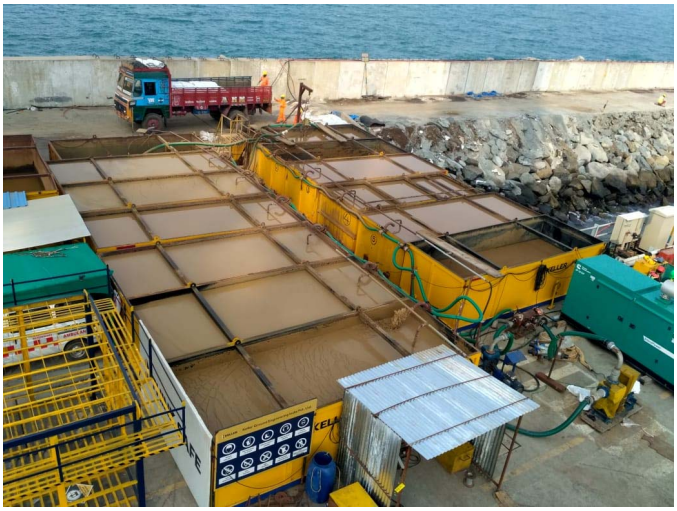


**Fig. 4. Bentonite storage tanks for hydration purpose**

## PERFORMANCE REQUIREMENTS

The quality, performance and properties of bentonite fluid vary widely based on its physical and chemical properties, water used, type of application, ground conditions, type of foundation and construction methodology. $P^H$. density, viscosity, sand content, silt content, filter loss, oxygen demand etc. are to be checked during the preparation of the fluid and during different construction activities/stages. Characteristics of bentonite suspension for usage at different stages are presented in Table 1. Following discussion on some of the characteristics of bentonite suspension are based on the testing on sodium-based bentonite.

**Table 1. Typical characteristics for bentonite suspensions (EFCC/DFI, 2019)**

| Property | Stages of usage | | | Test equipment (see Fig. 5) |
|---|---|---|---|---|
| | Fresh | Ready for re-use | Before concreting | |
| Density, gm/cc | < 1.10 | < 1.25 | < 1.15 | Mud balance |
| Marsh cone viscosity, sec | 32 to 50 | 32 to 60 | 32 to 50 | Marsh funnel |
| pH | 7 to 11 | 7 to 12 | NA | pH meter |
| Sand content, % | NA | NA | < 4 | Sand content set |
| Filtration loss (after 30min), ml | <30 | <50 | NA | API fluid loss test |
| Filter cake thickness (at 30min), mm | <3 | <6 | <3 | API fluid loss test |

### *pH*

$p^H$ is used as a quality control parameter for the bentonite as delivered to site. Most fresh slurries made with bentonite converted from calcium form to sodium will have a $p^H$ in the range of 9.5 to 10.5. Used slurries, unless contaminated by cement, often have a slightly lower $p^H$ than their fresh counterparts. Cement contaminated slurries may have very high $p^H$ values i.e., 11.5 to 12.5. Various field control testing on bentonite fluid is presented in Fig. 5.

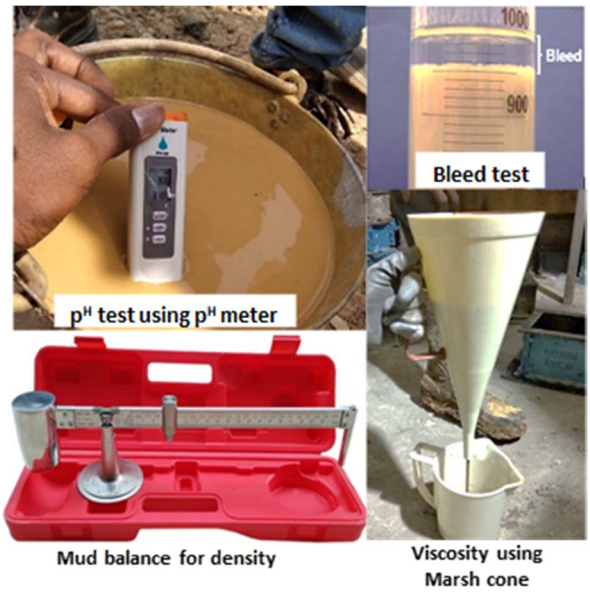



**Fig. 5. Field control testing on bentonite fluid**

### *Density*

Density should be enough to provide the necessary hydrostatic force required for the structural stability. If the density of bentonite is too high, workability is impaired. The standard instrument for density measurement is the mud balance. Fig. 6 shows the variation of density with the increase in percentage of bentonite by weight measured in field laboratory for different sources of bentonite.

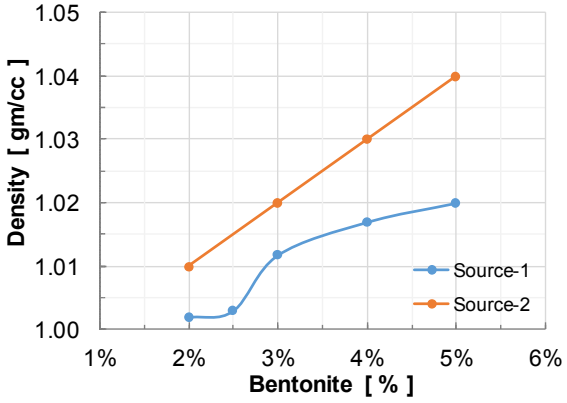



**Fig. 6. Variation in density with % of bentonite by weight**

***Bleeding***

In bentonite slurries, bleeding is a self-weight consolidation process that continues until the slurry has enough strength to support its own weight, this may also happen over a very long-time scale. Bleeding of bentonite slurries should be small (< 3%), once the slurry has been hydrated for about 24 hours. Severe bleeding in hydrated slurries normally suggests that the slurry mix is unstable. Usually, bleeding is measured on graduated cylindrical jar for a minimum observation period of 1 hour. Influence of mixer types on bleed can be seen in Fig. 7.

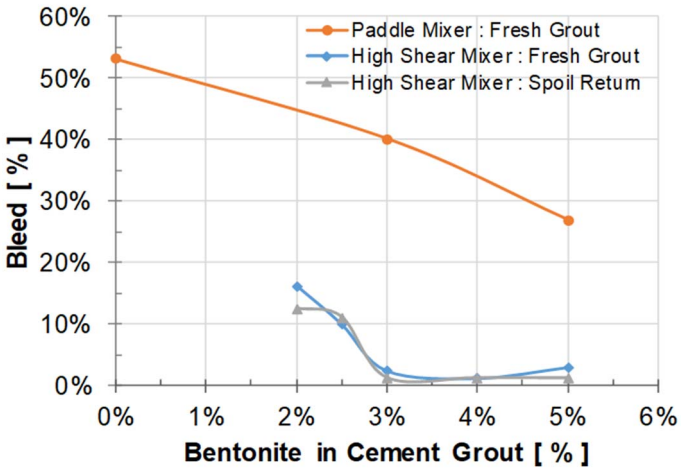



**Fig. 7. Influence of mixer types on bleed**

***Viscosity***

Bentonite slurry should have a minimum Marsh funnel viscosity of about 32 seconds (946 ml test volume) for working in sand to clayey soils. When working in gravel Marsh funnel viscosity of 40 to 50 seconds may be required to limit the filtration depth into the strata. De-sanding operation becomes difficult, if Marsh funnel viscosity exceeds 50 seconds. Also, pumping of bentonite slurry for long distances can become difficult and inefficient if the viscosity of the slurry is too high. Suspended soil particles can be removed from a slurry more easily if the slurry has low viscosity. Viscosity of bentonite slurry varies with concentration of bentonite, mixing mechanism, mixing speed, additives/chemical adjusters, re-mixing etc (Fig. 8). Variation of the viscosity of bentonite slurry with respect to hydration time is presented in Fig. 9.

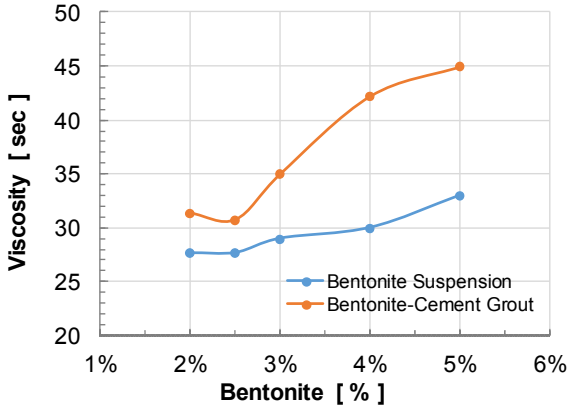



**Fig. 8. Variation of viscosity of bentonite slurry & bentonite grout**

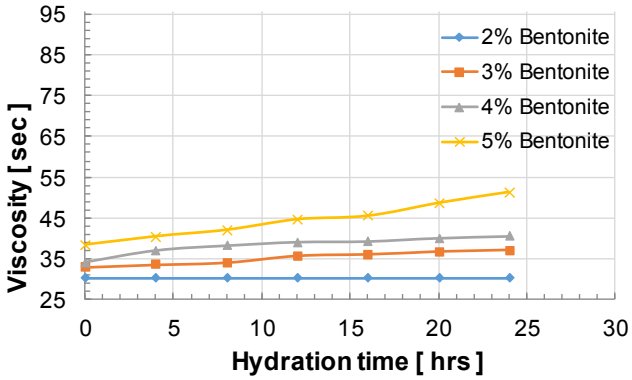



**Fig. 9. Variation of viscosity of bentonite**

In many occasions, due to lack of storage facilities for mixed bentonite slurry, additives are used to accelerate hydration time. Several laboratory and field tests were conducted to check variation viscosity and summary of the observed results are presented in Fig. 10 for different percentage of additives.

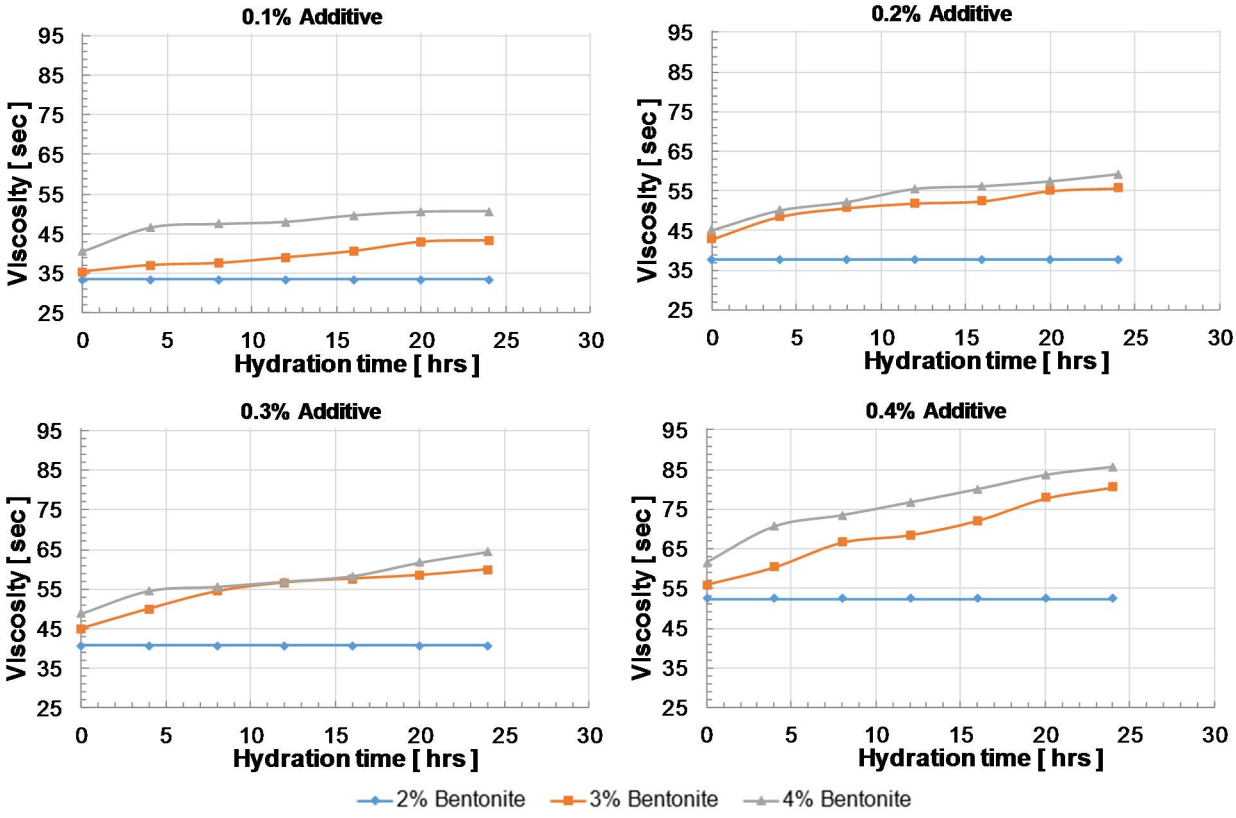



**Fig. 10. Viscosity variation with different % of additives and concentration**

## CONCLUSION

Based on the experiences in successful usage of bentonite as stabilizing fluid and grouting in large infrastructure projects, it is understood the following aspects shall be critically controlled.

1. Mixing of bentonite play a vital role in properties such as density, viscosity and bleed of the stabilizing fluid such as bentonite.
2. Bentonite suspension must be hydrated for minimum 12 hours after its mixing for the best results. This depends on water quality, mixing time, mixing type, shearing energy and fluid circulation. High shear mixers are recommended.
3. Bentonite suspension achieves viscosity with the hydration time. This gel formation helps for stability of trench.
4. The effect of viscosity of bentonite suspension varies with proportion of water to bentonite. Lower bentonite content leads to collapse of trench. Additives help improve the viscosity of the bentonite slurry and shall be chosen appropriately based on the encountered soil conditions and project site environment.

## ACKNOWLEDGEMENTS

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## REFERENCES

1. Bhattacharya, D. (2007). Collapse of soil in pile bore hole and its stabilisation using bentonite suspension. In Indian Roads Congress Technical papers (pp. 243-252).
2. EFFC/ DFI. (2019). Guide to support fluids for deep foundations.
3. FPS (Federation of Piling Specialists). (2006). Bentonite support fluids in civil engineering.
4. Jefferis, S. A. (1982). Bentonite-cement slurries for hydraulic cut-offs. In Proceedings, Tenth International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Sweden (Vol. 1, pp. 435-440).
5. Jefferis, S. A. (2013). Grouts and slurries. Construction materials reference book, 165-194.

# FORENSIC EVALUATION OF DEFECTIVE BORED PILES OF AN INDUSTRIAL PLANT

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## ABSTRACT

Settlement was observed in piles installed for an industrial facility in Haryana. The paper presents results of forensic evaluation of two piles. Low strain pile integrity tests (PIT) and parallel seismic tests (PST) were used to effectively identify the defects. To physically confirm the defects, coring through the pile concrete was done. The study effectively identified the nature of the defects and the depth at which the piles had major discontinuities.

**Keywords: defective piles, geotechnical forensic evaluation, pile integrity test, parallel seismic test, pile-concrete coring**

## INTRODUCTION

Bored piles of 600 mm diameter extending to 16 m depth below cut-off level of 1.3 m below GL had been installed for an industrial plant in Haryana. The piles were designed for an axial compression capacity of 50 tonnes. During the erection of the steel structure for the pre-engineered building, settlement was observed in some of the pile caps. These pile caps had two piles each below them.

A forensic investigation was undertaken by the authors to help the owner evaluate the severity of the problem and to develop remedial measures. The paper presents results of geotechnical investigation conducted at the site, coring through piles to assess the concrete quality and geophysical tests conducted to assess the problem. Results of the investigation on two piles (Pile Nos. 238 and 239) are detailed here to illustrate the methodology to evaluate the pile quality.

## PROJECT DETAILS

Loose sandy silt and silty sand was encountered to about 10-12 m depth at a factory for a heavy industry in Haryana. To transfer the loads safely, RCC piles of 600 mm diameter were installed to 16 m depth below the cut off-level of 1.3 m depth to support the pre-engineered buildings. Each pile group had two piles with each pile designed for an axial compression capacity of 50 tonnes.

During the erection of the steel structure for the factory, settlement was observed in some of the pile caps even before the full load was transferred through the columns. To evaluate the problem in detail and to reconfirm the stratigraphy, a geotechnical investigation consisting of boreholes and static cone penetration tests was performed in the vicinity of the piles that experienced failure.

This was followed up with geophysical tests that included low strain pile integrity tests and parallel seismic tests. Concrete coring through the piles provided the clinching evidence of the defects in the piles.

## SITE STRATIGRAPHY

The soils at the site are alluvial in nature and consist primarily of sandy silt of low plasticity from the ground surface to 9-12 m depth underlain by silty sand to 21 m depth. Sandy silt is met below the sand stratum to 30 m depth. Groundwater was encountered at about 5.8 m depth. SPT values are low to about


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10-12 m depth with some values as low as 1-4. Figure 1 presents typical borehole data drilled in the vicinity of the failed piles investigated. Fig. 2 presents photograph of borehole in progress in the factory premises.

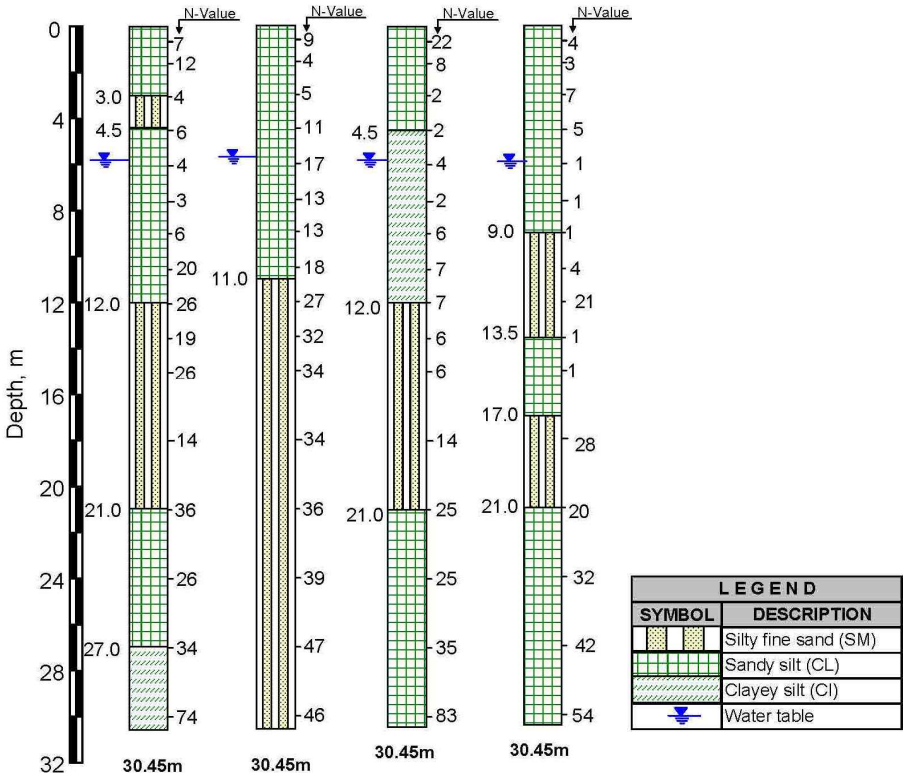



**Fig. 1. Typical Borehole Data**



**Fig. 2. Borehole in progress inside the factory structure**

The cone tip resistance values also show a similar trend with low $q_c$ values to 10-12 m depth. Figure 3 presents a photograph of the static cone penetration in progress. Plots of cone tip resistance versus depth are presented on Fig. 4.



Fig. 3. Static cone penetration test in progress

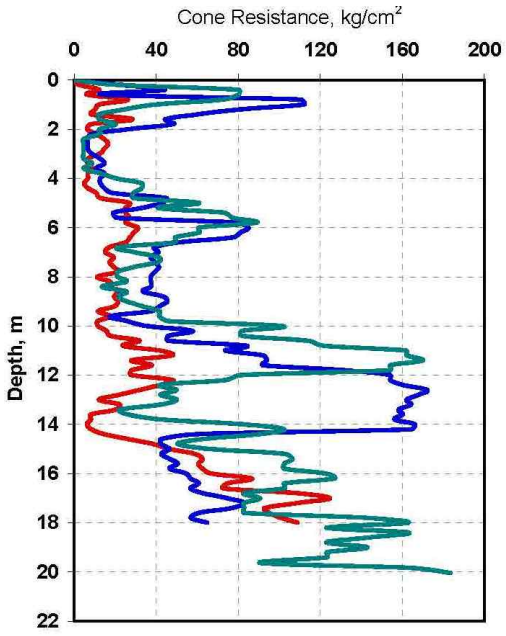



Fig. 4. Cone tip resistance versus depth

## NON-DESTRUCTIVE TESTS TO ASSESS PILE QUALITY

*Low Strain Pile Integrity Tests*

Low-strain pulse echo response test (pile integrity test, ASTM D 5882-00) using a handheld hammer was initially tried out by excavating a pit adjacent to the pile cap to expose the pile and impacting it from the side. But since the response was not good, the test was again carried out after dismantling the pile cap (Rausche et al, 1988). A small portion of the exposed pile top was leveled, and an accelerometer was placed on it. The hammer was impacted on the pile. A photograph of the test in progress is presented on Fig. 5.




Fig. 5. Pile integrity test in progress

The evaluation of PIT records was conducted in the time domain using the Pulse-Echo method. The pile top velocity is displayed as a function of time with an exponentially increasing magnitude such that the pile toe reflection is enhanced. The averaged, amplified velocity (in cm/s), averaged for several impacts, is presented on Figs. 6 and 7 as a function of the pile length. The input compression wave from the

hammer is reflected from pile toe (or a change in cross sectional area or pile material quality) and returns to the pile top at a time related to the speed of travel of the wave in the pile material. Pile length is computed based on the assumed wave speed (c) and measured wave arrival time (t) using the following relation:

$$c = 2L/t$$

PIT results for Pile 238 are presented on Fig. 6 for pile impacted from the side and on Fig. 7 for pile impacted on the top. Similar data for Pile 239 are presented on Figs. 8 and 9 respectively.

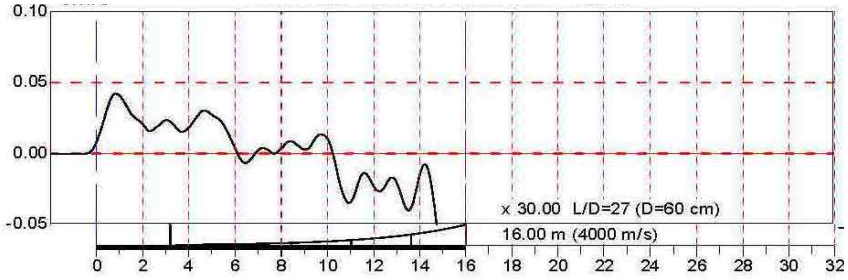



**Fig. 6: Typical PIT record - Pile 238 impacted from side**

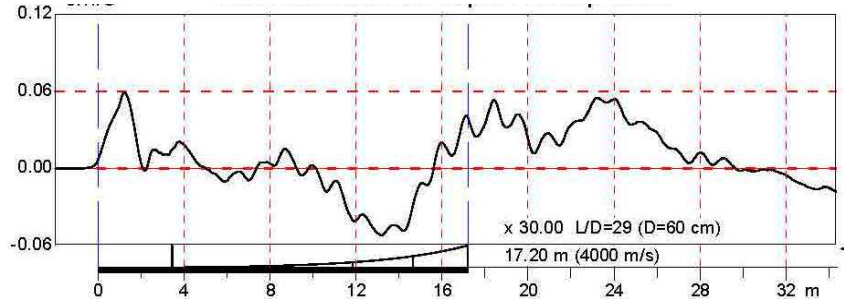



**Fig. 7: Typical PIT record - Pile 238 impacted from top**

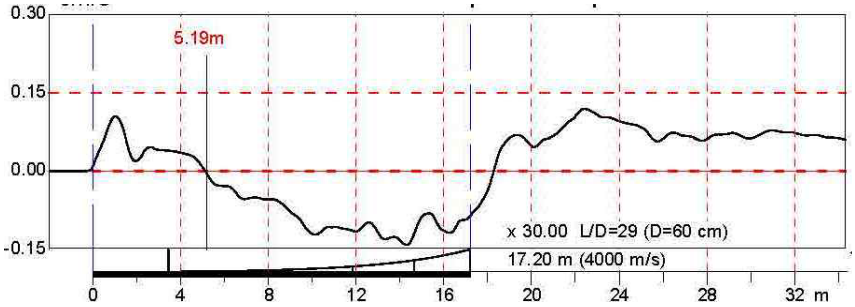



**Fig. 8: Typical PIT record - Pile 239 impacted from side**

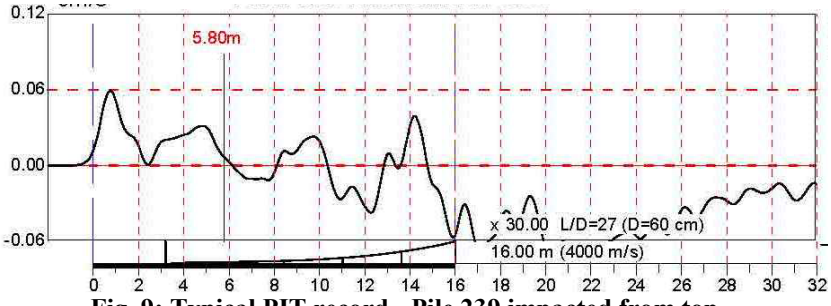



**Fig. 9: Typical PIT record - Pile 239 impacted from top**

Table 1 presents a summary of the interpretations from the pile integrity tests.

**Table 1. Results of pulse echo low strain Pile integrity Test (PIT)**

| Pile No. | Test Condition | Number of Tests Attempted* | Observations and Interpretations |
|---|---|---|---|
| P238 | Test done by excavating a test pit and striking the pile top from the side | 6 | Poor data quality. Test inconclusive. |
| | Test done from top of pile cap | 4 | Poor data quality. Test inconclusive. |
| | Test done from pile COL after dismantling the pile cap completely | 17 | Strong reflection at 3.5-3.6m depth, possibly due to bulging at this depth |
| P239 | Test done by excavating a test pit and striking the pile top from the side | 7 | Data indicates either a defect at 6m depth below pile COL, or the wave could not penetrate below this level. |
| | Test done from top of pile cap | 15 | Weak toe reflection seen at the anticipated pile tip level. But uncertain and needs verification |
| | Test done from pile COL after dismantling the pile cap completely | 19 | Strong reflection at 3.5-3.6m depth, possibly due to bulging at this depth |

** Each test being average of 6 blows*

It is noted that the variation in pile cross-section at 3-4m depths coincides with the presence of groundwater table. Since the pile boring was carried out using manual winch equipment without liners, it is likely that there might have been enlargement in the pile bore diameter at the groundwater table depth.

Due to significantly strong reflections from 3.5-3.6 m depths in Piles 238 and 239, it was not possible to determine the pile integrity below this depth. The authors suggested that the pile be exposed for physical inspection at this depth. However, the project owners refused to allow for the same, and chose to install additional piles at this location instead.

*Parallel Seismic Tests*

To assess the depth of the piles, parallel seismic test was performed in accordance with ASTM D4428/D4428M-00. The test can give reasonable assessment of depth of foundation and concrete quality (Sanjay Gupta et al 2015). The test involves impacting the exposed pile to generate seismic wave energy that travels down the pile and is sensed by a geophone receiver in a nearby borehole.

For this, a 150-mm diameter borehole was drilled about 1 m distance from the pile. The borehole was cased with 75 mm diameter PVC casing. The annular space between the borehole wall and the casing was backfilled using cement-bentonite grout.

A 5-kg hammer with a Teflon tip was used to impact the pile top. Triaxial geophones were lowered in the cased borehole to receive the compression and shear waves traveling down the foundation. Data acquisition to pick up the first arrivals (Primary wave) was done at every 1 m depth interval up to the final borehole depth of 20 m. A schematic of the test set-up is illustrated on Fig 10. Figure 11 is a photograph showing lowering of geophone into the hole.

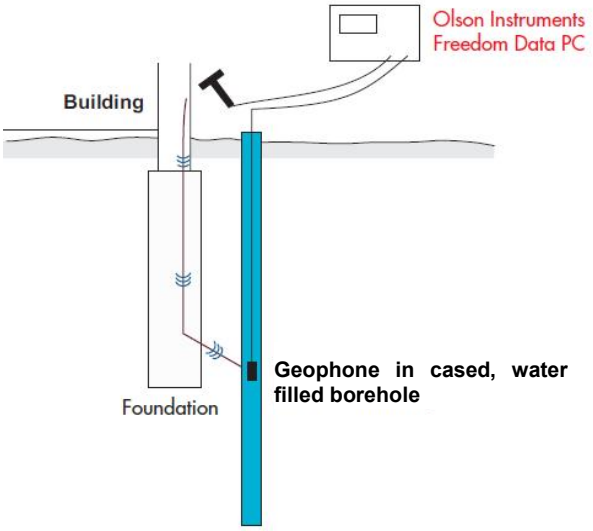



**Fig. 10. Schematic of parallel seismic test**



**Fig. 11. Borehole for parallel seismic test adjacent to pile cap**

Figure 12 presents a photograph showing the lowering of geophone into the borehole.

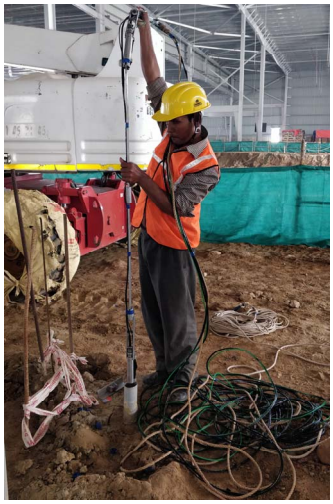


**Fig. 12. Geophone being lowered into borehole**

Figure 13 presents a plot of the measured wave velocity versus depth. The measured wave velocities are fairly low (~1000 m/s) in the top 12-15m depth, indicating that we are picking up tube waves- waves that travel down the water in the PS tube and not down the shaft. While there is no usable data obtained from P283, the sudden increase in measured wave velocity at P-239 at about 13.5m depth indicates a change of medium, i.e. earlier arrival of refracted waves travelling through the pile shaft, as compared to direct tube waves.

Table 2 presents the interpreted pile profile from the parallel seismic test.

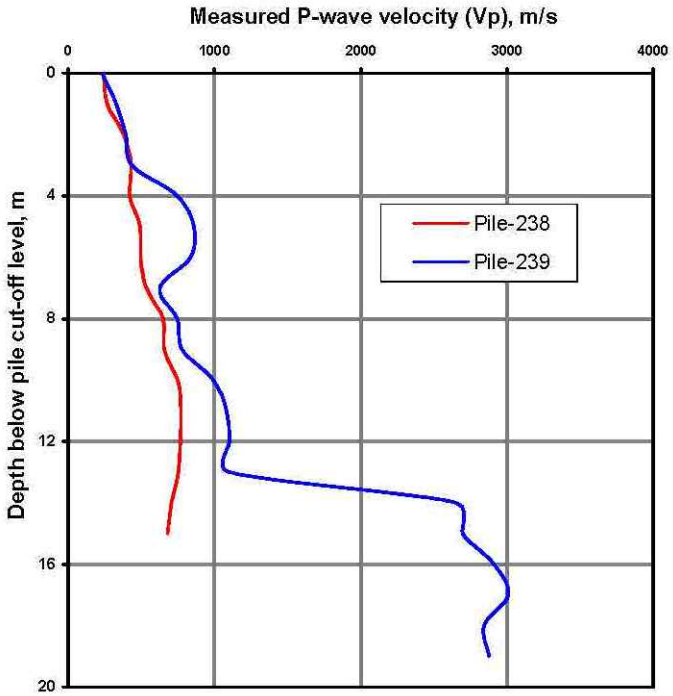



**Fig. 13. Parallel Seismic test: P-wave velocity vs. depth**

**Table 2. Interpretations from parallel seismic test**

| Pile No. | Possible Change in Impedance below Pile COL | Interpreted Pile Length below COL | Observations and Interpretations |
|---|---|---|---|
| 238 | ~ 5 & 11 m | Unclear | Pile length could not be ascertained due to poor quality PS data (possibly due to soil cavitation in the test vicinity) |
| 239 | ~ 3.5 m | Approx. 13.5 m | Some change in wave speed observed at 3.5m depth [matches PIT data].<br>Distinct change in wave velocity at 13.5 m depth [matches coring observations] |

## CORING THROUGH PILE

To confirm the pile quality and continuity, drilling was done through the piles using a hydraulic rotary core drilling rig. The drilling rig was set up at the pile-centre and leveled so as to drill vertically through the pile. A double tube core barrel with a 32-carat diamond impregnated core bit was used to maximize the core recovery and minimize mechanical fractures in the core. Fig. 14 presents a photo of the pile coring in progress.

Fig. 15 is a pictorial summary of the drilling records and the core recoveries and RQD values for Piles 238 and 239. The observations and interpretations based on the pile coring are discussed in Table 3.



Fig. 14. Coring through pile in progress

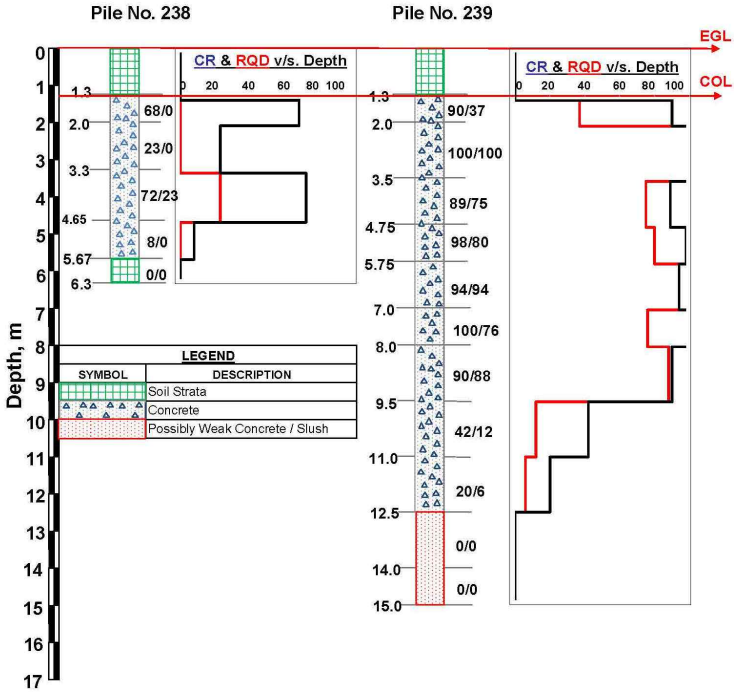



Fig. 15. Results of coring through pile

**Table 3. Observations and Interpretations from pile coring data**

| Pile No. | Maximum Explored Depth below COL | Depth at which soil was encountered | Observations and Interpretations |
|---|---|---|---|
| 238* | 6.3 m | 5.67 m | Bent longitudinal reinforcement pieces were found in last core run from 4.65-5.67 m depth. Soil was encountered in the pile bore at 5.67-6.3 m depth. and the coring was terminated in soil at 6.3 m depth.<br>The pile may be bent / damaged at about 5-6m depth. Alternatively, there may be a major defect (soil inclusion, necking, short pile) at this depth. |
| 239* | 15.5 m | 12.5 m | 0-9.5 m depth: Fairly good core recovery indicating expected pile concrete and continuity.<br>9.5-12.5 m depth: Core recovery % dropped significantly, which may be due to weak or discontinuous concrete.<br>Below 12.5 m depth: Nil recovery & RQD values.<br>Color of the return water was gray, *indicating that we were still drilling through concrete and not soil*. Possible presence of weak concrete or slush, possibly intermixed with soil.<br>Some reinforcement pieces (16 mm dia) were encountered in core run from 14 to 15m depth. Pile coring could not be extended below 15m depth due to reinforcement bars being encountered. |

| | |
|---|---|
| | Authors are of the opinion that the effective pile length may be only about 12.5m depth, below which weak concrete / pile bore muck is encountered, indicating poor bottom cleaning during pile installation. |

* *Expected pile length below cut-off level = 16 m*

## CONCLUSIONS

The study conclusively proved that the piles failed due to major defects in the piles. The combination of pulse echo low strain integrity test and parallel seismic test are effective non-destructive test methods to identify pile lengths as well as possible presence of defects / discontinuities. Coring through the concrete gave confirmatory evidence of the defects and pile quality. The combination of these tests is an effective forensic approach to assess the quality of the piles installed.

The findings and overall conclusions of the forensic investigation are summarized below.

***Pile 238***: Pile is bent or broken below 5.67 m depth, below which the testing results are uncertain due to variations in pile cross-section and possible cavitation in surrounding soil strata. The poor pile structural integrity, combined with the presence of loose soils in the pile vicinity, are a likely explanation for the pile settlement experienced at this location.

***Pile 239***: Effective pile length may be 12.5-13.5m, below which weak concrete / slush is encountered. Tests indicate poor bottom cleaning prior to pile concreting, which shall affect pile performance in-situ.

## REFERENCES

American Society for Testing and Materials. 2000. Standard Test Methods for Low Strain Integrity Testing of Piles, ASTM D 5882-00.

Rausche, F., Likins, G. E., Hussein, M.H., 1988. Pile Integrity By Low And High Strain Impacts, Third International Conference on the Application of Stress-Wave Theory to Piles, Ottawa, Canada; 44-55.

American Society for Testing and Materials. 2000. Standard Test Methods for Cross-hole Seismic Testing, ASTM D 4428-D4428M-00.

Sanjay Gupta, Ravi Sundaram, Sorabh Gupta 2015. NDT Techniques for Determining Depth of Foundations, Proceedings, Indian Geotechnical Conference IGC-2015. Pune.

Sanjay Gupta, Ravi Sundaram, Sorabh Gupta, (2017), Identifying Defects in Deep Bored Piles– Case Studies from India, Proceedings, 17th International Conference on Soil Mechanics and Geotechnical Engineering (17th ICSMGE), Seoul, Korea, 2637-2640.

# IMPLICATION OF SITE INVESTIGATION ON PILES SOCKETED IN WEATHERED ROCK

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## ABSTRACT

Bored cast-in-situ concrete piles socketed in rock offer best foundation solutions under situations involving heavy loads and under sub-surface conditions where a layer of loose soil overlies bedrock. The quality and reliability of rock data from the geotechnical investigation have become more important and the use of insufficient and incorrect data will have a huge impact on substructure design. This may lead to either uneconomical pile designs or on the contrary pose construction difficulties. This is well established in the development of one of the important major bridge structures across Balram River. This bridge in the state of Gujarat was designed as part of package 3R by the Dedicated Freight Corridor Corporation of India Limited. The initial stage investigation was performed by the poor quality of drilling resulted in poor core recovery and insufficient rock data. This has resulted in uneconomical geotechnical recommendations for substructure. Based on investigation data, the piling equipment mobilized at the site has difficulties to drill through this rock and form pile socket. Hence stage II investigation was proposed using hydraulic rig, double tube core barrel with controlled drilling at the site to get maximum core recovery. This improvement in investigation resulted in getting complete detail about the rock characteristics to plan suitable piling equipment & optimize the substructure design. Deploying suitable tools for investigation experienced drilling operator, performing drilling operation as per guidelines outlined in Indian Standards and capturing the drilling information is very important.

***Keywords: Forensic geotechnical engineering, weathered rock, Rock core sampling method***

## INTRODUCTION

Bored cast-in-situ concrete pile foundations are proposed for bridge structures located in rivers where the soil deposits underlain by rock were encountered. These piles are designed to resist loads from bridge structures and the effects of flowing water. The scour assessment in soil strata and weathered rock strata play a crucial role to derive the socket length of the pile. In these circumstances, getting a good quality of geotechnical investigation data is critical. Hence emphasis should be given in the geotechnical investigation stage to adopt good drilling procedures. Further based on the details of rock strata obtained from investigation, the contractor mobilizes the necessary tools for the pile socketing into rock. Poor quality of description of rock strata from investigation, results in uneconomical design and delay in project duration due to the change in the tools required for the pile socketing after initial mobilization at the site. Due to uncertainty in the behaviour of completely weathered to highly weathered rock strata and its variation along and across the bridge structure generally conservative designs in terms of socket length are recommended. However, shaft construction in a rock has an impact on project cost and an overdesign of socket length is not always advisable. Keeping this into consideration, the potential factors influencing the socket length need to be identified qualitatively which will help in optimizing the socket length into weathered rocks and hence to economize the pile design.


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This paper presents the quality of investigation data obtained from the initial stage of investigation and its effects on scour, pile toe levels and difficulties faced during the construction of the pile foundation. It is understood during pile construction that the rock strata encountered at the site are harder than expected from the data furnished in the initial geotechnical investigation. Hence confirmatory borehole investigation was proposed to investigate the rock characteristics at all the borehole locations. The investigation was carried out by using a hydraulic rig with a double tube core barrel and controlled drilling. This has showed a considerable variation in rock characteristics between the two investigations. The confirmatory boreholes showed better quality of rock compared to the initial stage investigation. The scour levels and pile toe levels were redesigned based on confirmatory boreholes. This was carried for one of the important major bridges across river Balram in Gujarat as a part of Package 3R (Western Corridor) by the Dedicated Freight Corridor Corporation of India Limited which is undertaking the construction, maintenance and operation of Dedicated Freight Corridors in the country. The Western Corridor is planned from Jawaharlal Nehru Port, Mumbai to Tughlakabad/Dadri near Delhi. The Western Corridor of the DFC project covers a length of 1,483KM (JNPT-Ahmedabad-Palanpur-Rewari-Asaoti-Dadri). The bridge is a span structure supported on 2 abutments & 5 Piers. This bridge is parallel to the existing Indian Railway (IR) Bridge spaced at 19m centers. The existing bridge is supported on an open foundation with strengthened substructure and foundation. To avoid the influence of foundations on the existing IR Bridge, it is recommended to adopt a pile foundation. The total length of the bridge is 194.7m with six spans each of 32.45m. The location photo of the proposed bridge is presented in Fig.1.

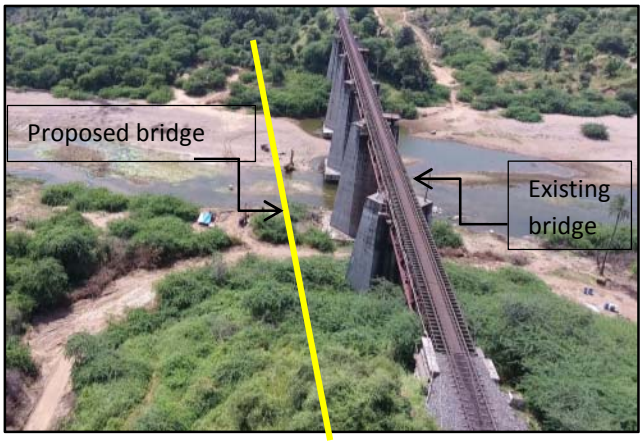



**Fig.1. Site location photo of Balram IR Bridge**

## GENERAL GEOLOGY

*Physiography*: The Balram River is situated in the North of Palanpur which has flat top topography. The hilly and mountainous topography is present in the East and has got a high relief.

*Geology:* The bridge site is covered with recent formation/overburden consisting of dark brown colour silty soil and brownish white colour medium to coarse-grained sand followed by brownish colour fine to medium-grained sand. Below the overburden, there is granite, pegmatite, and gabbro belonging to post Delhi intrusive rocks. These rocks have basic igneous plutonic origin. The geological setup and stratigraphy of the area are given in Table 1. Granite and pegmatite are very slightly weathered to fresh, strong to very strong, medium to coarse-grained, light pinkish to grey colour. These rocks are jointed in nature, inclined and have a dip of about 65deg to 70deg. Gabbro rock occurs below granite and pegmatite. It is dark or black colour, fresh, strong, medium-grained and jointed in nature, inclined and has a dip of about 45deg to 60deg.

**Table 1 Geological set up and stratigraphy**

| Era | Formation | Age |
|---|---|---|
| Quaternary | Soil, Sand, Older & New formation | Holocene |
| Purana | Gabbro, Granite, Pegmatite | Post Delhi Intrusive |

## INITIAL SITE INVESTIGATION

Initial site investigation with seven boreholes (Refer Fig. 2) at each support location was carried out by rotary drilling. SPT test was conducted at every 1.5m intervals and disturbed samples (DS) were collected. When the rock was encountered, NX size TC/diamond bits with single tube core barrel were used for drilling. The rock cores are used to characterize the quality of rock and engineering characteristics. The subsoil profile consists of alluvial soil (Silty SAND) with thickness varying from 0.6m to 4.5m in the river bed area followed by highly weathered (HW) to moderately weathered (MW) granite and HW to fresh (FR) gabbro rock. The alluvial soil is scourable when the flooding occurs. The soil profile along the bridge with core recovery (CR), rock quality designation (RQD) and weathering grades are presented in Fig 3.




**Fig.2. Google image with bridge and borehole location**

### *Quality of rock strata:*

The weathering grade, percentage of CR and RQD were studied to assess the quality of rock strata.

*Core recovery*: The core recovery is the ratio of the sum of lengths of core pieces to the total length of the core run. Fig.3 indicates that the material is predominantly completely weathered (CW) to HW rock to a depth of 170m, RL except for BH at P2 & P3. MW to fresh rock is available below +170m, RL.
*Solid core recovery (SCR)*: The solid core recovery is the ratio of the sum of lengths of solid, cylindrical core pieces to the total length of the core run. At BH P5, the solid pieces observed are below 8% up to +170m, RL. However at BH A1, BH P1, BH P4, and BH A2 there is an increase in solid cores with the increase in depth. At BH P2 & BH P3 CR and SCR are in close proximity for depth below +175m RL.
*Rock quality designation*: RQD is the ratio of the sum of lengths of core pieces that are >10cm measured along the centreline to the total lengths of core run. The very poor to poor quality of rock are present in all boreholes up to +170m except for BH P2 and BH P3 location where the quality of rock is fair to good below +175m, RL

Based on the above information, the quality of rock encountered across the bridge site is very poor to poor and completely weathered to highly weathered to a depth of +170m, RL except at BH P2 and P3.

### *Index properties of rock:*

Because of the vast range in properties of rocks, which reflect the structures, fabrics, components, we rely only on a number of basic measurements to describe rocks quantitatively. Certain properties that are relatively easy to measure are important in this regard and may be designated as index properties of rock specimens. As stated by Goodman porosity, density, sonic velocity together with petrographic data, permeability, durability, and strength are the index properties and attributes that need to be evaluated for engineering classification of rock and together they allow to arrive at useful correlations with experience for practical application. The tests conducted on rock pieces for porosity, water content and density give the results as 0.2% to 1.5%, 0.08% to 0.58% and 25.5kN/m$^3$ respectively. These index parameters indicate that the rock has low porosity, low water content and good density. The UCS strength varies from 20 to 36N/mm$^2$ which indicates the strength of rock is moderately strong. Strong rock was encountered at BH P2 below +174m, RL.

Based on section 6.6.2 of the IRS Substructure, the rock material with CR<35% shall be treated as soil for scour assessment. Hence the top loose silty SANDY deposit, CW and HW rock materials shall be used in scour calculation. The rock material with CR>35% shall be treated as non-scourable. Also, the pile toe level shall be derived by treating the CR<35% rock strata as soil material. This has a direct link with the quality of investigation carried out at the site. Based on these recommendations, the alluvium material, CW and HW rock with CR<35% shall be considered for scour calculations (Refer Table 2).

**Table 2 Elevation of rock having CR > 35% (Initial Investigation)**

| Location | A1 | P1 | P2 | P3 | P4 | P5 | A2 |
|---|---|---|---|---|---|---|---|
| Elevation (m,RL) | 173.000 | 175.250 | 179.070 | 181.230 | 186.100 | 173.450 | 179.480 |

### *Construction/Execution issues:*

As per the project requirement, the vertical pile load test was proposed at pier P1 & P5 location and lateral pile load test at A1 & A2 location. Pile load test was commenced with piling at Pier P5 with a hydraulic rig. The hydraulic rig was not able to make a progress in the rock strata at the site. Hence the designer recommended chiselling the rock through conventional chisel and bailer. The contractor informed that the chiselling through this rock was also difficult and requested an alternate solution. The designer advised the contractor to go for confirmatory borehole investigation at all support locations.

## STAGE II INVESTIGATION

Stage II investigation was conducted to ensure the quality of drilling at the site by using a hydraulic rotary drilling machine with double tube core barrel and NX size diamond core bit. The investigation was done with continuous supervision by an experienced geologist. Emphasis was given more on drilling methodology to get more core recovery by avoiding vibrations in the machine and slow drilling. As the initial stage investigation shows completely weathered rock to highly weathered rock up to a considerable depth, double tube core barrel was recommended to extract rock core samples. The double tube core barrel system contains an inner barrel that protects the core from contact with drill fluid and from erosion or washing from the circulating fluid is used. The main advantage of this system is to achieve better core recovery in the weak and fractured rock at the same time, less disturbance to core samples. In addition, diamond-impregnated drilling bits are used to retrieve the rock core samples. The rotational speed of the drill bit is effectively controlled at the site as guided by the geologist to produce a good amount of rock recovery by ensuring lesser wobbling and vibration effects in the drilling process. The rock samples collected at all borehole locations were preserved in core boxes after recording CR, SCR and RQD. The soil profile along the bridge with CR, RQD and weathering grades is presented in Fig 4.

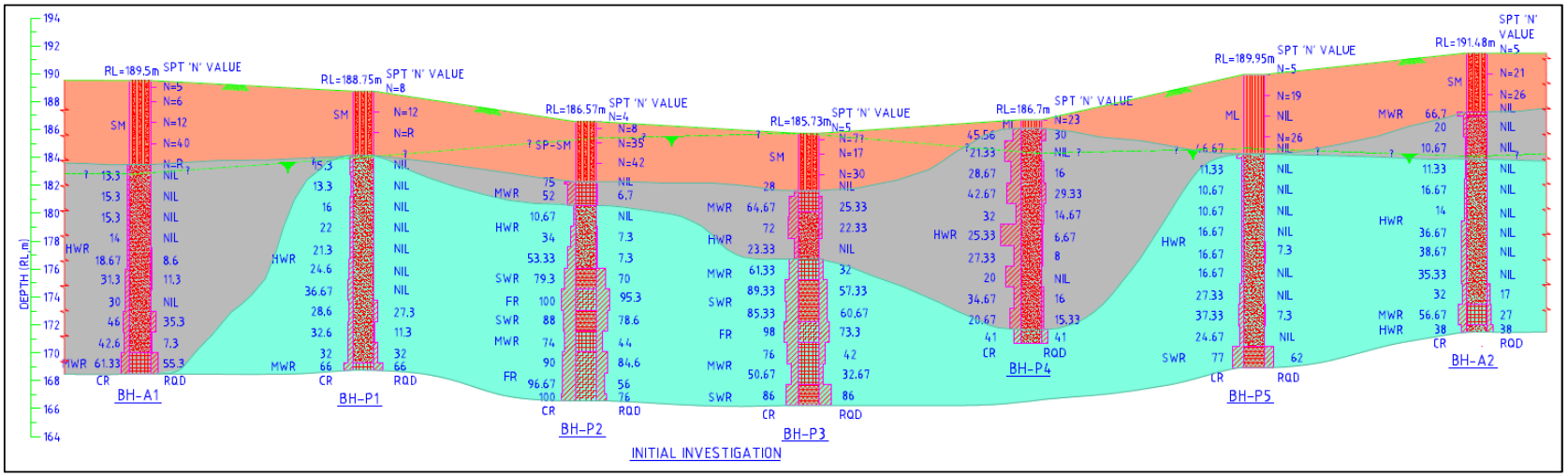



**Fig.3. Soil profile with CR, RQD & weathering grade along the bridge during initial investigation**

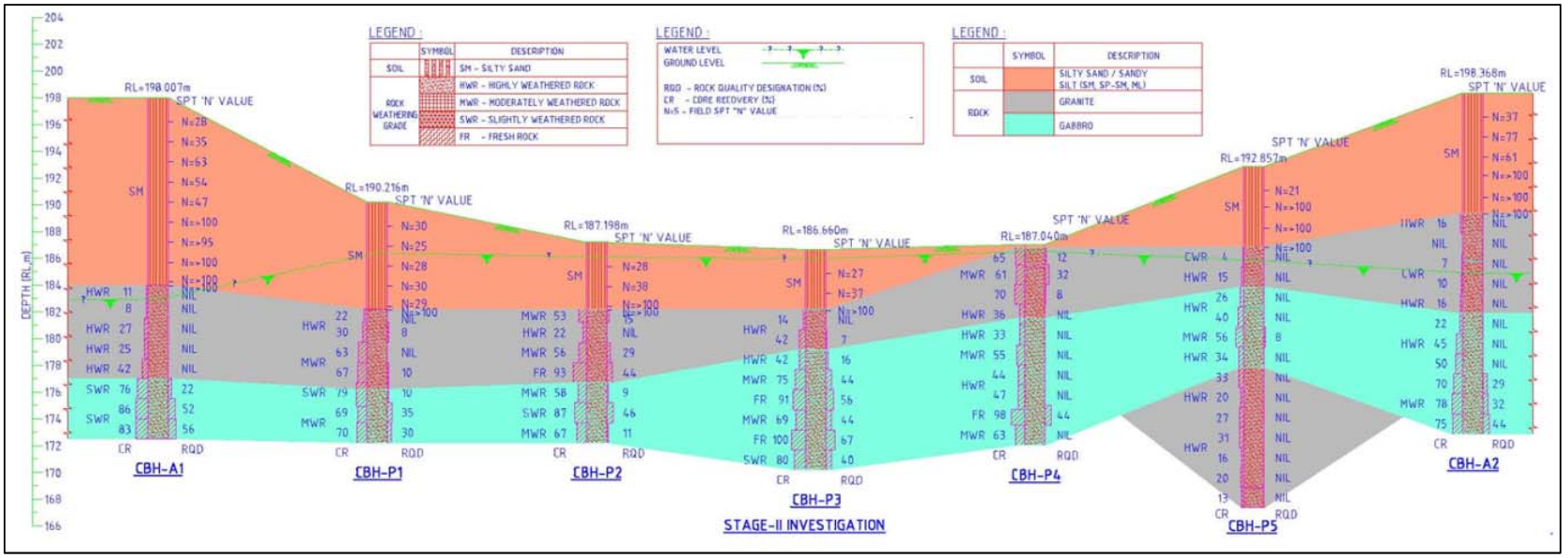



**Fig.4. Soil profile with CR, RQD & weathering grade along the bridge during stage II investigation**

The borehole location plan of stage II investigation boreholes are presented in Fig. 5. The variation of CR, SCR and, RQD with depth for both initial investigation and stage II investigation are presented in Fig.6 and Fig.7 respectively. In comparison with initial investigation, Good core recovery is attained in Stage II investigation which implies the sampling system used i.e. double core barrel has collected good amount of rock pieces than the single tube core barrel.




**Fig.5. Google image with confirmatory borehole locations**

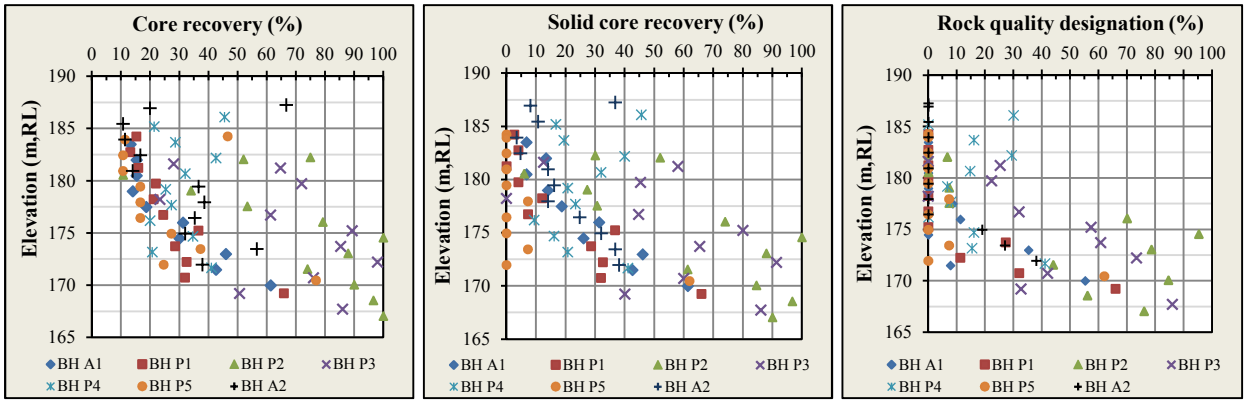



**Fig.6. Variation of CR and RQD with depth during initial investigation**

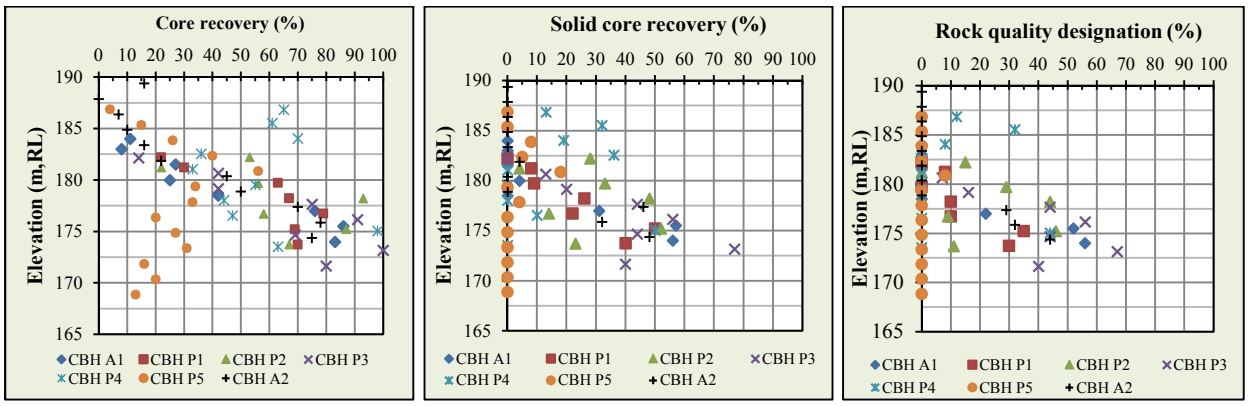



**Fig.7. Variation of CR and RQD with depth during stage II investigation**

***Subsoil profile:***

Subsoil profile observed in boreholes consists of embankment fill material at abutment locations & alluvium on river bed followed by gabbro and granite rock. The IR embankment fill is extended into nearby proposed DFCC abutment and pier locations which are indicated by the presence of embankment fill material in drilling information. The soil strata at the river bed consists of silty SAND (SM) with gravel/Sandy SILT (SM-ML) followed by granite/gabbro rock material (Refer Table 3).

**Table 3: Borehole wise thickness of layers**

| Borehole no. | Depth (m, below EGL) | Reduced level (m RL) | Strata | Remarks |
|---|---|---|---|---|
| $CBH\text{-}A_1$ | 0.0-0.5 | 198.007- 197.507 | Filled up soil | Approximately 8m thick soil is from IR embankment |
| | 0.5-3.5 | 197.507-194.507 | Silty SAND | |
| | 3.5-5.0 | 194.507- 193.007 | Sandy SILT | |
| | 5.0-14.0 | 193.007 - 184.007 | Silty SAND | |
| | 14.0-25.5 | 184.007 - 172.507 | HW to SW rock | |
| $CBH\text{-}P_1$ | 0.0-1.0 | 190.216 – 189.216 | Filled up soil | Approximately 4m thick soil is from IR embankment |
| | 1.0-8.0 | 189.216 - 182.216 | River bed material | |
| | 8.0-18.0 | 182.216 - 172.216 | HW to SW Rock | |
| $CBH\text{-}P_2$ | 0.0-5.0 | 187.198 - 182.198 | River bed material | Alluvial soil |
| | 5.0-15.0 | 182.198 - 172.798 | HW to FR Rock | |
| $CBH\text{-}P_3$ | 0.0-4.5 | 186.660 - 182.160 | River bed material | Alluvial soil |
| | 4.5-16.5 | 182.160 - 170.160 | HW to FR Rock | |
| $CBH\text{-}P_4$ | 0.0-0.2 | 187.040 - 186.840 | River bed material | Alluvial soil |
| | 0.2-15.0 | 186.840 - 172.040 | HW to FR Rock | |
| $CBH\text{-}P_5$ | 0.0-1.5 | 192.857 - 191.357 | Filled up soil | Approximately 4m thick soil is from IR embankment |
| | 1.5-6.0 | 191.357 - 186.857 | Silty SAND/ Sandy SILT | |
| | 6.0-25.5 | 186.857 – 167.357 | CW to MW Rock | |
| $CBH\text{-}A_2$ | 0.0-9.0 | 198.368 - 189.368 | River bed material | Approximately 5m thick soil is from IR embankment |
| | 9.0-25.5 | 189.368 - 172.868 | CW to MW rock | |

## *Quality of rock strata:*

The weathering grade, percentage of CR, SCR and RQD were studied to assess the quality of rock strata.

*Core recovery*: Fig. 4 indicates that the material is predominantly CW to HW rock to a depth of +179m, RL except at CBH at P5, wherein CW to HW is extended to a complete depth of drilling. Moderately weathered to fresh rock is observed below +179m, RL. This shows there is a considerable increase in core recovery with respect to the initial stage of an investigation.
*Solid core recovery:* At BH P5 the solid pieces observed are below 8% up to +168m, RL. However, at other locations, there is an increase in SCR with an increase in depth. The trend of the percentage of SCR is increased with respect to the initial stage investigation. A higher percentage (>60%) of SCR for BH P2 & BH P3 observed in the initial stage of investigation is not available in confirmatory borehole investigation. This shows there is a change in weathering between the two borehole locations.

*Rock quality designation*: Very poor to poor quality of rock is present in all boreholes within the depth of investigation.
Based on the above information, the quality of rock encountered across the bridge site is very poor to poor and completely weathered to highly weathered to a depth of +179m, RL. The rock core recovered from all the boreholes are presented in Fig. 9.

### *Index properties of rock:*

The UCS tests were performed under both soaked and un-soaked conditions. There is not much variation in strength; the UCS varies from 40 to 85MPa which indicates that the strength of rock is moderately strong to strong. Comparing the two investigations, the strength of rock is fairly uniform. However, the variation in strength is with respect to the sample preparation and least count of the dial gauges used in the laboratory.

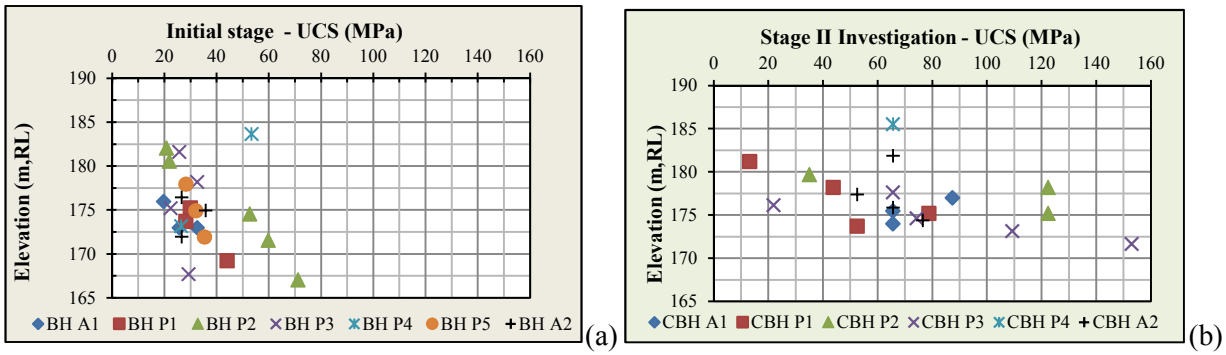



**Fig.8 Variation of UCS with depth (a) Initial stage investigation (b) Stage II investigation**

As discussed in the section above, section 6.6.2 of the IRS Substructure states that the rock material with CR<35% shall be treated as soil for scour assessment as well as pile capacity calculation. Based on these recommendations, the material to be considered in scour calculation is top alluvial soil followed by CW rock and HW rock with CR<35%. Hence the elevation of material to be used in scour calculation has increased and is presented in Table 4.

**Table 4 Elevation of rock having CR>35% (Stage II Investigation)**

| Location | A1 | P1 | P2 | P3 | P4 | P5 | A2 |
|---|---|---|---|---|---|---|---|
| Elevation (m,RL) | 178.507 | 179.716 | 179.698 | 182.160 | 186.840 | 182.357 | 180.368 |

With respect to the material to be considered to assess the depth of scour, the quality of drilling from the initial stage to confirmatory borehole investigation has a considerable improvement at A1, P1 and, P5, A2; remaining locations the core recovery fairly coincide between both the investigations. Considering the completely weathered rock strata as gravel material with a size of 4.74mm the scour assessments were recommended.

### *Scour recommendation:*

In the stage II investigation, with core photographs & excavation performed at the site to expose the rock surface, it is understood that the rock strata will give resistance to scour.

*Abutment A1*: the calculated design scour level itself is above the rock level;
*Pier P1 to P3*: the least rock level at P3 is +181.177m, RL which is below the design scour level of +183.121m, RL, so the rock level of +181.177m, RL is considered as scouring level at these piers duly considering the variation in top level of rock;




**Fig.9. Rock cores recovered from boreholes (Stage II Investigation)**

*Abutment A2*: Calculated design scour level is below the rock level, considering the nature of the rock, scour level is considered as rock level i.e. +185.811m, RL.

### *Foundation recommendation:*

As per the IRS Substructure section 6.6.2, the rock material with CR<35% shall be treated as soil with an angle of internal friction of 38deg. considering the strata with CR<35% as a granular material behaviour resulted in very less skin frictional resistance. However, the required capacity was mobilized by socketing the pile into rock strata with CR>35%. Hence bored cast-in-situ piles socketed into rock are provided. Piles of 1.5m diameter are proposed at the abutment to resist the lateral loads and 1.2m diameter piles are provided at piers. All piles are designed to resist both vertical and lateral loads.

## CONCLUSION

During the initial investigation stage single tube core barrel was used for collecting rock cores which have resulted in very poor core recovery leading to an uneconomical foundation design. During piling for pile load testing, the contractor faced difficulties in executing the test pile. Based on his request, stage II investigation i.e. confirmatory boreholes by using double tube core barrel was executed with complete supervision to get maximum core recovery. The core recovery has improved considerably at A1 & P1 and P5 & A2 locations. At the remaining locations, the core recovery fairly coincides between the two investigations. In order to comply with the foundation design guidelines as per the IRS substructure, the quality of investigation data with drilling methodologies recommended in IS 6926:1996 should have strictly adhered to the site. There was a gap in the presentation of factual investigation data from drilling agencies. Lack of quality data from investigation will lead to conservative designs which shall be avoided in the initial stage of investigation than going for confirmatory investigation. This can save time, money and delivers the project as per schedule. The investigations shall be done as per IS 1892 and IS 4464 respectively for soil and rock material. CR varies with respect to drilling tools and methodology adopted at the site. Hence using this parameter as a reference to consider the material as scourable or non-scourable may not be appropriate. Consideration of CW rock as fine gravel material may not be appropriate in scour assessment because of its origin from the rock. Hence deriving in-situ shear strength parameters through pressure meter test may be more appropriate than treating the strata with CR<35% as soil while assessing the scour and foundation recommendation. There is a large variation in rock weathering across the bridge site; less weathered material is observed at the middle portion of the bridge than on both sides. Hence variation of weathering between the boreholes is to be studied by using geophysical investigation combined with borehole investigation.

## REFERENCES

Indian Railway Standard Code of practice for the design of substructures and foundations of bridges.

IS: 4464-1985. “Indian Standard Code of practice for presentation of drilling information and core description in foundation investigation.”

IS: 6926-1996. “Indian standard for diamond core drilling- site investigation for river valley projects-Code of practice.”

IS: 1892-1979. “Indian Standard Code of practice for subsurface investigation for foundations.”

IRC: 78-2014. “Indian Railway Standard Code of practice for the design of substructures and foundations of bridges– Standard specifications and code of practice for road bridges, Section: VII Foundation and substructure.”

# IMPORTANCE OF COLLECTING SOIL SAMPLES IN HARD STRATA FOR SOIL CLASSIFICATION AND DETERMINATION OF SHEAR STRENGTH PARAMETER

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## ABSTRACT

This paper presents case studies of three railway bridges across river Saraswati, Umardasi and Pushpavati pertaining to the difficulties faced in sample collection and classification of soil strata with field SPT 'N'>100. These bridges in the state of Gujarat were designed as a part of package 3R by the Dedicated Freight Corridor Corporation of India Limited. Pile foundation was proposed at all the three bridge locations. The soil investigation was done in two stages: Stage I and Stage II. The stage I soil investigation consists of drilling borehole up to 50m depth by rotary drilling at each structure support location. It was observed that SPT 'N' is greater than 100 below 10-15m depth of investigation at all the borehole locations. The classification of soil with field SPT 'N'>100 was done using the disturbed sample collected in the SPT spoon sampler. The soil profile in all three structure locations was uniform and consists of silty sand (SM/SP-SM) followed by silt clay (CI/CH). Stage II investigation was carried with an attempt to collect undisturbed sample in strata with SPT 'N'>100 using mazier sampler for laboratory testing and hence validate the index and shear strength parameters considered for pile design during stage I investigation. On collecting and classifying the soil strata with SPT 'N'>100, the soil classification got changed from silty clay (CL/CI/CH) to clayey sand (SC). A comparative study between stage I and stage II investigation with respect to soil classification and shear strength parameters is presented in the paper.

***Keywords: Hard soil strata, Field SPT N value, undisturbed soil sampling, Mazier Sampler***

## INTRODUCTION

Soil investigation plays an important role in establishing the sub soil stratification for the geotechnical analysis. The most common approach in India for establishing the subsoil profile is by soil investigation through drilling of borehole along with standard penetration test (SPT) and sample collection for laboratory testing. The samples collected for laboratory testing can either be disturbed sample (DS) (split spoon sampler, continuous auger or bulk sampling) or undisturbed sample (UDS) (thin wall Shelby sampler, piston sampler and block sampler). It is not practical to collect UDS in very stiff to hard soils. In strata where it is difficult to collect UDS for laboratory testing, the most common method is to use the disturbed sample from SPT Sampler. But it may not be correct to establish the classification of soil based on the sample obtained from the SPT sampler in hard strata.

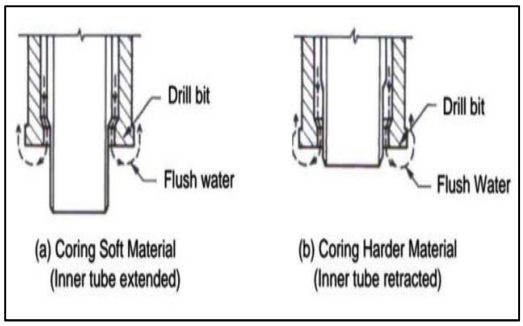
This paper highlights the inconsistencies in soil classification using samples from SPT spoon sampler in hard strata, where it is not possible to collect UDS with the support of three case studies. This paper presents case studies of three railway bridges across rivers Saraswati, Umardasi and Pushpavati. These bridges in the state of Gujarat were designed as a part of package 3R(Western Corridor) by the Dedicated Freight Corridor Corporation of India Limited which is undertaking the construction, maintenance and operation of Dedicated Freight Corridors in the country. The Western Corridor is planned from Jawaharlal Nehru Port, Mumbai to Tughlakabad/ Dadri near Delhi. The Western corridor of DFC Project covers a length of 1,483 km (JNPT – Ahmadabad – Palanpur – Rewari – Asaoti - Dadri).


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To support the bridge structure, pile foundation was proposed at all the three locations. In all the three case studies, the soil investigation was done in two stages: Stage I & Stage II. In stage I investigation, UDS sample was not collected in hard strata with SPT 'N' >100. The soil classification was done based on the disturbed samples collected from SPT spoon sampler. Stage II investigation was performed with an attempt to collect undisturbed sample in strata with SPT 'N'>100 using mazier sampler for laboratory testing and hence validate the index and shear strength parameters considered for pile design during stage I investigation.

## MAZIER SAMPLING

The method of sampling affects the degree of sample disturbance in soil specimens. The main use of mazier sampler is to obtain high quality undisturbed samples in very soft clay, stiff to hard cohesive soil and medium dense to dense sandy soil. Mazier core-barrel is a triple-tube core-barrel fitted with a retractable shoe [Ng & Leung 2007; GCO 2014 ;]. The cutting shoe and connected inner barrel projects ahead of the bit when drilling in soil and retracts when the drilling pressure increases in harder materials as depicted in Fig.1. This greatly reduces the possibility of any drilling fluid coming into contact with the core at or just above the point of cutting and hence minimizes sample disturbance. Moreover, the mazier core-barrel has an inner plastic liner that protects a cored sample during transportation to a laboratory. The sample in PVC pipe will be properly sealed in wax and taken to the laboratory. The core diameter is 74 mm, which is compatible with the laboratory triaxial testing apparatus. The sample is retracted from the tube and used for all the laboratory testing.



(a)

(b)

**Fig. 1: (a) Mazier sampler drilling technique (b) Mazier sampler used in the project**

## CASE STUDY 1: BRIDGE ACROSS SARASWATI RIVER

The clear span of the proposed bridge, parallel to the existing bridge, across river Saraswati is 1x17.671+16x17.600+1x17.684. The location photo of the proposed bridge is presented in Fig. 2.




**Fig. 2: Site location photo of the proposed bridge**

The stage I soil investigation consists of drilling 19 boreholes, one at each support location, to a depth of 50m below ground level. Field SPT 'N' value was measured at every 1.5m interval using SPT split spoon sampler. The soil profile is uniform along the bridge location and comprises of silty sand (SM)/clayey sand (SC) followed by silty clay (CL/CI/CH). The observed field SPT 'N' value is greater than 100 beyond 15m depth of investigation. Wherever possible, undisturbed sample (UDS) was collected within 15m depth of investigation using Shelby tubes. The dry sieve analyses and hydrometer analyses were conducted on both the UDS and split spoon soil samples. The following laboratory test was conducted on UDS collected within 15m depth of investigation:

- Bulk density;
- Specific gravity:
- Water content; and
- Direct shear test /unconsolidated undrained test based on type of soil.

Beyond 15m where the SPT 'N' is greater than 100, UDS was not collected up to the termination depth of the borehole. In these strata, the samples from SPT spoon sampler were used for the soil classification. As sufficient sample was not available, only dry sieve analysis and hydrometer analysis were conducted in the soil strata beyond 15m depth. The summary of field and laboratory test results is discussed in detail in the paper. In the design stage, wherever laboratory test results were not available, shear parameters were considered based on the correlation with extrapolated SPT 'N' values [Stroud 1989]. As there were no references on the limiting undrained shear strength ($c_u$) for hard clay, it was difficult to derive the shear strength by correlations using extrapolated SPT N value. In this scenario, a stage II investigation was proposed with the purpose of collecting UDS sample in the hard clay strata and validating the shear parameters considered for the design.

In the stage II investigation undisturbed sample was collected using mazier sampler for laboratory testing beyond 15m depth of investigation. The stage II investigation comprises of drilling four boreholes (BH-01, BH-02, BH-03 & BH-04) adjacent to the stage I boreholes (BH-194 P2, BH-194 P6, BH 194 P14 & BH 194 P17). The borehole location plans with both stage I and stage II boreholes are presented in the Fig.3. The SPT 'N' value was recorded at every 1.5m interval. It was observed that the soil profile was uniform along these four boreholes and the SPT 'N' was greater than 100 beyond 10-12m in all the four boreholes. The comparative study between the stage I & stage II investigation with respect to soil classification and shear strength parameters is summarised below.




**Fig. 3: Borehole location plan for the proposed bridge across river Saraswati**

***Field SPT N value***

The variation of SPT 'N' value with depth under both stages of investigation is presented in the Fig.4. It is observed that the field SPT 'N' value is consistent in both stages of investigation and is greater than 100 beyond 10-12m.

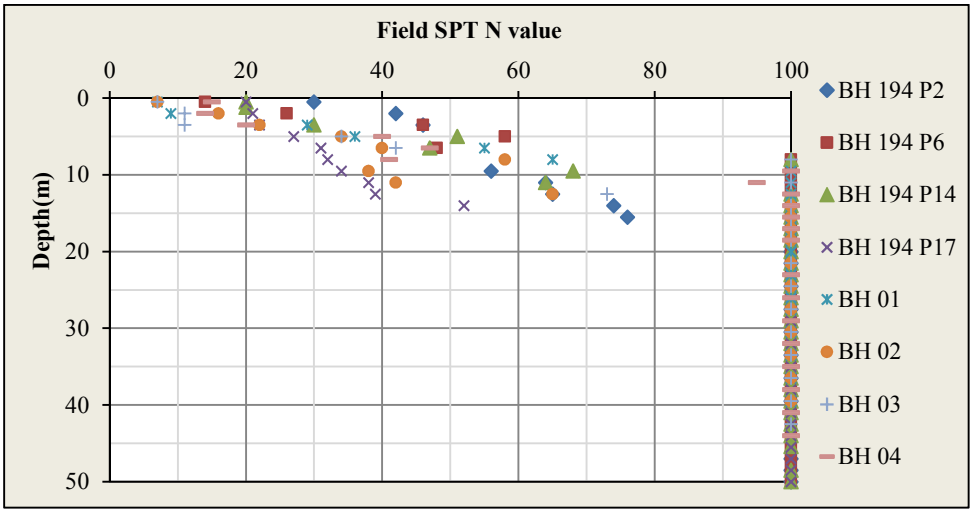



**Fig. 4: Variation of SPT 'N' with depth (Both Stage I & Stage II Investigation)**

***Ground water table***

Ground water levels were recorded during both the stages of investigations. The water table was observed between 8.4m to 17.5m below existing ground level (EGL) in Stage I investigation and 2.5m to 9.8m in Stage II investigation. The variation in water table is due to the difference in the month of the investigation. The Stage I and Stage II investigations were carried out in July 2016 and September-October 2017 respectively. The Stage II field investigation was carried out immediately after the flooding of the river. This has been reflected on depth of water level from ground level.

***Subsoil profile along the bridge***

Dry sieve analysis and hydrometer analysis were conducted on samples obtained from both the stages of investigation. The comparison of fine grained and coarse grained particles between two stages of geotechnical investigation is presented in Fig. 5. It can be observed that there is a change in soil strata classification where SPT 'N'>100. During the stage I investigation, the dry sieve analysis and hydrometer analysis for the strata with SPT>100 was conducted on samples from SPT sampler. The test results indicated that the soil is silty clay of low/medium/high plasticity (CL/CI/CH) with more than 50% fines content (silt + clay). To the contrary of stage I investigation, in stage II investigation, the dry sieve analysis and hydrometer analysis was conducted on samples collected from mazier sampler. The sample was retracted from the PVC pipe in laboratory for the testing. The laboratory results indicated that the stratum is clayey sand (SC) with more than 50% coarse grained particles (sand + gravel). This discrepancy in soil classification is mainly due to the poor sampling technique adopted in stage I investigation. In stratum where SPT N>100, the penetration into the layer was only about 7cm-10cm. During this penetration, it may be observed that mostly the fines in the stratum were pushed into the sampler. Hence the dry sieve and hydrometer analysis on the SPT split spoon samples showed more than 50% fines content.

***Atterberg limits***

In both the stages of investigation, Atterberg limit tests were conducted on the material passing 425μ sieve. The Fig. 6 indicates that there is a consistency in the liquid limit (LL) and plastic limit (PL) between both the stages of investigation. This confirms the uniformity of the index properties of the fines in the strata. As the UDS samples were collected by using mazier sampler in stage II investigation natural moisture content (w) was determined and is less than plastic limit which indicates that the in-situ condition of soil is in solid state.

***Consolidated undrained (CU) triaxial Test / Direct shear test (DST)***

The sample obtained from mazier sampler was retracted to establish the shear strength parameters. It is observed from grain size distribution that the gravel content is present in the sample. The remoulded samples are prepared to its natural moisture content by removing the gravel content. CU test was conducted to determine the effective shear strength parameters of soil.

Direct shear test (DST) were performed on clayey sand (SC) soil samples to determine the shear parameters of soil. The soil samples are completely saturated during testing. The results of both CU and DST tests are presented in the Fig.7. From figure, angle of internal friction of 31deg. is considered as design angle of internal friction for the clayey sand (SC).

***Design parameters for geotechnical design***

Pile foundation was proposed to support the piers and abutments. Pile design was done as per relevant standards. During design from stage I investigation data, shear parameters were considered from laboratory test results up to 15m depth of investigation. Beyond 15m, shear parameters were considered based on the correlation with extrapolated SPT 'N' values [Stroud 1989]. The extrapolated SPT 'N' value was varying in the range of 115 to 200. Hence the calculated undrained shear strength ($c_u$) of hard clay was varying from 500kPa to 1000kPa. For the pile design, the undrained shear strength of hard clay was limited to 550kPa.

After the stage II investigation, the soil strata with SPT 'N'>100 got changed from hard silty clay to very dense clayey sand. During stage II investigation, pile design was done based on the shear parameters obtained from the laboratory test results and the same is summarised in Table 1.

**Table 1: Summary of design parameters-Bridge across river Saraswati (Stage II)**

| Depth(m) | Soil Description | Density (kN/m$^3$) | Angle of internal friction (deg.) |
|---|---|---|---|
| 0-11.5 | Silty Sand(SM/SP-SM) | 19.0 | 31 |
| Below 11.5 | Clayey Sand(SC) | 19.7 | 31 |

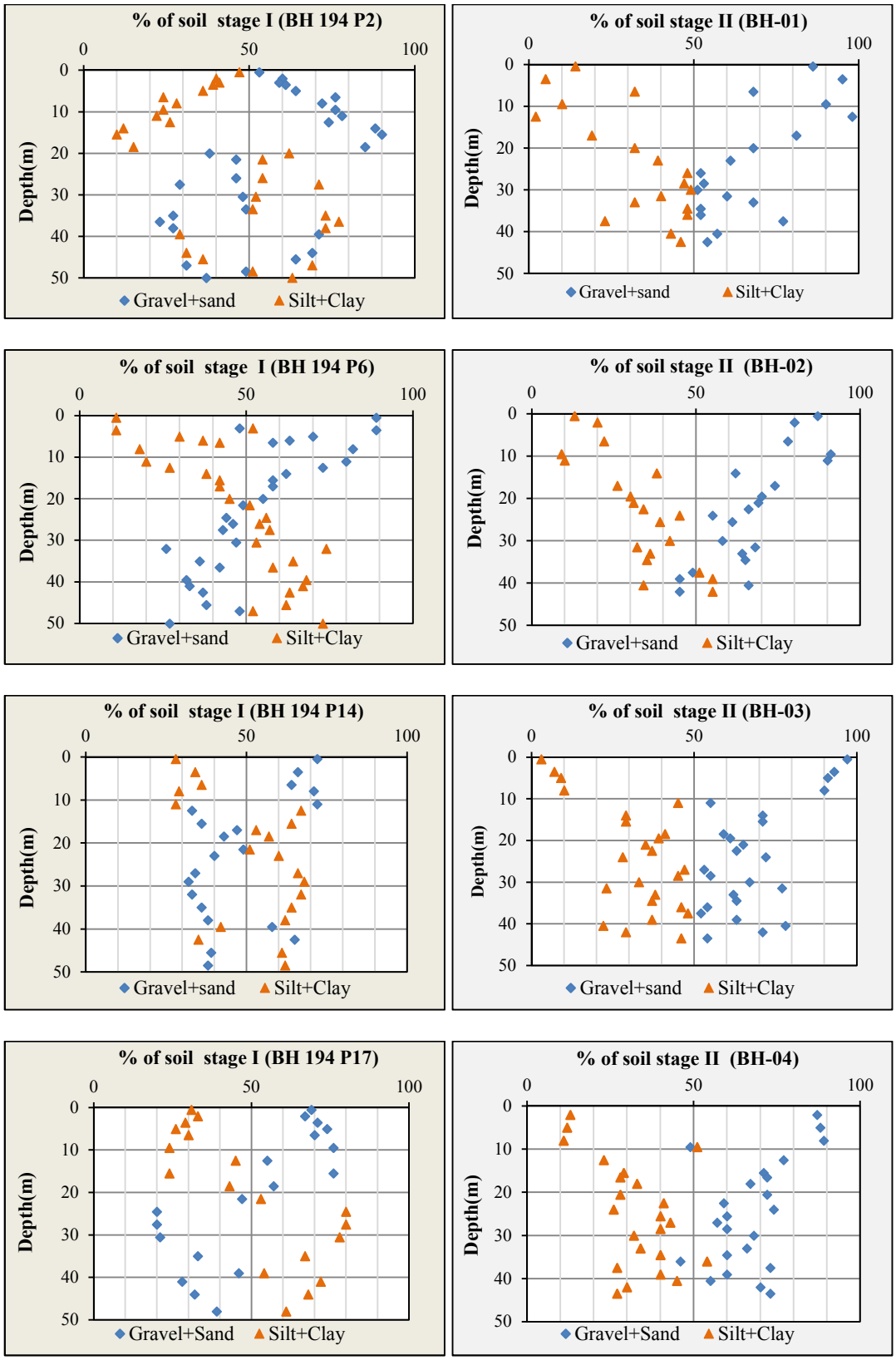



**Fig. 5: Comparison of percentage of fine grained and coarse grained particles between two stages of investigation (Saraswati River)**

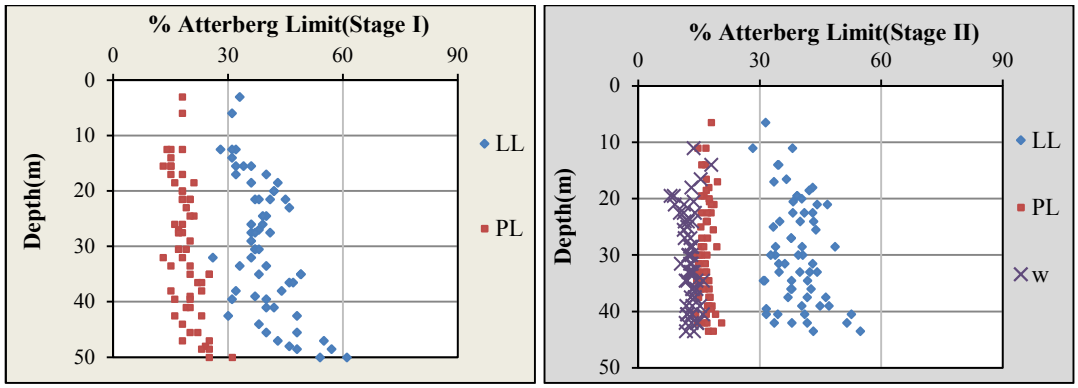



**Fig. 6: Variation of Atterberg limits in both the stages of investigation**

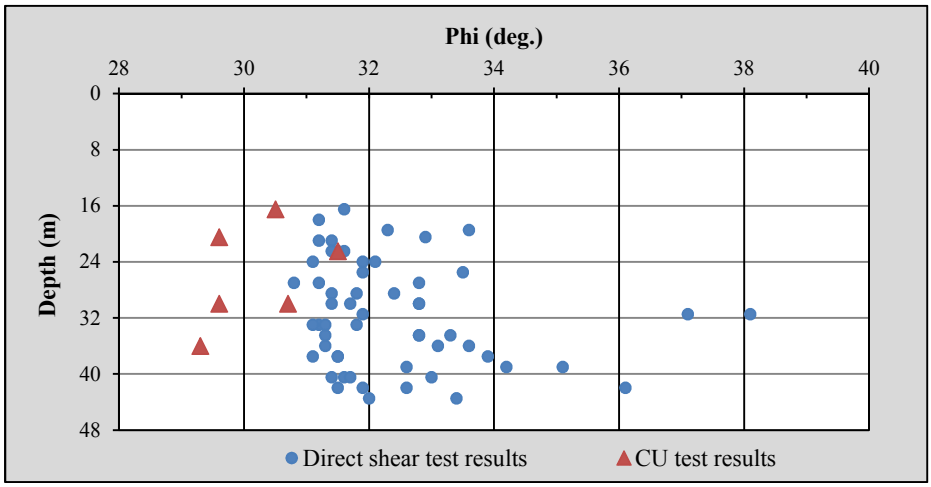



**Fig. 7: Summary of angle of internal friction with depth.**

**Case study II: Railway bridge across Umardasi River & Case study III: Railway bridge across Pushpavati River**

The clear span of the proposed bridge, parallel to the existing bridge, across river Umardasi is 1x18.626+12x18.500+1x18.614. The location photo of the proposed bridge location is presented in Fig. 8(a). The stage I soil investigation consists of drilling 15 boreholes, one at each support location, to a depth of 50m below ground level. The soil profile was uniform along the bridge and comprises of silty sand(SM)/clayey sand (SC) followed by silty clay (CL/CI/CH). The observed field SPT 'N' value is greater than 100 beyond 10m depth of investigation. Similar to the case study I, stage II investigation was carried out at four locations near to the existing boreholes. The stage II investigation comprises of drilling four boreholes (BH-01, BH-02, BH-03 & BH-04) adjacent to the stage I boreholes (BH-240 P13, BH-240 P10, BH 240 P7 & BH 240 P3). In stage II investigation, the SPT 'N' was greater than 100 from about 12m.

The clear span of the proposed bridge, parallel to the existing bridge, across river Pushpavati is 3x24.4. The location photo of the proposed bridge location is presented in Fig. 8(b). The stage I soil investigation consists of drilling 4 boreholes, one at each support location, to a depth of 40m below ground level. The observed field SPT 'N' value is greater than 100 beyond 15m depth of investigation. Stage II investigation was carried out at one location near to an existing borehole (BH 154 P1).



(a)


(b)

**Fig. 8: location photo of the proposed bridge across (a) Umardasi River (b) Pushpavati River**

The comparison of fine grained and coarse grained particles between two stages of geotechnical investigation for bridge across rivers Umardasi and Pushpavati is presented in Fig. 9 & Fig. 10 respectively. The following were observed on comparison of both the stages of investigation:

1.) There was consistency in SPT 'N' value and Atterberg limits with depth in both stages of investigation;
2.) The water table was observed at shallower depth in the stage II investigation as the investigation was done immediately after floods; and
3.) There was a discrepancy in the soil classification in hard soil strata (SPT N >100) from SPT split spoon sampler. The hard soil strata classification got changed from silty clay (CL/CI/CH) to clayey sand (SC) when analysed with soil samples from mazier sampler.

Pile foundation was proposed to support the piers and abutments. Similar to case study I, during the stage I investigation wherever laboratory test results are not available, the shear parameters of silty clay were derived based on the correlation with extrapolated N values [Stroud 1989]. In case study II, the extrapolated SPT 'N' value was varying from 112 to 161. Hence the calculated undrained shear strength of hard clay was varying from 500kPa to 850kPa. For the pile design, the undrained shear strength of hard clay was limited to 500kPa. In case study III, the extrapolated SPT 'N' value was varying from 115 to 190. Hence the calculated undrained shear strength of hard clay was varying from 550kPa to 1000kPa. For the pile design, the undrained shear strength of hard clay was limited to 550kPa.

In both case study II & case study III, after the stage II investigation the hard clay strata (CL/CI/CH) got changed to very dense clayey sand (SC). During stage II investigation, pile design was done based on the shear parameters obtained from the laboratory test results and the same is summarised in Table 2.

**Table 2: Summary of design parameters considered for pile design (Stage II)**

| **Design parameters- Bridge across river Umardasi** | | | |
|---|---|---|---|
| Depth(m) | Soil Description | Saturated density (kN/m$^3$) | Angle of internal friction (deg.) |
| 0-26 | Silty Sand(SM/SP-SM) | 19.0 | 32 |
| Below 26 | Clayey Sand(SC) | 21.0 | 32 |
| **Design parameters- Bridge across river Pushpavati** | | | |
| Depth(m) | Soil Description | Saturated density (kN/m$^3$) | Angle of internal friction (deg.) |
| 0-6.5 | Silty Sand(SM/SP-SM) | 19.0 | 31 |
| Below 6.5 | Clayey Sand(SC) | 20.0 | 31 |

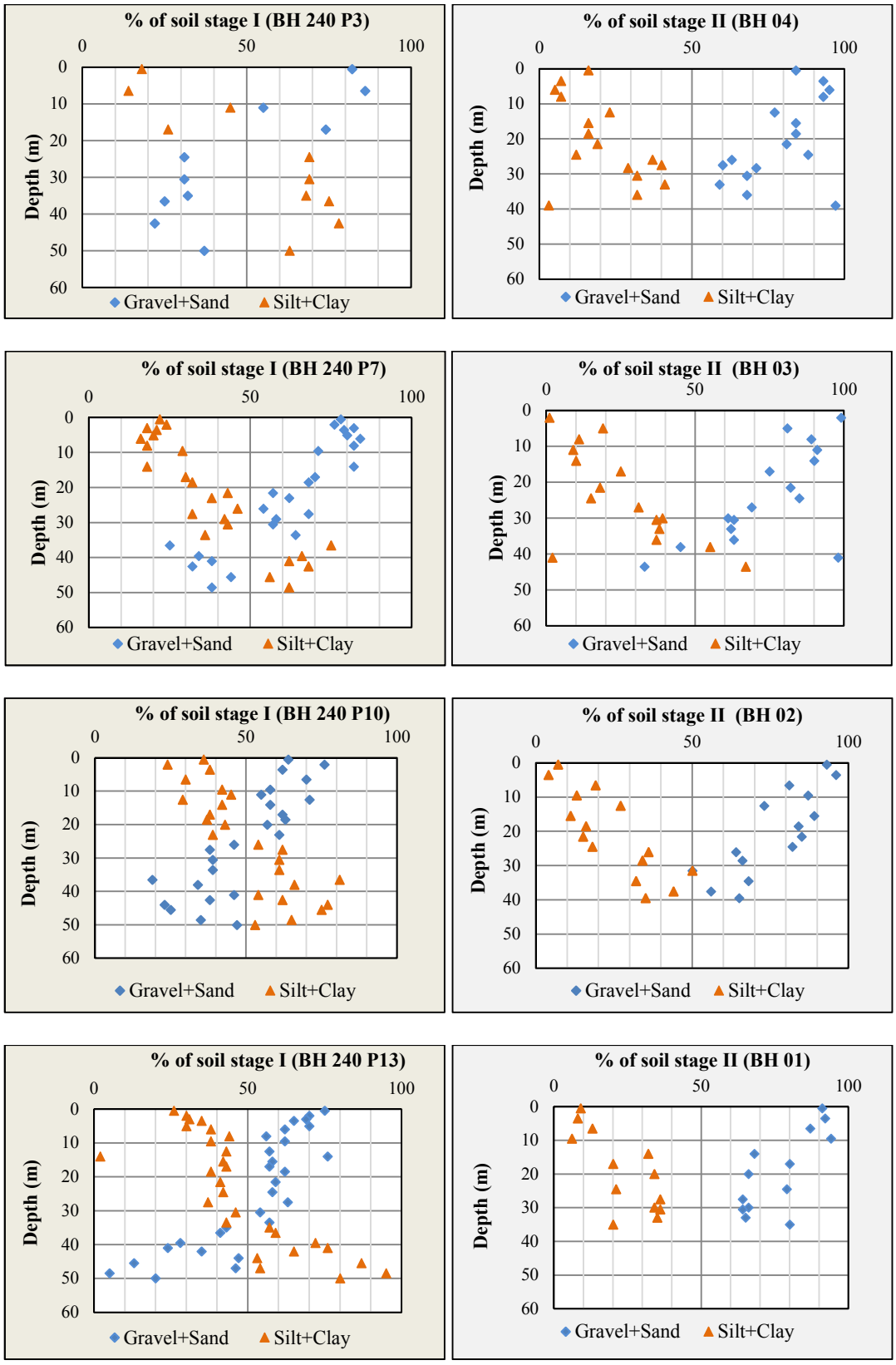



**Fig. 9: Comparison of percentage of fine grained and coarse grained particles between two stages of investigation (Umardasi River)**

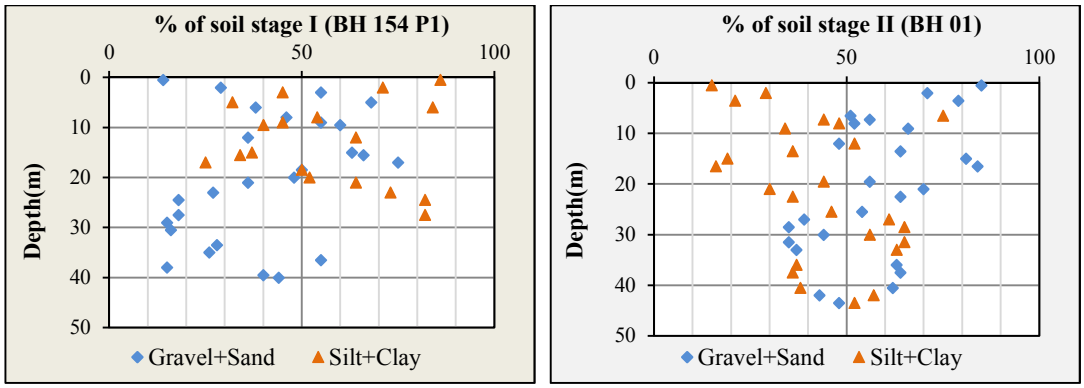



**Fig. 10: Comparison of percentage of fine grained and coarse grained particles between two stages of investigation (Pushpavati River)**

## SUMMARY AND CONCLUSION

The case studies presented in the paper emphasize on the importance of the application of proper sampling/investigation methods for establishing the subsoil stratification. In case of hard strata with SPT 'N'>100 and where the split spoon sampler penetration is very low, soil classification through samples from SPT split spoon sampler can give erroneous results. Two stages of investigation were performed at three bridge locations: Stage I where no UDS was collected in hard strata and stage II where UDS was collected with mazier samplers. In stage I investigation, the soil classification was done based on the samples from split spoon sampler. The shear parameters were derived based on correlations with extrapolated SPT 'N' value. In stage II investigation, soil classification and shear parameters was established from laboratory test results in undisturbed sample. It is observed that there was consistency in filed SPT N value and Atterberg limits in both stages of investigation but the IS soil classification of hard soil strata got changed from silty clay (CL/CI/CH) to clayey sand (SC) when tested with undisturbed samples(UDS) obtained from mazier sampler.

Discrepancy in soil classification will have effect on pile design and will lead to delays in the project schedule. Hence proper care should be taken in soil classification and shear strength parameters determination. This issues can be mitigated only through proper soil investigation and sampling into such soil stratum.

## REFERENCES

M.A.Stroud 1989. "The Standard Penetration Tests-Its Application & Interpretation" *Penetration testing in the UK,Thomas Telford,London,198.*

Ng, C.W.W.& Leung, E.H.Y. 2007b "Determination of Shear-WaveVelocities and Shear Moduli of Completely Decomposed Tuff." *Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 6, pp. 630–640. ASCE.*

Geotechnical Engineering Office 2014. "Guide to site investigation: Geoguide 2." *Geotechnical Engineering Office, Civil Engineering Dept., Government of the Hong Kong SAR.*

# PREDICTION OF MODULUS OF ELASTICITY AND ITS IMPACT ON SETTLEMENT

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## ABSTRACT

In coarse grained soils, often the allowable strength of foundation system is governed by settlements rather than shear strength of foundation soils. Hence, deriving the modulus of elasticity (E) of subsoil to evaluate the settlement below the foundation is one of the key tasks to decide the size of foundation as well as type of foundation. Numerous methods are available to arrive the modulus of elasticity viz. laboratory and field test (direct measurement-PMT/DMT and plate load test), correlations with SPT and CPT, empirical methods etc. In this paper, we attempted to correlate the modulus of elasticity values derived from in-situ footing load tests with available empirical equations, finite element methods (PLAXIS) and one-dimensional analysis using SETTLE$^{3D}$. The ratio of measured 'E' (from field tests) with respect to the inferred 'E' using different empirical correlations with SPT 'N' values is ranging from 0.8 to 9.3 times. The derived 'E' values from footing load tests is compared with available software tools, namely PLAXIS-3D and SETTLE$^{3D}$. The settlement from SETTLE program is slightly higher than the field measured settlement, whereas finite element analysis produced significantly higher settlement than SETTLE program and measured settlement at field.

**Key words: footing load test, modulus of elasticity, settlement, finite element method, elastic theory, Settle**

## INTRODUCTION

Prediction of modulus of elasticity of soil is one of the key tasks and it is primarily used for evaluating the immediate (Elastic) settlements for all type of foundations. During evaluation of allowable capacity of foundations, especially in cohesionless soils, the deformations are often governing rather than shear strength of soil. Hence, deriving representative modulus of elasticity (E) value of foundation soil is very vital and prediction of settlement is essential for all type of foundations. Foundation soil modulus shall be evaluated either directly from field tests (PMT/DMT) or correlations using other commonly available field penetration tests i.e. Standard penetration tests (SPT) and Cone penetration tests (CPT).

In the past, several empirical and analytical methods have been proposed over the years by various authors, to evaluate appropriate 'E' values SPT and CPT tests used. Settlement is function of soil modulus (for immediate settlement) and external pressure. Among various empirical correlation widely used, methods are Stroud[14], Bowles[5], Schmertmann[13], D'Appolonia[8], AASHTO[20], Papadopoulos[15], Trofimenkov[9], Parry[11], Schultze and Menzenbach[12], Kulhway and Mayne[10], Mezenbach[1], Chaplin[19], Clayton[16], Webb[17], Denver[18] and Al Sheyea[22]. In the current study, nearly 18 empirical correlations have been used to know the variation of E using different equations.

The aim of this study is to find the variation of 'E' value by comparing the back calculated using field tests and available empirical correlations and computer programs like finite element method (PLAXIS-3D) and one-dimensional analytical method (SETTLE$^{3D}$).

## SITE CONDITIONS AND SOIL PROPERTIES

SUBSURFACE STRATIGRAPHY


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Based on the available site investigation data and laboratory test results, the classification of soil and rock was arrived as per BS 5930 and IS 1498. The subsurface stratigraphy consists of clay layer of approximately 6.0m thick from EGL followed by a medium dense to dense sand layer with thickness of 6.5m. The dense sand layer is followed by completely weathered to highly weathered rock of 25.0 m thickness. Design ground model is arrived based on available bore logs (total 17) and previous experience in the vicinity of site. The ground water is observed at a depth of 2.30 m to 4.50m from EGL. Location of Boreholes and Foundation Load tests (FLT) are shown in Fig 1. Subsurface profile and derived parameters attained from geotechnical investigation and same used for the analysis are summarized in Table 1.

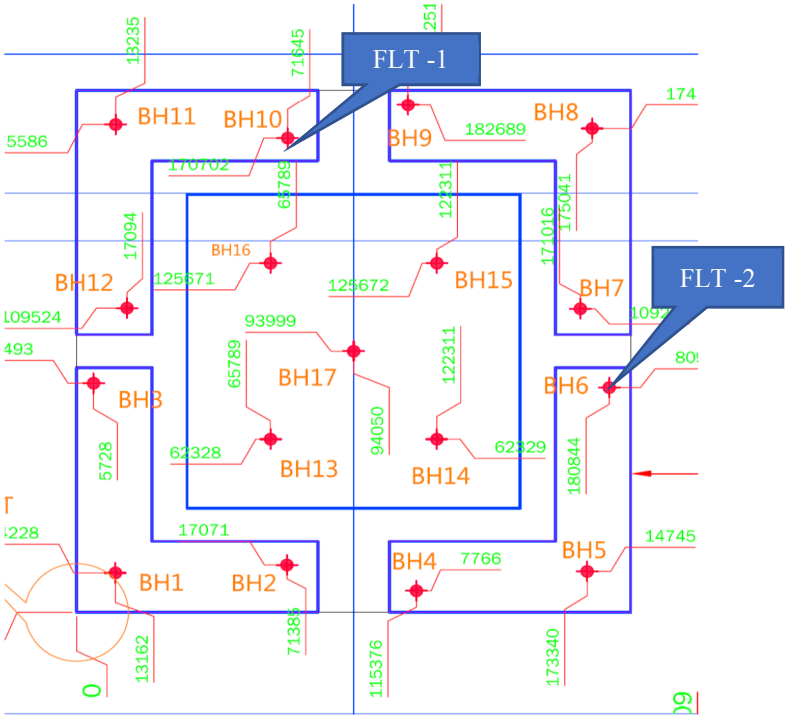



**Figure. 1. Borehole location plan**

**Table 1 Soil profile and Parameters.**

| Depth below GL | | Stratification | Average field N value | Unit weight (kN/m3) | *Modulus of Elasticity (MPa) |
|---|---|---|---|---|---|
| From (m) | To (m) | | | | |
| 0 | 6.0 | Clay | 9 to 17 | 18 | - |
| 6.0 | 9.0 | Silty Sand | 28 | 20 | 103/91 |
| 9.0 | 13.5 | Silty Sand | 30 | 20 | 103/91 |
| 13.5 | 39.0 | Completely to Highly weathered rock | >100 | 21 | - |
| Below 39 | | Hard rock | NA | 24 | - |

Note: *Modulus of Elasticity derived from Elastic theory (Footing load test data).

## FOOTING LOAD TEST

Footing load tests (FLT) were carried out at two locations near BH-06 and BH-10 as shown in the Fig 1. Tests were performed at the proposed foundation level. Square footing of 2.0 x 2.0m size with a thickness of 0.65m and embedment of 0.30 m was constructed. Test load was applied to an estimated allowable capacity of 350 kPa. Since, there is no standard specific code for FLT, IS code 1888 "Method of load test on soils" was followed for the loading sequence. The test load was applied to the square footing by a hydraulic jack, which was positioned at the center. The reaction to the hydraulic jack was provided by a kentledge platform carrying sandbags and it is supported by secondary and primary girders as shown in Fig 2 and 3. While testing, seating load (7 kPa) was applied and kept it for 24 hours. On next day, the load was applied to the footing in cumulative equal increments up to a pressure of 350 kPa. The settlement was monitored using four dial gauges for each increment of load after each time interval recommended in code. Typical schematic test setup i.e. footing, loading platform, hydraulic jack and dial gauges shown in the below Fig. 2 and Fig. 3.

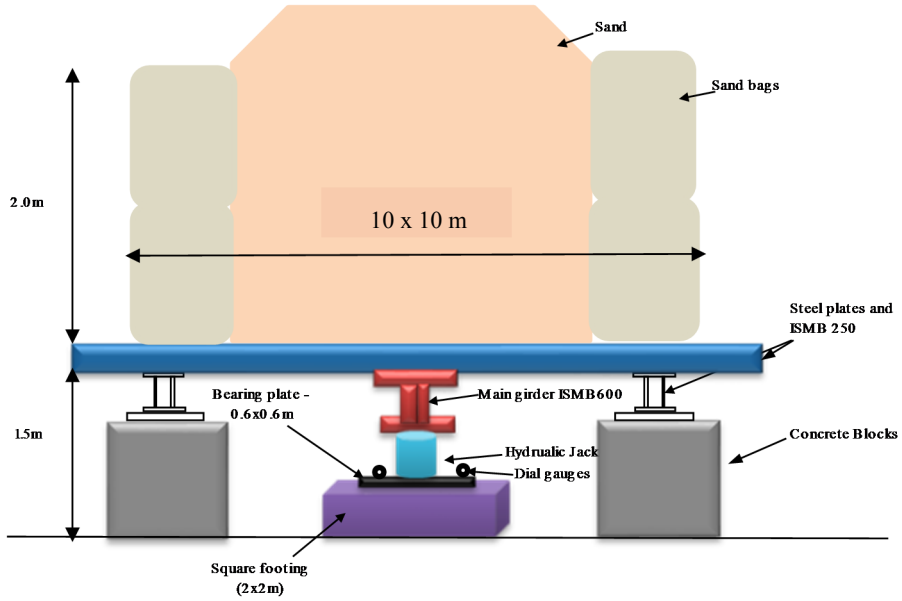



**Figure. 2. Schematic Loading platform arrangement.**



**Figure. 3. Hydraulic jack on square footing and loading platform.**

At each increment of load, constant pressure was maintained. After the completion of final load increment, the pressure in the hydraulic pump was released and the settlement of the square footing allowed to rebound. The summary of test results are summarized in Table 2 and load Vs settlement plot shown in Fig. 4 and Fig. 5

**Table 2 Footing load test results**

| Test | Test Location | Footing size L x B (m) | Depth (m) from EGL | Allowable pressure (kPa) | Settlement (mm) |
|---|---|---|---|---|---|
| FLT -1 | BH -10 | 2.0 x 2.0 | 7.5 | 353.5 | 4.70 |
| FLT -2 | BH - 06 | 2.0 x 2.0 | 7.5 | 353.5 | 4.14 |

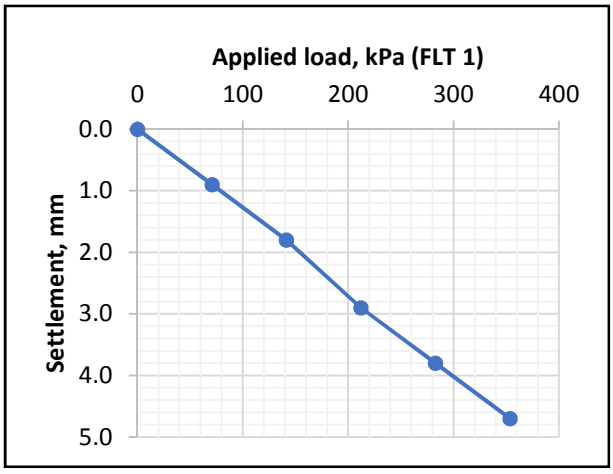



**Figure. 4. FLT-1 Load Vs Settlement**

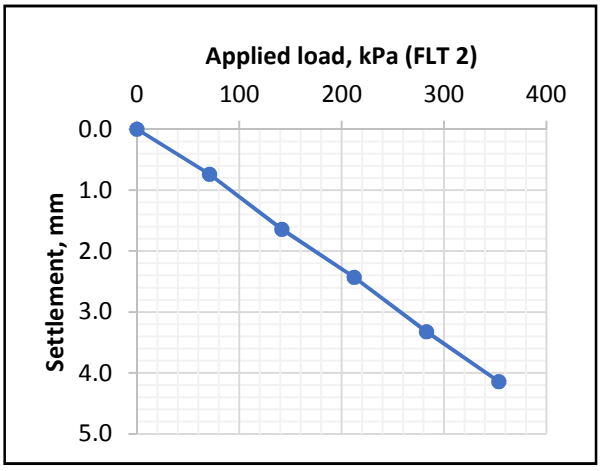



**Figure. 5. FLT-2 Load Vs Settlement**

## BACK CALCULATION OF MODULUS OF ELASTICITY USING ELASTIC THEORY FROM FIELD SETTLEMENT

Based on elastic theory [5 and 23] for uniform load applied on a footing foundation of dimension L × B at embedment depth $D_f$ in a deep elastic layer, the immediate settlement is calculated using the following equation.

$$S = q_o(\alpha B)\frac{1-\upsilon^2}{E_s} I_s\, I_f \qquad \text{---------- (1)}$$

Where,

$q_0$ = net applied pressure on the foundation

$\upsilon^2$ = Poisson's ratio of soil

$E_s$ = average modulus of elasticity of the soil under the foundation, measured from depth z = 0 to about z = 4B

B = B/2 for center of the foundation

= B for corner of the foundation

$I_s$ = Shape factor (Steinbrenner, 1934)

$I_f$ = depth factor (Fox, 1948)

α = a factor that depends on the location on the foundation where settlement is being calculated

for settlement under the center: α=4, m'=L/B, n'=H/ (B/2)

for settlement under the corner: α=1, m'=L/B, n'=H/B

The derived modulus of elasticity (E) for FLT-1 and FLT-2 values are summarized and provided in Table3.

**Table 3 Derived E values from Field test data**

| Field test data | | | | Average Modulus of Elasticity (E) kPa |
|---|---|---|---|---|
| Case | Footing size L x B (m) | Depth (m) | Allowable capacity $q_s$ (kPa) | |
| FLT- 1 | 2 x 2 | -7.5 | 353.5 | 91409 |
| FLT- 2 | 2 x 2 | -7.5 | 353.5 | 103373 |

## EMPIRICAL CORRELATION - MODULUS OF ELASTICITY USING SPT 'N' VALUES

Nearest boreholes to FLT are BH-06 and BH-05. To study the variation of inferred modulus of soil with SPT N (average), available standard empirical correlations suggested by different authors has been used. The derived modulus values and relevant empirical equations suggested by different authors along with equations are summarized and provided in the below Table 4.

**Table 4 Summary of Empirical correlations and derived Modulus of Elasticity**

| SI. No. | Soil Type | Relationship | Units | Modulus of Elasticity (E) kPa | | Reference |
|---|---|---|---|---|---|---|
| | | | | BH-06 (near to FLT-2) | BH-10 (near to FLT-1) | |
| 1 | Sand | E' = 1.0N | MPa | 28000 | 29500 | Stroud (1989) |
| 2 | Sand | E = 500 (N +15) | kPa | 21500 | 22250 | Bowles |
| 3 | silty Sand | E = 2400+530N+2100 | kPa | 19340 | 20135 | Schultze and Menzenbach (1961) |
| 4 | Sand | E = 766N | kPa | 21448 | 22597 | Schmertmann (1970) |
| 5 | Sand | $Es = a*N_{60}*Pa$ | kPa | 28000 | 29500 | Kulhway and Mayne (1990) |
| 6 | Sand | E = 18000+750N | kPa | 39000 | 40125 | D'Appolonia (1977) |
| 7 | Sand | E = 41600+1090N | kPa | 72120 | 73755 | D'Appolonia (1977) |
| 8 | Sand | E = 5N | $kg/cm^2$ | 14000 | 14750 | Parry |
| 9 | Sand | E = (15200 to 22000) ln N | kPa | 61679 | 62950 | Trofimenkof (1964) |
| 10 | Sand | $E = a * P_a (\sigma_3/P_a)b$ | kPa | 125566 | 125566 | Al Sheyea and K R Mohib (2011) |
| 11 | slightly silty Sand & Medium Sands | E = 7 N | Tsf | 18769 | 19774 | AASHTO, Sabatini et al. (2002) |
| 12 | Sand with little Gravel | E = 12 N | Tsf | 32175 | 33899 | AASHTO, Sabatini et al. (2002) |
| 13 | Sand | E = C1 + (C2*N) | $kg/cm^2$ | 16500 | 17175 | Mezenbach (1961) |
| 14 | silty Sand | E = 75+8N | $kg/cm^2$ | 29900 | 31100 | Papadopoulos (1992) |
| 15 | Sand | E=3.5N to 40N | MPa | 98000 | 103250 | Clayton et.al (1982) |
| 16 | Sand | $E = 7 N^{0.5}$ | MPa | 37041 | 38020 | Denver (1982) |
| 17 | Sand | E=(48+4N) | Tsf | 15322 | 15896 | Webb (1969) |
| 18 | Sand | E = (44N) | Tsf | 117976 | 124296 | Chaplin (1963) |

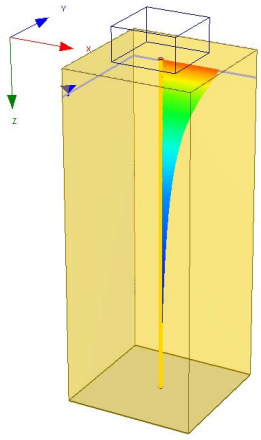
| SI. No. | Soil Type | Relationship | Units | Modulus of Elasticity (E) kPa | | Reference |
|---|---|---|---|---|---|---|
| | | | | BH-06 (near to FLT-2) | BH-10 (near to FLT-1) | |
| 19 | Sand | E = 500 (N +15) | kPa | 11125 | 10750 | Bowles |

Where,

C1 & C2 = Functions of type of sand.
Pa = Atmospheric pressure
a & b = Fitting parameters of Sand
$\sigma_3$ = Cell pressure

## PREDICTION OF SETTLEMENT USING ANALYTICAL METHOD (SETTLE-3D)

SETTLE$^{3D}$ [6] combines the simplicity of one-dimensional analysis with the power and visualization capabilities of more sophisticated three-dimensional programs. SETTLE$^{3D}$ is a 3-dimensional program for the analysis of settlement under embankment and foundations and it is widely used in the industry. In this analysis, Boussinesq' s method is considered to predict displacement in a soil mass against future structural loads which is equal to the applied pressure on FLT's. A typical settlement result of FLT-2 using modulus of elasticity value 103 MPa at a pressure of 350 kPa is shown in Fig 6 and 7.



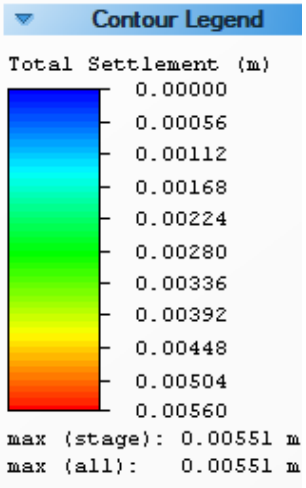



**Figure. 6. FLT-2 Soil 3D model**

**Figure. 7. FLT-2 Total Settlement**

## PREDICTION OF SETTLEMENT USING FINITE ELEMENT METHOD (PLAXIS-3D)

PLAXIS-3D (version 2018) [7] software is used to analyze the foundation performance under different pressure conditions. Footing and foundation soil is modelled using 10 noded triangular solid element. The size of foundation considered same as footing load test.

To simulate the construction sequence (in this case, footing load test), the below stages are defined during analysis.

1) First stage of excavation till -3.75m from existing ground level
2) Second stage of excavation till -7.5m from existing ground level

3) Activation of Footing and Interface
4) Activation of vertical load acting on footing

The footing was modelled as square plate with a width of 2m and thickness as 0.65m. The properties of footing are young modulus ($E_f$) =35 x $10^6$ kPa, unit weight of concrete ($\gamma_f$) = 25 kN/$m^3$and Poisson's ratio of concrete ($\nu_f$) = 0.2. In addition, interface elements were used between the footing base and soil with interface friction. Foundation soil engineering parameters considered during analysis is summerised in Table 5.

**Table 5 Soil Parameters for each footing load test**

| Field test data | | | | | Stratification | γ (kN/$m^3$) | E* (kPa) | C' (kPa) | ν | ϕ' (deg) |
|---|---|---|---|---|---|---|---|---|---|---|
| Case | B(m) | D(m) | $q_s$ (kPa) | $N_{av}$ | | | | | | |
| FLT- 1 | 2 | -7.5 | 353.5 | 28 | Sand | 18 | 91409 | 0 | 0.33 | 36 |
| FLT- 2 | 2 | -7.5 | 353.5 | 29 | Sand | 18 | 103373 | 0 | 0.33 | 37 |

Note: *Modulus of Elasticity derived from Elastic theory (Footing load test data).

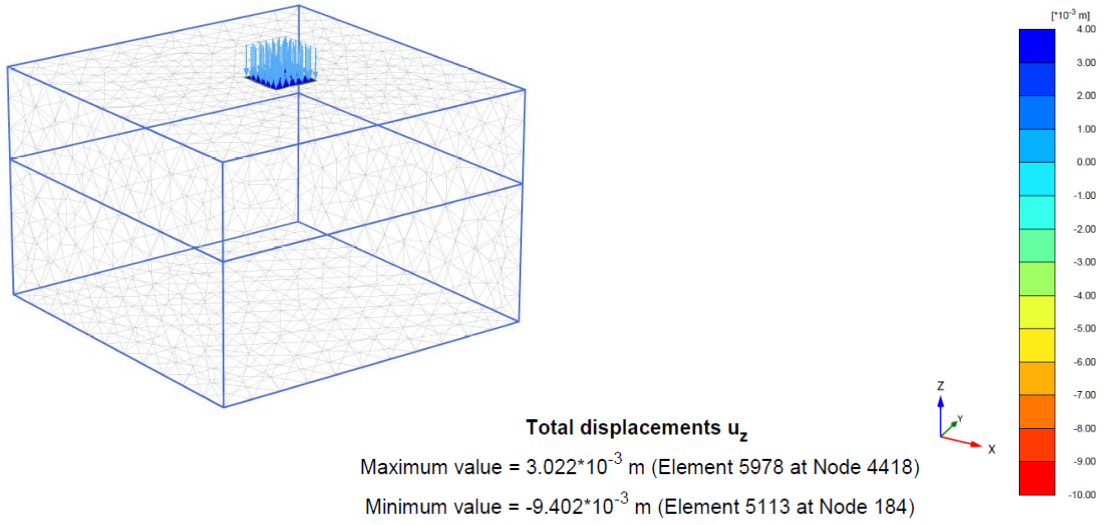



**Figure. 8. FLT-2 PLAXIS-3D model**

A typical settlement output (contour) result for the derived E using FLT-2 data (i.e. E=103 MPa and loading of 350 kPa) is shown in Fig 8

## COMPARISONS AND RESULTS

The inferred 'E' values using SPT average 'N' values for different equations commonly used in the industry is plotted and showed in the Fig 9.

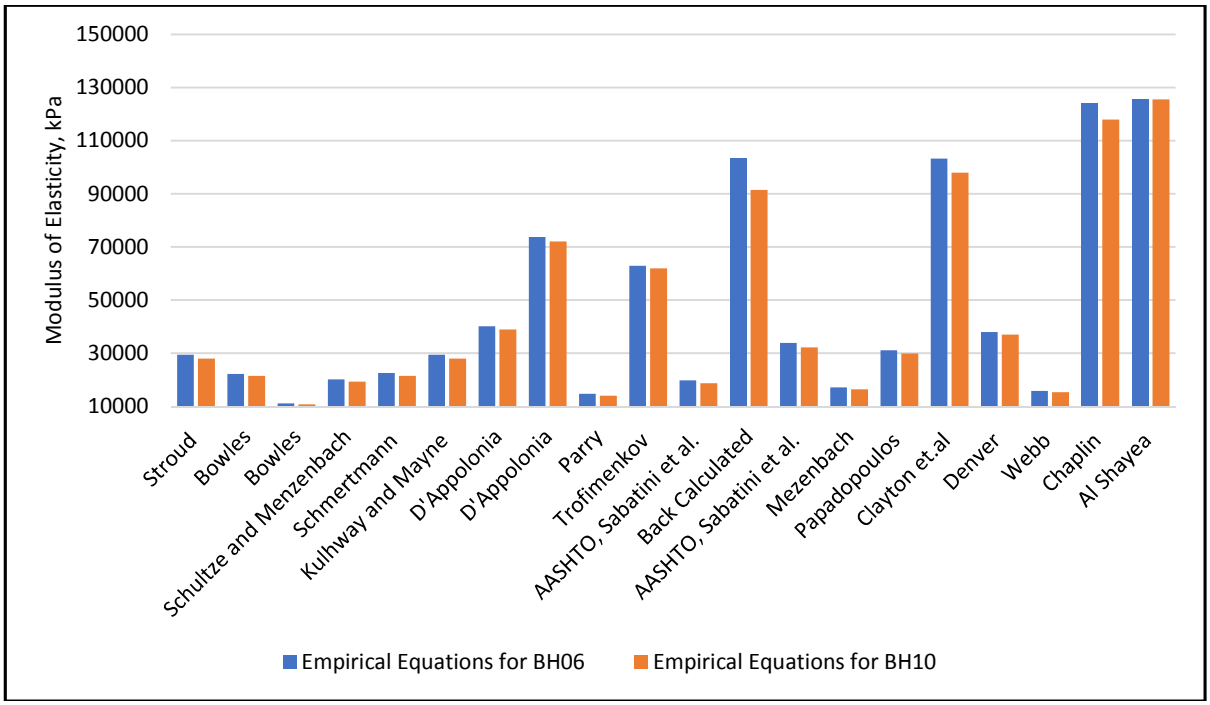



**Figure. 9. Empirical correlation of Modulus of Elasticity**

The estimated load vs settlements using analytical solution, finite element method and measured field data are plotted and provided in Fig 10 & 11. It is observed that, the settlement of the foundation is increasing continuously with the increase in pressure. The maximum displacement for FLT-01 is 12 % higher than FLT-02.

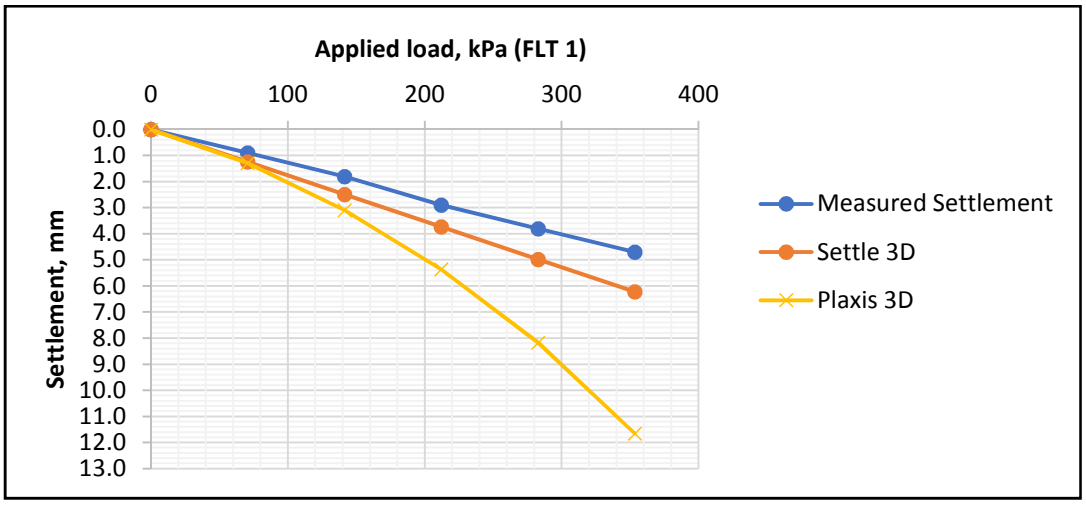



**Figure. 10. FLT-1 Load Vs Settlement graph.**

In Fig 10, the predicted settlement using SETTLE-3D software is slightly higher (approx. 6% at low pressures & 30 % at final pressure) than the measured settlement at field. The predicted settlement using PLAXIS-3D software is considerably higher (approx. 30% at low pressures & more than 100 % at final pressure) than the measured settlement at field.

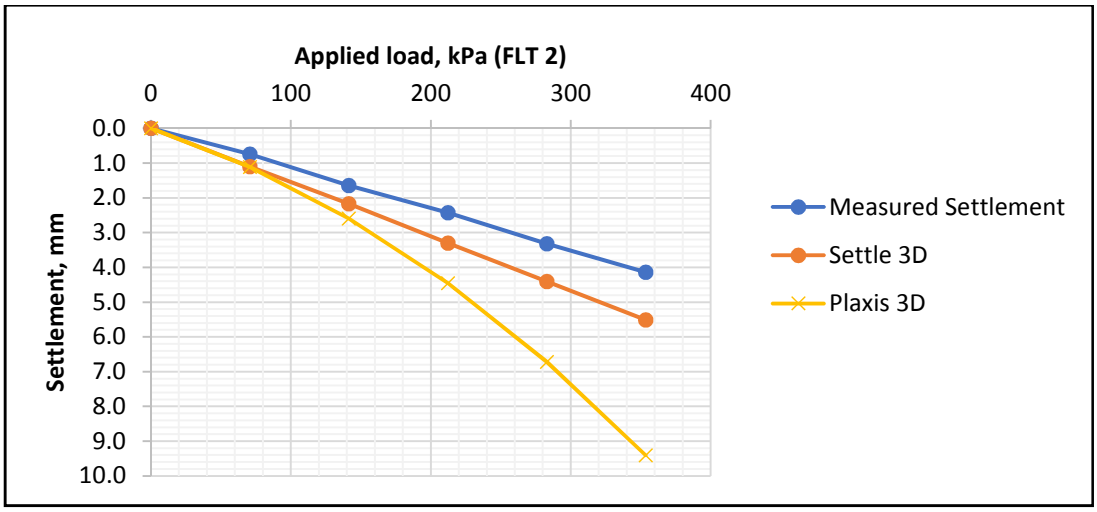



**Figure. 11. FLT-2 Load Vs Settlement graph.**

In Fig 11, the predicted settlement using SETTLE-3D software is slightly higher (approx. 2% at low pressures & 30% at final pressure) than the measured settlement at field. The predicted settlement using PLAXIS-3D software is considerably higher (approx. 30% at low pressures & more than 100% at final pressure) than the measured settlement at field.

## CONCLUSIONS

a) The ratio of measured 'E' (from field tests) with respect to the back calculated 'E' using different empirical equations using SPT 'N' values is ranging from 0.8 to 9.3 times and generally varies from 3 to 4. Based on these correlations, the modulus of elasticity (E) values ranging from 11 MPa to 125 MPa. Among the different empirical equations, Chaplin, D'Appolonia, Al-Sheyea and Clayton are consistent with derived 'E' values from the field tests.
b) It is observed that, the difference between the estimated settlement using SETTLE-3D and PLAXIS-3D, is varying from 15% to 85%.
c) The settlement from SETTLE-3D is slightly higher than the measured settlement, whereas finite element analysis gives significant higher settlement than SETTLE-3D & measured settlement.

## REFERENCES

[1] Arnold, M., 1980. "Prediction of Footing Settlements on Sand." Ground Engineering, Vol. 13, March, pp. 40-47.

[2] Burland J.B. and Burbidge M.C. (1985): "Settlement of foundations on sand and gravel." Proc. Instn. Civ. Engrs. 78 N.1 pp.1325-1381.

[3] "A new equation to predict the footings settlement on sand based on the finite element method" J. Requena, J. Ayuso, J. R. Jiménez and F. Spanish Journal of Agricultural Research 2012 10(4), 967-976

[4] "Estimation of guide values for the modulus of elasticity of soil" El-Sayed Abdelfattah El-kasaby, Benha University, Bulletin of the faculty of engineering, Assiut university, Vol-19, No.1, January 1991.

[5] Joseph E. Bowles, P.E, S.E "Foundation analysis and design" fifth edition, McGraw-Hill book company, New York.

[6] Rocscience Inc (2007-2009) – SETTLE-3D

[7] PLAXIS -3D. Finite element code for soil and rock analysis, version 2018, R.B.J. Brinkgreve and W.M. Swolfs, eds., Rotterdam, The Netherlands.

[8] D'Appolonia, D.J., E. D'Appolonia, and R.F. Brissette, "Closure to settlement of spread footings on sand". Journal of SMFD, ASCE, Vol.96, No. SM2, 1970, pp 754-762.
[9] Trofimenkov, Y.G., "Field methods for testing the structural properties of soils" Building literature publishing house, Moscow, 1964.
[10] EPRI Report EL-6800 by Fred Kulhawy and Paul W. "Estimating Soil and Rock Properties for Foundation Design"
[11] Parry, R.H.G. "Estimating Bearing of Capacity of sand from SPT values" Journal of geotechnical Engineering Division, ASCE, Vol. 103, GT9, September 1977, pp 1014-1019.
[12] Schultze, E. and Menzenbach, E., 1961. "Standard Penetration Test and Compressibility of Soils." Proceedings of the 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 527-531.
[13] Schmertmann, J.H., 1970. "Static Cone to Compute Static Settlement Over Sand." Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM3, pp. 1011-1043.
[14] Stroud, M.A., 1989. "The Standard Penetration Test-Its Application and Interpretation." Penetration Testing in the UK., pp. 29-46.
[15] Papadopoulos, B.P., 1992. "Settlement of Shallow Foundations on Cohesion!ess Soils." Journal of Geotechnical Engineering, ASCE, Vol. 118, No.3, pp. 377-393.
[16] Clayton, C.R.I., Hababa, M.B., and Simons, N.E., 1985. "Dynamic Penetration Resistance and the Prediction of the Compressibility of a Fine-Grained Sand - A Laboratory Study." Geotechnique, Vol. 35, No. 1, pp. 19-31
[17] Webb, D.L., 1969. "Settlement of Structures on Deep Alluvial Sandy Sediments in Durban, South Africa." Proceedings of the Conference on In Situ Investigations in Soils and Rocks, BGS., pp. 181-187.
[18] Denver. H., 1982. "Modulus of Elasticity for Sand Determined by SPT and CPT." Proceedings of the 2nd European Symposium on Penetration Testing, Vol. I, pp. 35-40.
[19] Chaplin, T.K., 1963. "The Compressibility of Granular Soils, with Some Applications to Foundation Engineering." Proceedings of the 2nd European Conference on Soil Mechanics and Foundation Engineering, Wiesbaden, Vol. 1, pp. 215-219.
[20] American Association of State Highway and Transportation Officials, LRFD Bridge design specifications, eight edition, November 2017. Publication Code: LRFD-8 • ISBN: 978-1-56051-654-5.
[21] IS code 1888 "Method of load test on soils"
[22] "Parameters for an Elasto-Plasto-Damage Model for the Stress-Strain Behavior of Dense Sand" Naser A Al Shayea and Khuram R Mohib, Department of Civil Engineering, KFUPM Dhahran 31261, Saudi Arabia, Article in International Journal of Damage Mechanics · January 2011 DOI: 10.1177/1056789509344041
[23] IS code 8009.1.1976 Code of practice for calculation of settlement of foundations

# ROTARY KELLY DRILLING FOR THE BORED PILE FOUNDATION OF THE HONG KONG ZHUHAI MACAO BRIDGE

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## ABSTRACT

The Hong Kong Zhuhai Macau Bridge (HZMB) is the completed "missing" link (see Fig. 2 below) in a Ring Road concept for the development of the Pearl Delta Region (Fig. 1). The HZMB is connecting Hong Kong with the southern part of Mainland China, Guangdong (formerly Canton), to the Cities of Zhuhai/Macau, Zhongshan, Dongguan and Shenzhen. The road will integrate the powerful industrial region of southern China (Guangdong) with the transport and trading hub of Hong Kong. Its total population counts around 105,000,000.00 people.

The construction of the bridge with a length of approximately 40 km, crossing the waters of the South China Sea, resulted in major challenges, particularly in the foundations. The construction of the bridge was spilt into two packages, the package on the territory of Main Land China and the package on Hong Kong territory. The Dragages-China Harbour–VSL Joint Venture (JV) was awarded to design + build the Bridge between PRC Border and Hong Kong. Save some land based piles the JV split the Bored Pile Foundation fundamentally into two areas, the shallow water and the deep water area. The following paper reports about the Bored Pile Construction for the bridge in the deep water area where piles turned out to be in excess of 110 m depth. The JV selected the Rotary Kelly Drilling (RKD) Bored-Pile Method offered by Bauer Hong Kong for these difficult piles. It is probably the first time in construction history in the world that the RKD Technique was used in such a big scale for a Mega Offshore Structure and was given the preference over the Reversed Circulation Drilling (RCD) Method.

## THE PROJECT

The HZMB-Link Road consists of various structures which are the bridge sections itself, tunnels and artificial islands to form finally the complete connection. An Artist Impression of the Key Structures is shown in Fig. 3.




Fig 1. Satellite View of Pearl Delta Region




Fig 2. Infrastructure Concept for Pearl Delta Region


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The major package on the Hong Kong side, the bridge structure leading through the Hong Kong Airport Channel to the PRC border (known as section Scenic Hill and HKSAR), was awarded to the Dragages–VSL-China Harbour Joint Venture. The bridge alignment and the location of the JV's package are shown in Fig. 4.



Fig. 3. Artist Impression of Key Structures

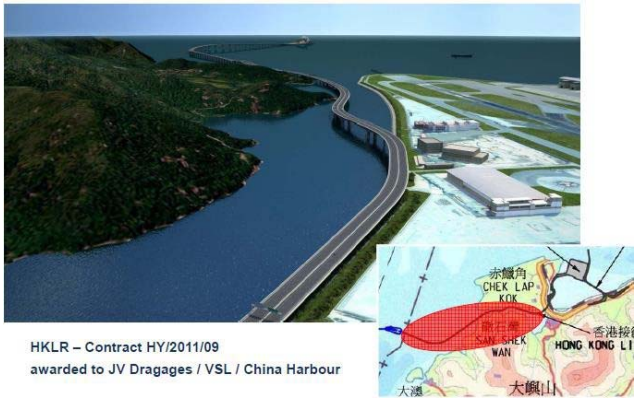



Fig. 4. Location of the JV's package

The JV decided already during the Tender Stage to work closely together with Bauer to explore the advantages of the Bauer RKD Method in regards to the Design and Construction of the Bored Piles. From the Geotechnical Baseline Report (GBR), it was expected that the Piles in the deep water section would be in the range of up to 80 m, maximum 85 m deep. The construction of such long piles with the RCD Method does usually take a very long time. The JV was from previous projects very familiar with the Bauer RKD Method and was aware of the much faster construction speed of this Method compared to RCD. The JV came to the conclusion that the overall cost of pile construction for the long piles was cheaper with the RKD compared to the RCD and in any case much faster.

The JV split therefore the offshore foundation works into two packages:

- the shallow water area close to the shore with shorter piles (RCD)
- the deep water section towards the PRC border with long piles (RKD)

The deep water section (western water) with the long piles was sub-contracted to Bauer Hong Kong. The deep water section is shown in Fig. 5.

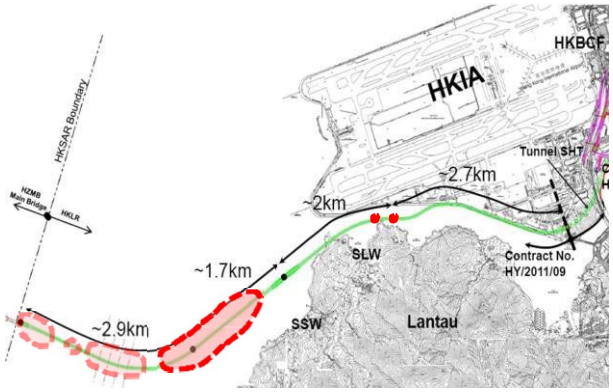



Fig. 5. Deep Water Section

## ROTARY KELLY DRILLING FOR THE DEEP WATER SECTION

### *Scope of Work and Program*

Bauer constructed for 32 piers in total 230 Bored Piles of 2,300 mm and 2,500 mm diameters between April 2013 and December 2014. In addition, one number of preliminary test pile was constructed using the Bi-Directional Load Cell Method. Bauer used four numbers of BG 40 rigs to complete the works. Working time was restricted. Due to noise limitations, generally 24 hours work was not allowed. Bored Piling Works needed to be suspended during the so called Dolphin Season (mating season) which was during the months of May and June. During this period, the Hong Kong Environmental Department did not allow rock drilling because of vibration and noise development which might disturb the dolphins (hence no Piling works). A summary of Bauer's total scope of work is shown in Table 1.

The average pile depth in the deep water section was around 72 m. The total drilled depth including empty bore was 16,638 m out of which Bauer had to excavate 1,892 m of hard rock. The Piles were at tender stage estimated to have a maximum drilling of 85 m. In the event Bauer had to construct Bored Piles to a depth of approximately 115 m.

**Table 1. Summary of Bauer's Total Scope of Work**

| | | ∅ 2300 as-build | ∅ 2500 as-build | Total as-build |
|---|---|---|---|---|
| Piles [nos] | *endbearing* | 111 | 81 | 192 |
| | *friction* | 31 | 7 | 38 |
| | *Total* | 142 | 88 | 230 |
| Drilling [m] | *Empty Bore* | 1,136 | 704 | 1,840 |
| | *soil* | 7,673 | 5,228 | 12,901 |
| | *mixed ground* | 780 | 621 | 1,401 |
| | *rock socket* | 271 | 225 | 496 |
| | *Total* | 9,860 | 6,778 | 16,638 |
| Services | *rebar [ton]* | 9,511 | 7,137 | 16,648 |
| | *concrete [m3]* | 42,122 | 33,974 | 76,096 |
| Resources | *rig month* | | | 58.3 |
| | *management* | | | 26.5 |

### *Productivity*

The analysis of the As-Built Program shows that Bauer needed in total 50.5 rig months excluding the time lost during the Dolphin Season (8 rig month) and the Lunar New Year (about 1 rig month). The productive drilling time was substantially reduced by logistical constraints on the water in particular the waiting time needed until the rig could move between pier locations and the moving time between the piers itself. The time loss, also called the "planned idling" time, was part of the original construction program. Planned idling was defined as:

- Time in which the Piling Rig was idling after drilling the last pile in a pier until concrete placing had been completed
- Time in which the Piling Rig was idling following the specification allowing excavation of a pile only 24 hours after the completion of casting concrete for any pile within a radius of 5 x diameter.

On land, such idling usually will not occur because the Rig can move on tracks from one pier location to the other. On the water, such flexibility does not exist due to the need of barge transport. Including all the unproductive time (like moving, planned idling etc.) the overall achieved production turned out to be 1.06 piles per week and rig or 4 piles a week using four numbers BG Rigs.

The actual drilling time varied between about three days (for shorter piles in the range of 50 m depth) and around 14 days for the very long piles up to 115 m. The drilling of the piles above 90 m was done with a Kelly Extension. The Kelly Extension was lifted into the hole and coupled to the Kelly by a separate crane. This process prolonged the drilling time substantially. The pictures in Fig. 6 do show the installation of the Kelly Extension for the deep pile drilling.



Fig. 6. Installation of Kelly Extension for the Deep Pile Drilling

Comparing the drilling speeds between the RCD and the RKD, it can be safely said that the RKD speed is approximately 3 times faster than the RCD. This comparison could be made only for piles up to a maximum length of approximately 50 m because that was about the maximum depth the RCD was drilling in the shallow water section. There is however no doubt that the factor for longer piles will be very much bigger. This is known from projects in Hong Kong on land where the completion of Piles close to/or around the 100 m mark drilled with RCD did take two months and longer.

## THE GEOLOGY

The sub-ground condition consists of a thick layer of Marine Clay over Alluvium and Granite. The Granite is in the upper layers completely too highly decomposed before turning into moderately strong G III to G II rock. The Hong Kong Granite G III to G II rock has commonly unconfined compressive strength in the range of 90 MPa to > 200 MPa. The Geological Profile as anticipated in the early stages of the contract is shown in Fig. 7. Figures 7 and 8 show that the thickness of the Marine Clay increases towards the deep water section and the granite rock head level dips down into depth of – 80 m below seabed level.

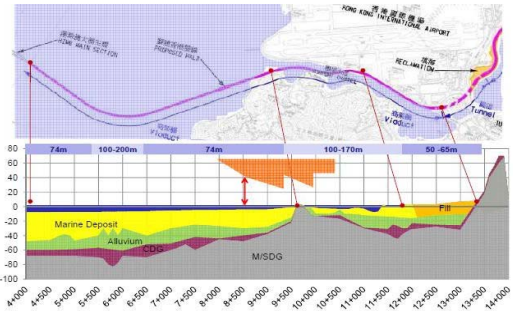



Fig. 7. Geological Profile

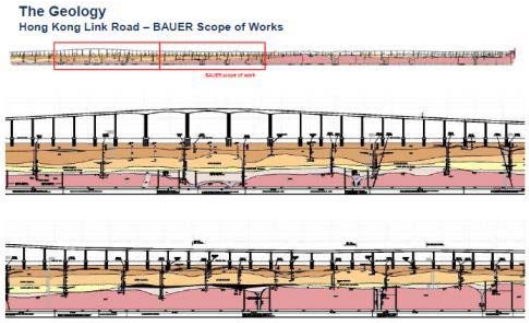



Fig. 8. Geology in Deep Water Section

The Engineer's requirement was that at every pile location a pre-drill had to be done from which cores were recovered. These cores formed the basis to determine the Engineering Rock Head and determine the founding level of the pile. Engineering Rock Head was defined in accordance to the Hong Kong Building Regulations. The definitions of the rock categories 1(a) to 1(d) are shown in Table 2 in the chapter Pile Design + Layout below.

The detailed predrilling at every pile ensured that all piles were constructed in defined (known) conditions and there is no risk of dealing with undetected fault structures or weathering grades. Such practice proved to be necessary. In fault zones the pre-drills did not find competent engineering rock until sometimes -100 mPD to -140 mPD. For these piers the pile design needed to be revised as explained further below.

Figures 9 to 11 show the soil investigation carried out and the result of the predrilling as an example which is done for every pile location.



Fig. 9. Soil Investigation at each Pile Location from Platform or Barges

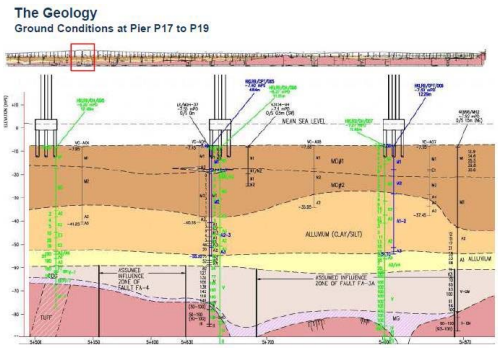



Fig. 10. Precise Investigation of Ground Conditions at Pile Location




Fig. 11. Rock Core Samples Retrieved from Predrilling

## PILE DESIGN AND LAYOUT
### *The Design Concept and Principles*

For the standard piers the bridge has a span of 75 m. The span for the navigation channel is 150 m, double of the standard span. Figures 12 to 14 show the typical pile cap arrangements. The Bored Piles have diameters of either 2,300 mm or 2,500 mm.

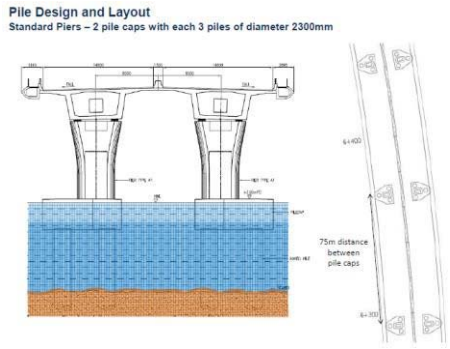



Fig. 12. Pile Cap Arrangement for Standard Piers

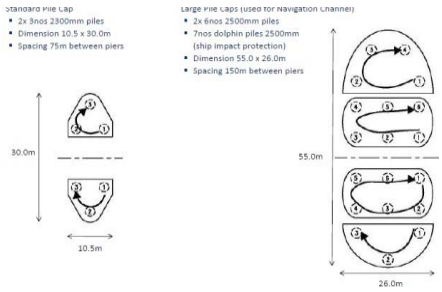



Fig. 13. Pile Cap and Bored Piles for Standard Pier and Navigation Channel

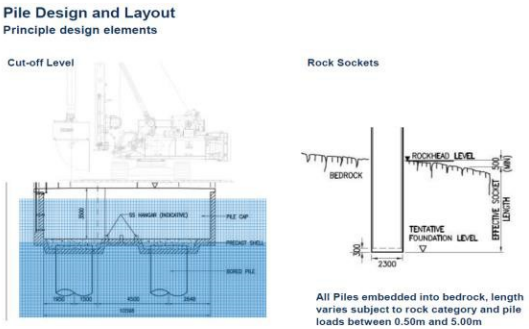



Fig. 14. Typical Socket Detail

The Bored Piles were typically designed as end bearing piles with friction allowed only in the engineering rock socket. The socket length of the Bored Piles varied, depending on weathering grade, between 0.5 m to 5.0 m per pile. The founding criteria in dependence of the rock categories encountered are summarized in Table 2 below. Allowed skin friction was usually 10% of the endbearing.

**Table 2. Founding Criteria for Endbearing**

Pile Design and Layout
Principle design elements - Founding criteria for bored piles

| Rock Category | Description | Allow. bearing pressure (kPa) | Min. embedment (m) | Effective socket length (m) | Design Socket Length (m) |
|---|---|---|---|---|---|
| 1(a) | Fresh strong to very strong rock of material weathering grade I, with 100% total core recovery (TCR) and no weathered joints (UCS ≥ 75 Mpa) | 10,000 | 0.50 | Ø2300mm 0.00<br>Ø2500mm 0.00 | Ø2300mm 0.50<br>Ø2500mm 0.50 |
| 1(b) | Fresh to slightly decomposed strong rock of material weathering grade II or better, with a TCR of more than 95% (UCS ≥ 50 MPa) | 7,500 | 0.50 | Ø2300mm 0.00 – 1.30<br>Ø2500mm 0.35 – 2.15 | Ø2300mm 0.00 – 1.80<br>Ø2500mm 0.85 – 2.65 |
| 1(c) | Slightly to moderately decomposed moderately strong rock of material weathering grade III or better, TCR >85% (UCS ≥ 50 Mpa) | 5,000 | 0.50 | Ø2300mm 1.50 – 3.35<br>Ø2500mm 2.60 – 4.40 | Ø2300mm 2.00 – 3.85<br>Ø2500mm 3.10 – 4.90 |
| 1(d) | Moderately decomposed, moderately strong to moderately weak rock of material weathering grade better than IV, TCR > 50% | 3,000 | 0.30 | n.a. | n.a. |

In the event the predrilling for some of the piers in the deep water section did not detect engineering rock head until over 125 m deep. Since the original design allowed friction and end bearing only below engineering rock head, an additional socket length would have been necessary and the length of piles for such conditions would have ended up at 130 m as a minimum. The construction of such deep piles is not impossible but would have been very time consuming and expensive (at the time of writing this paper, Bauer Malaysia constructs Bored Piles diameter 2.5 m diameter to design depths up to 150 m w/o the use of a Kelly Extension on Land in Kuala Lumpur).

The only option to overcome the difficult geological conditions without changing the cap structure and pile configuration was to construct piles for which friction would be considered already in the area above engineering rock head (Alluvium, CDG, HDG, Mixed Rock). However, using the friction values which are allowed in Hong Kong by the standards in similar soils the length of the piles would have been equally excessive. Therefore Bauer suggested to use shaft grouting to enhance the friction capacity.

Shaft grouting of Bored Piles in essence means to crack the outer concrete surface of the Bored Pile and pump grout under high pressure into the surrounding soil. The injected grout enhances the skin friction by increasing the bonding between soil and the Bored Pile.

At the end, in total 38 numbers of Bored Piles out of 230 piles were redesigned from rock socket piles to shaft grouted friction piles.

The difficulty was to establish the appropriate friction parameters for the shaft grouted areas in the different types of sub-ground materials. Hong Kong has experience with shaft grouting in Alluvium and Completely Decomposed Granite (CDG). For Alluvium, the ultimate friction of a shaft grouted pile is allowed to be taken as 6 x N (N=SPT blow count) and for CDG 2.4 x N with a limit of 200 KN/m².

In this case the shaft grouting was however to be done also in very heterogeneous material which was a mix of Highly Decomposed Granite (HDG) and mixed rock of various grades. For this material, no friction parameters from past experiences were known and needed to be determined.

To establish the safe friction parameters a full scale instrumented pile load test was constructed and tested in the offshore conditions.

*Full Scale Test on Shaft Grouted Bored Pile*

The pile was constructed by Bauer and the test was carried out by an independent party, Fugro Singapore Pte Ltd. The Test Method was the Bi-Directional Load Cell, which Fugro trades under the name Osterberg Cell Test. The test pile design and test arrangement is shown in Fig. 15. The pile and test was prepared for an ultimate load of 100 MN.

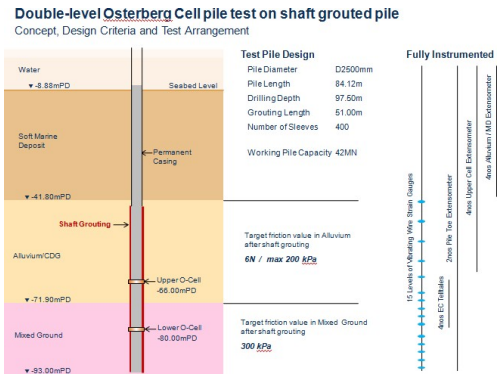



**Fig. 15. The Principle Test Pile Design and Test Arrangement**

The test was planned as a multiple stage loading test using two Osterberg Load cells (O-Cells) in order to split the pile into three distinct sections.

The upper O-Cell was located at a level of -66 mPD so as to mobilize the ultimate shaft resistance within alluvium and marine deposit and the upper part of the CDG. The lower O-Cell was located at a level of -80 mPD which resulted in a 14 m thick separation between the two O-Cells in the middle section of the test pile.

The ground conditions within the middle section of the test pile comprised of 6 m CDG underlain by 8 m of HDG. Figure 16 shows the schematic configuration of the test pile and the location of O-Cells. The ground conditions adjacent to the bottom section of the test pile include a mixture of HDG with a limited amount of "mixed rock" (ie. Grade III rock).

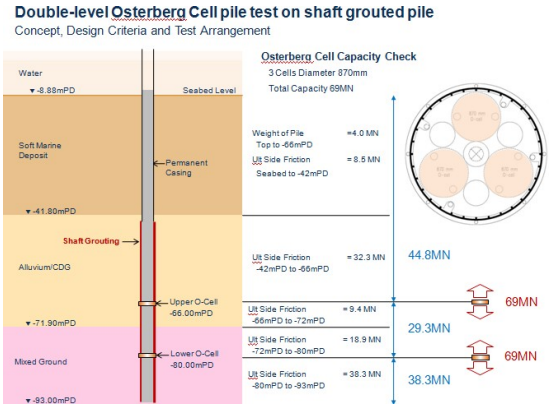



**Fig. 16. Configuration of Test Pile and O-Cell Location**

Three 870 mm diameter O-Cells were installed at both the upper and lower O-Cell locations with a maximum rated capacity of 69 MN (i.e. 3 times 23 MN). The rated capacity of 69 MN was well in excess of the expected capacity of the upper and middle sections of the test pile. Although the O-Cells had a rated capacity of 69 MN, in practice it is possible to achieve a higher load. During the actual load test, the lower O-Cells failed at a load of approximately 114.3 MN, almost double of the nominal capacity.

The Test Pile was fully instrumented to enable the determination of both, the load distribution along the pile length and the movement at the top and bottom of each section of the test pile.

The Pile Test was carried out in three distinct stages. The overall aim of each stage was to determine design parameters for the upper, middle and lower sections of the pile at each subsequent load stage. The loading procedures are summarized in Table 3. The testing sequence is shown in Fig. 17.

**Table 3. Loading Procedures**

| Stage | Action | Envisaged Behavior |
|---|---|---|
| 1 | Top O-cells are Active & Loaded<br>Bottom O-Cell are Locked | Side friction resistance failure of ALL and CDG<br>Additional test data of shaft friction from middle section |
| 2 | Top O-cells are Open<br>Bottom O-cells are Loaded | Side friction resistance of CDG and HDG in middle section<br>Additional test data on shaft friction and end bearing below Lower O-cells |
| 3 | Top O-cells are Closed<br>Bottom O-cells are Loaded | Additional test data on shaft friction and end bearing below Lower O-cells |

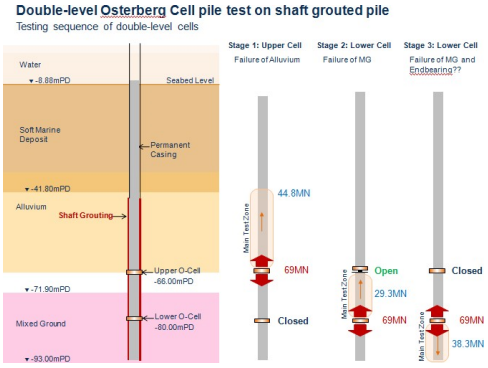



Fig 17. Testing Sequence of O-Cells

The bearing performance of the test pile was very good. The upper part of the pile reached a load of approximately 62 MN. The ultimate load which was put on the lower part of pile was higher than the O-Cell capacity at about 109 MN. Even until this high load still the lower section of the pile did not fail.

The analysis of the test readings showed that the upper part of the pile in the Alluvium / CDG had an average friction of about 300 kPa. This was significantly greater than the standard design values. The lower part of the CDG had an average friction of 680 kPa. The achieved side friction in the mixed ground was 760 kPa. The measured end bearing at this stage was 7.3 MN, far below the ultimate capacity. The test results are summarized in Figure 18. Figure 19 shows the installation of the rebar cage with the double O-Cells and Pile Instrumentation.

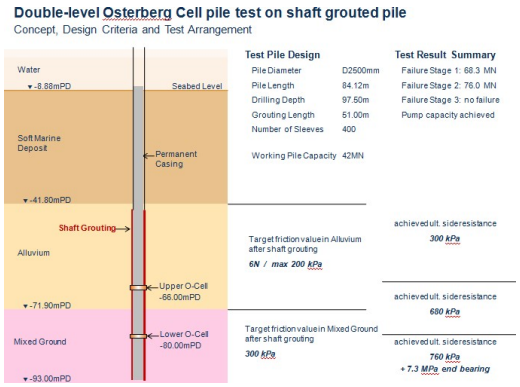



Fig. 18. Test Pile Results




Fig. 19. Installation of Rebar Cage with Double O-Cell and Instrumentation

## PRINCIPLE WORKING METHOD AND EQUIPMENT SET UP FOR OFFSHORE ROTARY KELLY DRILLING (RKD)

### *Set Up & Work Preparation*

The selected method for the Bored Pile construction in the deep water section was RKD. In principle the method is onshore and offshore the same. The significant point is that the construction of large diameter 2,300 mm and 2,500 mm piles need heavy equipment which in turn needs a stable platform whether on land or offshore. Offshore the platforms for the BG Drilling Rigs can be either Jack Up Barges / other

type of barges depending on water condition or Steel Deckings/Platforms. For the HKZMB, the JV selected to construct steel deckings as platform for the BG – Drilling Rigs. The installation of the platforms was done well in advance of the Bored Pile Equipment mobilization to the Pier location. The steel deckings were designed and constructed in a manner that they could be dismantled after the completion of the Bored Piles at one Pier and installed at the next Pier (reused). This reduced substantially the numbers of platforms needed. The permanent casings needed for the Bored Piles were vibrated into the seabed once the platform was completed. The permanent casing length was substantially shorter than for the RCD Method because for RCD the casing serves as the carrier of the rig (substantial cost savings). Hong Kong usually requires for RCD that casings reach rock material. The casings for the RKD could be stopped in the upper layer of the Alluvium below the Marine Clay. The typical construction steps of erecting the piling platform and installing the permanent casing is shown in Fig. 20 and Fig. 21.

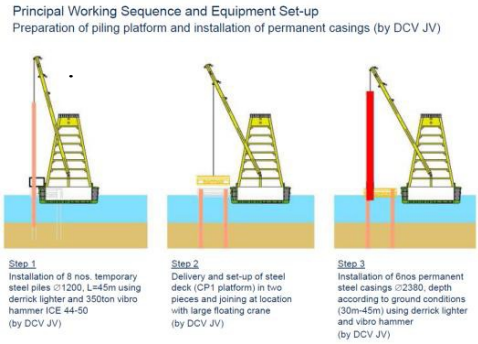



Fig. 20. Erection of Piling Platform and Casing Installation

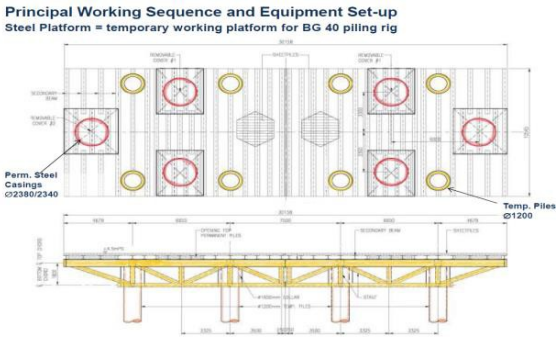



Fig. 21. Layout & Section through Piling Platform with Casings

After the installation of the platform the Piling equipment is mobilized to the pier location. The BG Drilling Rig type BG 40 was mobilized by a RoRo barge equipped with a movable ramp to the platform. For moving the BG 40 Drill Rig to the platform tidal levels and wave actions needed to be considered carefully. Fig. 22 and Fig. 23 show the movement of BG 40 Rig on to the piling platform.

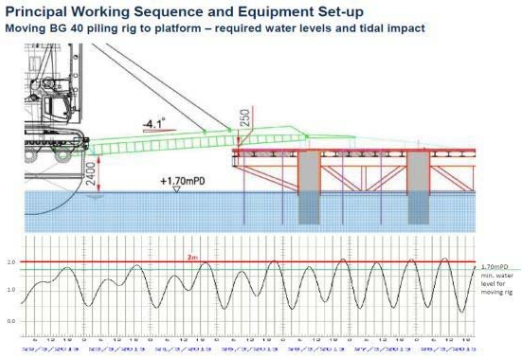



Fig. 22. Sketch of Moving of BG 40 to Piling Platform




Fig. 23. Actual Moving of BG 40 to Piling Platform

Following the Rig set up on the platform the supporting equipment needed to be placed next to the platform to facilitate the Bored Pile construction. The supporting equipment consisted of a spoil barge, crane barge, tool barge and bentonite slurry barge. A typical set up for a standard pier is shown in Fig. 24 and Fig. 25 as sketches.

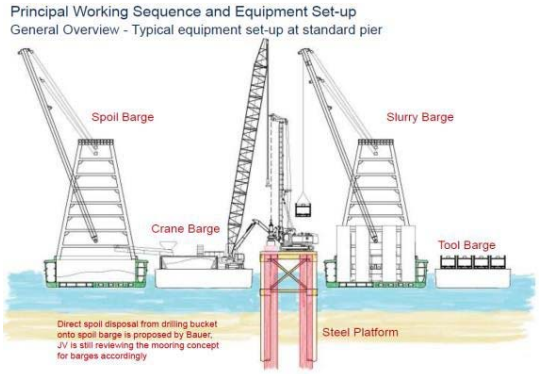



Fig. 24. Sketch of Equipment Set Up

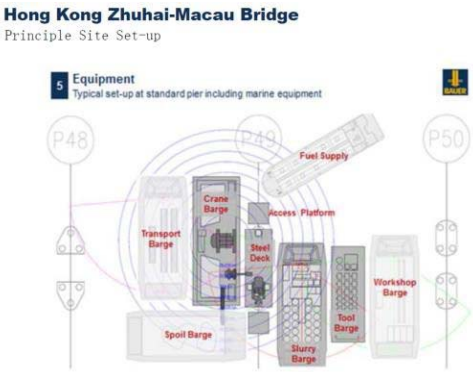



Fig. 25. Sketch of Layout of Working Set Up

The actual site situation and working set up are shown in the Figures 26 + 27 as aerial view and picture from the site.



Fig. 26. Aerial View Working Set Up



Fig. 27. Picture of Barge Set Up

The Key element for off-shore piling is work planning, preparation and provision of back-up facilities. The equipment was modified to be fit for offshore drilling. Barges needed to be specifically designed and modified to be suitable for the RKD method. 3D studies were used to arrive at the optimal barge design and set up.

Every construction step needs to be preplanned in detail. The planning needed to be reviewed almost on a daily basis to organize the timely provision of temporary and permanent materials, consumables and last not least engineering supervision. Expensive inefficiencies were to be avoided. Bauer provided an experienced logistic and planning team for this purpose. At the peak Bauer had a workforce of 340 staff for 4 operating Rigs working 24 hrs out of which almost 100 staff were placed onshore.

Sufficient spare parts and complete modules like a spare BG Rig, Spare Kellies, spare tools needed to be on standby in order to prevent interruptions of the construction process in case of equipment breakdown or regular maintenance. The parts and modules were stored in Yard which was fully equipped to serve also as a Workshop for manufacturing and maintenance of tools and equipment. This facility was set up close to the shore with berthing facilities.

Fig. 28 shows the Bauer Workshop set up which was located far from the workfront. In order to deliver parts from the workshop to the workfront it needed 4 hours of boats ride.



Fig. 28. Aerial View of Bauer Site Workshop

***Pile Drilling and Installation of Permanent Materials (concrete + reinforcement)***

Once the site set up was completed the Bored Pile construction did commence. The working steps on water follow in principle the same sequence as on land. One difference was in this case that the environmental regulations required the removal of the Marine Clay to a designated dumping ground. The Marine Clay was not allowed to be mixed with bentonite slurry. Therefore the excavation through the Marine Clay was done within the permanent casing without water or slurry (dry). The excavation of the Alluvium commenced under water surcharge and continued with Bentonite slurry (partly Hybrid Slurry using Polymer additives) at a depth when the uplift forces (boiling) at the pile bore base were too big but in any case always when the bottom of the permanent casing was reached. The steps of borehole stabilization are shown in Fig. 29.

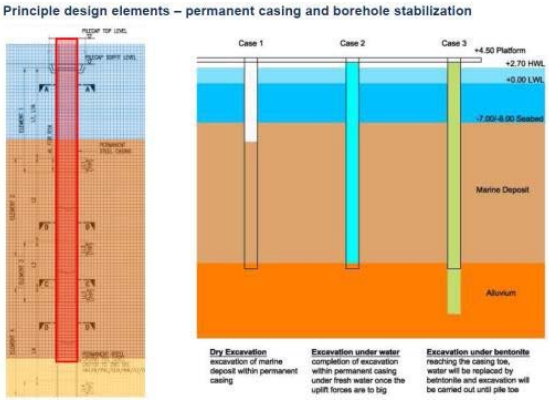



Fig. 29. Steps of Borehole Stabilization

The principle steps of Bored Pile Construction are as follows:

- Soil excavation with bucket or auger within casing
- Continue excavation under Stabilization Fluid
- Rock excavation using Core Barrels, Cross Cutter or other Tools
- Cleaning of Pile Toe with Cleaning Bucket
- Recycling of Stabilization Fluid
- Installation of Rebar Cage
- Concreting of Bored Pile using high slump self compacting tremie concrete Prior to material

installation, Koden Verticality Tests were carried out.

Site Photos of individual working steps are shown in Figures 30 to 33.



Fig. 30. BG 40 Drilling on Platform




Fig. 31. Emptying Bucket into Spoil Bin




Fig. 32. Rock Drilling and Recovered Rock




Fig. 33. Recycling of Slurry Cores

The installation of the permanent material required major logistic planning and arrangement. Reinforcement Cages had a weight of up 100 tons and the concrete volume could reach 500 m³ per pile. The heavy rebar cages, equipped with sonic logging pipes, needed to be coupled over the borehole.

Concrete was delivered either using truck mixers or alternatively was delivered directly from a floating batching plant and pumped from a concreting barge to the pile location. Fig. 34 and Fig. 35 show the concreting on site.

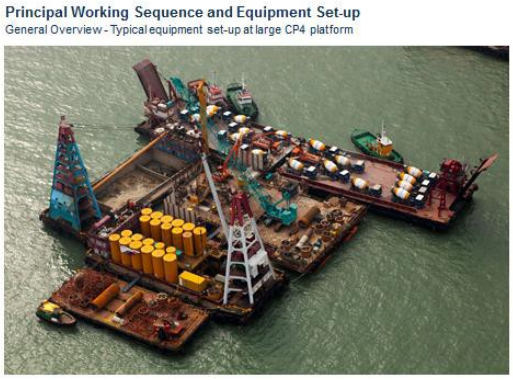



Fig. 34. Aerial View of Concreting



Fig. 35. Concreting of Bored Piles

## POST CONSTRUCTION TESTING

The specification required the following testing of the Bored Piles:

- Sonic Logging of all piles
- Interface Coring of all piles
- Full concrete coring of selected piles (8%)

The sonic logging tests showed all sound quality of the concrete. The rock interfaces similarly showed a clean basis of the pile toe without bentonite contaminations. In cases where minor imperfections of interfaces were suspected or the rock quality at the interface below the pile base showed fracturing a post toe grouting was carried out through access pipes (sonic logging pipes) which were installed into the pile at the rebar cage prior to concreting. Photographs of typical interfaces are shown in Fig. 36. and Fig. 37.




Fig. 36. Interface between Concrete and Rock at Pile Toe

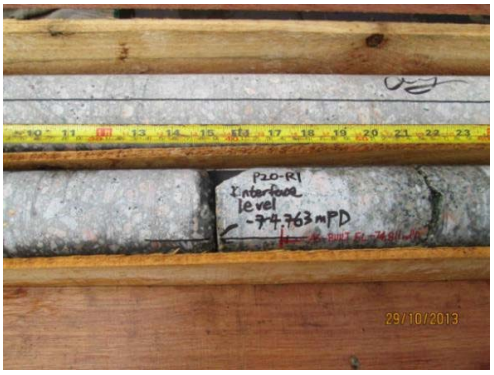



Fig. 37. Interface between Concrete and Rock at Pile Toe

This paper is concluded with adding two photos of the completed bridge in Fig. 38 and Fig. 39 of this impressive Mega Structure.



Fig. 38. Bridge before entering the tunnel



Fig. 39. Bridge Section with regular span

## SUMMARY

The paper served to report about the successful completion of the Foundation in the Deep Water Section of the HKZMB using the RKD Method of Bauer to install large diameter Bored Piles diameter 2,300 mm and 2,500 mm to a depth of about 115 m. It is the first time that Bauer RKD Method was used for a Mega Offshore Project worldwide. After the completion of all Piles with a construction speed faster than

original planned it is proven that RKD is a cost effective Method to construct Large Diameter Bored Piles for Sea/Water Bridges. The additional cost for platforms can be minimized by proper planning and are compensated by considerable cost savings due to much greater construction speed (savings in marine facilities) and by savings in length and thickness of permanent casings needed for RCD. The strict quality control and testing showed that all piles were constructed to the full technical requirements and strict specifications. The success for the construction of such a technical and logistical demanding project is however always the outcome of excellent teamwork between Designer, Main Contractor, Foundation Contractor, Suppliers and other sub-contractors and of course the Employer.

# CAPACITY OF PILES IN POND ASH BASED ON MODEL PILE LOAD TESTS

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## ABSTRACT

The electric power requirements of this country at present are primarily met from coal based thermal power, which in turn generates huge quantity of coal ashes as by-product. The un-utilised coal ash is generally disposed in to ash ponds. The ash ponds abandoned after their capacity is reached, are occupying vast areas of land. As these areas are generally not preferred for cultivation, there is great necessity for their utilization. Solar plant installation is one of the ways of utilizing these abandoned ash ponds. In view of the wind induced uplift forces acting on these solar installations, the preferred foundation type is pile foundations. However, it is possible only if the interaction of piles in pond ash is well understood. In view of this efforts are made in this study by carrying out a series of model pile loads tests in laboratory and understand the interaction.

The model piles made of cement concrete with mesh reinforcement are installed in to the ash pond test bed and its ultimate capacity is determined by applying monotonically incremented loading. The length of embedment, the density of the test bed and saturation state are studied systematically.

The study indicated that, the nature of failure is brittle in pond ash and the effect of other variables is similar to that of cohesionless soils such as Sands. A comparative study is also conducted on sands to have a correlation among different soils. This study gives insights into the behaviour of the pile in the pond ash and it's utilisation in the field.

**Keywords: Pond Ash, Model Pile load tests, Ultimate capacity**

## INTRODUCTION

Pond Ash is one of the three types of ash formed at thermal power plants namely, Fly Ash, Bottom Ash, Pond Ash. "Pond Ash" as the name suggests comes from the vast area of Ash pond formed by flushed ash from the power plants. The power needs of this country are primarily met through thermal power plants, ash ponds occupy vast areas of land, as, it is evident that these ash ponds are going to increase over period of time. In the time when acquiring lands has become difficult, efforts must be put to efficiently utilize these increasing ash pond sites.

The scope of this paper is to present the ultimate capacity results of model piles in model test bed by variation of model test bed density, length of embedment of pile, diameter of pile, saturation condition.

## LITERATURE REVIEW

Research has initially dealt pond ash as a replacement to soil, precisely as a fill material and therefore early works have been inclined towards determination of shear properties under Pandian NS, Sridharan A, Chittibabu (2001), Jakka RS, Datta M, Ramana GV (2010). Following this the strength aspect pertaining to bearing capacity of pond ash with or without surcharge and reinforcement has been conducted and reported with respective to each case under Bera, A.K. Ghosh, A (2007) (2009), M. V. S. Sreedhar, G. Venkatappa Rao, R. Ramesh Reddy (2011). The research until now has made progress pond ash as a bearing stratum for shallow foundation, embankment fill material, as a part of Mechanically Stabilized Earth Walls etc.


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## METHODOLOGY

The methodology adopted in this study includes performing a series of model Pile Load Tests on the test bed prepared with varying densities and saturation states. The experimental results were analysed to evaluate the effect of reinforcement on the resonant frequency and the magnitude of peak displacement. The detailed procedure is presented in the following sections.

## MATERIAL CHARACTERISATION

The pond ash used for this model test study is collected from ash pond site at National Thermal Power Corporation Limited (hereafter NTPC)-Ramagundam, Telangana, India. The pond ash is collected from a depth 2m below the surface level of ash pond. Ordinary Portland Cement (hereafter OPC) is used with steel mesh reinforcement for casting of circular model piles with varying embedment lengths, i.e., 250mm, 200mm, 150mm respectively. Coarse sand and cement mortar are used and placed into mould with reinforcement fixed in place. Aforementioned dimensions are considered to incorporate the testing conditions in the laboratory.

The characteristics of the materials used in this study are as summarized below in "Table 1", tests performed are as per Indian Standards (IS). (IS Heavy Compaction Test hereafter IS HCT)

**Table 1. Characteristics of the Pond Ash**

| S.no | Parameter | Value |
|---|---|---|
| 1 | Specific gravity | 1.92 |
| 2 | Gravel sized particles (%) | 4 |
| 3 | Sand sized particles (%) | 67.67 |
| 4 | Silt sized particles (%) | 20.93 |
| 5 | Clay sized particles (%) | 7.4 |
| 6 | Consistency | Non-Plastic |
| 7 | Coefficient of uniformity, Cu | 19 |
| 8 | Coefficient of curvature, Cc | 3.37 |
| 9 | Maximum Dry Density (MDD) (IS HCT) in (gm/cc) | 1.29 |
| 10 | Optimum Moisture Content (OMC) (IS HCT) as a (%) | 22 |
| 11 | Classification (IS:1498-1972) | SM |

## EXPERIMENTAL SETUP and PREPARATION OF TEST BED

Efforts were made to simulate the test procedure prescribed in IS 2911-part 1- section 1(2010). The test bed is prepared in volume control technique, where the amount of pond ash/soil and water to be added is calculated for controlled depth and area dimensions. The amount of dried pond ash/soil is calculated along with the amount of water required for the particular target dry density. The pond ash/soil is mixed with the calculated amount of water (in litre) and placed in the tank. The placed pond ash/soil is compacted until the control depth assumed (5cm in this case) and the process is repeated layer after layer, to achieve proper compaction throughout the test bed. The model pile load tests are conducted in tank of dimensions 45cm x 45cm x 45 cm. The model tank sizes incorporate the pressure bulb of model pile generated by loading in both the cases, with sufficient space on all sides. The schematic representation and a view of the test bed is shown in Fig. 1 and Fig. 2.

The model pile load test is conducted by preparing pond ash bed of required dry density as a percentage of MDD i.e., 85%, 90% and 95% of MDD with optimum moisture content for the above-mentioned cases. The test has been conducted for zero moisture content and saturation moisture content for dry density as 85% of MDD, to observe the variation with moisture content. The model pile will be driven in every case and monotonically increasing loads will be applied. The loading and settlement of pile

will be recorded simultaneously with the help of proving ring of given capacity and dial gauge. A similar test with sand will be conducted for a model pile, for a test bed dry density as 95% of MDD, to have a comparative result.

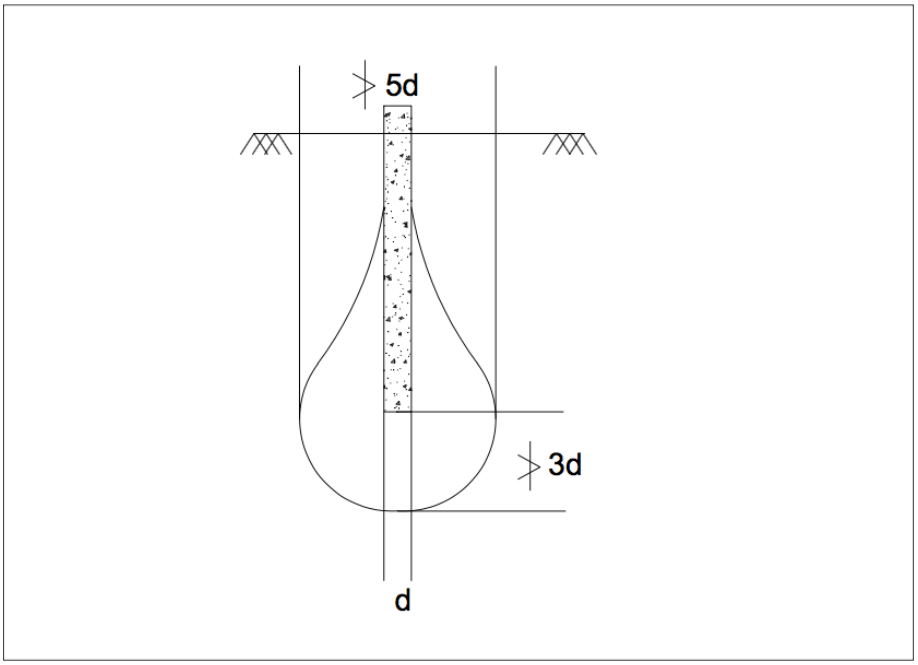



**Fig-1: Depiction of pressure bulb for a pile.**



**Fig-2: A view of the experimental setup.**

## TEST PROCEDURE

The test procedure is of Initial Test type generally in accordance with IS: 2911-part 4. The model pile will be driven in every case and monotonically increasing loads will be applied. The loading

and settlement of pile will be recorded simultaneously with the help of proving ring of given capacity and dial gauge. The Load vs settlement graph is plotted for every case, i.e., different model pile embedment length, different dry density as percentage of MDD and different saturation states.

The procedure is repeated for 3 different embedment lengths as following: 250mm, 200 mm, 150mm with varying aforementioned parameters in 3 cases respectively. The variation is plotted and represented in graphs in the following section.

## EXPERIMENTAL RESULTS

### *Load Vs Settlement for Each Case*

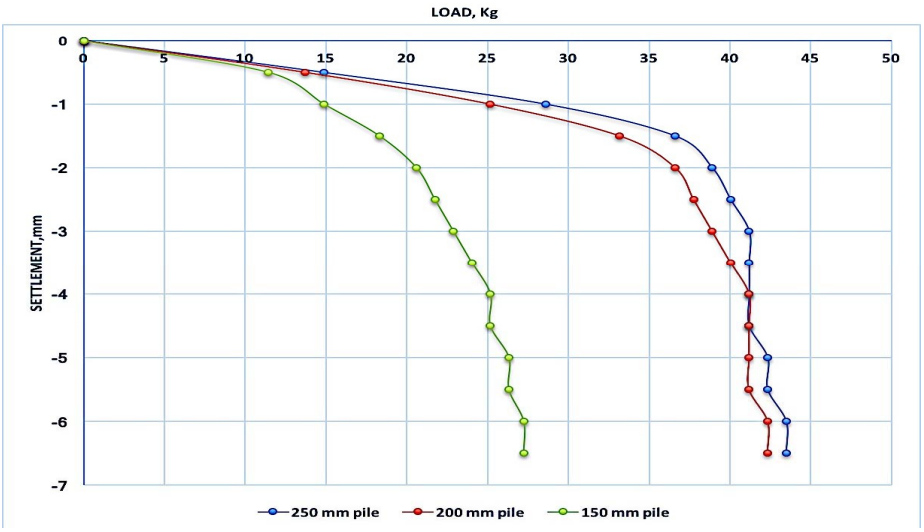



**Fig-3: Model pile load test for Dry density as 85% of MDD.**

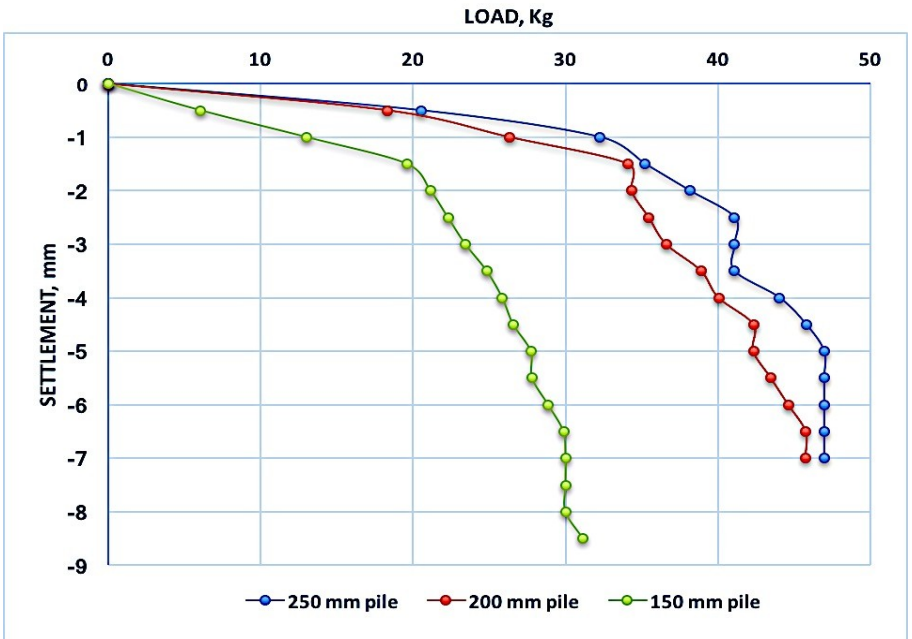



**Fig-4: Model pile load test for Dry density as 90% of MDD.**

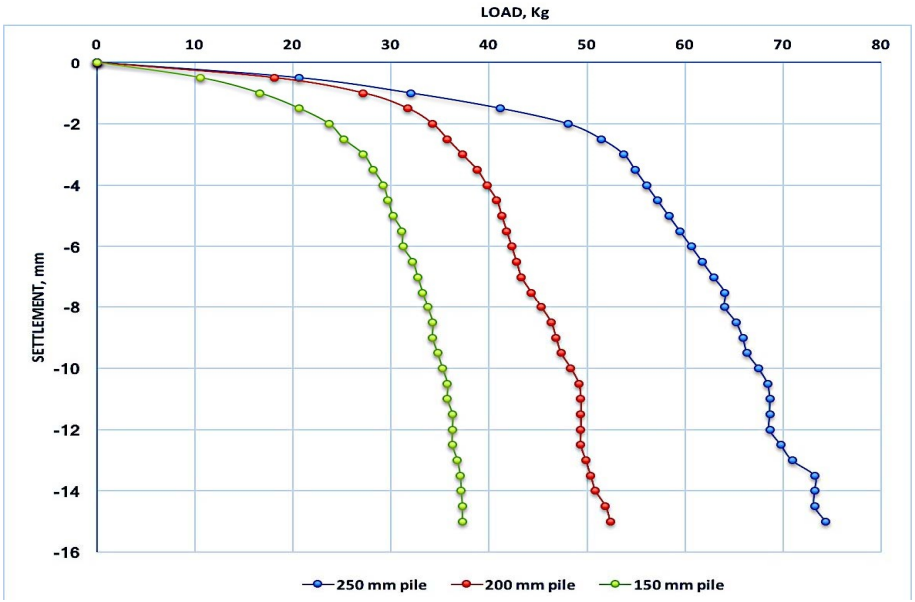



**Fig-5: Model pile load test for Dry density as 95% of MDD.**

*Load Vs Settlement Comparison Between Sand & Pond Ash*

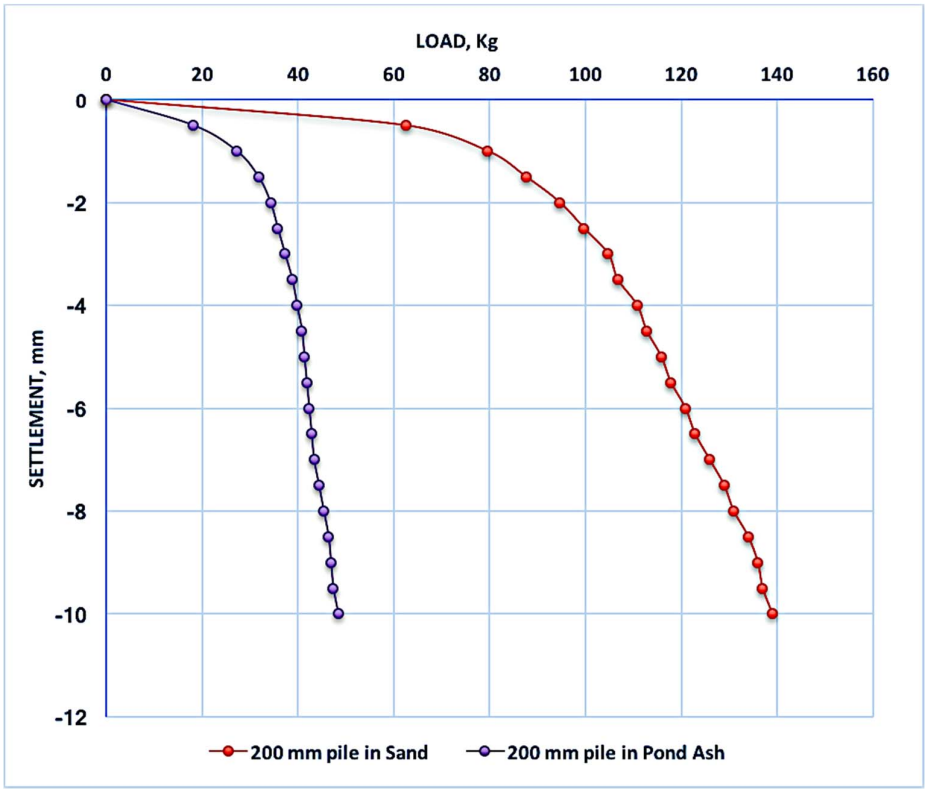



**Fig-6: Comparison between model pile load test for sand and pond ash at Dry density as 95% of MDD respectively.**

*Load Vs Settlement Comparison between all cases*

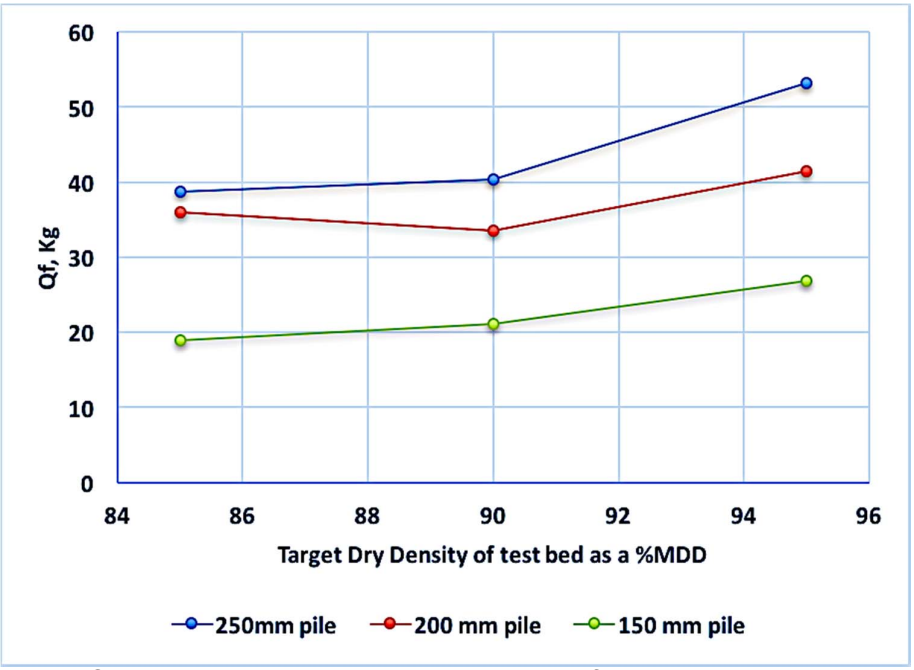



**Fig-7: Comparison between model pile load test for sand and pond ash at Dry density as 95% of MDD respectively.**

*Load Vs Settlement Varying Saturation State*

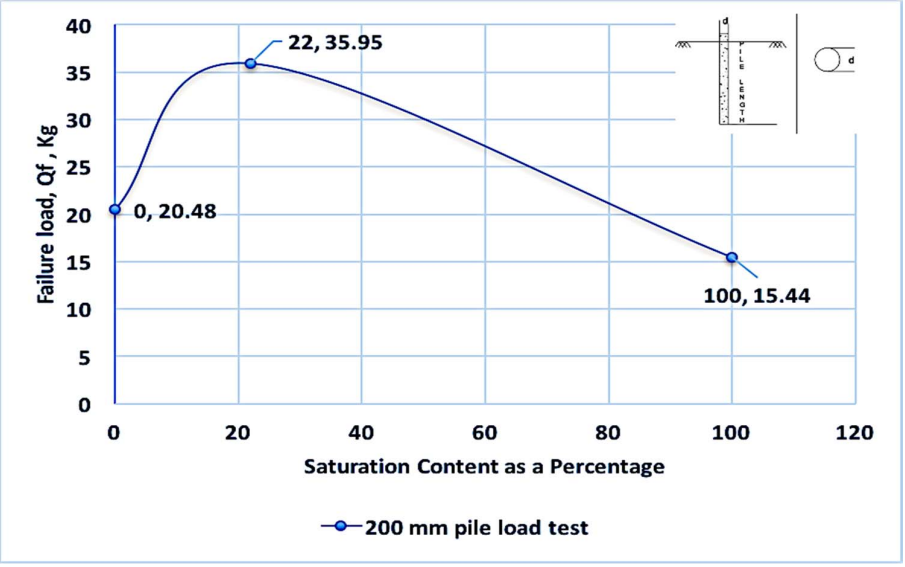



**Fig-8: Ultimate load vs Saturation as a percentage at Dry density as 95% of MDD respectively.**

## OBSERVATIONS

Based on the results, the following observations can be made.

- The failure behaviour is similar to that of cohesion less soils, i.e., the failure is brittle.
- Increase in ultimate load capacity with increasing density is observed similar to piles founded in dense/hard stratum relative to loose/soft stratum.
- Greater the length of embedment of pile, greater was the ultimate load carrying capacity. For the test conditions adopted in the present study, for a 67% increase in length of embedment of the pile has resulted 97% increase in capacity.
- The relative comparison of ultimate carrying capacity of model pile in pond ash was found to be 46% of that of Sand under similar test conditions namely the pile geometry, the relative density and moisture state of the test bed.
- The analysis of driven type of model piles indicated cast in situ type of installation will have better response in both bearing and frictional resistance of pile.

## CONCLUSIONS

Based on the experimental investigations carried out in this study, the following conclusions can be drawn.

1. The interaction of pile in Pond Ash is found to be similar to that of soils depicting the load-settlement plot as elasto-plastic in nature with distinct point of inflection.
2. Increase in embedment length and density of strata have shown significant increase in ultimate load carrying capacity.
3. The variation of ultimate load with varying moisture was found as follows: for completely saturated case ultimate load was recorded 57.05% less than that of OMC case and 24.60% less than that of completely dry case.

## SCOPE FOR FURTHER STUDIES

This has been an initial study and scale effect is planned for future studies. Field study will be conducted with desired size and composition of full scale piles to have a comparative analysis.

## REFERENCES

Bera, A.K., Ghosh, A. And Ghosh, A. (2005), Regression model for bearing capacity of a square footing on reinforced pond ash, Geotextiles and Geomembranes, 23, No. 3,261–285.

Bera, A.K., Ghosh, A. And Ghosh, A. (2008), Bearing Capacity of square footing on reinforced pond ash, Ground Improvement, 161, GI 1, pp 17-22.

Jakka, R.S., Datta M., Ramana G.V, (2010) “Shear Behaviour of Loose and Compacted Pond Ash”, Geotech Geol Eng (2010).

Pandian N.S, Sridharan A., Chittibabu G. (2001) Shear strength of coal ashes for geotechnical applications, Proceedings of the Indian geotechnical conference, Indore, India, pp 466–469.

Sreedhar, M.V.S., Venkatappa Rao, G., Ramesh Reddy, R. (2011) “A Constitutive Model for Bearing Capacity of Pond Ash Based on Laboratory Model Tests”.

Sridharan A., Pandian, N.S., and Rajasekhar, C., Geotechnical Characterization of Pond Ash, Ash Ponds and Ash Disposal Systems (V.S. Raju Et Al., Eds), Narosa Publishing House, New Delhi, Pp. 97–110 (1996).

# CHALLENGES ENCOUNTERED FOR FOUNDATION OF DURGAM CHERUVU EXTRADOSED BRIDGE IN HYDERABAD– DESIGN AND CONSTRUCTION

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## ABSTRACT

India's largest extradosed bridge (on completion) is being constructed over Durgam Cheruvu Lake to connect Madhapur and the heavily populated resident area of Jubilee hills.
Madhapur, itself is heavily populated owing to the presence of numerous IT hubs of HITEC city and Ambedkar Open University, both of which infringe with the bridge alignment on Madhapur approach.

The surrounding areas of Durgam Cheruvu are a part of undulating topography with elevation ranging from 520 to 630 mean sea level (MSL).
The main geomorphic units are residual hills, pediment inselbergs and valley fills. Based on the details of geotechnical investigation and survey, it was observed that the hard massive boulders of pink / grey granite are filled throughout the bridge alignment.

Providing the foundation for the bridge in the valley slope is quite challenging. Mobilization of the machinery and supply of materials to the project area is restricted because of heavy traffic regulations and limited access due to the terrain.

Negating the aforementioned overwhelming challenges, the open foundation for the bridge has been successfully completed. The present study encompasses the various challenges encountered for the design and construction of the foundation for the extradosed bridge.

**Keywords: extradosed bridge, pediment inselbergs, Granite, open foundation**

## INTRODUCTION

The bridge across Durgam Cheruvu lake is in its final stage of construction. Once completed, it would be the longest extradosed bridge in India. Extradosed bridge is a form of cable-stayed bridge, in which the stiffness of the girder is significant compared to that of the cables, thereby ensuring that the girder carries majority of the live load. The bridge is proposed as a part of the infrastructure development of Greater Hyderabad Municipal Corporation (GHMC) under the aegis of Municipal Administration & Urban Development Department (M.A&U. D). The engineering-procurement & construction (EPC) contract of this project has been awarded to L&T construction.

The project comprises extra-dosed main span bridge portion, approach span viaduct and RE wall approaches. The total length of the bridge is 735.63m including the cable stay portion which has a span of 233.80m. The general arrangement drawing of the bridge is shown in Fig.1.


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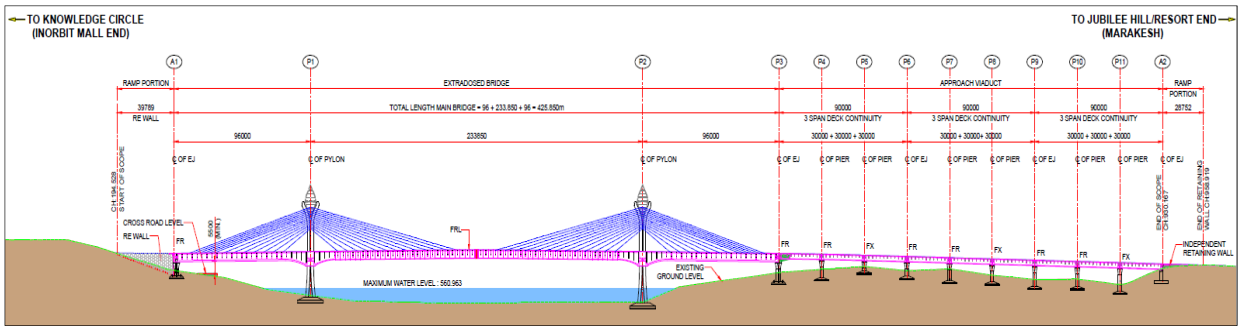



**Fig.1General arrangement drawing**

## SUB-SURFACE DETAILS AND AVAILABLE INFORMATION

The project site is surrounded with hillocks and towering granite. The main geomorphic units are residual hills, pediment inselbergs and valley fills [4]. And based on the details of geotechnical investigation and survey, it was observed that the hard massive boulders of pink / grey granite are occupied throughout the bridge alignment as shown in Fig.2.



**Fig.2. Pink and grey granite exposure around Durgam Cheruvu**

The detailed geotechnical investigation was carried out at proposed pier and abutment locations. The objective of the investigation is to interpret the engineering properties of soil/ rock for the purpose of design of foundation. The soil investigation consists of drilling, sampling, field tests and collection of disturbed/ undisturbed soil and rock core samples.

Geotechnical investigation was carried out comprising17 boreholes (Figs3. (a), (b) & (c)) to recognise the sub-surface stratification including various field and laboratory test results [1]. As per the details assembled from the sub-surface investigation, it was observed that initial 10 to15 meters stratum is filled up by completely weathered and disintegrated rock, having core recovery (CR) up to 33 per cent and NIL rock quality designation (RQD). This layer is formed by the complete in-place disintegration of parent bed rock material, but still partially retains the original rock mass structure. SPT conducted in this layer encountered with refusals.

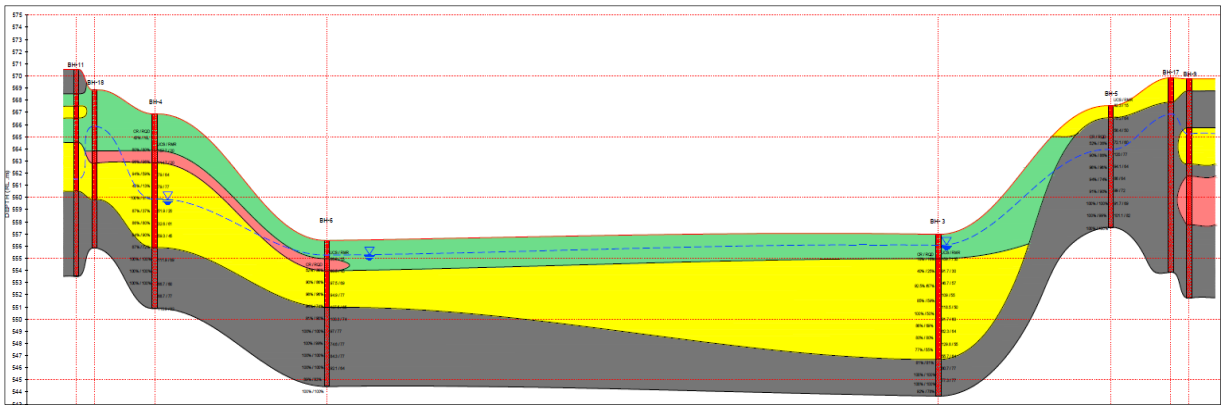


**Fig.3 (a). Sub-soil profile for main bridge portion (A1 to P3)**

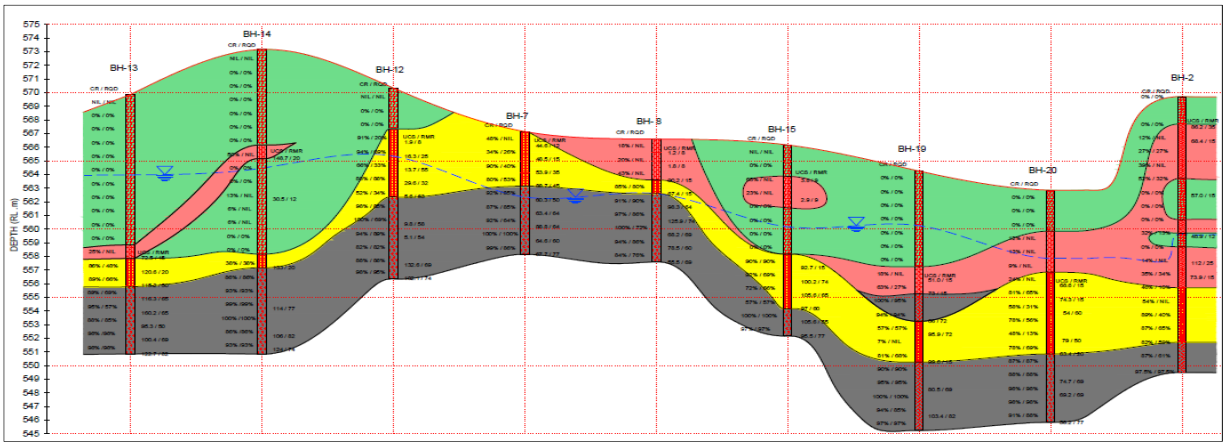


**Fig.3 (b). Sub-soil profile for viaduct portion (P4 to A2)**

| SYMBOL | DESCRIPTION |
|---|---|
| | COMPLETELY WEATHERED DECOMPOSED ROCK |
| | HIGHLY WEATHERED GRANITE |
| | MODERATELY TO SLIGHTLY WEATHERED GRANITE |
| | UNWEATHERED GRANITE |

| SYMBOL | DESCRIPTION |
|---|---|
| | COMPLETELY WEATHERED DECOMPOSED ROCK |
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| | MODERATELY TO SLIGHTLY WEATHERED GRANITE |
| | UNWEATHERED GRANITE |

**Fig.3 (c). Legends used for sub-soil profiling**

The underlying bedrock layer was moderately weathered to sound fresh rock. The CR of this stratum varies between 33% and 100%, while RQD ranged between 0 and 100%.The stratification at the project site showing top disintegrated/ completely weathered rock layer and deep fresh bed rock is shown in Figs.4&5 respectively.



**Fig.4. Weathered granitic moorum up to 2-3m with granitic boulders at Pier 6 location**



**Fig.5. Weathered & fresh granitic boulders-steep jointing at Pier 7 location**

The detailed analysis of the strength and quality of the rock was carried out and the consolidated analysis of UCS and RMR values based on the geotechnical investigation data is presented in Figs.6&7 respectively.

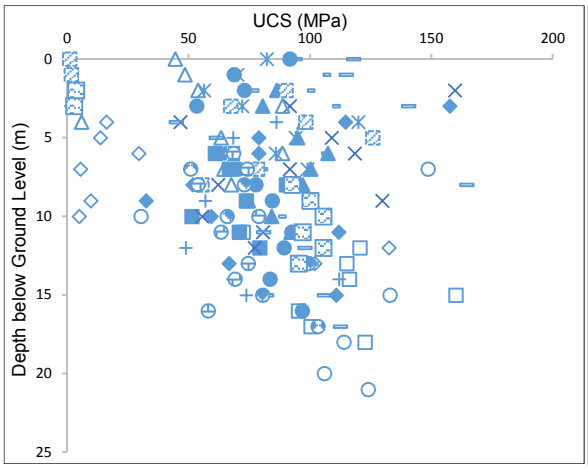



**Fig.6. Variation of UCS along depth**

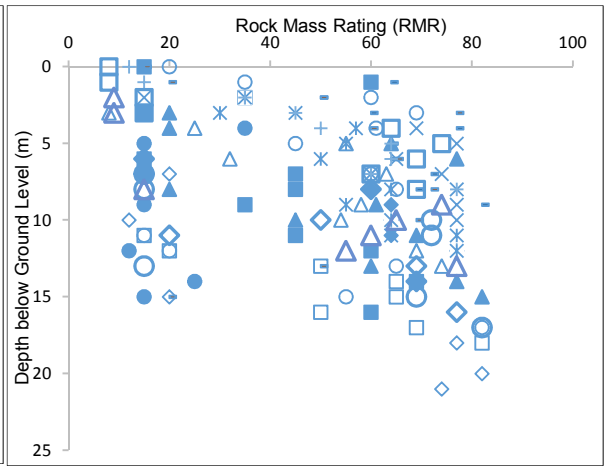



**Fig.7. Variation of RMR along depth**

It can be seen from Fig.6 that in few boreholes, moderately weak to strong rock was predominant in the top 5.0m having UCS values ranging from 0 to 50 MPa, and below that the rock was found to be strong to very strong having UCS value ranging from 50 to 165 MPa.

## TOPOGRAPHY AND REGIONAL GEOLOGY OF PROJECT SITE

The preliminary survey and topographical studies revealed that the Western flank is a plateau type, which follows cliff with steep slope towards the Durgam Cheruvu as can be seen in the contour map for the site presented in Fig.8. The surrounding areas of Durgam Cheruvu, forms a part of undulating topographical area with elevation ranging from 520 to 630 m from mean sea level (MSL). Mobilisation of machinery

and transportation of construction materials were adversely affected by this abrupt change in ground elevation.

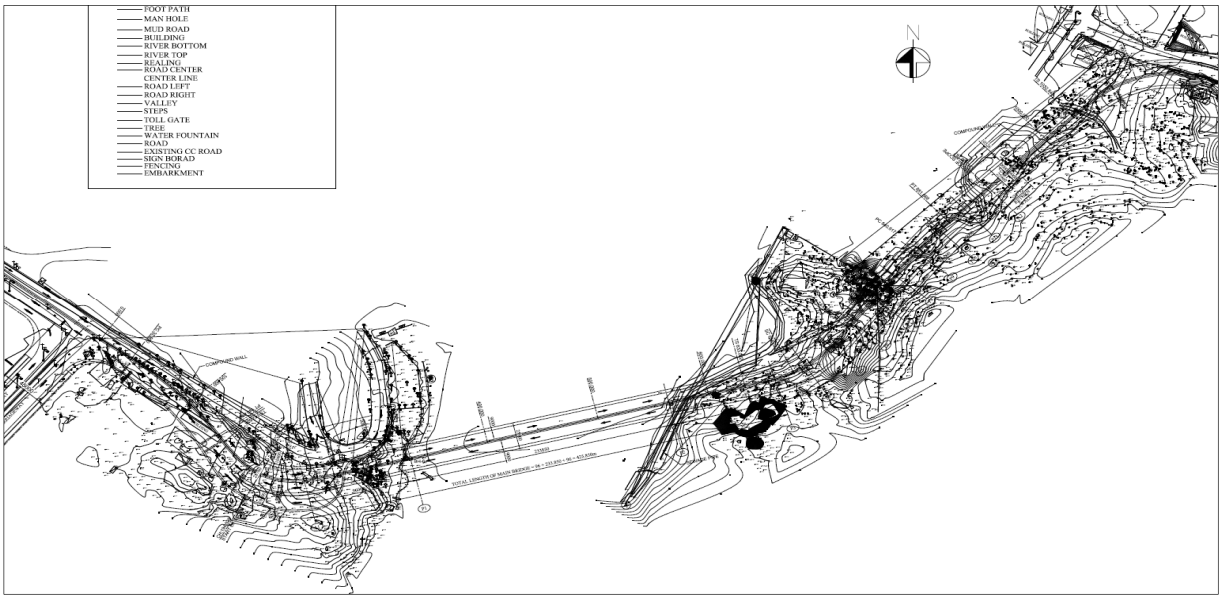



**Fig.8. Topography of the project site**

Sub surface geological studies were carried out to understand the geology of project location. Pits were excavated and it facilitated to study sub–surface geological features. These features exhibit top most layers with jointed rocks, boulders and decomposed soil (top 10-15m). Fresh granitic rocks were noticed at deeper depth (below 15m). The water is stagnated in the lake which resulted in the top layer as marshy. The towering granites, as shown in Fig.9, were formed by means of erosion of neighbouring softer materials in due course of time. And the lesser resistant rocks/areas become rounded to create geological forms known as inselbergs or tor [2]. These geological formations or structures are the factual justifications for undulating terrain of the project location.

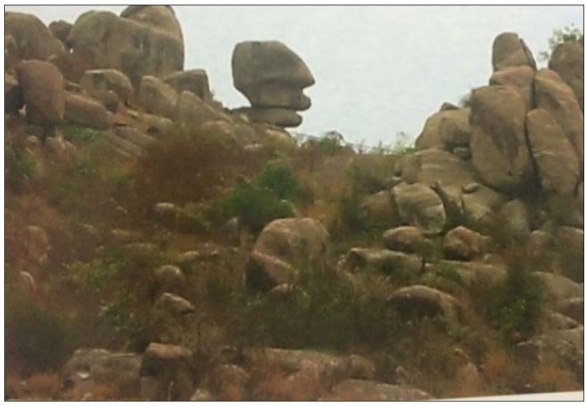


**Fig.9. Boulder granites with well jointed rocks**

## PROJECTCONSTRAINTS

The foundation for bridges is designed to ensure the safe transfer of heavy loads from girders to a suitable competent stratum, preferably rock, and hence rock socketed pile was the conventional choice for foundation. However, this option was unfavourable for the undulating topography of the project location. The Kelly/rotary drilling method was not suitable for present terrain since the mobilization of machinery requires large amount of cutting and filling of ground and furthermore, the movement of undercarriage over boulders may cause the overturning of machines. The tripod chiselling method was not favourable because of noise pollution adjacent residential areas and due to its prolonged time for completion. And, reverse circulatory drilling (RCD) method requires heavy duty cranes as support for machinery and its mobilization was very tough for the project site.

Additionally, the project site is situated near the highly populated residential area of Jubilee hills where the frequency of vehicles movement is very high and traffic regulation becomes challenging. In that location further deployment of heavy machinery and cranes for piling activities becomes inaccessible. The regular chiselling method would have been time-taking and since the project site is located in the urban area, thus heavy residential complexes, it was restricted to proceed with conventional blasting method.

## FOUNDATION SOLUTION/RECOMMENDATIONS

Foreseeing the quantum of constraints and the difficulties of the foundation, L&T EDRC recommended that the shallow foundation shall be an apt option instead of rock-socketed pile foundation. There were lot of challenges in construction of the shallow foundation as it included huge excavation. For instance, the maximum foundation dimensions were observed to be 20m x 18m x 4m, which includes the excavation of 1440$m^3$ of disintegrated / completely weathered rock material. These huge foundation dimensions demanded large scale levelling of the profile. The availability of levelled intermediate geo-material (IGM) / rock surface was tough because of the hilly terrain and undulated topography. Any unevenness found below the foundation slab was levelled using PCC. The exaction pit for the foundation at P-11 location is shown in Fig.10.



**Fig.10. Excavation pit at P-11 location**



**Fig.11. Cracked piece of granitic boulder showing SCDA filled drilled hole**

For blasting boulders, an innovative method was the need of hour which could be used to overcome project site constraints, and for the sake of ease, chemical cracking was the best available option. In

chemical cracking, soundless cracking agents (SCDA) used in cracking consist of a cementitious powdery substance and when it is hydrated at an appropriate temperature its volume expands through the formation of ettringite and calcium oxide (CaO) [3&5]. In this process, the material is poured into the pre-drilled holes in the rock and the subsequent chemical reaction increases the temperature of the compound up to 150°C, as presented in Fig.11. This allows the material to volumetrically expand over a period of several hours. Under the existing confinement in a drilled hole, this volumetric expansion generates an expansive pressure and when it exceeds the tensile strength of the rock, cracks form.

## SUMMARY AND CONCLUSION

This paper presents the various challenges handled while designing and constructing the foundation for extradosed bridge across Durgam Cheruvu lake. Geotechnical characterisation was undertaken using conventional site investigation, laboratory testing and in situ testing techniques. Sub-surface studies were conducted using excavation pits and encountered that the project site is having an undulating terrain with geological formations like inselbergs and towering granites.

The project location is surrounded by heavily populated residential area and HITEC city, thereby further traffic regulations was there for heavy machineries. Proposal of rock socketed pile as the foundation for Durgam Cheruvu bridge was adversely affected by the undulating topography and limited access to the project site. L&T EDRC designed shallow foundation resting on weathered granitic rock as an appropriate alternative and the huge foundations, maximum size of 20m x18m x 4m, were constructed at project site successfully. Blasting of granitic outcrops was not permitted as it creates the noise pollution in residential area and hence chemical cracking was adopted.

## REFERENCES


1. Geotechnical Investigation Report (2017) furnished by M/s SKW Soil And Survey Co., Navi Mumbai.
2. Balakrishna. S (1964) Some studies on granites of Hyderabad from Proceedings of the seminar on peninsular Geology, Geology Dept, University Hyderabad, pp232-253.
3. Hinze, J.; Brown, J. (2017) Properties of soundless chemical demolition agents. J. Constr. Eng. Manag.,120, 816–827.
4. Kanungo, D.N., Rama Rao, P., Murthy, DSN. And Ramana Rao, A V., (1975) Structural features of granites around Hyderabad, A.P. Geophys. Res. Bull., 13: pp 337-358.
5. Laefer, D.F.; Ceribasi, S.; Wortman, J.; Abrozevitch-Cooper, N.; Huynh, M.-P.; Midgette, J. (2010) - Expansive fracture agent behaviour for concrete cracking. Mag. Concr. Res., 62, 443–452.
6. M.L. Gupta, S R Sharma, A. Sundar and S.B Singh (1987) Geothermal studies in the Hydrated granitic region and the crustal thermal structure of the Southern Indian shield., Technophysics, 140, pp 257-264.
7. Rao,Y.J(1963) The Pink and grey granites of Hyderabad and the problems of their age relationship .Proc Symp on Stratigraphy Age and correlation of the Archaean Provinces of India.

8. Robet Mala Chemenss John M (1995) Are enclave's significance in granite petro genesis Third Hutton symposium on Granite.
9. IS: 12070 -1987 – Code of Practice for Design and Construction of Shallow Foundation on Rocks
10. IS: 6403-1981– Code of practice for determination of bearing capacity of shallow foundations.
11. IS: 8009 (Part 1) – 1976 – Code of practice for calculation of settlement of foundations.
12. Foundation analysis and design by J.E Bowles, Fifth edition.

# CONSTRUCTION OF THE FOUNDATIONS OF MEGA SEA BRIDGE OF INDIA MUMBAI TRANS-HARBOUR LINK

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## ABSTRACT

The new Sea link Bridge over Main land Sewri on Mumbai side to Ulwe on Navi Mumbai side is characterized by some aesthetically appealing elevations of the Bridge and large spans. This paper gives an insight to design and construction technique adopted for foundation construction of the Trans-Harbour Link Package-1 at Sewri end, Site being located in intertidal zone had its unique geological characteristics and own set of challenges during construction.

**Key words: Pile foundation, Deep rock drilling, Marine piling, Intertidal zone, Navigational zone, Reverse circulation Drilling, Bracket support system and precast shell**

## INTRODUCTION

The Mumbai Trans Harbour Link (MTHL) is one the mega project under construction in India. The high volume of traffic burden on present routes in Mumbai City has created the need of fast pace completion of projects. The MTHL would be an important infrastructure project to improve the connectivity between Greater Mumbai and Navi-Mumbai, facilitating the economic development in Mumbai Metropolitan Region. MTHL passes through both the land and above the water. Obstacles in the marine sections include mangrove forest areas, and on the land sections railway line crossings and roads.

## OVER VIEW OF THE PROJECT

The proposed 21.8 km long six lane wide Mumbai Trans-Harbour sea Link bridge connects Sewri on the Mumbai side with Chirle on the Navi Mumbai side. About 16.3 km of viaduct is over sea/creek and about 0.5 km is land via-duct on Sewri side & 4.9 km of viaduct on Navi Mumbai side is over land. The MTHL is constructed in three packages. This fully access-controlled highway will have four interchanges, toll collection facilities and other necessary facilities and building.

The MTHL commences from Sewri, where it connects with the Eastern Freeway. The alignment of the MTHL traverses on MbPT land for about 500 m near Sewri,from where it enters Mumbai harbor and then continues in the same direction over mudflats and then crosses the Pir-Pau jetty, the Thane Creek, the Panvel Creek and the intertidal zone before turning south to touch the main land at Shivaji Nagar in Navi Mumbai. The alignment then meets the main land villages of Jasai and Gavan to end at the NH 4B near Chirle. The alignment on the main land on Navi Mumbai side also crosses the Coastal Road proposed by CIDCO, the railway line under construction near this location, the State Highway (SH) 54 and the existing railway line before connecting to the NH 4B.

The MTHL alignment for civil works is divided in following three packages;
**Package-1** includes the Sewri Interchange on land on the Mumbai side and the adjoining 10.380 km (CH 0+000 km to CH 10+380 km) bridge above sea/creek (Fig. 1).

**Package-2** includes a 7.798 km long bridge (CH 10+380 km to CH 18+187 km) above sea/creek and the Shivaji Nagar Interchange on land on the Navi Mumbai side.


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**Package-3** includes a 3.613 km long road bridge and earthwork section (CH18+187 km to CH21+800 km) having interchanges at the SH 54 and the NH 4B near Chirle and Railway-over-Bridges at two locations in Navi Mumbai.
This paper discusses the salient features of MTHL Package-1 only.

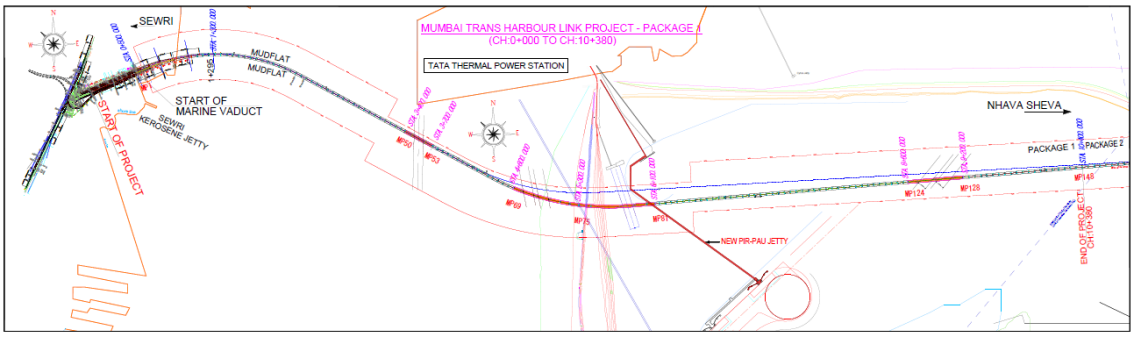



**Fig.1 MTHL PKG-1 Bridge Alignment**

The entire section of Package-1, except a small section, is composed of bridges, comprising of typical Pre Cast (PC) box girders (fig. 2a) and Orthotropic Steel box girders Decks (OSD) (fig. 2b) at obligatory navigational channel.

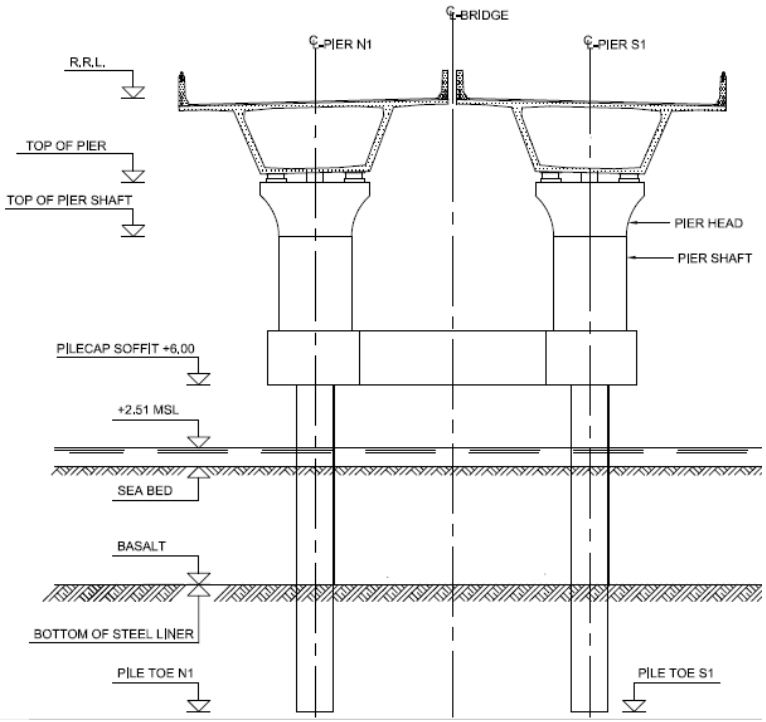



**Fig. 2 a) Typical Bridge Cross section with PC Box-girder**

The general layout of the MTHL PKG-1 has 147 spans. The typical span for intertidal zone and sea zone is 60 m. Also, there are 4 types of Obligatory navigational spans with length ranging from 85 m to 180 m. Spans adjoining to obligatory spans are varying from 40 m to 53.75 m length. As per technical specification minimum deck continuity of 200 m is required, therefore the P C Box girders are generally 6 to 7 spans continuous. The typical width of the carriageway is approx. 14 m for each side separated by 0.5 m, with provision of an additional span at rescue bay. Erection shall be done by span-by-span method utilizing Overhead Launching system. The bridge is having a design speed of 100 km/hour.

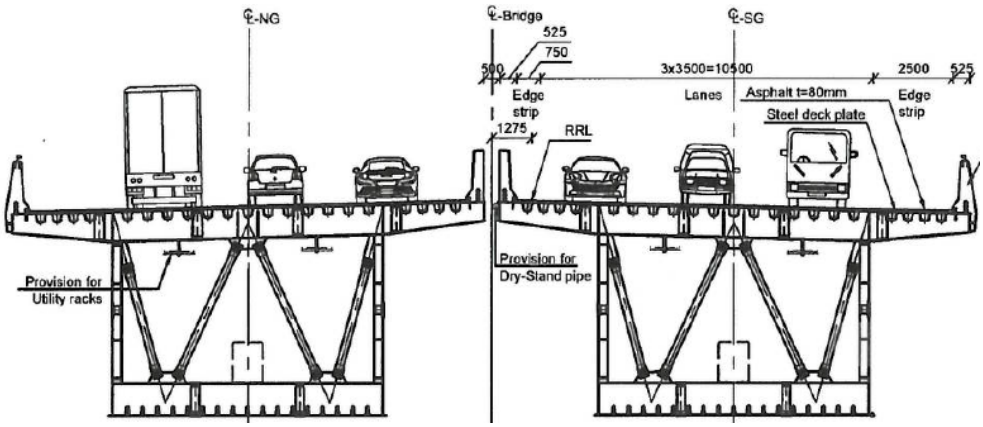



**Fig. 2 b) Typical Cross section of OSD Box-girder**

## SUB SOIL CONDITIONS:

The Subsurface profile generally consists of fill overlying residual soil/marine deposits/Boulders underlain by Basalt/Breccia/Tuff Bedrock. In general, sea bed has very soft to soft clay layer on top. However, at some locations there are sediments of medium dense to dense Sand entrapped in the above strata. Below the soft Marine Clay layer, very weak to weak completely weathered to highly weathered Basalt is encountered. The thickness of the superficial deposits varies along the main alignment from less than 5 m to 21 m (fig. 3). Under laying this is moderately to slightly weathered dark grey Basalt with RQD 10-60%. The Basalt encountered is massive, hard, fractured and dark greenish grey in colour with RQD >60%. Compressive strength of rock core samples ranges between 30 MPa and 120 MPa.

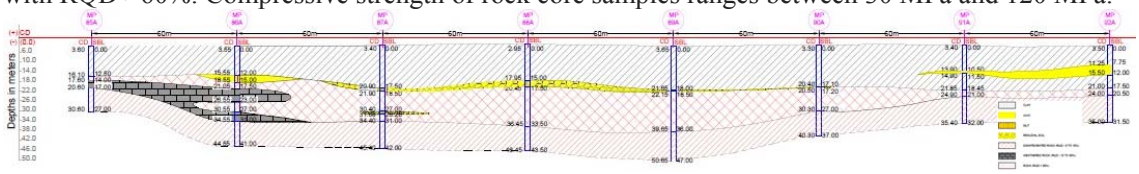



**Fig. 3 Representative subsoil profile**

## SUBSTRUCTURE:

Main bridge consists of 148 nos. of foundations from MP1 to MP148 with pile dia. mainly as 2.2 m and 1.5 m with 16 mm thick liner upto rock level. Typical pile group has 4 nos. of pile with exception of 6 piles in rescue bay span at MP 98 – MP99. The pile cap is typically dumble shaped with maximum size of 14.9 m X 19.4 m and for Navigational Spans it is of rectangular size 12 m X 30 m (fig. 4a & 4b). Pile foundations are resting on rock strata at a depth ranging from 20 m to 40 m from sea bed level. The Lowest tide level (L.T.L) is at - 0.1 m Chart Datum (CD) and Highest high tide level (H.H.T.L) at +5.8 m CD. The proximity to ecologically sensitive surrounding has posed a real challenge to the design and construction of the bridge. And has resulted in the innovative dumble shaped design of typical pile cap, which has minimal impression on ground in mangrove zone.

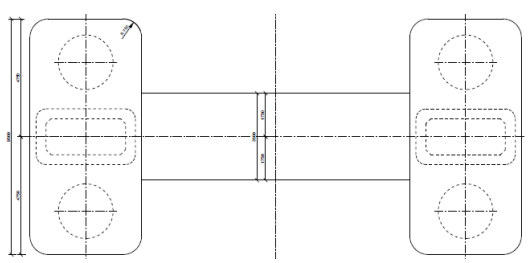
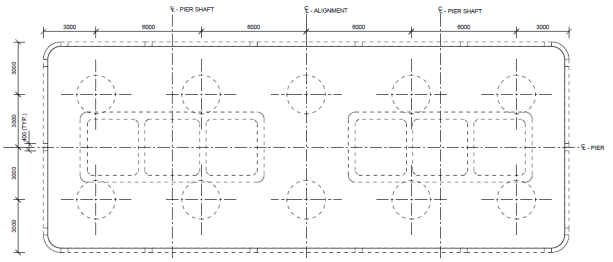


**Fig. 4 a) Typical span Pile Cap**



**b) Typical Navigational span Pile Cap**

## CONSTRUCTION:

This paper presents challenges faced in foundation and substructure construction above sea/creek. It details out use of lean construction method, adopted for pile & pile cap without causing any disturbance to the surrounding environment. The MTHL passes across the port facilities, navigation channel and seabed pipelines in the Mumbai Bay, which need to be considered obligatory in the design and as well as during execution of the works. In addition, the necessary minimum vertical and horizontal clearances need to be followed during construction.

### INTERTIDAL ZONE:

The entire length of the bridge has two zones viz. Intertidal and Marine zone. In intertidal zone there is no water during low tide, so no marine fleet can operate during this period. So, to facilitate equipment, material and workforce movement a Temporary Access Bridge (TAB) is constructed along the bridge alignment in this zone (fig. 5a).




**Fig. 5 a) Intertidal zone -Temporary Access bridge**

The Temporary access bridge is 8 m wide with finger platforms at each pile-cap location projecting from it. The fingers provide working area of about 30m X 12m (fig. 5b) for carrying out pile and substructure works.



**Fig. 5 b) Finger platform projections**

Typical cross section of two lane TAB is seen below (fig. 6).

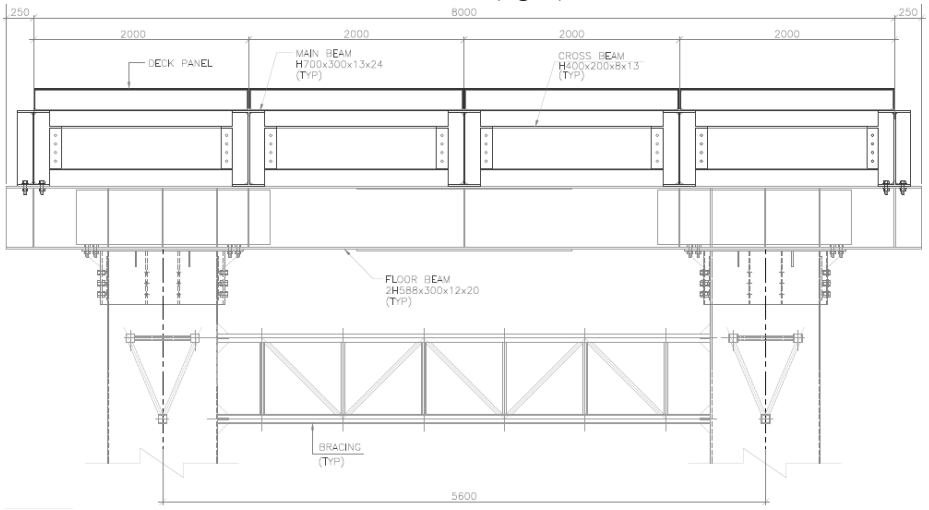



**Fig. 6 Typical C/S of TAB**

The TAB piles (10 mm thick liners of 1m dia.) are constructed using a Cantilever Piling Gantry (fig. 7a) moving along the alignment, followed by erection of TAB superstructure. TAB superstructure has floor beams, Main girders prefixed with cross beams and over it modular deck panels, erected using 1500 KN cap. crawler crane (fig. 7b). Once TAB and finger is completed at a location permanent bridge pile construction starts.



**Fig. 7 a) Cantilever piling gantry** **b) Erection of TAB**

***Pile Construction:***

Generally followed methodology involves:

- Carrying out survey and lowering of twin pile outer guide frame (fig. 8) to nearest coordinates of pile location.
- Securing the guide frame in position by driving six temporary liners in collar.
- Adjusting it to appropriate working level above high water level by lifting with crane
- Locking it in position.
- Then seating of inner guide frame to precise pile coordinate.




**Fig. 8 Outer guide frame**

Permanent Liner (2.2 m inner diameter) of 18 to 24 m length is brought and lowered into position using vibro - hammer. Liners are driven up to refusal. Then Reverse Circulation Drilling machine (RCD) is mounted over the pile liner. During low tide there is no water available, meeting water demand of approx. 16 cum/min for drilling process is a challenge. In addition, due to stringent environmental regulation excavated soil needs to be disposed suitably. Therefore, an extensive system for water supply, sludge collection & disposal is planned for RCD operation (fig. 9). It includes provision of 2 nos. sludge/sedimentation tanks of 50 cum. cap. each and 2 nos. of water tanks of 48 cum. cap., thus, ensuring 196,000 Lit. of water for recirculation during low tide operation of RCD. The excavated soil is collected in a split hopper barge and is disposed at designated deep sea location. One setup of twin pile guide frame allows construction of two piles together, thus speeding up the entire process. Once drilling is complete, flushing is done, reinforcement cage is lowered, and concreting is carried out, upto a window kept above the cut off level of pile in the liner. After concrete is set the pile liner is cut to pile cutoff level.

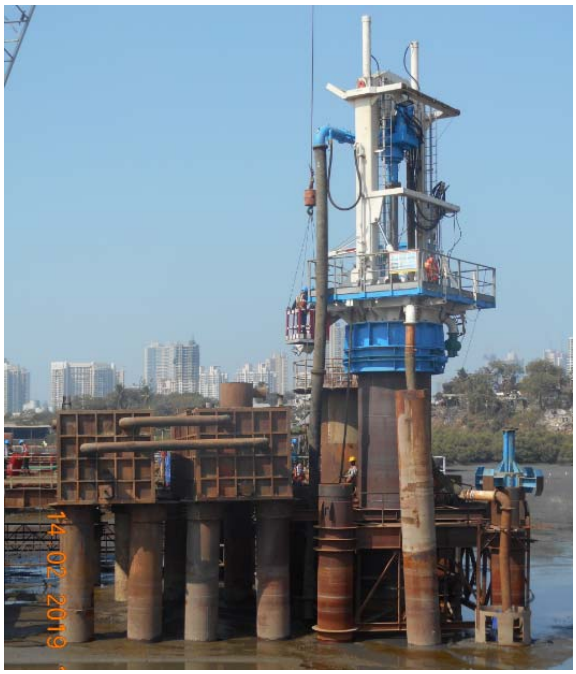


**Fig. 9 RCD in operation**

***Pile cap construction:***

The L.T.L is at - 0.1 m CD and H.H.T.L at +5.8 m CD. The proximity to ecologically sensitive surrounding has posed a real challenge to the design and construction of the bridge. As the pile cap soffit is at +6.0 m CD and being eco sensitive zone any support system from the ground is not possible

for the pile cap. Therefore, a lean method of construction is adopted, wherein steel brackets are welded to pile liner directly, over which hydraulic jacks supporting the formwork system (fig. 10) are placed. Pouring of approx. 400 cum concrete is done in 300 mm layers from opposite ends. Controlled permeability form liner are used for improving surface finish and durability of exposed concrete surface. No tie rods are used in vertical form system, as designed props and turn buckles support the entire system, giving a neat surface devoid of any impressions (fig. 11).



**Fig. 10 Pile cap Reinforcement**



**Fig. 11 Pile cap after removal of form**

In intertidal zone water un-availability throughout the day is a bigger challenge to piling process. Therefore in order to match project schedule requirement additional piling rig are put to use. A temporary extension of the finger is done and using Rotary Auger machine boring is completed, ensuring proper disposal of the excavated soil.



**Fig. 12 Piling with Rotary rig in intertidal zone**

**MARINE ZONE:**

Considering the area of operation and limited accessibility from the shore, a well-planned and initiated logistic plan are put in place to meet the day to day requirements of the operating fronts under the supervision of dedicated Logistics Manager. A deep-sea Marine job demands high degree of control and precision with tight tolerances, therefore MTHL PKG-1 has a centralized survey system under control of the Survey Manager.

***Pile construction:***

The marine zone is subjected to exposure of swell, breaking seas and tidal currents. In addition, at times strong wind affect the barge movements. Hence, barge movement from one to other Pier location is restricted and governed by natural forces. Therefore, to be able to face and operate under such conditions Jack up Barges (JUB) (3 nos.) are used. Each barge is of 32m X 21m X 3 m size loaded with a 2600 KN cap. Crawler crane, Vibro-hammer, powerpack, RCD machine, RCD power pack, air compressors and water pumps.

Pile operation starts with Jack up Barge positioning. Before every movement voyage plan, charts, site survey reports, tidal height predictions, navigational hazards, weather prediction and procedures are studied and observed. Jack-up is towed to pile location with help of tug boats and guidance of navigational software-HydroPro. For each location positioning of anchors in correct position and within tolerance is done as per anchoring/mooring plan. Safe placement of anchors is a challenge, care is to be taken so that anchors do not damage seabed utilities i.e. pipelines, cables etc., identified during bathymetry and geophysical survey. Once arrived at location, such that pile guide slots can be used accurately, legs of JUB are lowered and pinning is done. Considering swell and tidal variation, air gap needs to be maintained for smooth operation. So, the JUB is jacked to a minimal air gap as required for safe piling operation. For driving permanent pile, 4 reference points are marked for guide frame beams (fig. 13). Guide frame is adjusted to get the precise pile co-ordinate. Then pile liner is driven using Vibro-hammer up to refusal.



**Fig. 13. Jack Up Barge in Marine zone**

Then RCD is mounted on the liner and boring is carried out up to design founding level. Sometimes, due to highly weathered rock conditions sinking of liner occurs. Managing this condition is a challenge, owing to heavy loading on the liner. So, bore-logs are studied and verified with observed soil sample. Accordingly, extreme care is exercised during boring in such strata. Once boring is complete flushing is carried out and reinforcement cage in 12 m long pieces is lowered into the pile and connected to each other with mechanical couplers. Material barges are used to supply pile reinforcement and form work material for Pile-cap.

For concreting Marine Batching Plant (MBP) (2 nos.) of 60 cum cap each sized 60m x 22m x 3m having concrete pump, transit mixer, power pack and stationary tower placer boom on deck are used (fig. 14).

A 750 KN cap. crawler crane with clam shell placed on MBP is used to transfer aggregate material into the bins from the material barge. Using same JUB location all piles of the pile cap are constructed.



**Fig.14 Batching Plant mounted on Barge**

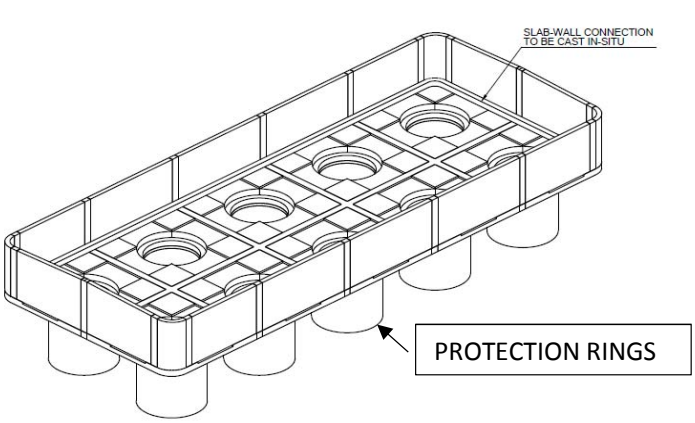
***Pile cap construction:***

In marine zone for typical dumble shaped pile cap, formwork system (fig. 15a) similar to intertidal zone is used which is supported by steel brackets welded in place to the pile liners. Over it primary beams are provided, which support tie beams and secondary beams. Once bottom form is in place reinforcement is fixed in situ due to heavy congestion of reinforcement bars and concreting is carried out with MBP. In Marine zone mainly all foundation have typical pilecap (fig. 15b) except the odd twin double shaped pile cap for the rescue bay span which require approx. 600 cum. of concrete and obligatory spans.



**Fig. 15 a) Typ. Marine pile cap-formwork system b) Typ. Marine completed pile cap**

The obligatory Navigational spans have rectangular pile cap of size 12m X 30m X 3.3m. As the pile cap soffit is at +2.7 m CD and top is at +6.0 m CD the pile cap structure lies in intertidal heights. It is proposed to use a precast shell (fig. 16a) as sacrificial form for pile cap construction. The Pile cap has protection rings (fig. 16b) around the pile to protect the pile from barge or vessel impact.



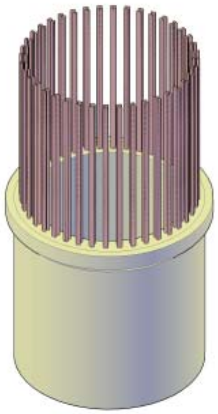



**Fig. 16. a) Navigational Span pile cap –Precast shell** **b) Precast Protection ring**

The protection rings weighing 450 KN shall be precast and put in place after welding of seating bracket. Seating bracket shall be welded using a sealing compartment positioned temporarily using a crane on the liner (fig. 17a, b &c). Then precast slab panels followed by wall are to be erected and in situ stitch shall be cast to form a closed element. Then reinforcement shall be placed in situ and concrete of 1100 cum. be done using MBP.

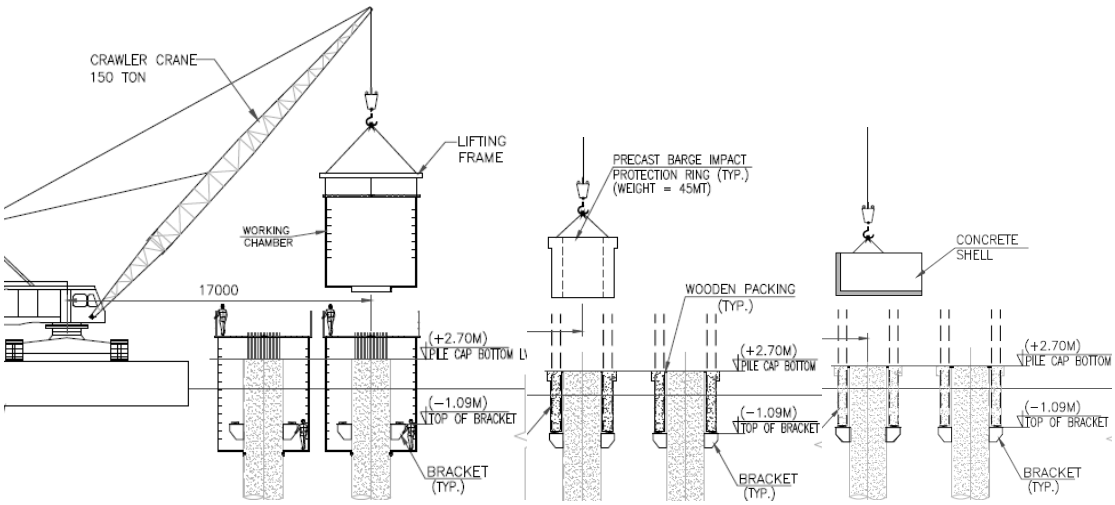



**Fig. 17. a) working chamber fixing b) protection ring lowering c) Precast shell fixing**

Once pile cap construction is over pier and pier cap is completed, followed by superstructure erection.

## CONCLUSION

The construction system for the project were designed based on geographical, geological and ecological sensitivity requirement. It was evident that overall performance of the system is highly vulnerable to difficult and challenging site conditions. Apart from few liner settlement problems no significant construction issue was observed.

Challenging conditions lead to lean and innovative method of construction in difficult site conditions. Method adopted has set a trend for taking up challenging Long span bridges over sea / intertidal rivers.

## ACKNOWLEDGEMENTS

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## REFERENCES

Subba, R.T.N., 1988. Foundations and piers of Second Hooghly Bridge, International Seminar on Cable stayed Bridge, ING-IABSE, Bangalore.

Subba, R.T.N., 1999. Pylon foundations of four cable stayed bridges- The Indian experience, ING-IABSE Colloquium, New Delhi, February 22-24, pp. 153-168.

S, Kashima,. M, Kitagawa., T. Moritani., K Yamada, 1999, Design and construction of foundation for Akashi Kaikyo bridges, ING-IABSE Colloquium, New Delhi, February 22-24, pp. 265-270.

# MICROPILES AND GROUND ANCHORS INCREASE THE CAPACITY OF EXISTING BRIDGE ABUTMENTS

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**Hub Foundation Co., Inc, Chelmsford, MA, USA**

## INTRODUCTION

The existing Route 145 (Winthrop Avenue) bridge over the MBTA commuter rail line and the Pan-Am Railway was originally built in 1899. The superstructure was reconstructed in 1958. The combination of increased traffic which resulted in increased vertical loadings and unbalanced lateral loads, derating of its current capacity due to old age and deterioration plus new design standards required the replacement of the entire bridge. This bridge is located in the City of Revere, a few miles north of Boston, Massachusetts. The steel truss and bridge deck was supported directly on the existing abutments comprised of stacked granite block gravity walls. The abutments will remain after the existing bridge is removed and replaced. Photo 1 shows the existing bridge over the rail tracks. However, the granite block abutments needed to be "reinforced" to allow for the additional vertical and lateral loads to be supported in accordance with current highway and railroad standards.



Photo 1. Existing Bridge over the Commuter Rail Tracks

## SITE INVESTIGATION

Documents of the original design and construction of the bridge abutments could not be found. A site investigation was performed to determine the depth and width of the granite block abutments and the subsurface conditions below.

A total of 10 soil borings were drilled. Four borings were drilled directly through the abutments to depths of up to 130 feet below the roadway. The abutments were 26.5 to 30 feet deep. A 2.5 foot deep void was discovered directly below the eastern abutment. Top of rock was found to range from 95 to 120 feet below the roadway. Figure 1 provides the subsurface information


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based on borings at the existing bridge. Figure 2 shows rock coring results of Boring BB-1A. The rock is the Cambridge Argillite and the overall quality at this site was excellent.

Several horizontal probes of the abutments were made to determine their width. Based upon these probes, the base of the abutments were estimated to be 12 to 14 feet wide.

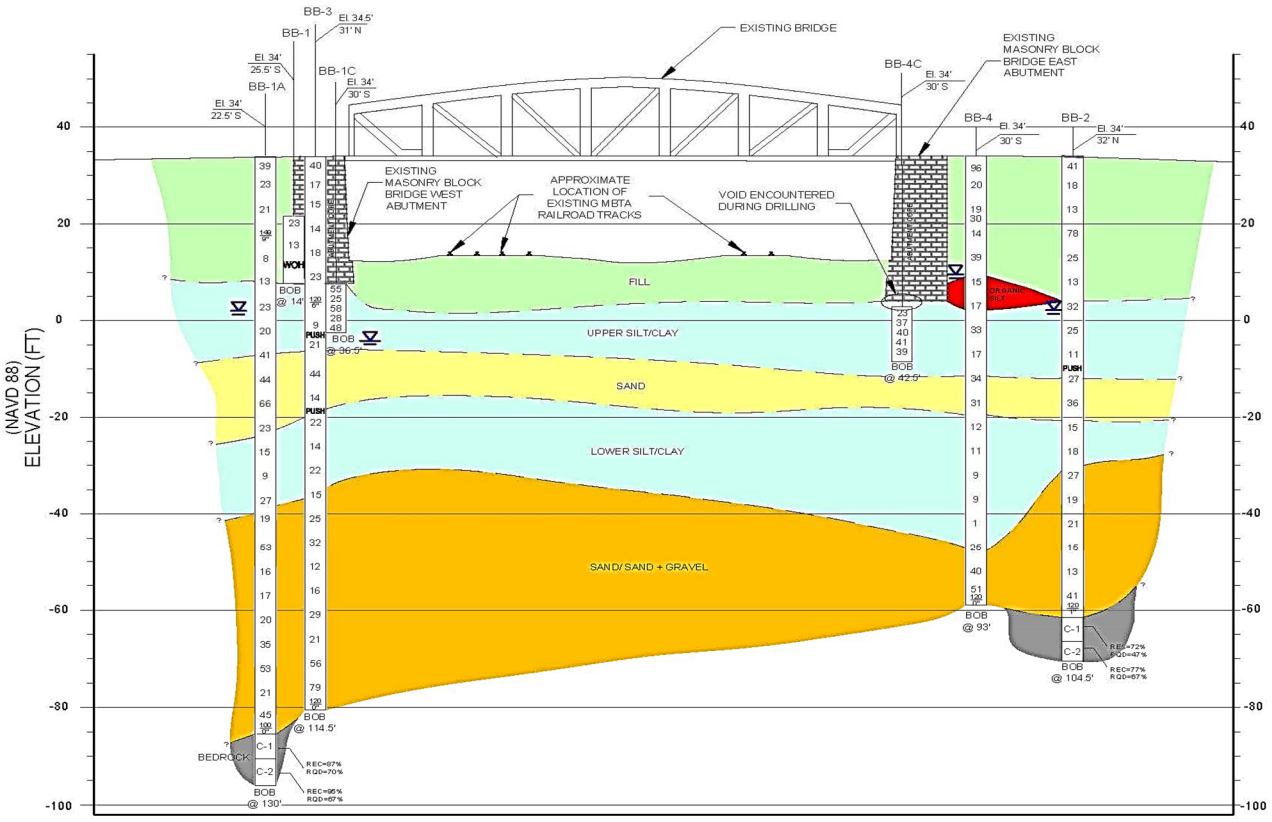



Figure 1. Subsurface Profile at Existing Bridge

| Boring Number | Core Run | Core Depth Below Roadway Surface (ft) | Approximate Elevation (ft) | Recovery (%) | RQD (%) |
|---|---|---|---|---|---|
| BB-1A (west abutment) | C-1 | 120 to 125 | -86 to -91 | 87 | 70 |
| | C-2 | 125 to 130 | -91 to -96 | 95 | 67 |

Figure 2. Boring BB-1A Rock Coring Details

## DESIGN SOLUTION

The reinforcement of the existing granite block abutments consisted of 20 micropiles with an axial design capacity of 265 kips in compression and 41 tiebacks with a design load of 50 kips. Figure 3 shows the foundation system layout. Figure 4 is a detail section taken from the bid documents for the project showing both elements and the granite block abutments. Both elements were drilled thru the granite blocks using a rotary-percussion drilling system.

The lock-off loads of the tiebacks were distributed over the face of the granite block walls using a steel Load Transfer Unit (LTU) affixed to the wall. The project engineers provided the general design guidelines and Hub provided the final design for both elements. The micropiles were designed to support the entire bridge loads without transferring any additional loads onto the abutments.

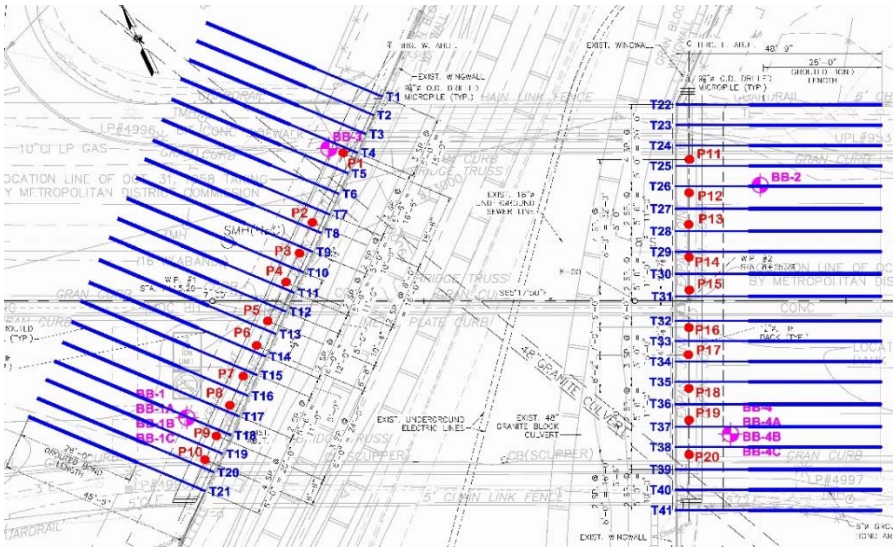



Figure 3. Foundation System Layout

All the micropiles were designed with 7.0" OD permanent steel casing drilled into stable rock. The original design by the DOT required a 20 feet long rock socket. After the required verification tests were completed and analyzed, the final design of the rock socket length was reduced to 10 feet based upon a conservative ultimate bond value of 200 psi. The reinforcement for the piles was a No. 18 Gr 75 bar within the rock socket plus a 4.5 feet lap above the tip of the

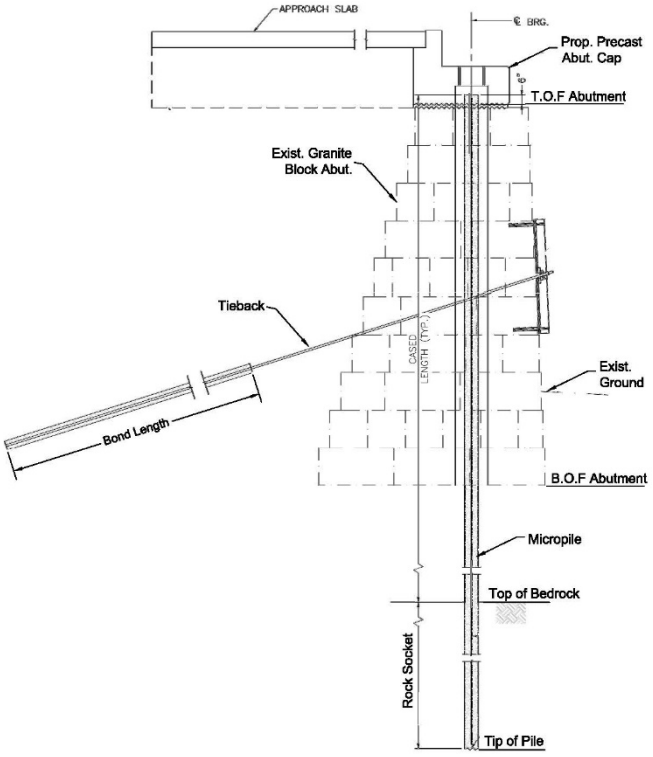



Figure 4. Abutment Section (as per Bid Documents)

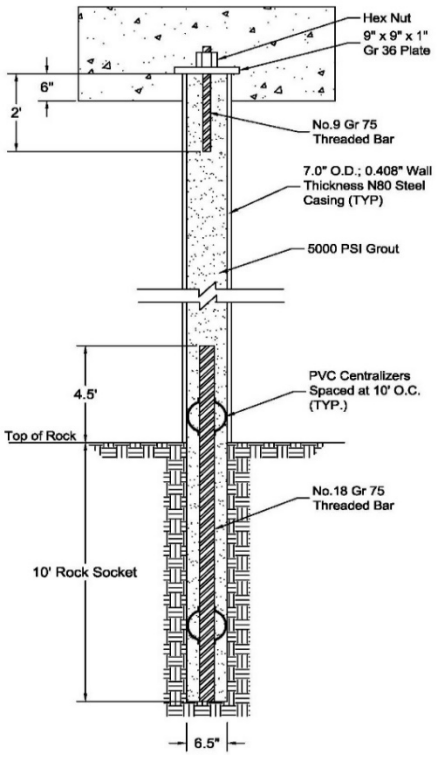



Figure 5. Drilled Micropile Detail (Hub Design)

casing. Figure 5 shows the design details of the micropiles. The micropiles were installed during night shifts.

The tiebacks were drilled with 5.5" OD temporary casing at 10 degree and 15 degree angles into medium dense granular fill behind the granite walls. The shallow angles were critical to the overall design to ensure that the vertical loads imposed from the tiebacks be as small as possible. The 2.5 foot deep void under one of the abutments was a major factor in this decision. Also, the tiebacks were installed directly under the bridge. The shallow angles allowed the installation to be performed with a conventional drill rig. Typically, permanent tiebacks are not bonded in fill materials. However, the relatively low design loads, the consistent thickness of the structural fill layer, the overall economics of relatively shorter tiebacks (as compared to the much longer tiebacks that would have been required to bond them into the silt layer below the fill) and the lack of space for the longer tiebacks made this solution the only one that was feasible.

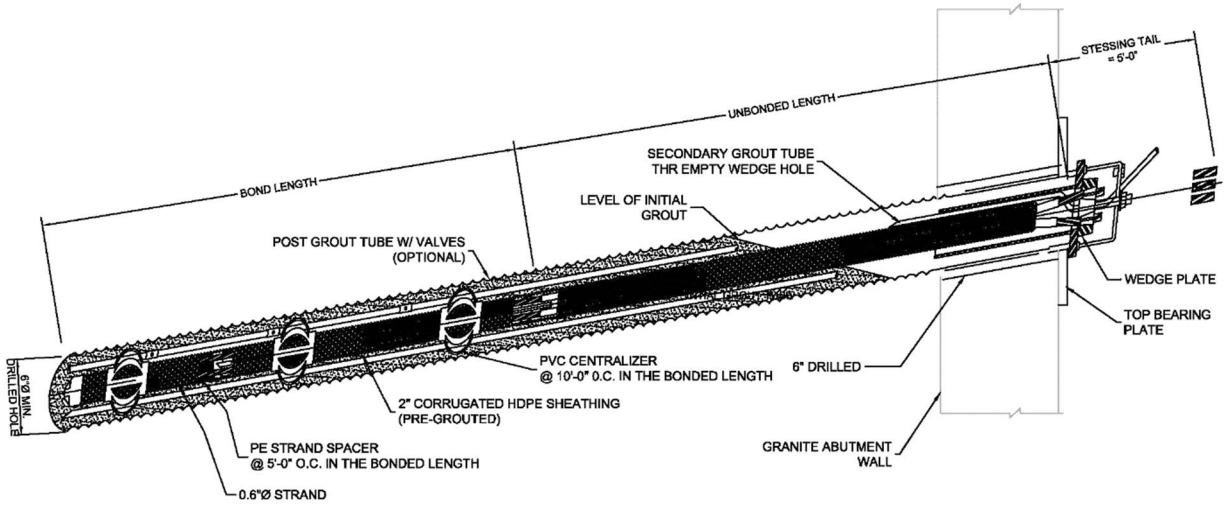



Figure 6. Tieback Detail

The tiebacks consisted of 2-0.6" diameter, Grade 270 low relaxation strand anchors. Hub would have preferred to use 3 strands for redundancy, however our vendor (Dywidag Systems International) indicated that Post Tensioning Institute (PTI) recommends that permanent wedge seating to be done at minimum 50% of GUTS (Guaranteed Ultimate Tensile Strength) of the strand anchor to ensure the strands will not slip after lock-off load. The anchors were fully encapsulated with double corrosion protection or Class I per PTI to provide the required level of corrosion protection specified. The drill casing sections were extracted after the grouting process. Additional grout was added under pressures of up to 200 psi as the casing was extracted. Figure 6 shows the details of the various components of the tendons.

The total length of the tiebacks ranged from 47 feet to 50 feet. The unbonded length was determined as the length of the tieback from the face of the abutment wall to the failure plane plus 5 feet. The theoretical failure plane is inclined at an angle of $45°+\phi/2$ which is 61° from the

horizontal, where ϕ is the friction angle of the sandy fill. The inclination of the failure plane was projected 4 feet above the bottom of the abutment. Figure 7 show the location of theoretical failure plane.

A conservative bond value of approximately 7.5 psi was selected for the sandy fill resulting in a bond length of 28 feet. The impact of working close to an active commuter train line into Boston did not allow for any possibility for tieback failures. To further ensure that this would not happen, a post-grout tube was added to each tendon to ensure that additional tiebacks would not be required.

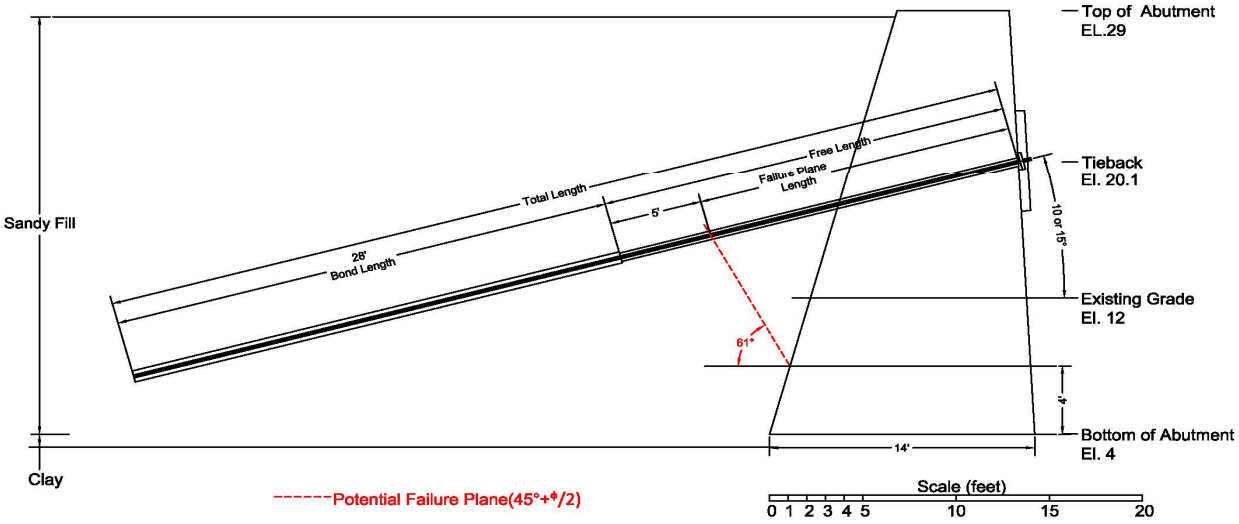



Figure 7. Location of Theoretical Failure Plane

## PILE LOAD TESTING

Two verification pile load tests were performed on sacrificial piles prior to the installation of the production micropiles. The test piles were installed and tested on ground surface level close to the abutments. The test piles (TP-1 and TP-2) were installed on March 26, 2015 and were tested on March 30 and April 1. The test piles were reinforced with No. 24 Gr 75 bars for the full length of each test pile. Strain gauges were utilized to monitor very small movements to correlate to load at various depths in the socket. Tell-tales were utilized to determine movement at the top and bottom of the rock sockets. The test piles were subjected to the loading procedures in general conformance to ASTM A3689. The tension loads were applied in increments using a calibrated test jack and were monitored using a pressure gauge and a load cell. The movement at the pile head was measured using three dial gauges. Figure 8 shows a design detail of the pile load test. Photo 2 shows the test pile setup.

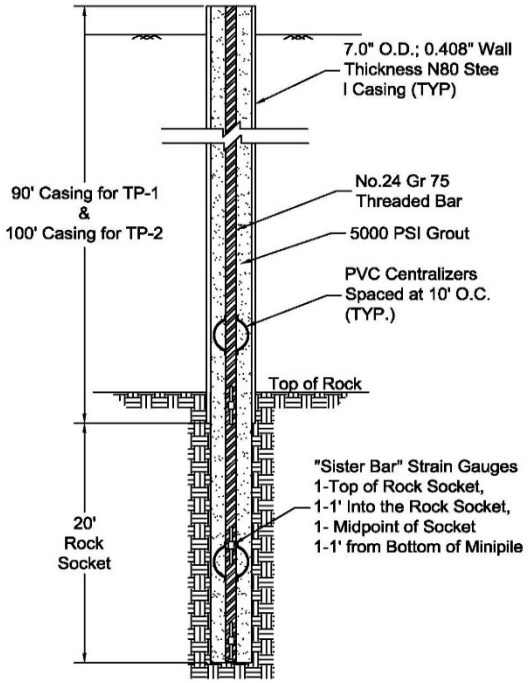



Figure 8. Verification Test Pile Detail



Photo 2. Verification Test Pile Setup

Test pile TP-1 was initially loaded to 477 kips and was held for a 10 minute creep test. After the test pile was unloaded, it was reloaded to 530 kips to record how the pile would react to another cycle of a higher load. During the reloading cycle, the pile movements followed the theoretical elongation line of the test bar. The net settlement of the pile at 477 kips was 0.116 inches and 0.252 inches at 530 kips. The full test loads were transferred directly to the top of the rock

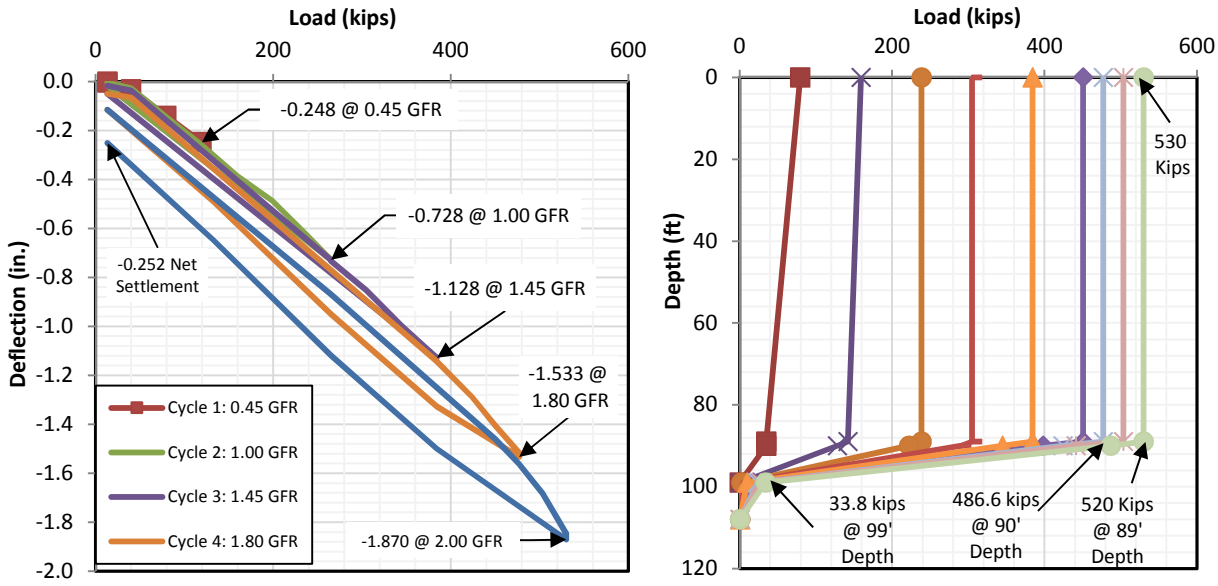



Figure 9. Settlement v Load Test Plot and Load Distribution Plot for TP-1

socket. Only about 34 kips remained when it reached the midpoint of the rock socket and almost zero kips at one foot above the pile foot tip. Figure 9 provides the settlement v load test plot and load distribution plot for TP-1.

Test pile TP-2 was loaded and performed similar to TP-1. The net settlement of the pile at 477 kips was 0.172 inches and 0.387 inches at 530 kips. Almost 92 percent of the applied test load was transferred directly to the top of the rock socket. Less than one kip of load remained at the midpoint of the rock socket length and at the pile tip. Figure 10 provides the settlement v load test plot and load distribution plot for TP-2.

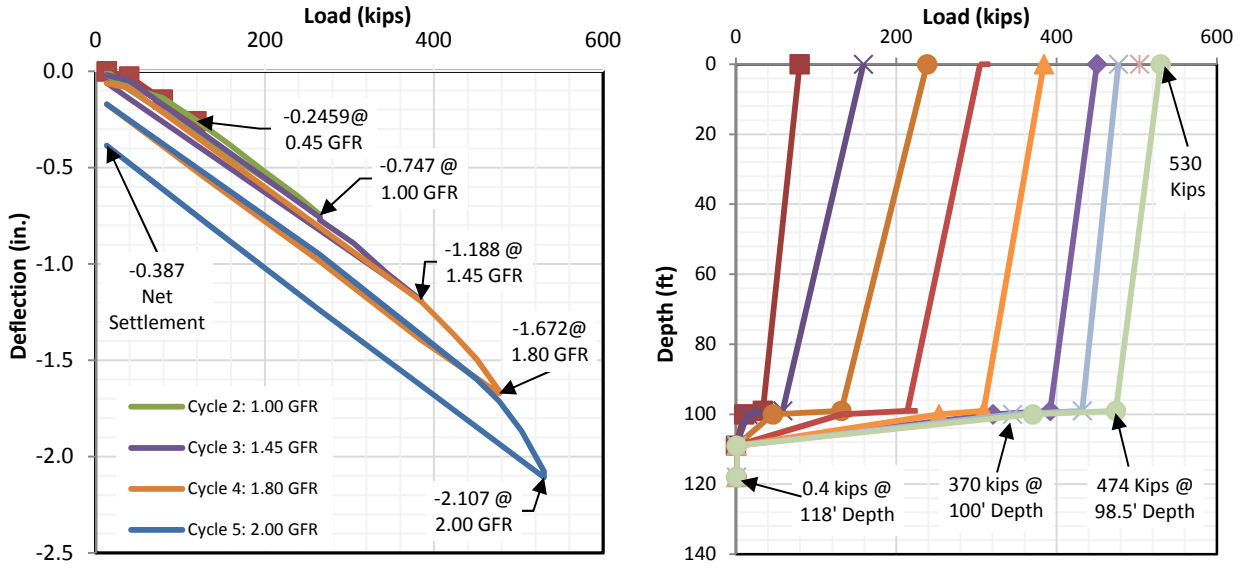



Figure 10. Settlement v Load Test Plot and Load Distribution Plot for TP-2

Both verification pile load tests performed extremely well within all the project acceptance criteria. The test results indicated that a minimum ultimate bond value of 280 psi had been demonstrated. Using a conservative bond value of 200 psi resulted in a reduced socket length of 10 feet.

All DOT project require that proof tests be performed on 5% of the production piles. In this case, a minimum of one proof test would be required at each abutment. The site logistics were not favorable to the placement of the test frames and support mechanisms at any production pile location without imposing significant reaction loads onto the existing bridge. On of the reasons that we performed the two verification tests to much higher test loads required was to demonstrate that a higher factor of safety would exist within the rock socket such that proof test could be waived. The combination of the very successful test results plus the inability of the existing bridge to support the reaction loads (without significant shoring and detouring of the traffic on and off the bridge) allowed the DOT to accept our recommendation to waive the two proof tests.

TIEBACK TESTING

A tieback test program was performed in advance of the installation of the production tiebacks to confirm that the required capacity could be achieved in the fill with a reasonable factor of safety. The installation of the tiebacks, placement of the LTU and the testing of each tieback was performed during both day and night shifts. Photo 3 shows the forms for LTUs at each tieback location. A performance test was conducted on the tieback in accordance to The Recommendations for Prestressed Rock & Soil Anchors, Post Tensioning Institute, PTI, loading schedule. The tension loads were applied cyclically and in increments up to 133% of tieback design load (66.5 kips) and was held for a 10 minute creep test. The creep at 66.5 kips test load was measured as 0.006 inches at 10 minutes, well below the 0.04 inch acceptance criteria. Figure 8 provides the load v settlement plot for the tieback performance test. The production tiebacks were proof tested in accordance with PTI standards. All the proof tests performed similarly to the performance test.



Photo 3. LTU Forms for Tiebacks

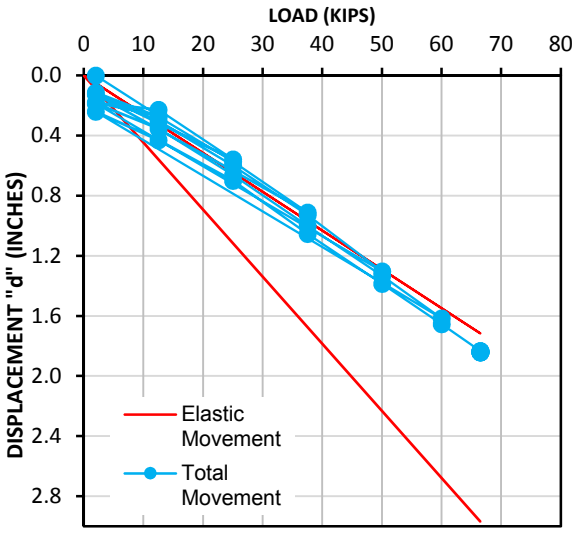



Figure 8. Load v Settlement Plot for Tieback Performance Test

After each tieback was tested successfully, an initial lift-off test was performed to confirm that the lock-off load was within 5% of the specified load. After each tieback met the required acceptance criteria, it was unloaded to allow for the construction of the LTU (refer to Photo 3). After the concrete reached the required strength, the LTU was installed and bolted to the granite wall. The anchorage hardware was then installed and the tendon was loaded to the required lock-off load. A cover box was then placed over the exposed tendon to completely seal the tendon from the elements.

## CONCLUSION

Photo 4 shows the installation of the micropiles during the night shift alongside active traffic lanes. Photo 5 shows the installation of the tiebacks directly below the existing bridge superstructure. The drill rig is next to a spur track which was not active during the day shifts. The project presented significant challenges, especially very difficult logistics requiring the micropile installation to be performed close to vehicular traffic and the tiebacks adjacent to the extremely active commuter tracks. Drilling through significant thickness and depth of the granite block abutments proved to be difficult at times, especially when the granite blocks were loose and shifted while drilling.

Notwithstanding the above, all aspects of the micropiles and tiebacks performed successfully, and the overall installation was completed within the allotted schedule.

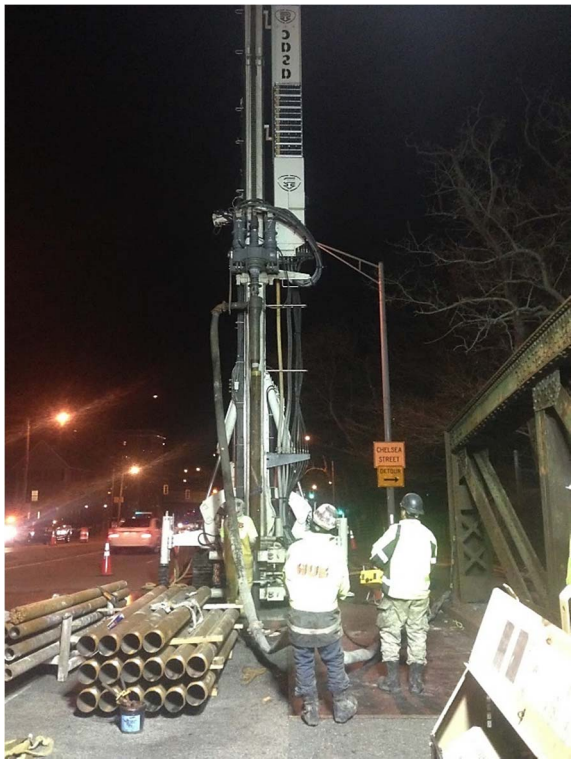



Photo 4. Micropile Installation at Night



Photo 5. Tieback Installation under the Existing Bridge

# PILE FOUNDATION FOR CABLE SUPPORT STRUCTURE IN BROWNFIELD SITE WITH UNDERGROUND OBSTRUCTIONS – A CASE STUDY

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## ABSTRACT

The execution of projects in brownfield site conditions with restricted space throws up additional challenges especially in foundation construction and design. Engineering design for foundations of structures in such projects involves not only correctly applying the principles of science but also meeting unknown challenges posed by variations in assumed sub-surface conditions. This paper talks about a case study in an existing plant being retrofitted with new equipment where the pipe and cable supporting structures connecting the various units had to be located, designed and executed considering existing structure in the vicinity. However, even after careful planning and ground survey to locate the facilities at the most convenient location to meet design and construction requirements, there were unknown surprises thrown up during construction from unknown subsurface utilities. The location of the tower structure foundation was found to have numerous underground utility services like pipes and cables in live condition that could not be disturbed. There was no place to install the piles required for the foundation at the design location; also, it was not possible to reroute the new pipes or find alternate location for the structure. Solution to this seeming impasse was arrived at by innovative design of the foundation and superstructure that avoided the interference to piling operation posed by existing live service lines and without altering the location of the tower structure to preclude modification in the already fabricated pipe and cable gallery structure. The structure and foundation were redesigned to avoid the critical zone of interference to piling operation. This solution could be executed successfully through innovative concepts, careful planning, collection of survey data and continuous interaction with site.

**Keywords: process plant, cable bridge/ rack, pile foundation, underground utilities**

## INTRODUCTION AND BACKGROUND

The project comprised design and construction of a process plant where improved and more environment friendly technology was being implemented to reduce emission and reuse waste heat for process operations. The project required installing various process equipment and supporting auxiliary units and structures adjacent to operating units of the existing process plant. Space availability for locating the various units was a difficult issue for the project. The locations of the new units were planned based on two seemingly opposing concepts. The units had to be located to be close enough to the existing process units to supplement the operations and minimize the length of utility service connection. On the other


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hand, the new units had to be located provide adequate space to enable construction of foundations and substructures and ensure safe erection of the new facilities.

To locate the plant units in most efficient manner various surveys both over-ground and underground were carried out. Overground survey was done through total station mapping all existing structures and available corridors; underground survey by ground penetrating radar was done to locate probable routing of sub-surface utilities like pipes and cables. Locations of units were planned based on the available survey data and then syndicated with the operation department of the owner from the aspect of constructability with complete safety in vicinity of existing plant.

## LAYOUT OF THE PLANT

The units of the new plant are located within the operating zone of existing process plant and adjacent to many of the existing facilities in operation. The new plant consists of various process and ancillary units, conveyor tunnels, dust extraction system equipment, dust storage structures and various utility structures for conveying pipes and cables. The plant layout is shown in Fig.-1 below:

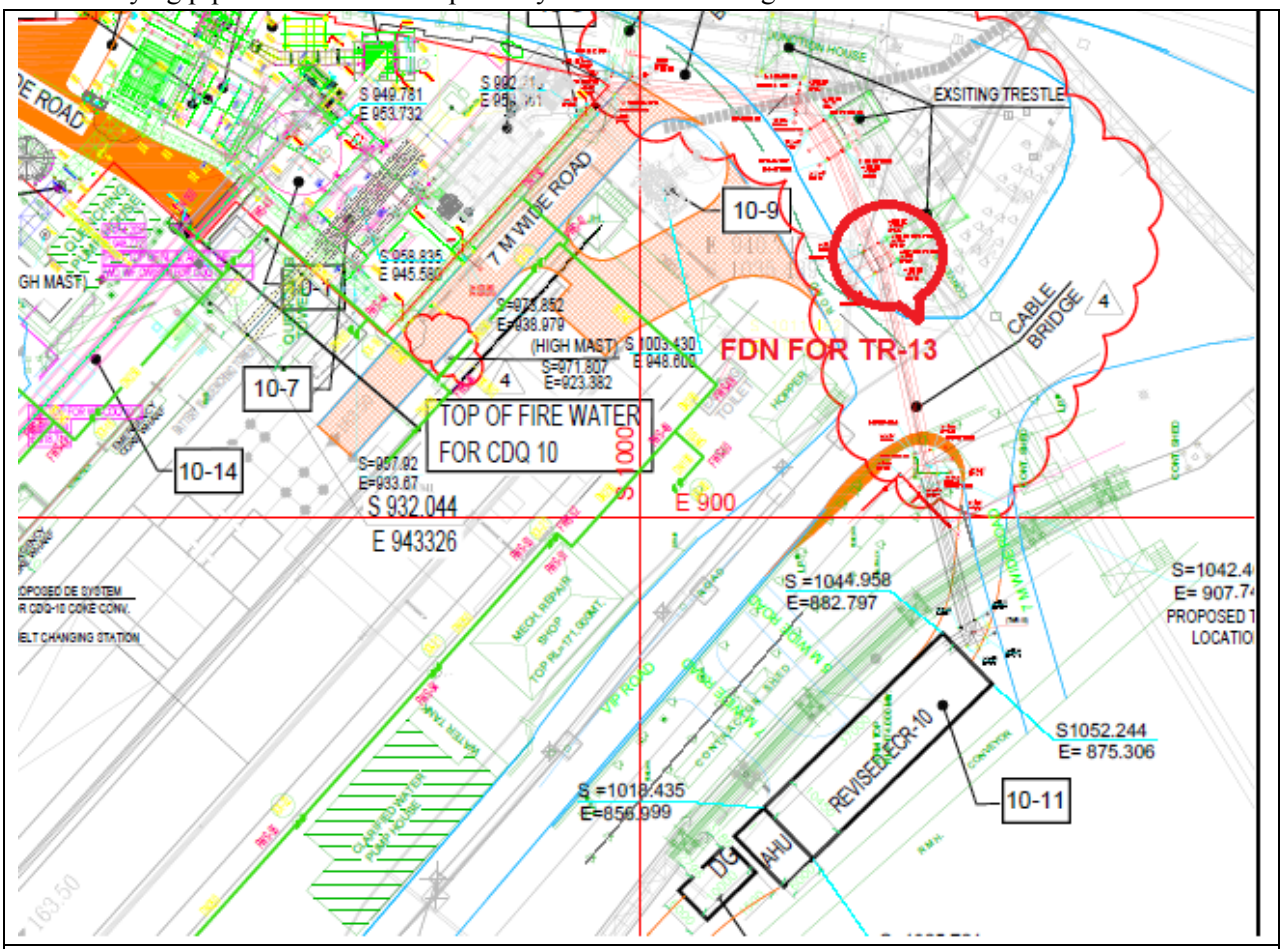



**Fig. 1. Layout of the Cable Bridge structure**

The unit that is the subject of study in this paper is a structure to convey pipes and cables between the electrical switchgear room for the new plant and the various equipment at different locations of the project site. The cables and pipes are carried through an over-ground bridge structure of steel

construction, supported on four-legged towers and two-legged trestles at suitable intervals. The locations of the supporting towers/ trestles were chosen so that there was minimum interference with existing structures. The route for this cable bridge structure is in a narrow corridor with a plant road on one side and very heavy structure of conveyor support on the other side. The additional constraint was that the adjacent road serves to carry dumpers for transporting materials in operating plant and maintenance equipment the road could not be closed for construction. The new structures and their foundations had to be designed and constructed with these limitations.

## LAYOUT OF THE CABLE SUPPORT STRUCTURE

The cable supporting structure comprises of bridge or rack of steel construction and steel lattice towers at suitable locations. The locations of the towers are chosen to suit existing plant facilities and the length of the cable bridges vary from about 18m to 30m. The most critical tower location is the tower marked TWR-13. This tower support is adjacent to a heavy structure of Junction House of conveyor structure towards east side and plant road towards the west. The design, supply and erection of the steel structure for cable support was in the scope of another vendor. The foundation design for this structure was based on the structure load data furnished by the other vendor and was quite critical in the initial phase itself due to restricted space available below ground in the space available between the foundation for conveyor junction tower and edge of road. The 3D view and general arrangement of the structure are shown in figures 2 and 3 respectively.

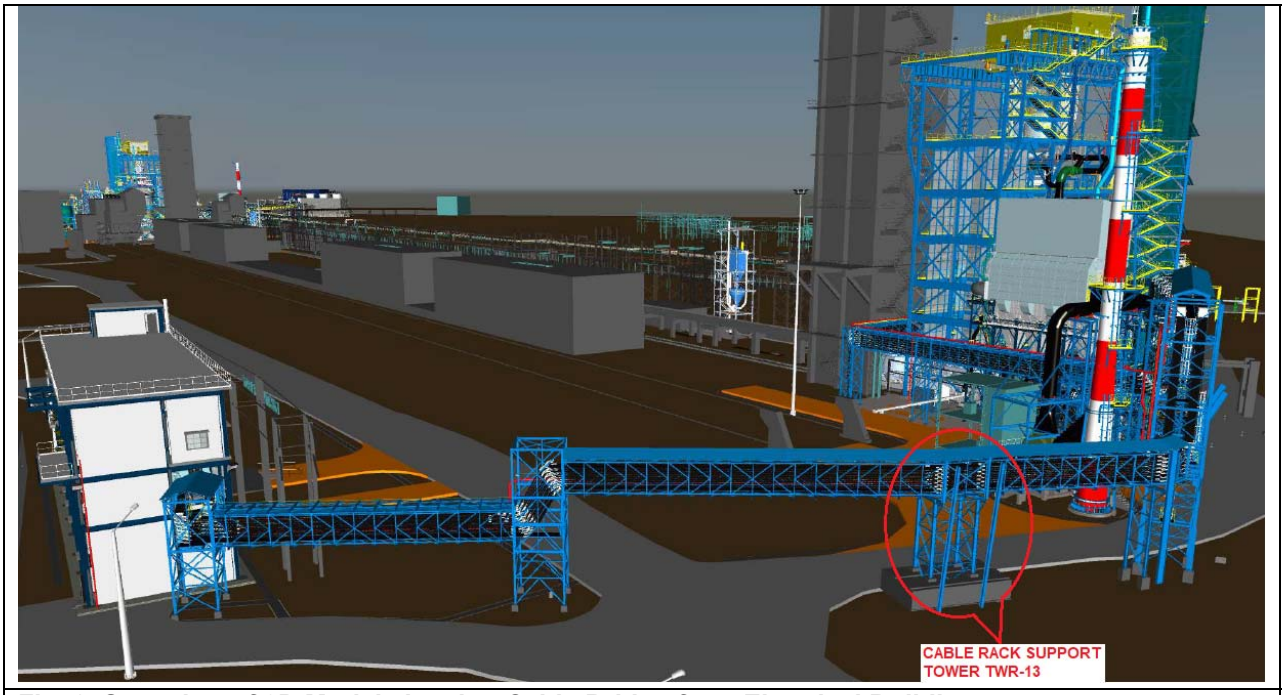



**Fig. 2. Snapshot of 3D Model showing Cable Bridge from Electrical Building**

The initial foundation design was carried out as a shallow raft foundation. The contact area of the raft was provided as required to distribute the loads from the superstructure safely to the foundation. The designed solution for the foundation had already considered the site constraints known at the time of design – proximity to adjacent existing conveyor gallery supporting tower foundation, existing road and other obstructions. The foundation design had been carried out with due consideration for these constraints by

accommodating in available space. It was agreed with construction contractor and owner that excavation protection systems in the form of structural shoring would be provided to protect the adjacent structure foundations and the road.

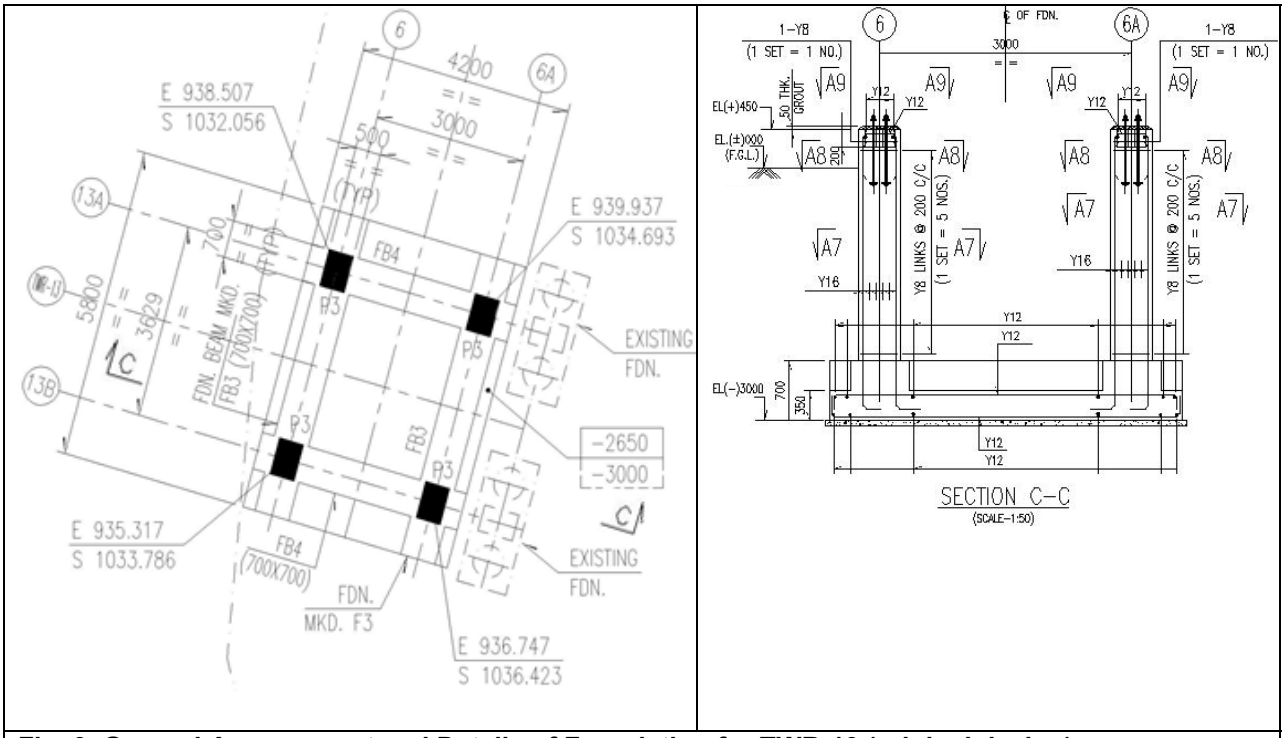



**Fig. 3. General Arrangement and Details of Foundation for TWR-13 (original design)**

## SITE CONDITIONS AND CONSTRAINTS

Preparatory work for foundation execution was started at site. The prevailing work safety standards required the use of cable detectors and making trial pits to determine the presence of underground utilities – buried pipes and cables in the zone of the foundation. The process of trial trenching is done in stages of 1.5m depth at each stage. Cable detectors sensed large number of cables in the foundation footprint zone. There was also a service water pipe conveying water to other plant units. Neither of them could be disturbed or re-routed since required shutdown of the services could not be obtained. The site conditions are depicted in Fig. 4 showing presence of cables and pipe.



**Fig. 4. Constraints at site – adjacent Junction tower structure and underground utilities**

## FIRST STAGE MODIFICATION OF DESIGN

After reviewing the site condition, it was evident that though the first solution attempt would have provided an economic foundation design, execution constraints would not permit the implementation of this design. Since the available space was restricted, the concept was changed from spread foundation type to deep foundation so that the base area required for the transfer of load from superstructure would be minimum. Previous geotechnical characterization of the site showed that moderately weathered rock was available at relatively shallow depth of 8 – 10 metres which would enable bored cast in situ rock socketed piles as suitable foundation type. New design for the foundation with piles was prepared and drawing was provided for execution at site. Since it was difficult to go below the existing utilities for construction of the pile cap, the pile cap was planned to be put over ground avoiding any probability of interference and damage to these utilities. Pile locations were designed to avoid interference with the anticipated layout of utilities. The details of the modified design are shown in Fig. 5.

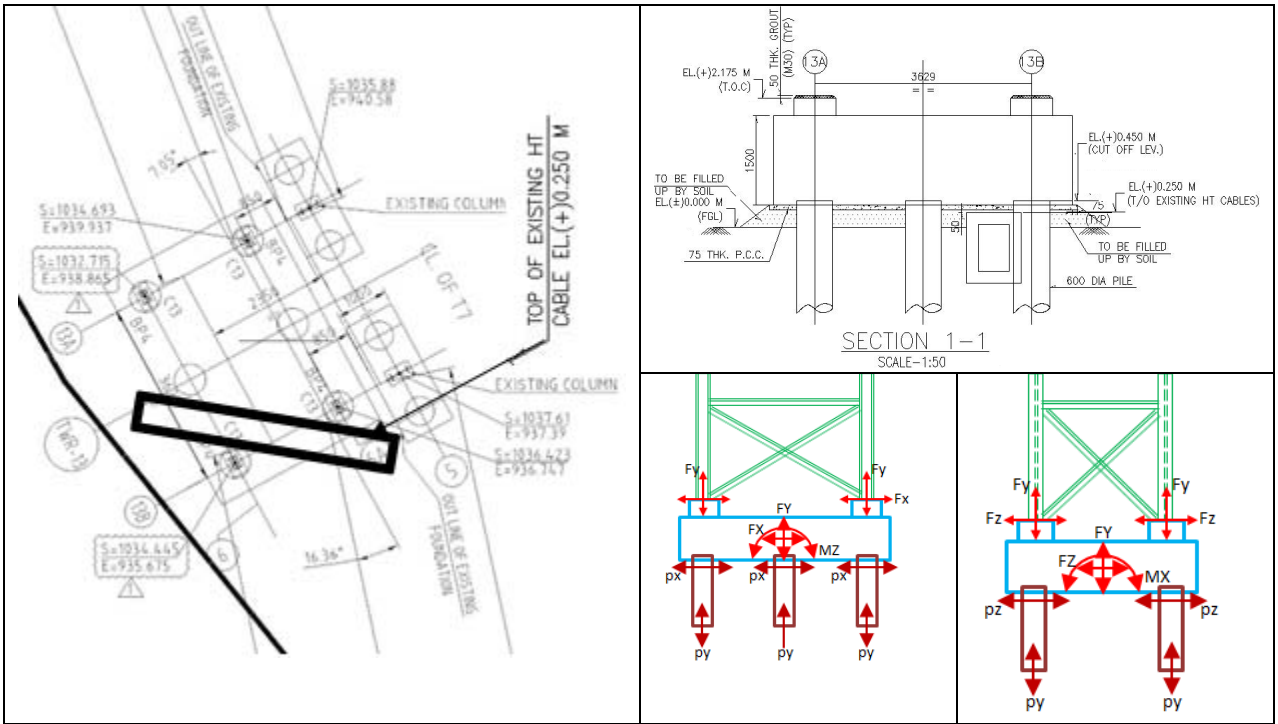



**Fig. 5. Modified foundation design for TWR-13 and load distribution to piles**

## ADDITIONAL CONSTRAINTS

Though the first stage modified design considered the presence of underground utilities, more surprises were unearthed on further progress of excavation at site. When the trial trench excavation reached the second stage, another set of cables running almost perpendicular to the set of cables detected in first stage was revealed. It was obvious that the pile layout as designed in the first stage modification could not be executed. Attempts were made to relocate the piles and modify the pile cap to suit the pile locations so as not to alter the structure which was being supplied by others; however, these were unsuccessful due to safety concerns in driving piles close to live cables. Attempts were also made to check out alternate location for the tower on the other side of the road with associated change in route of the cable bridge, but it was not accepted by plant owner. Shifting of tower location towards south to clear the obstruction zone was also not feasible as the structure would then be located at the road junction restricting vehicle movement which was again not acceptable for plant operation. It became evident that the tower location could not be changed and though pile foundation was the only solution for the case, conventional design concept would not work.

## INNOVATIVE DESIGN CONCEPT

Summarizing the situation and the design constraints, the following critical aspects were noted:

a) The previously selected route for the cable bridge structure and the location of the tower (refer Fig. 1) cannot be changed.
b) The solution of pilecap resting on six piles supporting the tower (refer Fig. 5) could not be executed due to safety concerns of pile executing close to live utilities. The load distribution to piles considered all vertical and horizontal loads and the overturning moments arising out of the lateral loads at base and are also shown in Fig. 5.
c) Modification of the cable bridge structure was not feasible since it had already been fabricated and any alteration would involve additional cost and time.
d) Pile locations could not be dictated from design perspective; rather the design had to suit the available locations where piles could be installed safely.

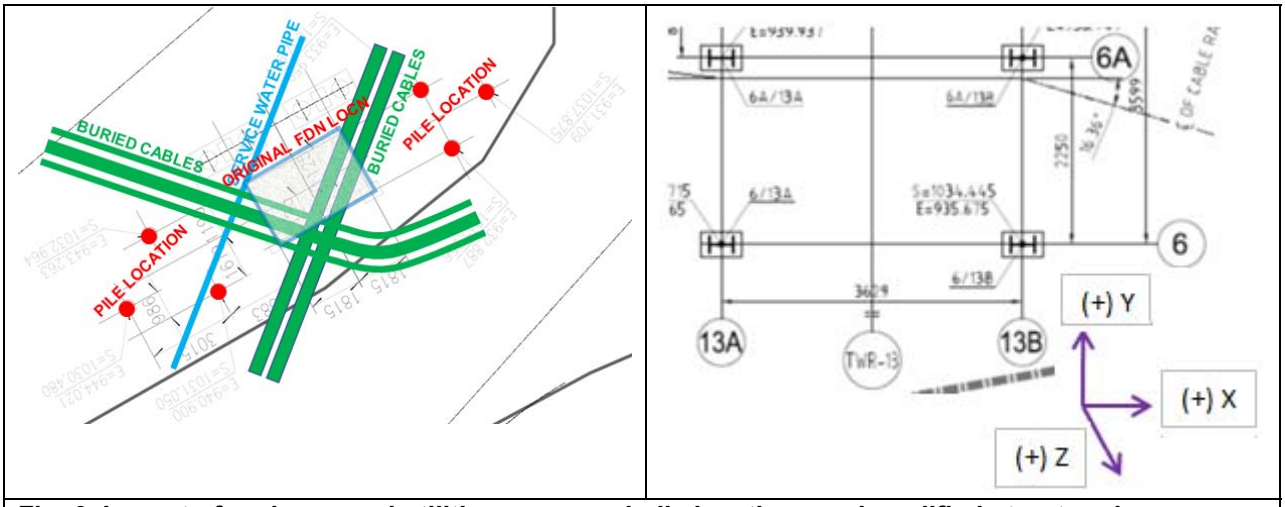



**Fig. 6. Layout of underground utilities, proposed pile locations and modified structure base**

The strategy adopted to provide a feasible solution under these constraints was based on the following concepts:

a) Foundation design would be completely modified to suit site conditions. Piles would be installed at locations where it was feasible to be executed at site in completely safe manner and without any interference with the existing underground cables and pipes.
b) The pile cap design would be performed using the actual pile locations furnished from site. It was planned that two groups of three piles would be provided without any regular pattern. The pile cap design would account for the force distribution to the pile group.
c) The lower portion of the fabricated tower structure would be modified. The tower would rest on a portal formed of heavy steel beams with the columns resting on the two separate pile caps. The arrangement of structure and foundation and the load distribution to the piles is shown in Fig. 6.
d) The portal structure would be designed with required rigidity so that the deflection at the location of the tower would not exceed the value of expected foundation settlement of 25mm considered in the original foundation design with tower base resting on spread foundation. The settlement at the pile cap locations are negligible with the rock socketed piles.

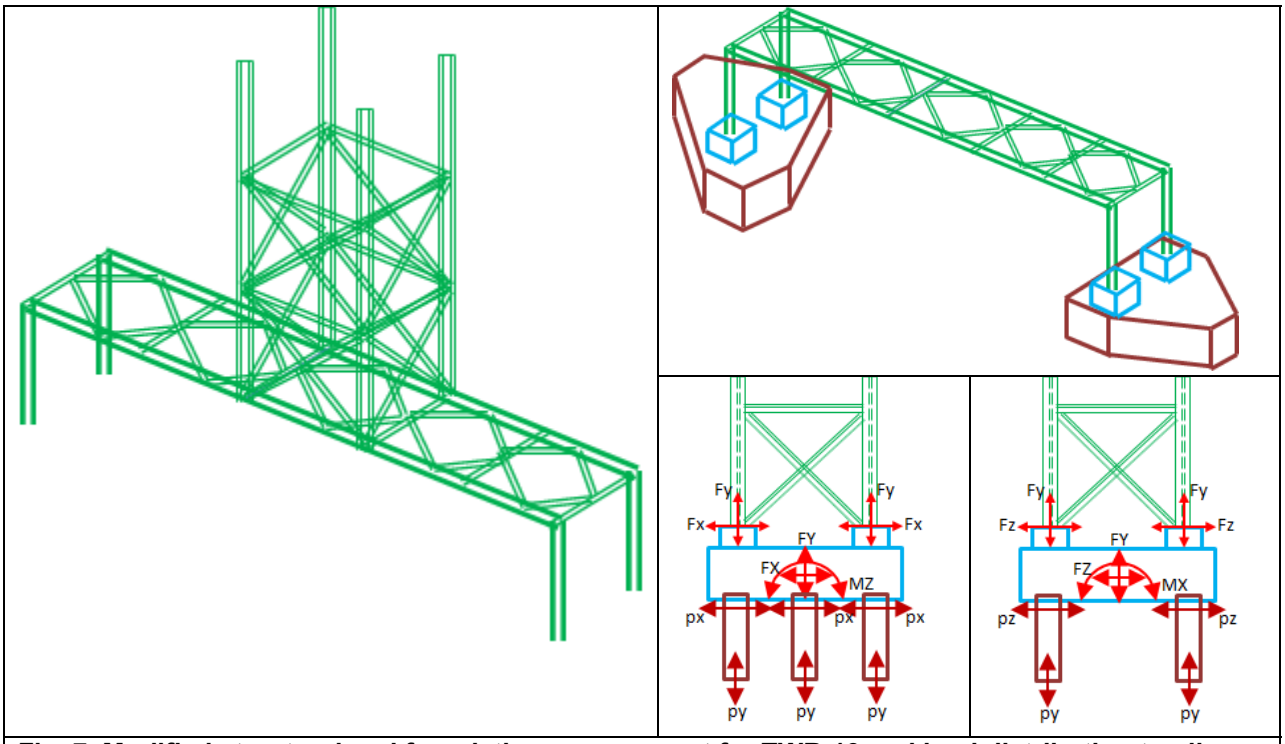



**Fig. 7. Modified structural and foundation arrangement for TWR-13 and load distribution to piles**

***Soil stratification and pile design***

The soil stratification as revealed by the exploration work is as follows:

**Table 1. Soil stratification at the project site**

| Strata | Thickness (m) | SPT-value | Description |
|---|---|---|---|
| Stratum I | 0.75-4.0 | 6-10 | Heterogenous fill of moorum, quartz pieces, mica schist rock fragments with hard slag with coal pieces, cinder, concrete |
| Stratum II | 2.0-3.0 | 23-42 | Lateritic soil with silt, micaceous sand and quartz pieces |
| Stratum III | 1.5-7.2 | 42(3cm) | Highly weathered micaschist rock with quartz pieces |
| Stratum IV | 4.0-10.0 | - | Partly weathered micaschist rock with quartz veins |
| Stratum V | 12.0-25.0 | - | Slightly weathered micaschist rock with quartz veins |

The pile type recommended in the geotechnical report is bored cast-in-place piles socketed into the weathered rock of stratum IV. Piles will derive their resistance from skin friction and socket resistance in rock stratum IV. Point resistance is not considered since the chiselling operation will fracture the rock at tip of pile. Socket length recommended is 4 times pile diameter based on the recommendation of maximum socket length given in standard pile design references. The pile resists vertical compression and tension loads by socket friction. Recommended pile capacities are:

**Table 2. Recommended pile load capacity**

| Pile dia (mm) | Pile safe load capacity (kN) | | | |
|---|---|---|---|---|
| | Vertical compression | Vertical Tension | Lateral | |
| | | | Free head | Fixed head |
| 450 | 850 | 380 | 12.95 | 34.37 |
| 550 | 1000 | 570 | 17.85 | 47.38 |
| 600 | 1300 | 650 | 20.52 | 54.45 |

The pile chosen for this foundation is 600mm diameter pile of length 12m which would include 2.5m socketing into rock.

***Redesign of structure and foundation***

The tower structure was reanalysed for the loads imposed by the cable bridge to extract the design forces at the foundation level. The basic loads considered were dead load (DL), live load (LL), cable load (CBL), and wind load (WL); the loads are summarized in Table 3 for the structure configuration and force axis definition in Fig. 7.

**Table 3. Load Data for Columns of Cable Bridge supporting tower structure**

| Load combination | Loads for 6/13A & 6A/13A | | | Loads for 6/13B & 6A/13B | | |
|---|---|---|---|---|---|---|
| | Fx(kN) | Fy (kN) | Fz (kN) | Fx(kN) | Fy (kN) | Fz (kN) |
| DL+LL | -140.026 | -216.741 | -13.315 | 140.046 | -245.821 | -6.682 |
| DL+LL+CBL | -242.986 | -351.588 | 32.91 | 243.016 | -409.595 | -6.68 |
| DL+LL+CBL+WLZ+ | -242.986 | -351.587 | -12.62 | 243.016 | -409.594 | 108.731 |
| DL+LL+CBL+WLZ- | -242.981 | -351.585 | -103.679 | 243.021 | -409.597 | -122.101 |
| DL+LL+CBL+WLX+ | -242.981 | -351.585 | -60.366 | 243.021 | -409.597 | -122.101 |
| DL+LL+CBL+WLX- | -192.611 | -276.257 | 37.394 | 289.211 | -478.864 | -1.461 |
| 0.9DL+WLZ+ | -75.243 | -128.661 | 0.006 | 75.243 | -140.35 | 115.413 |
| 0.9DL+WLZ- | -75.238 | -128.659 | -47.74 | 75.248 | -140.353 | -115.418 |
| 0.9DL+WLX+ | -24.868 | -53.331 | -0.783 | 121.438 | -209.62 | 5.221 |
| 0.9DL+WLX- | -126.392 | -204.857 | -1.251 | 28.272 | -70.213 | -5.217 |

Upward loads are with +ve sign, Downward loads are with (-)ve sign

Based on these foundation forces the reactions in individual piles for the two pilecaps were worked out. The results are tabulated below:

Maximum compression in pile = 688kN < compression capacity of 1300kN

Maximum tension in pile = 260.36kN < tension capacity of 650kN

Maximum shear in pile = 54.04kN (this includes the provision for additional 25% load capacity due to loads generated by wind loads, refer IS 2911 Part 1/ Section 2). This is marginally close to allowable value for fixed head pile but is just adequate. The weight of the pile is already discounted for in the load capacity for compression; for pile in tension, only socket friction has been considered to resist the upward force due to bending moment on the pile group.

The revised analysis with modified structure and pile foundation was able to achieve the desired result of placing the foundations without causing disturbance to the underground utilities.

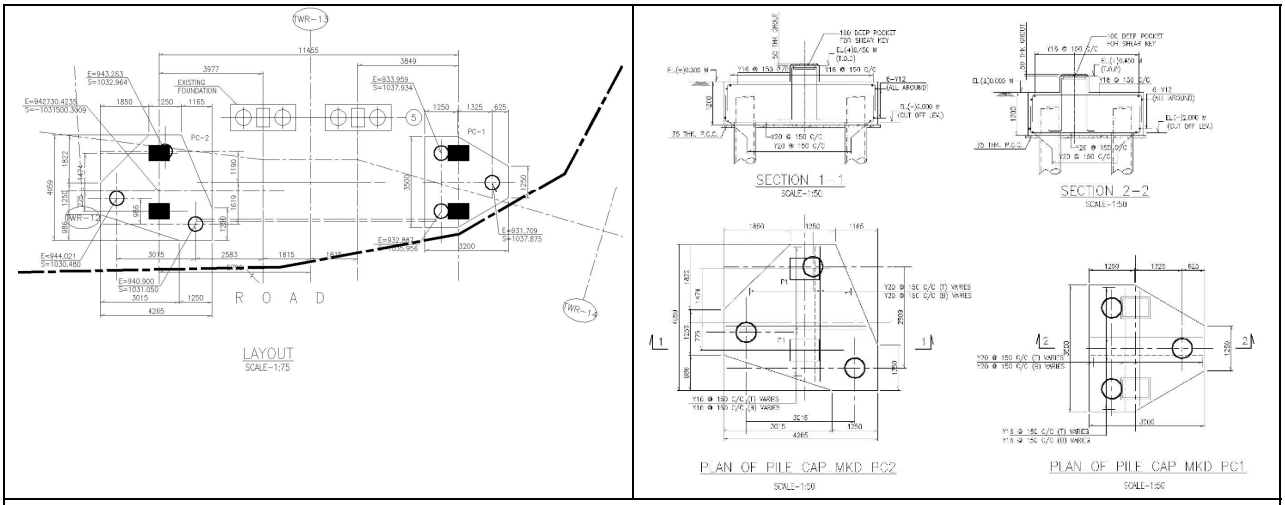



**Fig. 7. Modified layout and design of split pilecap foundation**

## SUMMARY

The component of design presented in this study reflects a typical problem encountered in brownfield site conditions where there are many challenges to construction due to interferences from existing facilities with numerous above and below ground obstructions and restrictions imposed by plant operations. The solution presented for the problem is a case study in application of engineering principles to overcome such constraints. This solution enabled the foundation design to be performed with required safety of supported structure and equipment and also to meet the challenges faced by in construction and enable successful implementation of the solution. The modified layout enabled construction to be completed without disturbance to existing utilities, with no shutdown of the road thus maintaining regular operation activities in the plant and without compromise of safety of the construction work. The final layout and details are shown in figure 7.



**Fig. 8. Pilecap and Erected structure of cable gallery and supporting tower**

## CONCLUSION

The case study presented shows the importance of proper and comprehensive planning for locating facilities in brownfield site conditions in operating plant. Facility locations should consider not only over ground survey but also the layout of substructures and underground utilities. In majority of cases, drawings for underground units are not available and while use of over ground survey work is very common, underground survey work is mostly neglected. This absence of data severely affected the foundation design; multiple and piecemeal exchange of information between site and design teams resulted in repetition of engineering work on schemes that could not be executed. There was considerable delay that incurred towards the work and though this activity was not in project critical path, the resources were engaged both in design and construction site for this work for much longer than required resulting in constraints for all stakeholders. This loss of time and resource could have been avoided through layout planning based on complete and detailed information of underground utilities.

## ACKNOWLEDGEMENTS

We would like to thank Tata Steel Limited, Jamshedpur, India for facilitating necessary approvals to Tata Consulting Engineers to work as "Consultant" in such a complex project and continuous support on site specific inputs throughout project execution.

## REFERENCES

Geotechnical Investigation Report by KND Engineering Technologies Limited

Pile Design and Construction Practice by M. J. Tomlinson

IS 2911 (Part 1/ Sec 2): Design and Construction of Pile Foundations – Code of Practice, Part 1 Concrete Piles, Section 2 Bored-Cast-in-Situ Piles

# STRENGTHENING OF EXISTING BUILDING FOUNDATIONS BY MICROPILES

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## ABSTRACT

Strengthening of foundations with micro piles is progressively being used due to major advantages that this technique can be adopted with limited space availability for execution and with less disturbance to the nearby existing structures/foundations. Current codes do not present specific design and detailing guidelines for this retrofitting situation. In one project, the process plant building is being presently operated with a 20Ton (T) capacity Electric Overhead Travelling (EOT) crane. Since the existing crane had served its design life, owner decided to install a new 25T capacity crane. To carry the new increased loads, retrofitting of foundations for few columns was required for higher loading. In this case, micro-piles are designed to underpin the existing foundations for upgrading load bearing capacity of the existing foundations. The work was to be done keeping the plant in running condition and no civil construction work for foundation jacketing is allowed inside the building. Micro piles were chosen for underpinning the foundations by installing adjacent to the existing raft outside the building and connecting the two rafts. Hollow TITAN bar has been used as the reinforcement as well as grouting facilitator and no other separate casing was required for constructing the micro piles. This solution achieved the objective of in-situ strengthening of foundations to carry the higher column load without need for foundation jacketing.

**Keywords: process plant, EOT crane, micropile, foundation underpinning**

## INTRODUCTION

In today's scenario, most of the projects are OPEX based and retrofitting the existing facilities to enhance its capacity for higher requirement is a very common phenomenon. Additionally, the plant owners are preferring the solutions which will not affect the operation of the plant during retrofitting. This is a case study where a solution is made for foundation retrofitting without hampering the plant activity. In this case, the load carrying capacity of a foundation has been enhanced by micro piling to make the structure safe for introducing a higher capacity crane required for the increased production from the process plant.

Micropiles are a type of deep foundation element that are used to provide structural support, for underpinning of existing foundations, transfer loads to deeper ground layers through liquefiable soil strata or for ground improvement by binding the soil mass. Micropiles are preferred in many situations with restrictions on accessibility, insufficient headroom space available, difficult underground conditions and in locations where noise and vibration issues for installing large diameter piles are of major concern.


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## BACKGROUND AND PROBLEM STATEMENT

The process plant building is a multiple bay shed building consisting of four bays where different manufacturing activities are performed. In one of the shops, the manufacturing process is being presently operated with a 20T capacity EOT crane and a semi gantry crane below the existing EOT crane one. Since the existing EOT crane has completed its design life, plant owner decided to replace this crane with a 25T capacity EOT crane. Adequacy of the existing column foundations of the building was checked for this increased crane load. It was observed that although the foundations along other column rows are safe, some of the foundations of e-row columns have lower factor of safety. Altogether, five numbers of footings from grid-16 to grid-22 of e-row, except foundations of grid-18 and 19 require upgradation of load carrying capacity. Foundations of grid-18 and 19 have larger dimensions (4.0 m x 2.0 m) and are placed at a greater depth (bottom of footing at -2.0 m) than the other foundations. These foundations are analysed and found to be safe in bearing pressure against worst combination of design loads. The building layout is shown in Fig.-1 below:

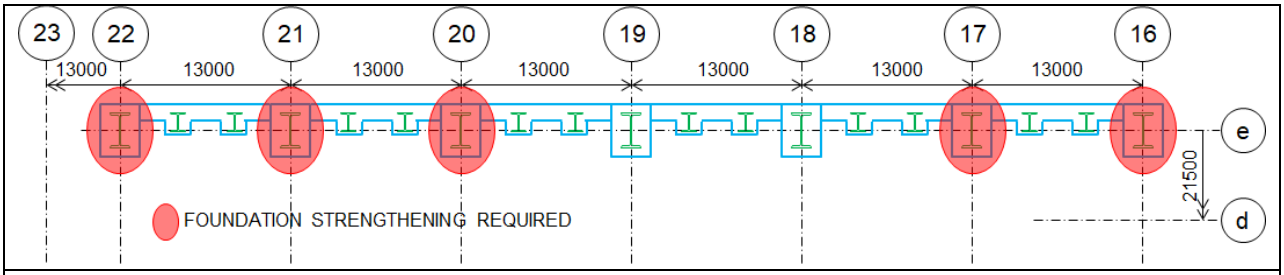



**Fig. 1. Layout of Columns for the Process Plant Shop**

For the balance five foundations, three following schemes for strengthening were initially conceptualized:

i) Grouting below the foundations.
ii) Extension of the plan dimensions of the existing foundations.
iii) Strengthening of existing foundations by micro-piles.

The strengthening proposals were discussed with the plant operation team. Shut down of plant and construction work inside the shop was not allowed by the operation group. The first two proposals would have required disruption of plant operation and are hence ruled out from constructability issues. In this case, micro-piles are designed to underpin the existing foundations for upgrading load bearing capacity of the existing foundations. The strengthening of foundation was envisaged by installing micro-piles from outside of the building. This would enable execution of the work for strengthening of the foundation without any shut down or carrying out construction activities inside the shop. Specialized drilling equipment would be required to install these micro-piles because of limited headroom facilities and other site constraints.

## DESIGN OF THE MICROPILE

The diameter of micropiles generally vary between 100 mm to 300 mm. The geotechnical report for this project furnished the allowable load carrying capacities of 150 mm, 200 mm and 250 mm diameter micro piles of varying tube thickness. For the underpinning purpose, selection of pile diameter depended on the

structural arrangement of the existing foundations and column pedestals, quantum of additional load to be carried and space available for micropile installation and foundation strengthening work.

The base slabs of existing foundations were of size 2.0 m x 1.5 m and pedestals for supporting structural columns were of size 1.0 m x 1.0 m. Since space available was small and work had to be executed for existing shop in quickest possible time, it was decided to adopt higher diameter micro piles with lesser numbers. In most cases, foundation strengthening work is done by increasing the base area uniformly on all sides. However, this special situation did not permit any work inside the shop and construction work had to be planned completely from outside the building. Additionally, available space in existing foundation on the outer side of column was 500 mm, which was not enough to drive 250 mm diameter micro pile.

The micro piles were thus planned to be installed in the extended portion of foundation. The new foundation was planned to be extended towards outer side of the building. Extended part of footing size was envisaged as 1.525 m x 1.5 m. This would provide required space for installing the micropiles, connecting the added portion to existing foundation and enable the whole construction work to be carried out from the outside of the roll shop.

Minimum clearance of 575 mm has been kept between nearest micro pile edge and face of pedestal to facilitate construction and meet the project standards. Micropiles of 250 mm diameter and 18 m length have been proposed As per the geotechnical investigation report of the shop area, the load carrying capacity of 250 mm diameter microplie of 18 m length are as follows
Compression capacity = 420 kN
Tension capacity = 310kN
Please refer Fig. 2 below extracted from the geotechnical investigation report[2]:

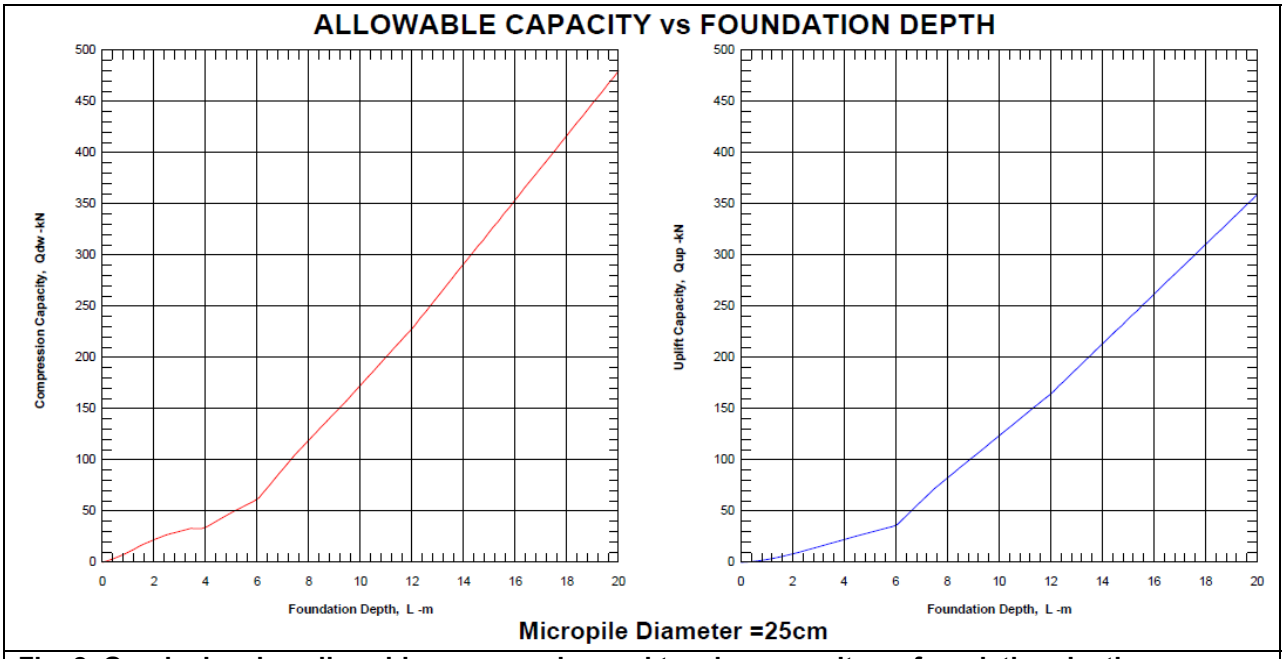



**Fig. 2. Graph showing allowable compression and tension capacity vs foundation depth**

The factors used to determine the centre to centre spacing of micropiles for underpinning of foundations include existing footing condition, accessibility for the work and the load to be carried by the micropiles. However, in all cases, it should not be less than three times of the diameter of the micropiles. Based on the loading consideration, soil characteristics, type of micropiles adopted for the project, it is envisaged that four numbers of micropiles will need to be installed for each foundation. Accordingly, center-to-center spacing of micropiles has been considered as 1050 mm along one direction and 950 mm along other direction. The foundation layout for strengthening scheme is depicted in Fig. 3.

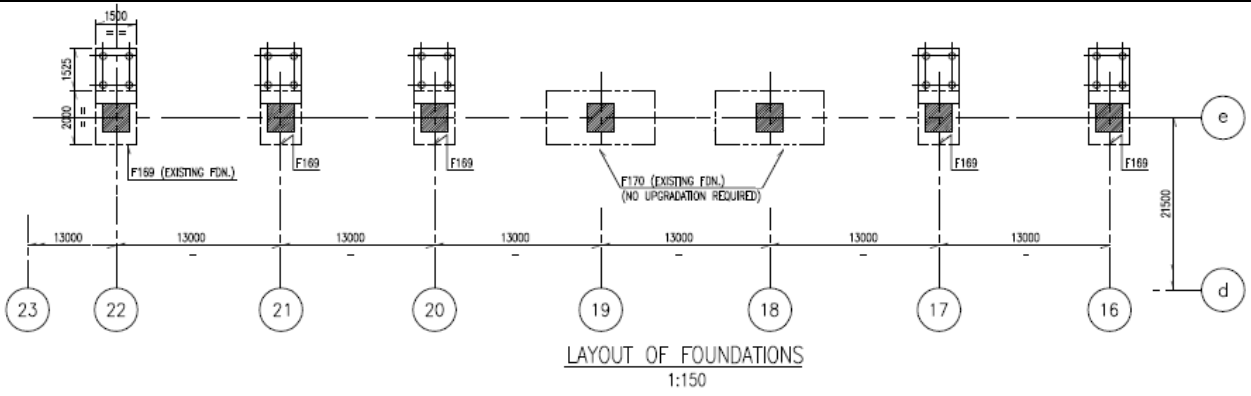



**Fig. 3. Layout of Foundations for Process Plant Shop**

## TYPE OF MICROPILE

The reinforcement in a micropile may be in the form of a reinforcement bar, a steel pipe, structural steel rolled section or CTS/Titan IBO®. CTS/Titan IBO® micropiles has been used for this case as these are used by locally available contractors.

CTS/Titan IBO® micropiles consist of a continuously threaded, hollow bar as reinforcement tendon, combined with a Portland Cement grout body of a minimum 25 N/mm2 strength. The rough, profiled surface of the grout body transfers tension and/or compression loads to the ground[1].

Ischebeck hollow CTS/TITAN BAR™ type are used as reinforcement to resist the required loading in the worst condition. The material of the hollow bar, as well as the thread deformations comply with ASTM A-615[1].

Initially, following type of TITAN bar is adopted:
Bar type. = 40/16
Nominal outside dia = 40 mm
Nominal inside dia = 16 mm
As per the loading consideration, 40/16 Ischebeck hollow CTS/TITAN BAR™ is adequate. But to meet the minimum requirement of the manufacturer's combination of rod size with bit type and achieving the required capacity as well a different bar size was chosen. Hence, 73/56 Ischebeck hollow CTS/TITAN BAR™ has been provided as the reinforcement in the micro piles. Details of the micropile are shown in Fig. 4.

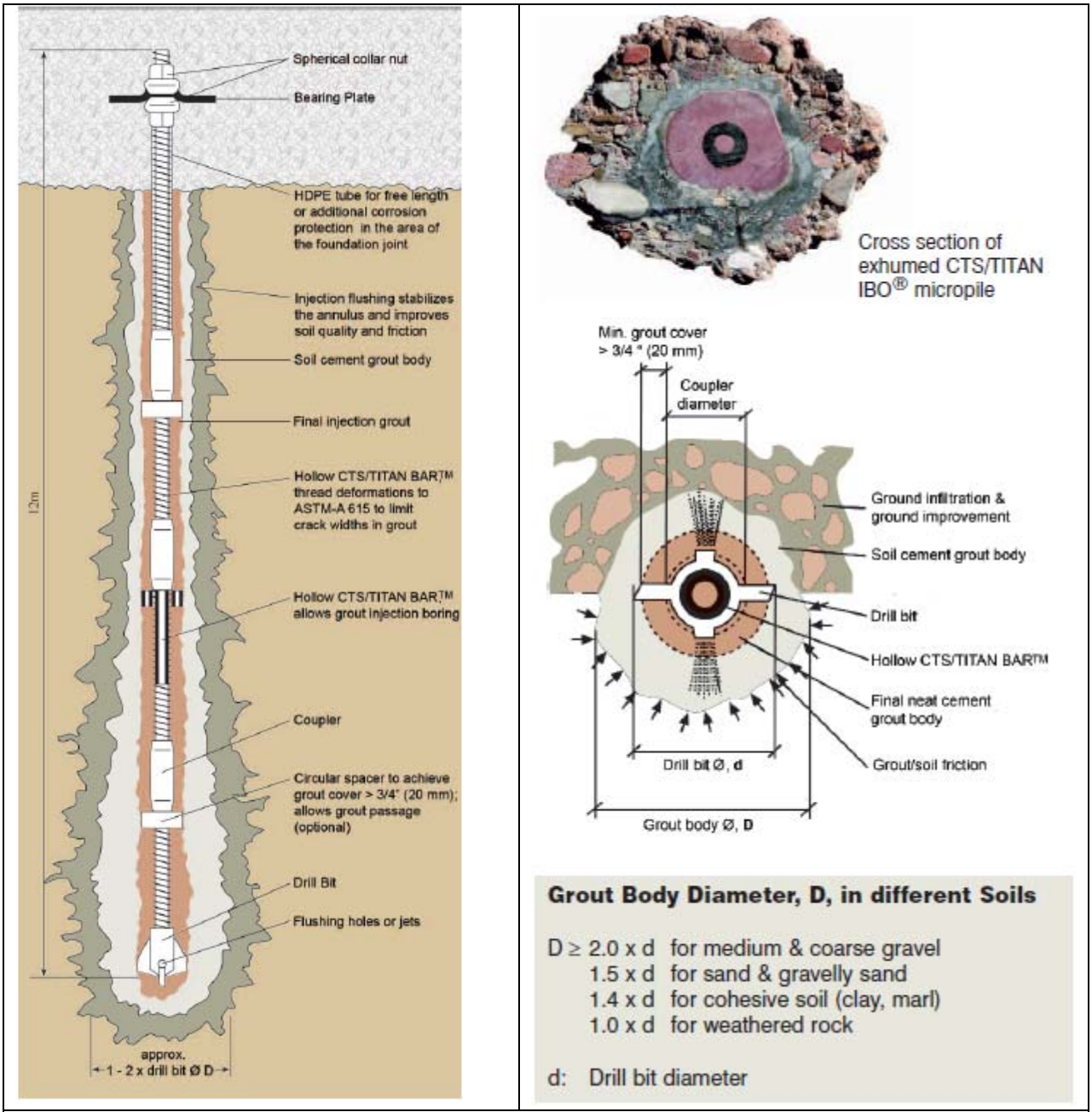



**Fig. 4. Details of Micropile – Elevation and Cross Section**

Key features of Titan Bars

Utilization of a steel of hollow bar geometry: A hollow bar behaves better than a solid rod of the same cross-sectional area with respect to flexural requirement and surface bond criteria.

The threads on hollow TITAN BAR™ are formed in similar way as the ribs on a reinforcement bar fabricated according to DIN 488 & ASTM-A 615.

Continuous threads guarantee the TITAN BAR™ can be cut or coupled anywhere along its length.

## GEOTECHNICAL AND STRUCTURAL DESIGN OF MICROPILE

The compressive and tensile load carrying capacity of the micro pile obtained from the latest Geotechnical investigation report are as follows:
Allowable compression capacity of 250 mm dia. Micro pile of length 18 m = 420 kN
Allowable tension capacity of 250 mm dia. Micro pile of length 18 m = 310 kN
Please refer figure 1 above.
As per brochure of TITAN bars, Structural load carrying capacity of
73/56Titan Bars are as follows:
Allowable tension capacity :1194 kN
The theory of lateral load behaviour of bored pile or drilled shaft can also be used to design a micropile for lateral loads and overturning moments. Typically, a micropile behaves as a long pile due to its relatively narrow cross-sectional dimension compared to its embedment depth. The stiffness factors are governed by the stiffness (EI value) of the pile and the compressibility of the soil. Permissible deflection at the pile head is considered as 5 mm. The soil parameters considered for pile design are presented in Fig. 5.

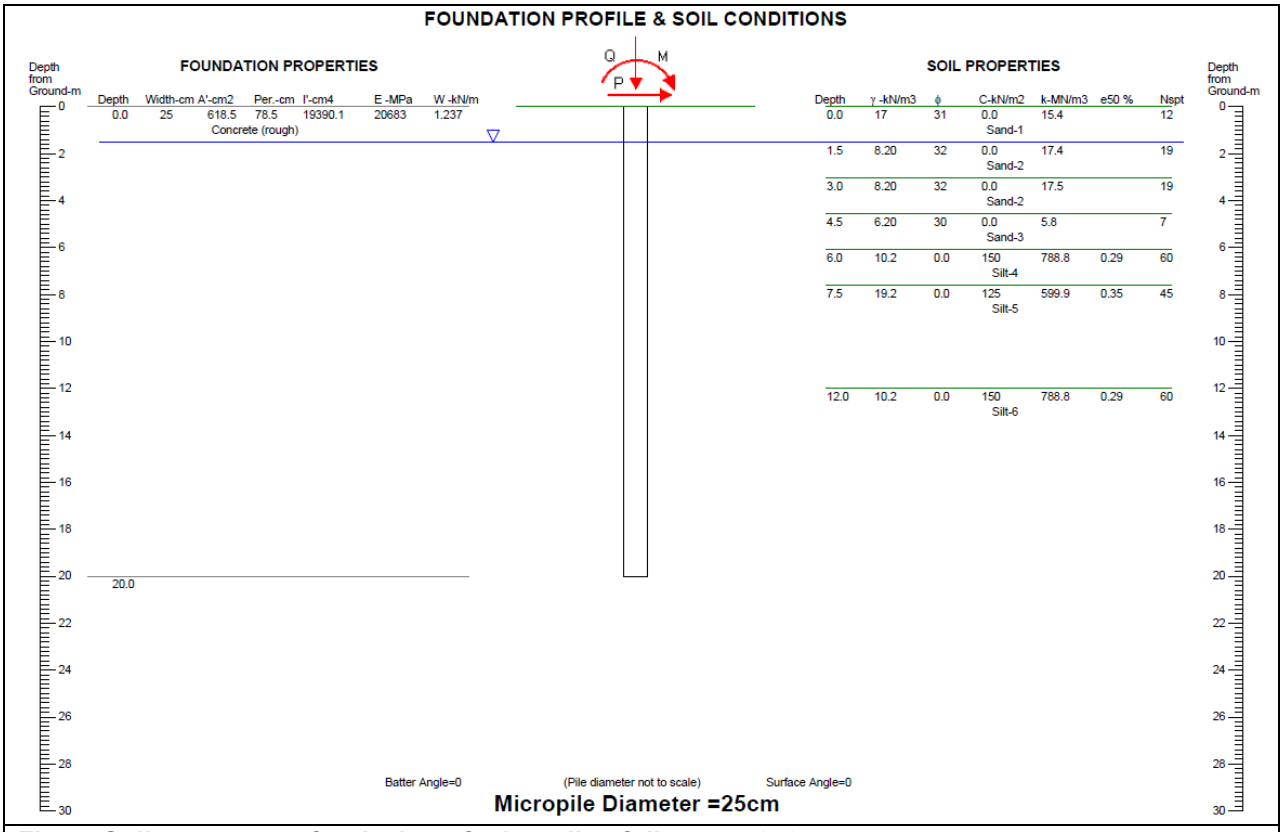



**Fig. 5. Soil parameters for design of micropile of diameter 250mm**

As per latest geotechnical report, Lateral load carrying capacity of 250 mm diameter micro pile with permissible deflection of 5 mm = 13 kN. The graph for lateral load capacity of micropile extracted from the geotechnical report[2] is shown in Fig. 6. Efficiency factor for the group pile has been considered as 1.

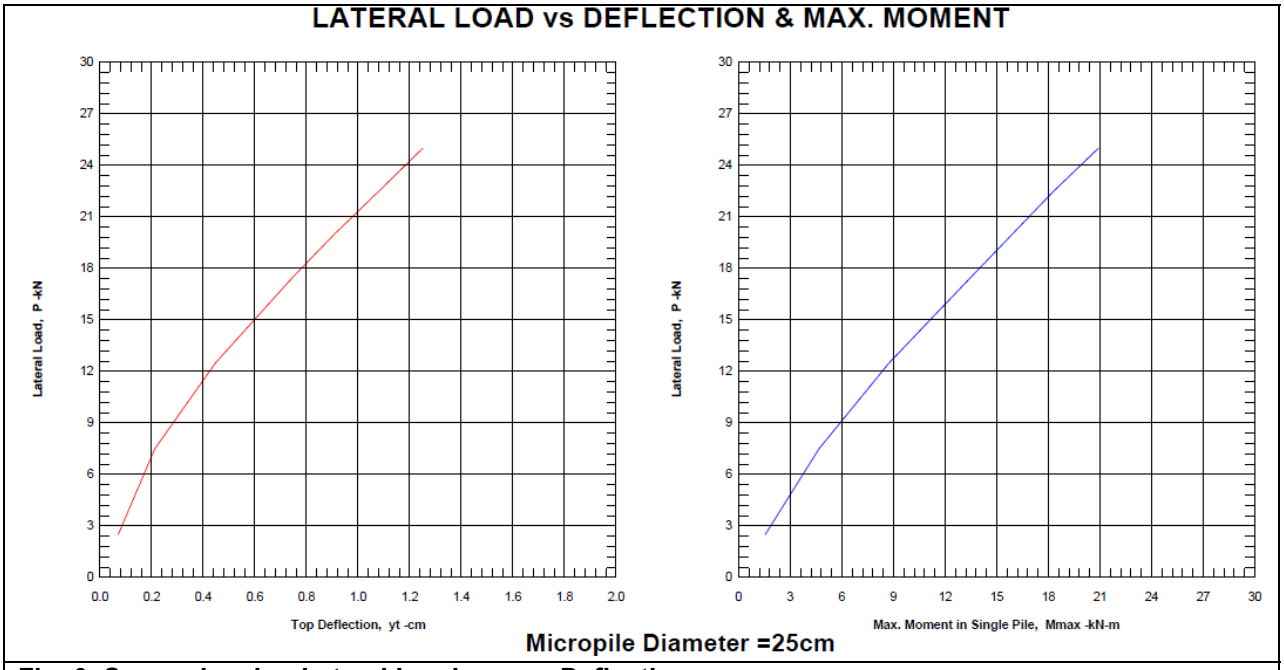



**Fig. 6. Curve showing Lateral Load versus Deflection**

## DESIGN OF REBAR DOWELS

The final component of the design was integrating the pilecap of the newly installed micropiles to the existing foundation. The pile cap has been designed for normal flexural requirements and dowel bars have been provided to ensure the proper bonding between exiting and new concrete. The rebar dowels have been checked against shear and tension and these dowels are chemically anchored as per the material HIT-RE 500-SD. The foundation detail and arrangement of dowel bars is shown in Fig. 7.

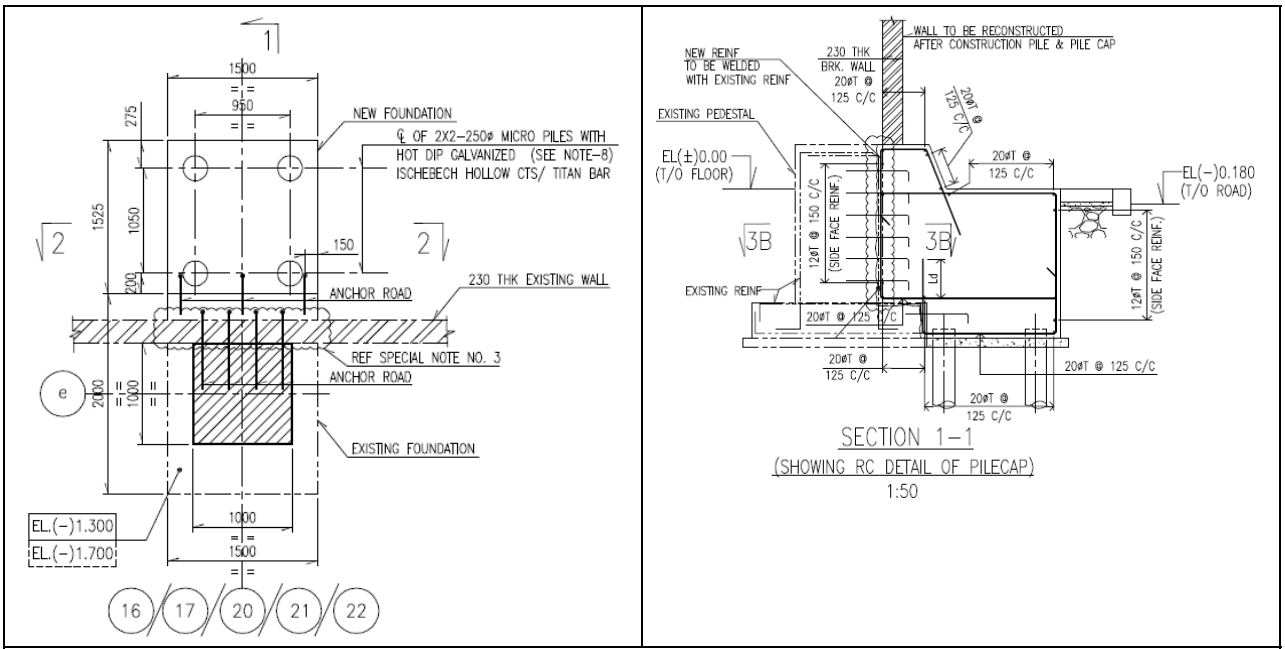



**Fig. 7. Arrangement of Dowel Bars to connect new Pilecap with Existing Foundation**

## CONSTRUCTION ENABLERS

To execute the designed solution with micropile support, the following recommendations were made:

i) Extensive site preparatory work is to be carried out before installation of micropiles. These include excavation/demolition of wall foundations and wall near the existing footings required to be modified.

ii) The existing EOT crane within the shop building shall not be operated during the construction period and shall be parked beyond the influence zone of the columns under modification.

iii) Design pile capacities are to be verified at site by conducting initial load tests on test piles. After verification of the design assumption and installation procedure through load test working piles are installed.

iv) After installation of micropiles and construction of new foundations/pile cap, the wall footings, wall which are demolished before shall be reconstructed to the operational condition.

## SUMMARY AND CONCLUSION

Use of micropiles for strengthening of existing foundations is a commonly used practice at present across many types of project applications. The use of hollow TITAN BAR in the micropile has eliminated the requirement of providing temporary casing and thus resulted in reduced cost of the micropile. The hollow bar is also used as grouting facilitator. The pile is designed to carry both compression and tension loads optimizing the number of piles required in a foundation. The pile is easy to install using lightweight rotary percussion drilling equipment and can be installed in confined spaces. The installation process generates almost no vibrations or noise and produces little spoil for discharge. The installation procedure can be completed in quick time. With all these advantages, the chosen micropiles could achieve the

required strengthening of the existing foundation with minimal disturbance to the operation activities in the process plant shop. For this project, the micropiles have been used for increasing the load carrying capacity of the foundation but the site constraints for execution required further innovative approach where the existing foundation is extended in plan and the extended portion is supported on micropiles after ensuring integral connection of the new foundation to the existing. The revamped foundation achieved the required increase in capacity to support the additional load from the new EOT crane but the work could be executed without affecting the plant operation.

## ACKNOWLEDGEMENTS

The project team would like to thank the customer for facilitating necessary approvals to Tata Consulting Engineers to work as "Consultant" in such a complex project and continuous support on site specific inputs throughout project execution.

## REFERENCES

1. Ischebek TITAN Micropile Brochure from Con-Tech Systems Ltd.
2. Geotechnical Investigation Report