# Optimization of bored pile foundations

T.A. Peiris
Professor,
Dean/Faculty of Engineering
SLIIT,

H. S. Thilakasiri
Professor,
Department of Civil Engineering,
University of Moratuwa.
Consultant, SLIIT

## What is the function of a foundation?

- Foundation is the key interface element between the superstructure and the ground which facilitates safe transfer of superstructure loads to the ground.
- Failure of the foundation of a structure either due to:
  - shear failure of the ground; or
  - excessive settlement
- makes the structure unusable.

Most famous foundation failure in the world

Leaning tower of Pisa

## Future challenges faced by foundation engineers

- Increased rate of urbanization has twofold effects on the types of structures constructed:
- people tend to built tall structures to maximize the use of the available land; and
- also they tend to build small to medium size structures on very weak grounds.

# Future challenges ……

- Both these scenarios pose different challenges to foundation engineers:
  - Tall large structures transmit extremely high loads to the ground and the foundations should be capable of resisting such heavy loads using economical pile foundations.
  - Small to medium size structures on weak grounds require economical foundation solutions to match the cost of the structure.
    - How to build my home on a plot of land having poor ground conditions with minimum cost?

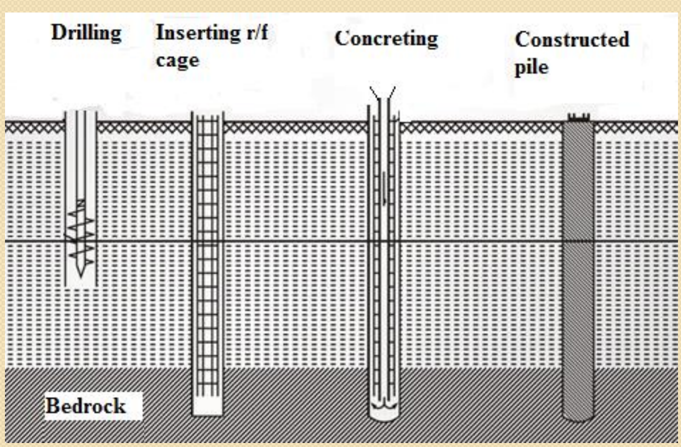
# This seminar focuses on optimization of high capacity bored piles

- Foundations for tall large structures:
  - How to install high capacity piles at an economical cost.
  - Should investigate:
    - Design;
    - Construction; and
    - Testing.

# Foundations for tall large structures

- Loads from tall structures are generally transmitted to the ground using deep foundations resting on hard layers and in Sri Lanka rock socketed bored and cast in-situ piles are used for this purpose.
- Then, what are the challenges of having a pile on bedrock?

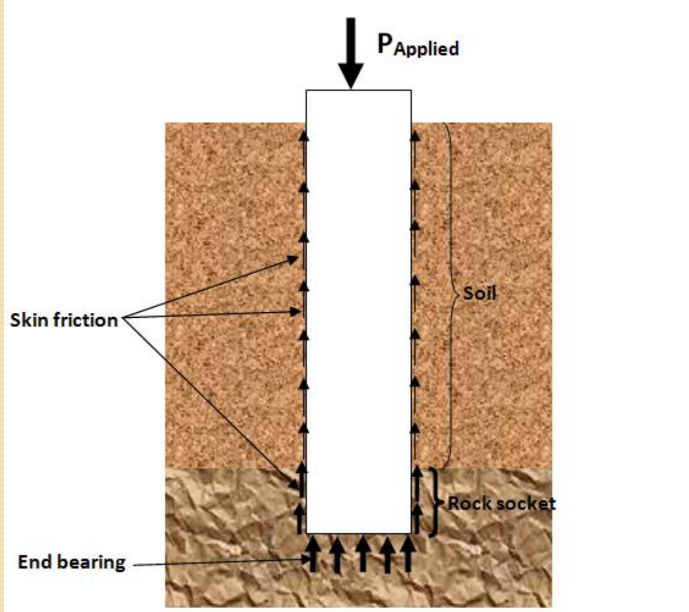
## Construction sequence of a bored and cast in-situ pile




Construction activities happen underground and cannot be observed

# Load carrying capacity of a pile

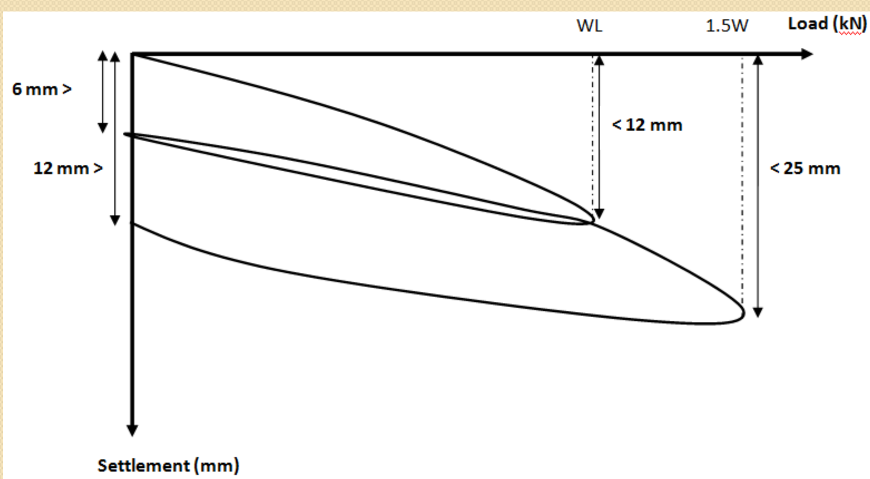
- The structural capacity of a 1.5m diameter pile constructed using 40 grade concrete is in the range of 17700 kN (≈ 1770 tons).
- The geotechnical capacity of a pile is coming from:
  - Skin friction along the pile shaft; and
  - End bearing at the toe of the pile
- Very often geotechnical capacity is the governing condition.




Geotechnical capacity of a rock socketed bored pile

# Performance specifications of a tested pile

- The observed load-settlement data are checked against the 'failure criteria' given below (ICTAD/DEV/16):
  - For loading cycle upto the working load,
  - Maximum allowable gross settlement is 12mm;
  - Maximum allowable net settlement is 6 mm.
  - For load cycle upto 1.5 x working load,
  - Maximum allowable gross settlement is 25 mm
  - Maximum allowable net settlement is 12 mm.




**Load – settlement curve showing performance specifications**

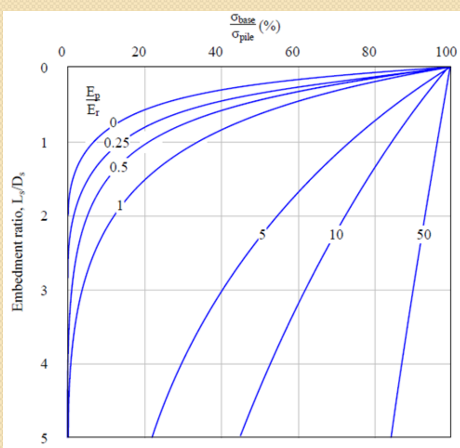
## Performance specifications of a pile

- Due to high degree of risk associated with failure of piles, the performance specifications are rather severe.
- A pile violating any of the above specified conditions during load testing is considered to have reached 'failure'.
- Therefore, design and construction of high capacity economical pile foundation to meet the above specifications is a though challenge.

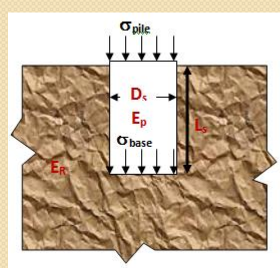
# OPTIMIZED DESIGN

# Skin friction in the bedrock

- Skin friction in the pile socket depends on:
  - Type and strength of the bedrock;
  - Weathering state of BR;
  - Depth of embedment in the bedrock;
  - Construction methodology:
    - Use of bentonite as drilling fluid;
    - Time delay between drilling and concreting; and
    - Cleaning of the pile bore before concreting.
- Depends very much on the state of the bedrock and the construction practice adopted.

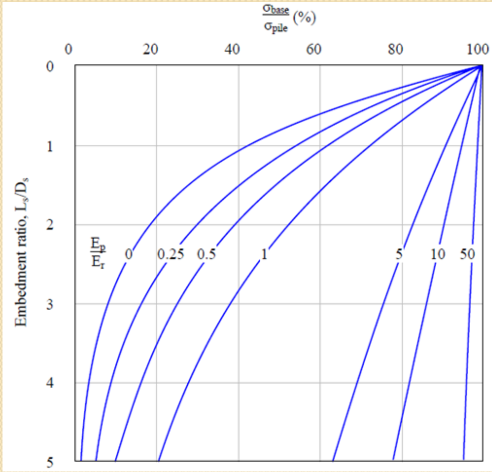



**Load Distribution in Rock Socketed Piles, ϕ' = 70° using Finite Element Analysis (Based on Kulhawy & Goodman, 1987)**

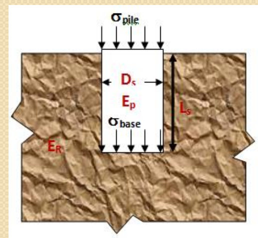



Skin friction depends on the stiffness ratio of the pile material and the bedrock.

When the stiffness of the bedrock is higher, having a rock socket longer than the pile diameter is not economical.




**Load Distribution in Rock Socketed Piles, ϕ' = 40° using Finite Element Analysis (Based on Kulhawy & Goodman, 1987)**



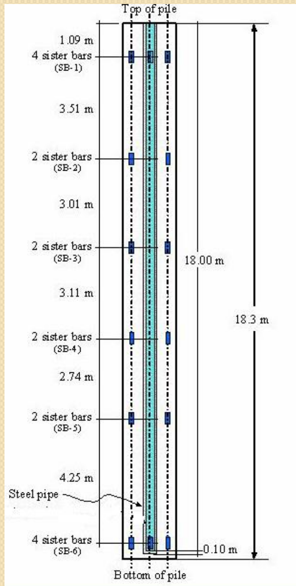

Skin friction depends on the stiffness ratio of the pile material and the bedrock.

When the stiffness of the bedrock is higher, having a rock socket longer than twice the pile diameter is not economical.

# Measured skin friction in the bedrock in Sri Lanka

- Skin friction in the rock can be measured using instrumented static load tests or Osterberg cell tests.
- Skin friction distribution obtained from dynamic load testing is vey approximate.
- Only one O-cell test is done in Sri Lanka and the results of the test are not made available yet.
- Only very few instrumented pile load tests are done upto now(less than five).

## Instrumented pile load test




Strain gauges attached to reinforcement

Loaded kentledge


Strain gauge readout unit

Hydraulic jacks

Load cells

Loading arrangement inside the kentledge

Load cell readout unit

# Bidirectional Cell Test in Sri Lanka

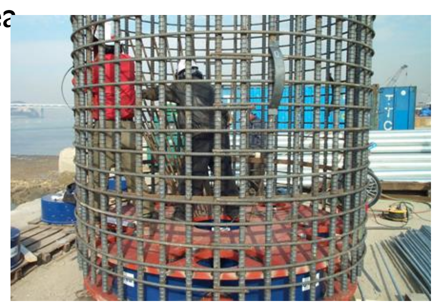
# Bidirectional Cell Test in Sri Lanka

# Bidirectional Cell Test in Sri Lanka

# Osterburg Cell Test

- The current world record for the highest test load of 279 MN (28000 tons) applied on a pile in Incheon, Korea using an O-cell by LoadTest inc., USA.

- O-cell used for the world record test



## Measured skin friction in the bedrock in Sri Lanka

- The unit skin friction in the socketed region of the Pile 1 and Pile 2 are 1600 kPa and 600 kPa respectively.
- The ultimate skin friction specified in ICTAD guideline, 200 kPa, is very conservative and should be revised.
- Research should continue in this area to develop a reliable skin friction estimation method.

# Use of Limit State Design in Foundation Engineering

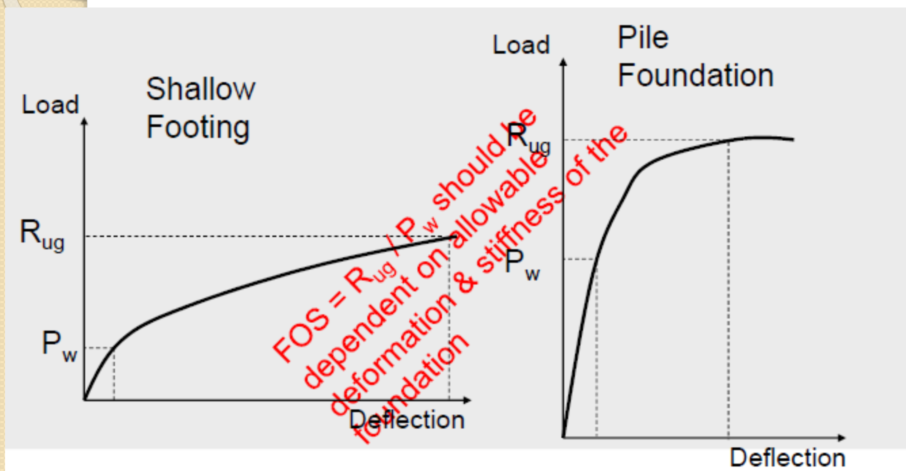
## Working Stress Design Method

Structural Engineers often ask the Geotechnical Engineers to provide:

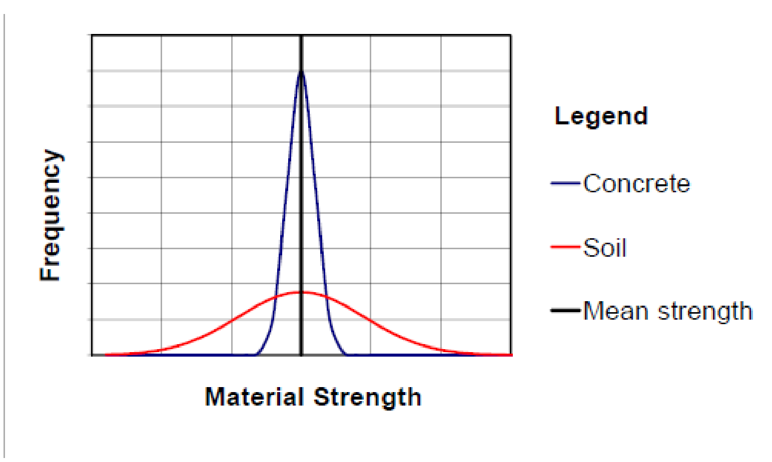
- "**allowable**" bearing pressure for footings
- "**allowable**" shaft friction and **"allowable"** end bearing pressure for piles

- **what does "Allowable" mean?**
  - Ultimate Capacity / Factor of Safety (FOS) ? or
  - FOS can be rewritten as:
    FOS = Ultimate Capacity / Working Load

# What is the appropriate FOS (2, 2.5, 3)?




# Choice of FOS is also affected by uncertainty in material properties



- **Working Stress Design Summary**

  "Allowable" load = Ultimate Capacity / FOS

  FOS should be dependent on:

  tolerable deformation

  foundation stiffness (linear or non-linear?)

  uncertainties in material properties/behaviour

- **The Ultimate Limit State Inequality Formula**

ULS Loads ≤ Design Geotech Resistance

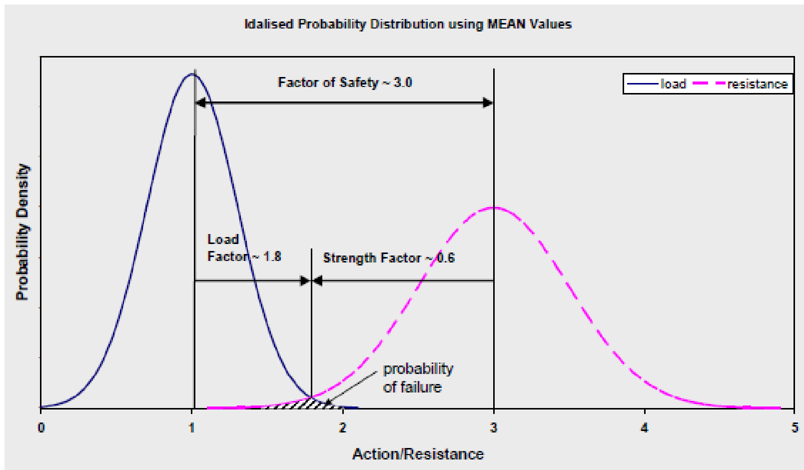
$$S^* = \Psi S \leq \Phi_g R_{ug} = R_{ug}^*$$

Load factor typically > 1

(Resistance) Strength factor typically < 1

Assumes probability approximates to a constant factor
Assumes loads and resistance are independent

# Idealised Probability Distribution




# Limit State Design Requirements

- **Part I - Strength Limit State**

$$R_{ug}^{*} \geq S^{*}$$

$$\Phi_g R_{ug} \geq \Psi S$$

Failure mechanism does not form due to deflections

- **Part 2 - Serviceability Limit State**

  Under the serviceability loading, the resulting deflection does not exceed the tolerable limit.

- **Deformation is a Key Design Issue?**

  In both methods, likely settlement should be assessed but rarely done in Working Stress Design

Why???

# Strength Reduction Factor, $\phi_g$

- AS2159-2009 Australian Standard: Piling – Design & Installation

$$\phi_g = \phi_{gb} + (\phi_{rf} - \phi_{gb})K \geq \phi_{gb}$$

  - $\phi_{gb}$ = Basic Geotechnical Strength Reduction factor
  - $\phi_{rf}$ = Intrinsic **test** factor

- $\phi_{gb}$ vary from 0.4 to 0.76
- $\phi_g$ vary from 0.4 to 0.85

- $\phi_{gb}$ = Basic Geotechnical Strength Reduction factor

Depends on:

- Site Specific:
  - Geological Complexity
  - Extent of Ground Investigation
  - Amount & Quality of Geotechnical data
- Design Specific
  - Experience: Similar foundation & Geological conditions
  - Method of assessment of geotechnical parameters
  - Design method adopted
  - Use of in-situ test data and installation data
- Installation

- Installation
  - Level of construction control
  - Level of performance monitoring –during and after construction

# Design Methodology

- **Ultimate Limit State Analysis**
  - ***Assess overall stability of the foundation***

    Analyse entire foundation system with factored-down resistances, and subjected to the ULS load combinations

    Design is OK if system does not collapse

  - ***Estimate maximum pile responses***

    Compute axial load, bending moment etc. for structural design purposes

# Design Methodology

- **Serviceability Limit State Analysis**
  - Assessment of foundation settlements
  - Assessment of pile vertical stiffness to be used in structural design purposes

| Sand-stone Rock Class | Unit | Working Stress Design Values | | Limit State Design Values | | |
|---|---|---|---|---|---|---|
| | | Allowable End Bearing Pressures (MPa) | Allowable Shaft Adhesion (kPa) | Ultimate End Bearing (MPa) | Ultimate Shaft Adhesion (kPa) | Elastic Modulus (MPa) |
| V | 4A | 1 | 50 | 3 | 150 | 100 |
| IV | 4B | 1.5 | 150 | 10 | 500 | 500 |
| III | 4C | 3.5 | 400 | 20 | 800 | 1000 |
| II | 4D | 8 | 800 | 80 | 2000 | 2000 |

Note: FOS ranges from 3 to 10

- Preliminary design using "allowable" design values resulted in long socket lengths (e.g.10m in Class II for 2.4m diameter piles)
- Detailed design using Limit State Design method, with rock socket settlement performance analysed using numerical methods:
  - Base alone would achieve adequate "Strength" for the ULS load
  - Design was governed by "serviceability" limits
  - Rock socket required to provide sufficient stiffness
  - Rock socket lengths reduced to 50% to 60% of the preliminary design lengths

# Overall Stability

For all load cases, foundation system does not collapse

| Wind Load Case | % of Non yielded piles |
|---|---|
| LC1 | 68% |
| LC2 | 68% |
| LC3 | 74% |
| LC4 | 77% |
| LC5 | 85% |
| LC6 | 86% |
| LC7 | 82% |
| LC8 | 82% |
| LC9 | 62% |
| LC10 | 62% |
| LC11 | 54% |
| LC12 | 56% |
| LC13 | 59% |
| LC14 | 60% |
| LC15 | 58% |
| LC16 | 58% |
| LC17 | 92% |
| LC18 | 96% |
| LC19 | 93% |
| LC20 | 93% |
| LC21 | 92% |
| LC22 | 96% |
| LC23 | 93% |
| LC24 | 92% |

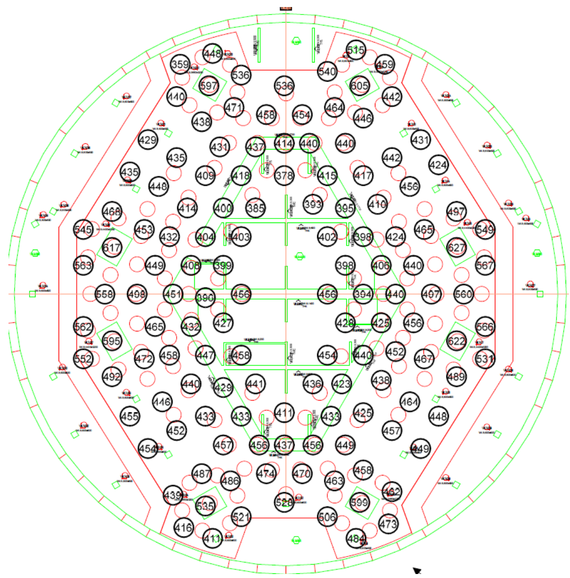
# Pile Vertical Stiffness

- Pile group vertical stiffness

  359MN/m to 627MN/m

- Note that single pile stiffness is 4400MN/m

Important effects of pile interaction



# EVOLUTION OF FOUNDATION DESIGN

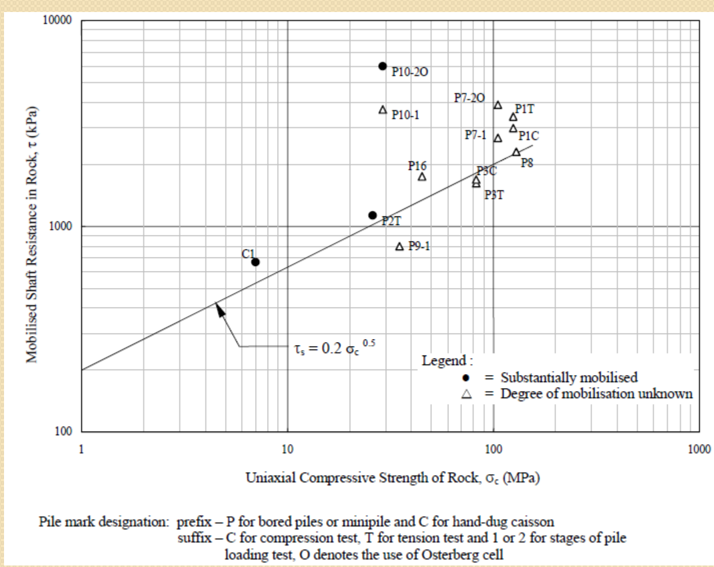
| Design Stage | Max. Total Axial load (factored) (MN) | Pile configuration | Settlement (mm)/Differential settlement |
|---|---|---|---|
| Concept Design | | 2.1m diameter 80m long | |
| Scheme 1 | 5,939 | 139 Nos. | **150** |
| Scheme 2 | 6,345 | 148 Nos. | **160** |
| Scheme 3 | 6,672 | 160 Nos. | **175** |
| | | | |
| Schematic Design | 6,873 | 136 Nos. 2.1m diameter, 55m long | **117**<br>**$\Delta L_x/L_x$= 1/540**<br>**$\Delta L_y/L_y$= 1/450** |
| | | | |
| Design Development (Final Design) | 6,113 | Total 136 Nos<br>36 Nos 2.4m diameter, 40m long & 100 Nos 2m diameter, 55m long | **87**<br>**$\Delta L_x/L_x$= 1/460**<br>**$\Delta L_y/L_y$= 1/600** |

- Euro code "Supervision of the construction process, including workmanship, and any monitoring of the performance of the structure during and after construction, shall be specified in the 'Geotechnical Design Report'
- Geotechnical investigation report should not be misunderstood as the 'Design report'.

- Section 3.4.1 of Euro code 7 states that The results of a geotechnical investigation shall be compiled in a Ground Investigation Report, which shall form a part of the Geotechnical Design Report.
- The assumptions, data, methods of calculation and results of the verification of safety and serviceability shall be recorded in the Geotechnical Design Report.

# Estimation of skin friction

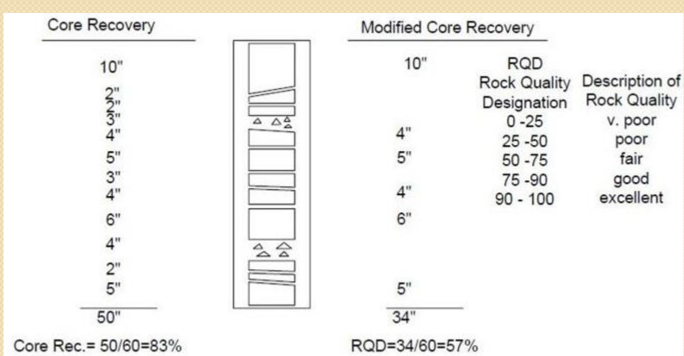
- High percentage of the skin friction is carried by the rock socket.
- ICTAD/DEV/16 specifies the ultimate (maximum) skin friction in the rock socket as 200 kPa.
- Measured skin friction of the socketed region of the piles into rock types similar to Sri Lanka, shows much higher ultimate skin friction.



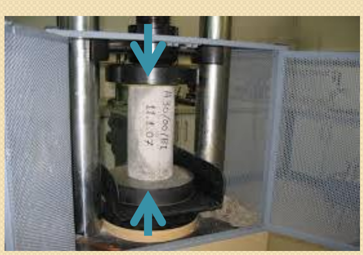

Measured skin friction from Hong Kong guidelines

# End bearing capacity

- In Sri Lanka, the end bearing capacity is estimated using:
  - Rock Quality Designation (RQD); and
  - Unconfined compression strength of intact rock specimen.
- No consideration is given to:
  - Core recovery;
  - State of the fracture in the bedrock;
  - Thickness and stiffness of In-fill material in the fractures.

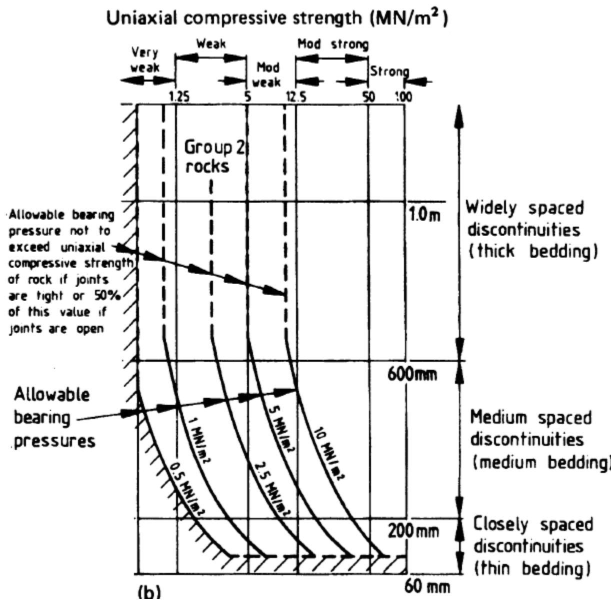



**RQD**



**Unconfined compression test**

## Chart used to estimate the allowable end bearing capacity of piles in Sri Lanka



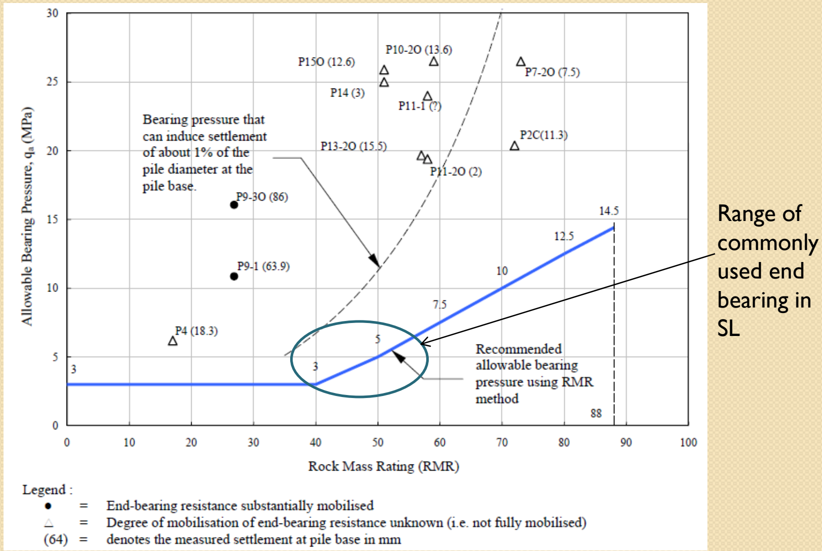

- The same chart is given in Euro code 07 under

**"A sample method for deriving presumed bearing resistance for spread foundations on rock"**

- Where did this chart come from?
- What condition was it developed for?
- Are the parameters considered sufficiently characterize the rock?
- With the higher concrete grades, can't we go for higher end bearing in good rock?

## End bearing capacity

- Characterization using RQD and unconfined compression strength is not sufficient.
- A more meaningful characterization of the rock, such as rock mass rating (RMR), considering:
  - Strength of Intact Rock;
  - Rock Quality Designation (RQD);
  - Spacing of Joints;
  - Conditions of Joints;
    - Separation rating;
    - Roughness rating;
    - Infilling rating; and
    - Weathering rating
  - Groundwater




Hong Kong Guideline using RMR to estimate the allowable end bearing capacity

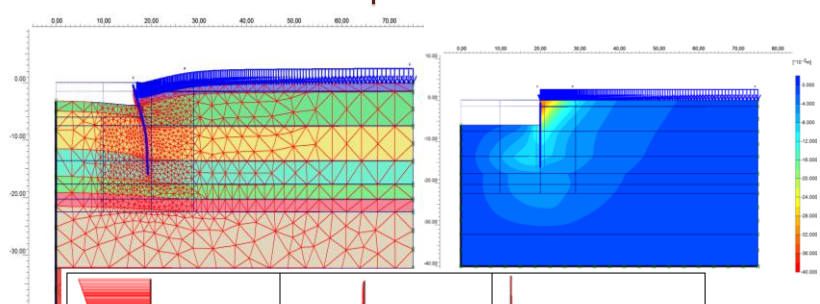
# End bearing capacity

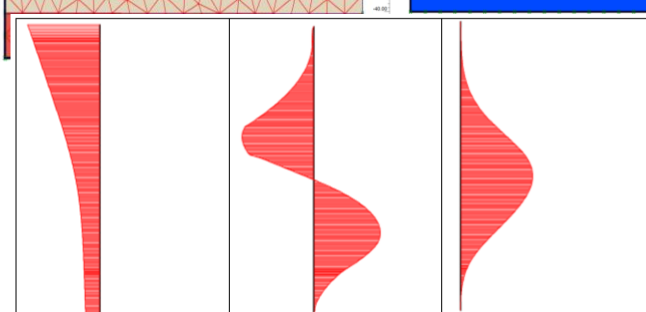
- A methodology similar to the HK guidelines should be developed to suit our bedrock conditions and construction practise.
- The designers may get the confidence to go for much higher allowable carrying capacities than the currently used 3 to 5 Mpa.

# Case study of the optimization of a secant pile wall

- Deeper the wall better performance?
- Three cases analysed for different wall depths

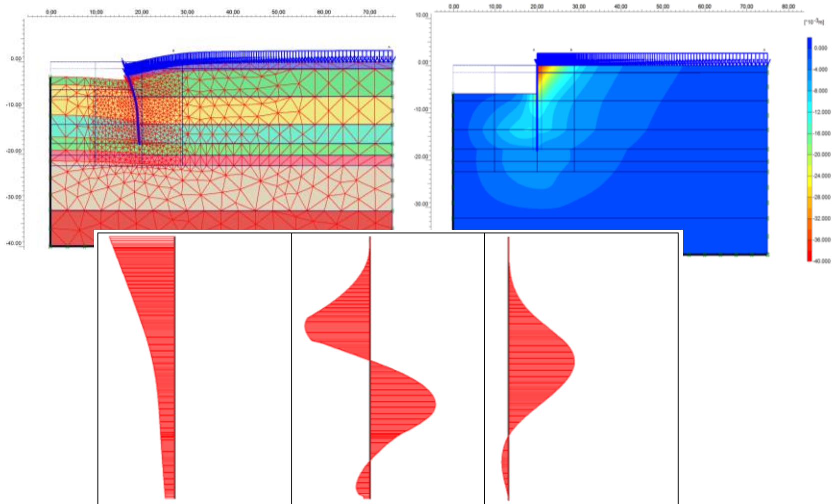
## Wall bottom -16m depth below EGL





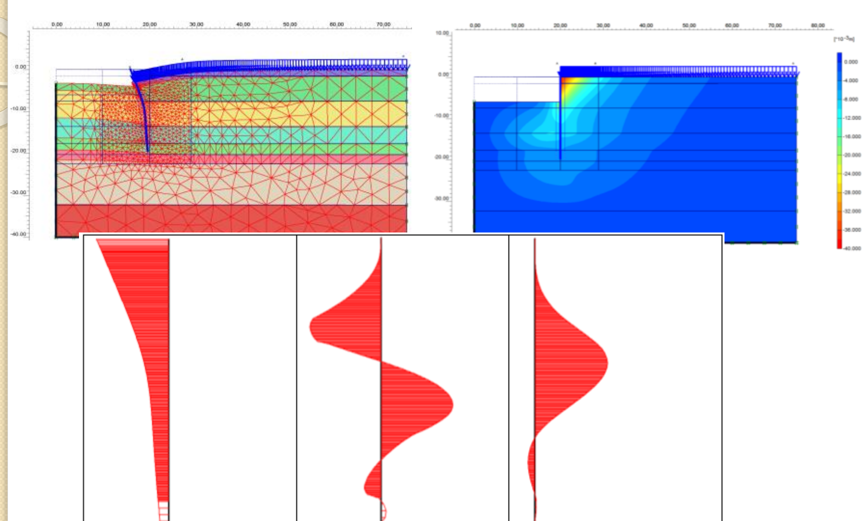
| Horizontal displacement 39 mm | Shear force diagram , Maximum shear force is 149 kN/m | Bending moment diagram, Maximum bending moment is 533 kNm/m |
|---|---|---|

## Wall bottom -18m depth below EGL



| | | |
|---|---|---|
| Horizontal displacement 40 mm | Shear force diagram , Maximum shear force is 150 kN/m | Bending moment diagram, Maximum bending moment is 528 kNm/m |

## Wall bottom about -20m depth below EGL



| | | |
|---|---|---|
| Horizontal displacement 40 mm | Shear force diagram , Maximum shear force is 151 kN/m | Bending moment diagram, Maximum bending moment is 532 kNm/m |

- No apparent advantage of going for deeper wall
- Do manual calculations to fine effective depth.
- Do detail analysis to find the optimum depth
- Consider effects of soil saturation

# CHALLENGES FACED DURING CONSTRUCTION

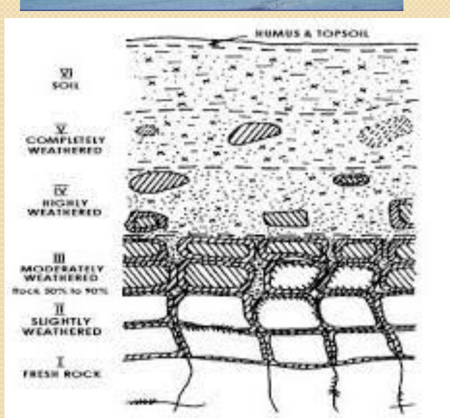
- Euro code “Supervision of the construction process, including workmanship, and any monitoring of the performance of the structure during and after construction, shall be specified in the ‘Geotechnical Design Report’
- Geotechnical investigation report should not be misunderstood as the ‘Design report’.

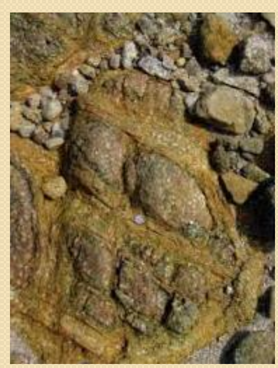
## Challenges faced

- Termination of the piles on the appropriate rock layer (Termination criteria)
- Cleaning of the pile bottom before concerting.
- Concreting piles without structural defects in its shaft.

## Termination criteria

- Piles should be terminated on the bedrock assumed in the design.
- Identification of the quality of bedrock during drilling for the piles is extremely difficult as only small rock pieces are coming out during drilling.
- Therefore, quality of the bedrock obtained during site investigation stage and indirect measurements made during drilling for piles should be used to assess the quality of the bedrock.



**Typical weathering profiles of the bedrock**

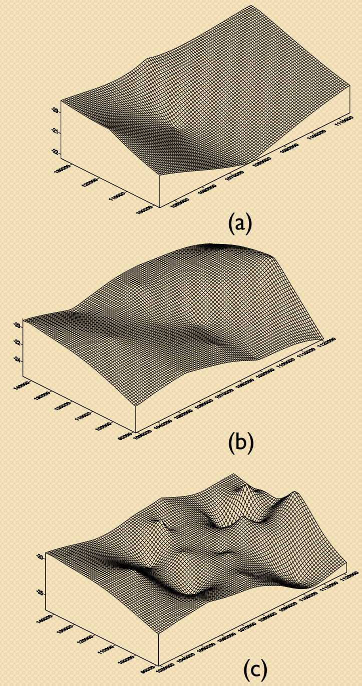
## Termination criteria

- Weathered rock layer consists of:
  - completely decomposed rock;
  - highly weathered rock;
  - partially weathered rock, and
  - fractured rock layers;
- Large variations in the thickness of these sub-layers and the overall thickness of the weathered rock layer within a very short distance;
- During drilling for the piles the residue coming out of the borehole doesn't give any indication of the quality of the bedrock;
- As such identification of the sound bedrock during drilling for the pile is a difficult task.

## Termination criteria

- Site investigation is needed not only for design of pile foundation but also for construction quality control as well.
- Site investigation programme should be modified if thick weathered rock layers with rapid spatial variations are found.
- Rock coring should be carried out at a reasonable number of locations within the site to establish the bedrock profile.

***3 D model of the rock profile using (a) 6 boreholes; (b) 16 bore holes; and (c) 16 bore holes & additional data at pile locations***




## Interpretation of the test results

- Very often only settlement performance limits of the pile are checked.
- If the settlement performances are satisfied well within the limits, optimisation should be attempted.
- Shape of the load-settlement curve carries some hidden information regarding the performance of the pile.

## Termination criteria

- Drilling rate is another method used to identify the quality of the bedrock
- Some organizations terminate the piles when a specific drilling rate is achieved (for e.g. 800 mm/hr)
- Drilling rate depends also on
  - the pressure applied on the rock;
  - the torque mobilize by the machine on the Kelly bar; and
  - Abrasive resistance of the rock
- The condition of the drill bits is also an important parameter

## Termination criteria

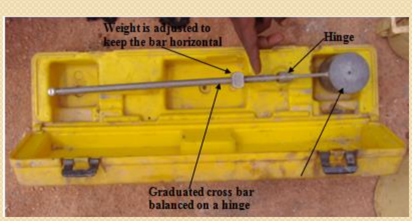
- Both mapping of the bedrock and the drilling rate should be considered in termination of the piles
- If the mapping technique is used alone, it may not identify sudden variations in the bedrock profile.
- If only the rate of drilling method is used to terminate the piles, the piles may be terminated on isolated boulders above the bedrock level.
- The pile termination criterion, for a site with varying bedrock profile preferably should be done after installation of a test pile near a location of a borehole used for field investigation.

## Cleaning the pile bottom

- Sometimes when large rock pieces are covering the bottom of the pile bore, cleaning bucket may be used to clean the bottom.

- The debris that is present may consist of:
    - Deposition of granular material from the drilling operation through rock and soil;
    - Dislodging and falling small block-like portions of soil and rock from the unlined wall of the borehole; and
    - Ground water percolated through the silty and sandy layers.

## Cleaning the pile bottom




Cleaning by circulation of drilling mud should be carried out after adding fresh bentonite or cleaning of drill mud after using a de-sander

# Other types of drilling fluids

- Use of modified bentonite and polymer.

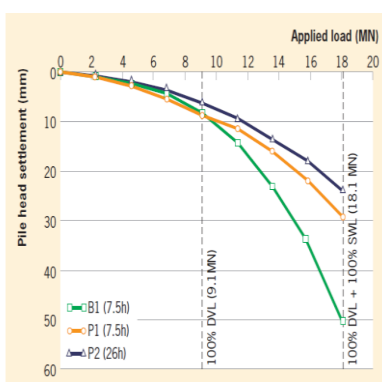
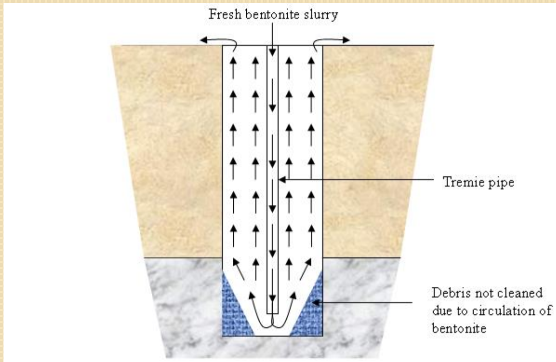


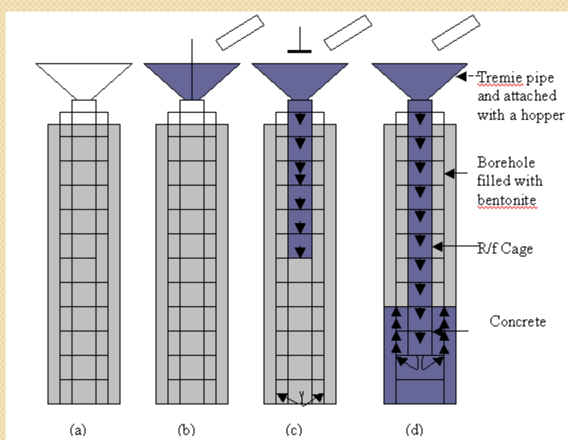

*Figure 6: Load-settlement curves for the test piles*




**Circulation of bentonite slurry through the tremie pipe**

This method of cleaning may leave certain amount of debris at the bottom of the pile as shown in the above Figure.
The filter cake formed may be dislodged and removed from circulation to certain extent

## Concreting of Cast in-situ Bored Piles




Concreting a borehole using tremie pipe:
(a) Tremie is assembled in the borehole;
(b) (b) A plug is placed at the bottom of the hopper and filled with concrete;
(c) (c) Plug is removed and concrete moving through the tremie replacing bentonite slurry in the tremie; and (
(d) d) Concreting continued with bottom of tremie pipe immersed in fresh concrete

# Concreting of Cast in-situ Bored Piles

- Concreting should be done in a continuous operation without any interruptions.
- During the time period, from initial charging of the pile to end of concreting, the bottom of the tremie pipe should be always kept below the top surface of the concrete inside the borehole.
- Concrete should be:
  - Low degree of segregation;
  - Self compacting under its own weight;
  - High workability and fluidity throughout the entire placement operation;
  - Required strength; and
  - Resistance against aggressive environment surrounding the pil

**A comprehensive survey was carried out by Jayasekara et al. (2003) to investigate the quality control measures used by the piling contractors in Sri Lanka.**

| Item | Methodology adopted | % of the sites |
|---|---|---|
| Setting out | Using Total Station | 33 |
| | Using Theodolite and Tape | 44 |
| | EDM | 11 |
| | Tape only | 12 |
| Method of drilling | Winch and chisel | 36 |
| | Auger | 64 |
| Measurement of the inclination of the pile | Measured during boring | 22 |
| Use of casing | Permanent casing | 0 |
| | Casing upto 2m (temporary) | 30 |
| | Casing for full length (temporary) | 40 |
| Drilling fluid (Bentonite) | Used | 70 |
| | Not used | 30 |
| Checking the density of bentonite before concreting | Tests carried out | 44 |
| | Estimated by experience | 56 |

**A comprehensive survey was carried out by Jayasekara et al. (2003) conti…..**

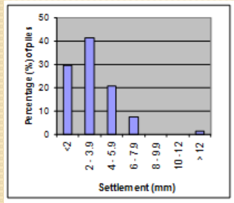
| | | |
|---|---|---|
| Cleaning of the borehole before concreting | Using cleaning bucket only | 33 |
| | Bentonite circulation only | 23 |
| | Cleaning bucket and bentonite circulation | 44 |
| | Cleaning with water | 0 |
| Workability measurement of fresh concrete | Slump measured for each truck load | 89 |
| | Other method | 11 |
| No of test cubes | 3 cubes per pile | 66 |
| | 6 cubes or more | 33 |
| Measurement of pile length | Using tape | 9 |
| | Using a tape with a plumb bob | 45 |
| | DMC rod | 18 |
| | Tremie pipe length | 19 |
| | Reinforcement cage | 9 |
| Determination of the rock level | Rock sample | 72 |
| | Rock contours | 9 |
| | Penetration rate | 19 |

**A comprehensive survey was carried out by Jayasekara et al. (2003) conti…..**

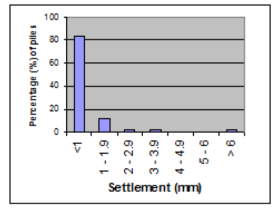
| | | |
|---|---|---|
| Cover for reinforcement cage | Improper cover blocks | 55 |
| Spacing between cover blocks | Less than 2m | 33 |
| | Greater than 2m | 67 |
| Spacing between hoop reinforcement | 150mm or less | 66 |
| | 200 mm | 22 |
| | > 200mm | 12 |
| Spacing between the longitudinal reinforcement | 200 mm or less | 11 |
| | 300 mm or more | 89 |

* only at 22% of the sites, pile verticality was checked;
* Temporary casings were not used in about 30% of sites;
* Only in about 44% of the sites, the density of bentonite slurry is checked before concreting;
* About 72% of the piling contractors used the rock samples obtained during the drilling process to determine the bedrock level

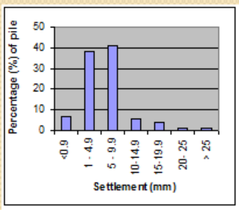
**A survey was carried out to collect the results of the static pile load tests done on bored and cast in-situ piles. The results are analyzed to investigate the violation of the above specification by the piles tested.**



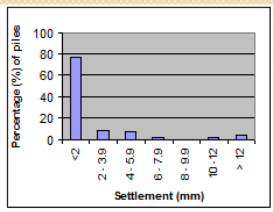
Percentage of piles verses settlement at the working load



Percentage of piles verses the settlement after unloading from the working load.



Percentage of piles verses settlement at 1.5 times the working load



Percentage of piles verses the settlement after unloading from the 1.5 times the working load

# Survey results

- 5% of the piles are violating one or more of the settlement specifications.
- A detailed analysis by Thilakasiri et al. (2005) established that the reason of 'failure' of the piles is the 'weak toe' condition of the piles.
- This finding agrees very well with the lack of quality control measures adopted during termination of the pile and cleaning the pile bore, reported by Jayasekara et al. (2003).

# OPTIMIZATION BASED ON TESTING OF PILES

## Testing of piles

- Testing of piles is done mainly due to the following reasons:
  - To evaluate the performance of pile at the preliminary or later stages in terms of settlement and carrying capacity;
  - To assess the structural integrity of the pile; and
  - To obtain additional information required for the pile design such as: total skin frictional capacity; distribution of the skin friction along the pile shaft and mobilized end bearing.

# Testing of piles

- Testing of the piles could be carried out at mainly two different stages:
  - Testing of test piles prior to the construction of working piles; and
  - Testing of piles during construction stage of the working piles.
- Piles tested during the construction stage could be further subdivided into
  - (a) Piles for preliminary testing and
  - (b) Routine proof testing of piles.

# Load testing of piles

- Depending on the type of load applied:
  - Compression load testing
  - Tension load testing; and
  - Lateral load testing.
- Compression load testing:
  - Static Load Testing;
  - Dynamic load testing; and
  - Statnamic load testing.
- Static load testing:
  - Conventional static load testing;
  - Static load testing of an instrumented pile; and
  - Static load testing using Osterburg cell.

# Load testing of piles

- Number of piles tested depends on many factors but generally about 1% to 4% piles are load tested.
- Selection of the type of load testing method depends also on many factors
  - Among them information expected to be obtained from the testing.
- Consider:
  - Information obtainable; and
  - The accuracy level of the information.

# STATIC LOAD TESTING OF PILES

- In this test, load is applied at the top of the pile with or without incrementing the pile shaft.
- Instrumented pile load tests should be done to obtain more information regarding the pile.
  - Accurate mobilized skin friction distribution and end bearing;
  - Load deformation behavior of the pile toe and the state of the mobilized skin friction; and
  - Skin friction mobilized in the rock socket.
- Even though highly accurate, very time consuming and expensive.

Reaction systems for static load tests

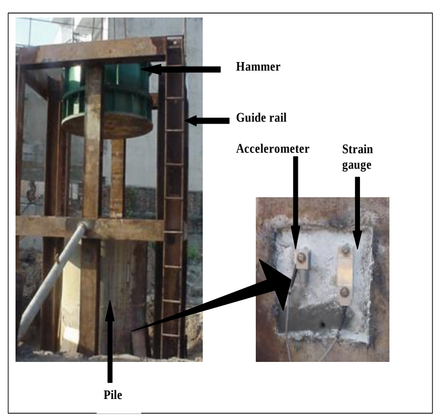

Anchor cable system

Loaded platform

Anchor pile system

# Dynamic load testing of piles

- The pile is loaded by application of a dynamic impact through a hammer blow.
- Pairs of strain gauges and accelerometers should be attached diametrically opposite sides of the pile.
- More than a single pair may be needed for large diameter piles.

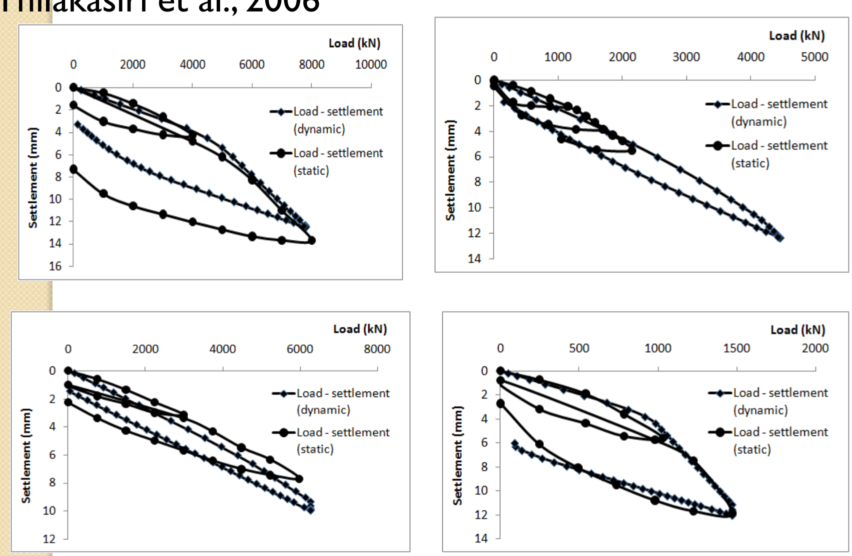


# Dynamic load test

- Measured strain and acceleration at the top of the pile may be used in the field to obtain:
  - Carrying capacity;
  - Integrity of the pile;
- A more rigorous analysis, referred as CAPWAP analysis, may be done in the office to obtain the static response of the pile.
  - Static load – settlement curve;
  - Skin friction distribution along the pile shaft;
  - Integrity of the pile.

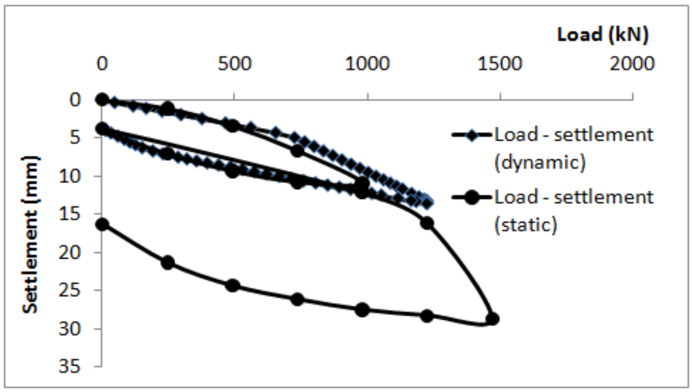
## Comparison of dynamic and static load test results

- Thilakasiri et al., 2006



## Comparison of dynamic and static load test results

Thilakasiri et al., 2006
Capacity is not fully mobilized in the dynamic load test




# Dynamic load test

- During late 90's and early 2000, dynamic load testing was done by the foreign engineers.
- Sri Lankan engineers were educated on dynamic testing through seminars, CPD courses and research presentations.
- Now almost all dynamic load testings in the country are done by Sri Lankan firms and Sri Lankan engineers.

## Advantages of dynamic load tests

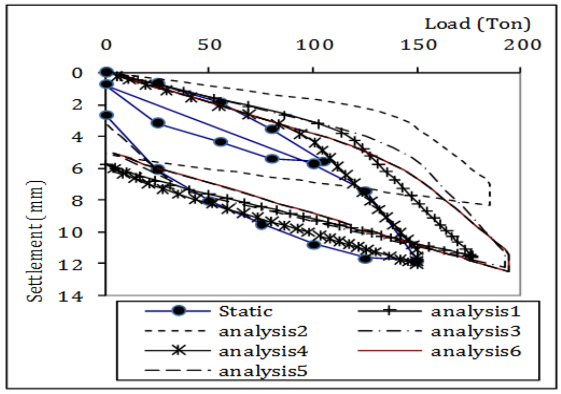
- Low cost in comparison with the static load tests;
- Very quick compared to static load test, few piles can be tested within a day;
  - Testing of more number of piles compared to static load testing
- Additional information such as: integrity; approximate distribution of skin friction and end bearing etc. regarding the pile can be obtained.
- Can be performed even in congested sites.

## Disadvantages of the dynamic load test

- Indirect interpretation methods involving wave propagation theories.
- The accuracy level of the results depend on the data interpreter.
  - Automatic optimum solution given but any software has its own limitations.
  - Interpretation of the geotechnical engineer.
- Even though most of the parameters given are verified against the direct field measurements, skin friction distribution, and the estimated mobilized end bearing are not.
- Very well established throughout the world but interpret within the accuracy levels.

# CAPWAP Signal matching

Thilakasiri et al., 2006
Showed that best match is NOT when the match quality is the minimum
Consider:
- Dynamic soil parameters;
- Soil resistance distribution; and
- Blow count




| Analysis No | Match quality | Blow count observed | Estimated blow count. | Capacity by CAPWAP (kN) |
|---|---|---|---|---|
| I | 2.55 | 667 | 591 | 1731 |
| II | 2.94 | 667 | 584 | 1812 |
| III | 4.53 | 667 | 652 | 1884 |
| IV | 4.76 | 667 | 621 | 1472 |
| V | 3.09 | 667 | 563 | 1472 |
| VI | 3.71 | 667 | 569 | 1910 |

# Estimation of the characteristic compressive strength of piles from dynamic load test

(3)P The impact energy shall be high enough to allow for an appropriate interpretation of the pile capacity at a correspondingly high enough strain level.

(4)P The design value of the compressive resistance of the pile, $R_{c;d}$ shall be derived from:

$$R_{c;d} = R_{c;k}/\gamma_t \tag{7.10}$$

with

$$R_{c;k} = \text{Min}\left\{\frac{(R_{c;m})_{mean}}{\xi_5}; \frac{(R_{c;m})_{min}}{\xi_6}\right\} \tag{7.11}$$

where $\xi_5$ and $\xi_6$ are correlation factors related to the number of piles tested, $n$, and are applied to the mean $(R_{c;m})_{mean}$ and the lowest $(R_{c;m})_{min}$ value of $R_{c;m}$ respectively.

NOTE The values of the partial factor and correlation factors may be set by the National annex. The recommended values are given in Table A.11.

Table A.11 - Correlation factors $\xi$ to derive characteristic values from dynamic impact tests[a, b, c, d, e,] ($n$ - number of tested piles)

| $\xi$ for $n$ = | ≥ 2 | ≥ 5 | ≥ 10 | ≥ 15 | ≥ 20 |
|---|---|---|---|---|---|
| $\xi_5$ | 1,60 | 1,50 | 1,45 | 1,42 | 1,40 |
| $\xi_6$ | 1,50 | 1,35 | 1,30 | 1,25 | 1,25 |

[a] The $\xi$-values in the table are valid for dynamic impact tests.

[b] The $\xi$-values may be multiplied with a model factor of 0,85 when using dynamic impact tests with signal matching.

[c] The $\xi$- values should be multiplied with a model factor of 1,10 when using a pile driving formula with measurement of the quasi-elastic pile head displacement during the impact.

[d] The $\xi$-values shall be multiplied with a model factor of 1,20 when using a pile driving formula without measurement of the quasi-elastic pile head displacement during the impact.

[e] If different piles exist in the foundation, groups of similar piles should be considered separately when selecting the number n of test piles.

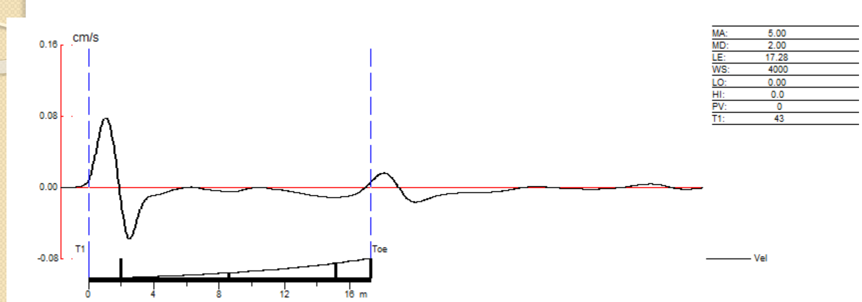
# Integrity testing of piles

- Direct methods;
  - Excavation or extraction of the pile; and
  - Drilling through the pile.
- Indirect methods (NDT).
  - Surface reflection methods (small strain integrity test (PIT);
    - Pulse Echo (or sonic echo) Method (PEM); and
    - Transient Dynamic Response (or impulse response) method (TDR).
    - Conventional high strain load testing of piles
  - Direct transmission methods:
    - Crosshole sonic Logging (CSL);
    - Single-hole Sonic Logging (SSL)

# Integrity testing of piles

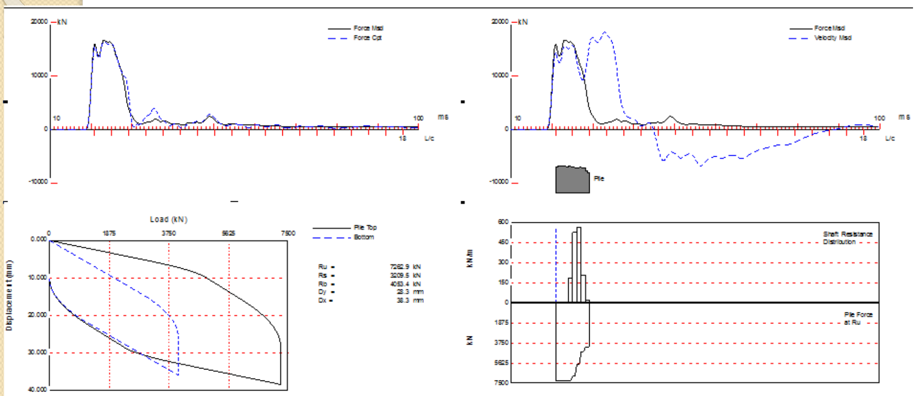
- Identify the limitations of the testing method.
  - PIT is a preliminary test and has certain limitations
  - PIT does not give the carrying capacity of piles.
  - PIT Might give some indication about the load carrying capacity but not conclusive

# Soft toe was identified in this record

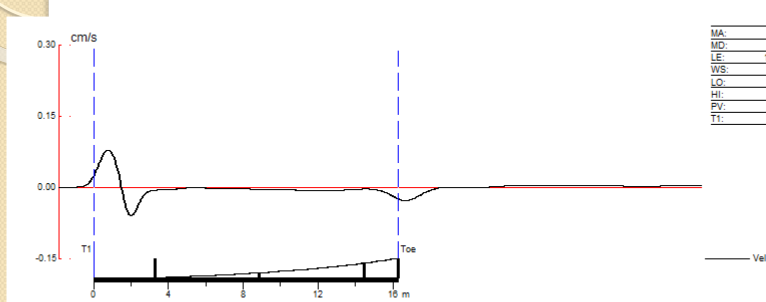



- 1.5m thick sand fill under the pile toe
- The working load of the pile is 11300 kN but PDA testing gave only 7263 kN capacity

# CAPWAP results



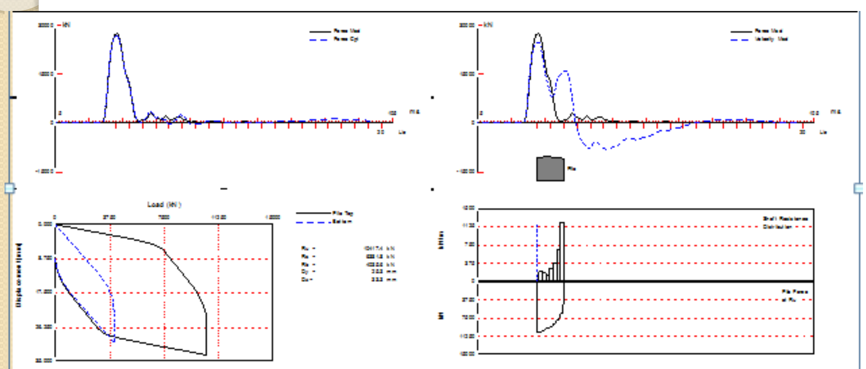

# Soft toe was NOT identified in this record




| | |
|---|---|
| MA: | 5.00 |
| MD: | 3.25 |
| LE: | 16.27 |
| WS: | 3800 |
| LO: | 0.00 |
| HI: | 0.0 |
| PV: | 0 |
| T1: | 50 |

- 200mm thick sand fill under the pile toe
- The working load of the pile is 11300 kN but PDA testing gave only 10417 kN capacity

# CAPWAP results



# Integrity tests

- Cross hole sonic logging (CSL) is more accurate
  - Can identify the magnitude and location of the defect;
  - Depth limitation of PIT not with CSL
  - Soft toe can be identified
  - Better to use CSL on high capacity piles in foundations without redundancies

## Conclusions

- To optimize pile foundations:
  - Use of the Limit state approach
  - Detail design with reliable design approach
  - Use of instrumented test piles
  - Construction methodology with appropriate quality control methods

## Conclusions

- Strengthened quality assurance programme
  - Limits of the testing methods
  - Selection of appropriate tests
- Appropriate changes to the guidelines and the national annexure
- Overall change in the way of thinking

# *Thank you*