SRI LANKAN
GEOTECHNICAL SOCIETY

ANNUAL CONFERENCE

10th September 2009

Wimalasurendra Auditorium
Institution of Engineers, Sri Lanka

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Principal Sponsor – Engineering Laboratory Services (Pvt.) Ltd.

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Welcome Message from the President, Sri Lankan Geotechnical Society

The Sri Lankan Geotechnical Society (SLGS), formed in 1986, provides a forum for promoting cooperation among the engineers, geologists and other scientists in Sri Lanka for the advancement of knowledge in geotechnical engineering. Being a Member Society of the prestigious International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE), we have been successful in getting cooperation also of the international community of geotechnical professionals in our efforts in sharing and disseminating geotechnical knowledge by way of conducting conferences, seminars, workshops, geotechnical forums and publishing journals and newsletters.

However, we are yet far away from understanding ourselves and convincing others on the important role of a geotechnical professional in development. Almost all structures and facilities are founded ultimately on soil or rock. Be it a house or a skyscraper, a culvert or a suspension bridge, a road embankment or an earth dam, a ground reservoir or a dockyard, an excavation or a slide prone hill slope, a quarry or a landfill, it is not only the safety, stability and economy that matters but also the aspects of health, environment and sustainability which must be given primary attention. Knowing the soil and rock we build upon, and understanding their characteristics and behaviors, we geotechnical professional should be playing a leading guiding role in planning, design and construction. For this we should constantly familiarize ourselves with the updated knowledge, techniques and practices in geotechnical and related engineering and scientific fields.

This Annual Geotechnical Conference is yet another activity that exposes both international and local experiences highlighting the geotechnical professional’s role in development. Associate Professor Ken Kawamoto of Saitama University, Japan will enlighten us about geo-environmental problems associated with landfills and gas and liquid transport within soil mass. Eng. Dr. W.A Karunawardena of National Building Research Organisation and Eng. Dr. H.S. Thilakasiri of the University of Moratuwa will share their experiences respectively on Geotechnical practices in Highway Construction and Role of the Geotechnical Engineer in the Piling Industry.

On behalf of the SLGS I sincerely welcome the three resource personnel and all the participants of this conference. I am hopeful that this forum will be a great opportunity for your interaction in sharing experiences and enhancing your knowledge.

Eng. Kirthi Sri Senanayake
President
Sri Lankan Geotechnical Society.

SLGS ANNUAL CONFERENCE

10th SEPTEMBER 2009 AT WIMALASURENDRA AUDITORIUM, IESL

PROGRAMME

1:00 - 1:30 Registration

1:30 - 2:30 Presentation 1:

“Environmental Risk Assessment At Waste Disposal Landfill Site: Emerging Soil Physical Processes And Properties”

- *Prof. Ken Kawamoto*

2:30 - 2:40 Discussion

2:40 - 3:15 Presentation 2:

“Geotechnical Practices in Highway Construction”

- *Eng. Dr. W.A. Karunawardena*

3:15 - 3:25 Discussion

3:25 - 3:45 Tea

3:45 - 4:20 Presentation 3:

“Role of the Geotechnical Engineer in the Piling Industry”

- *Eng. Dr. H.S. Thilakasiri*

4:20 - 4:30 Concluding Remarks

4:30 - 6:00 13th Annual General Meeting of the SLGS (for members only)

SRI LANKAN GEOTECHNICAL SOCIETY

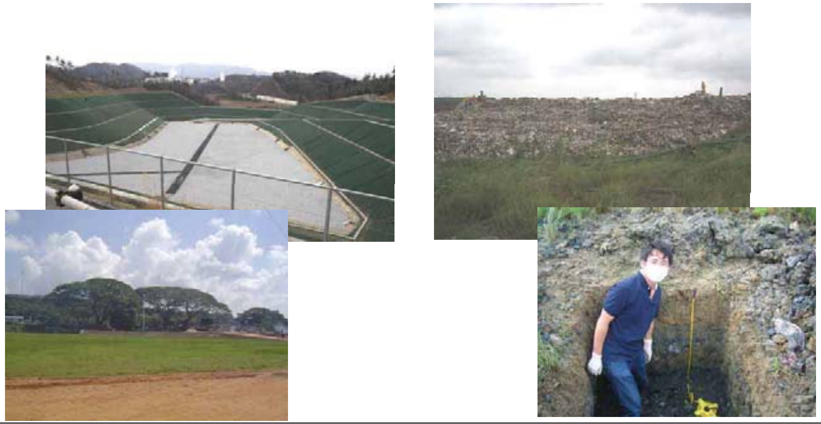
EXECUTIVE COMMITTEE *for the term 2009*

PRESIDENT	Eng. K.S. Senanayake
VICE PRESIDENT	Prof. S.A.S. Kulathilaka
PAST PRESIDENT	Prof. B.L. Tennekoon
PAST PRESIDENT	Eng. D.P. Mallawaratchie
HONY. SECRETARY	Eng. K.L.S. Sahabandu
ASST. SECRETARY	Eng. A.J. Amarasinghe
TREASURER	Eng.W.A.A.W. Bandara
ASST. TREASURER	Eng. R.M. Rathnasiri
EDITOR – JOURNAL	Dr. W. A. Karunawardena
EDITOR – NEWSLETTER	Dr. U.P. Nawagamuwa
COMMITTEE MEMBERS	Dr. L.B.K. Laksiri
	Ms. T.J.Jayasundara
	Mr. M.L.I. Abeysinghe
	Mr. H.R. Maduranga

SECRETARIAT: National Building Research Organisation,
99/1, Jawatte Road,
Colombo 05.

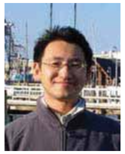
Environmental Risk Assessment at Waste Disposal Landfill Site: Emerging Soil Physical Processes and Properties

Ken Kawamoto
Geoenvironmental Engineering & Soil Mechanics Laboratory
Saitama University, Japan



Geoenvironmental Engineering / Soil Mechanics Lab.
Graduate School of Science and Engineering, Saitama University

Professor: Toshiko Komatsu
Assoc. Professor: Ken Kawamoto
Lab. Technician: Kunihiro Kobayashi
Secretary: Ritsuko Shibata

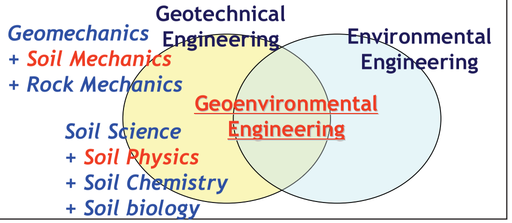


Graduate Students:
Shoichiro, H. (D3 Japan) --> (Research Assoc.)
Anurudda (D2, Sri Lanka)
Anu (D1, Nepal) Praneeth (D1, Sri Lanka)
Chamindu (D1, Exchange Ph.D student, Sri Lanka)
Tshering (M2, Bhutan)
Taiki, H. (M2, Japan) Yuichi, S. (M2, Japan)
Rouf (M1, Bangladesh) Naveed (M1, Pakistan)

And, 2 undergraduate students.
2 new Ph.D students (Sri Lanka & Nepal) will join on Oct. 2009.

Geoenvironmental Engineering

- Soil & groundwater contamination
  - Fate and transport of hazardous chemicals in soil
  - Remediation of contaminated soil
  - Environmental risk assessment
- Global warming
  - Transport/Emission of greenhouse gases in/from soil
  - Carbon cycle in soil-water system
- Land conservation
  - Soil erosion
  - Soil salinization



Geomechanics
+ Soil Mechanics
+ Rock Mechanics

Geotechnical Engineering

Environmental Engineering

Geoenvironmental Engineering

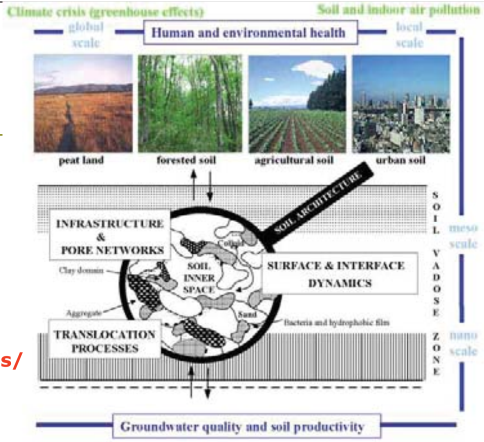
Soil Science
+ Soil Physics
+ Soil Chemistry
+ Soil biology

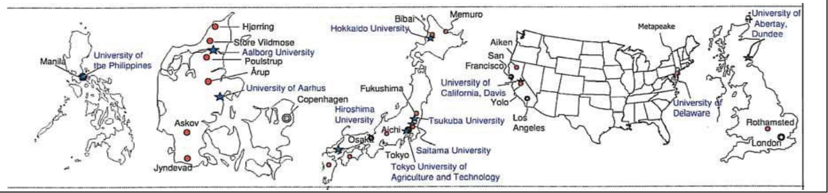
Global Research (1)

"Soil-it-is" project 2007-2012

Soil Infrastructure, Interfaces & Translocation Processes in Inner Space

http://www.agrsci.dk/soil-it-is/

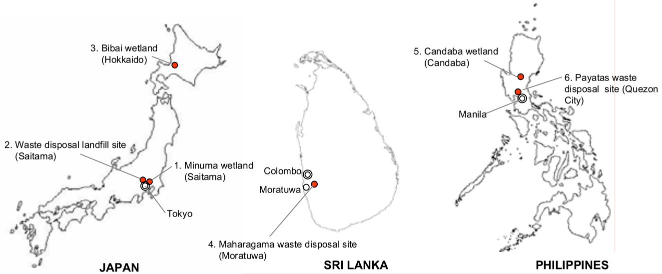




Global Research (2)

Waste disposal landfill site
and Wetland Projects
(2008-)

- + Student Exchange Program
- + Joint Seminar
- + Co-Supervising Graduate Student



Waste Disposal Landfill Sites

Payatas, Quezon City, Philippines

Maharagama, Sri Lanka

Waste in Japan

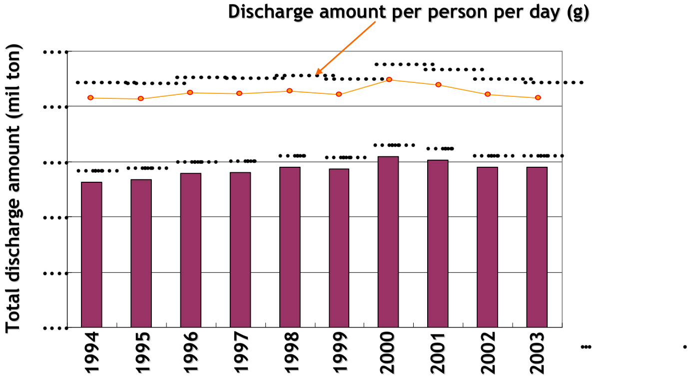
FY 2003	Municipal Solid Waste	Industrial Waste
Discharge amount (mil ton)	51	412
Incineration rate (%)	78	75
Final disposal (%)	3.6	7.3
Recycle rate (%)	17	49
Numbers of landfill site	> 2000	>1000

(Source: Aoyama & Ikeda, 2006)

Municipal Solid Waste (FY 2008)
Total Discharge: 53 millions ton
Total Cost: more than 2 trillions yen (20 bil USD)
Total Risk: unknown

Residual Capacity of Landfill Sites (FY 2003)
Capacity: 150 mil m^3
Residual Time: 14 years

Annual Amount of Municipal Solid Waste in Japan(1994-2003)

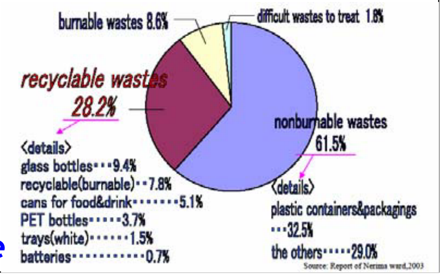


(Source: MOE, press release, 2006)

Details of Burnable and Non-Burnable Wastes

Burnable

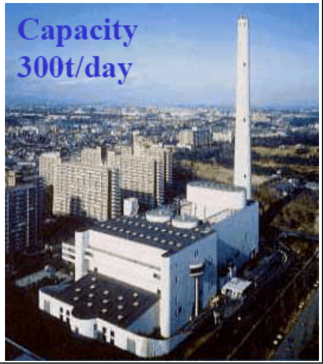
Non-Burnable



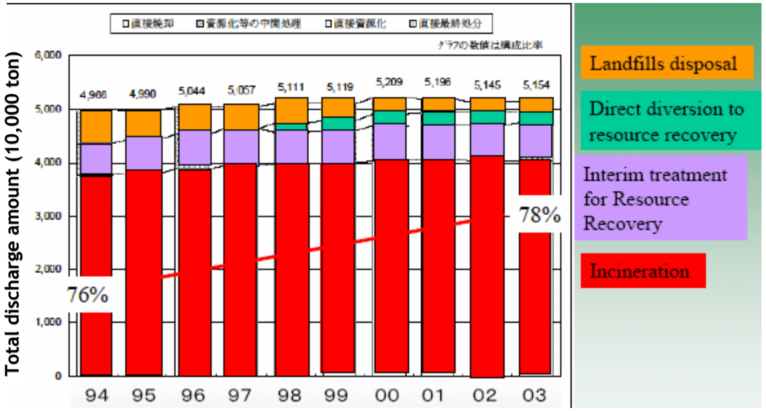
Incineration

Incineration ash (Yorii, Saitama)

Incineration plant (Nerima, Tokyo)
Constructed in 1983
10.5 bil yen (100 mil USD)



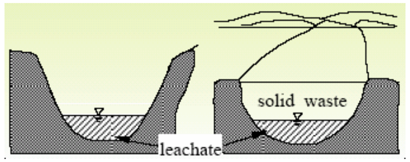
Annual Change of Waste Treatment in Japan(1994-2003)



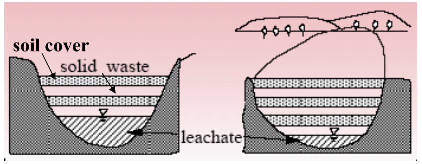
(Source: MOE, press release, 2006;
Aoyama & Ikeda, 2006)

Classification of Landfill Structure : Municipal Solid Waste (1)

Anaerobic landfill

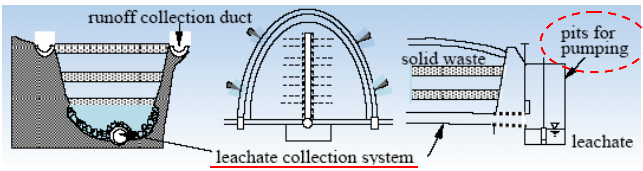


Anaerobic sanitary landfill

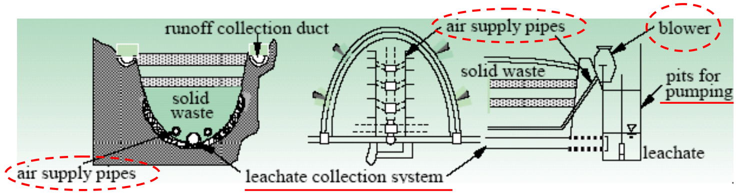


Classification of Landfill Structure : Municipal Solid Waste (2)

Semi-aerobic landfill



Aerobic landfill

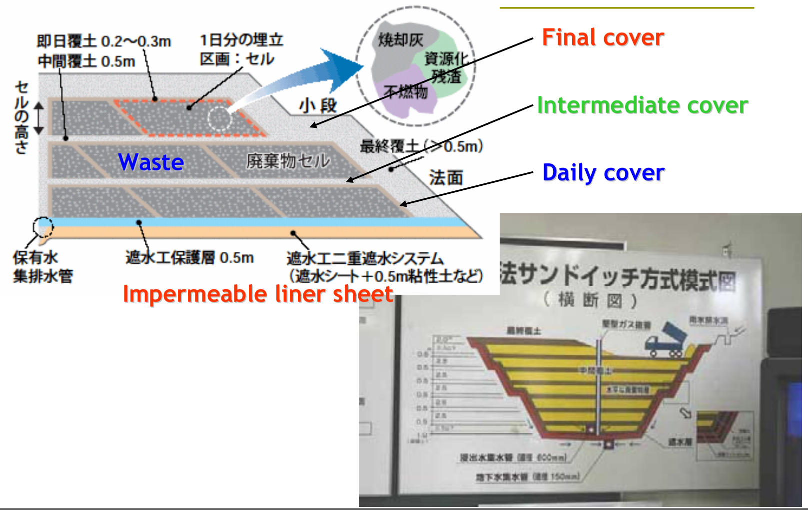


Semi-Aerobic Landfill (Fukuoka Method)

Nakata landfill site, Fukuoka, Japan
(First semi-aerobic landfill in 1975)

Yorii landfill site, Saitama, Japan
(1990•)

Daily and Final Covers at Landfill Site



What is the Risk of Landfill Site ?

Human health and surrounding (local) environment
- •• Soil & groundwater pollution
- •• Water pollution
- •• Air pollution
- •• Noise pollution
- •• ••••••

Leachate

Landfill gas

Global environment
- •• Greenhouse gas emission
- •• ••••••

Merits
- •• Resources (Recycling materials, rare metals, ...)
- •• Energy generation
- •• ••••••

How to assess the Risk and do the corrective action?

STEP1: Collect information

STEP2: Evaluation

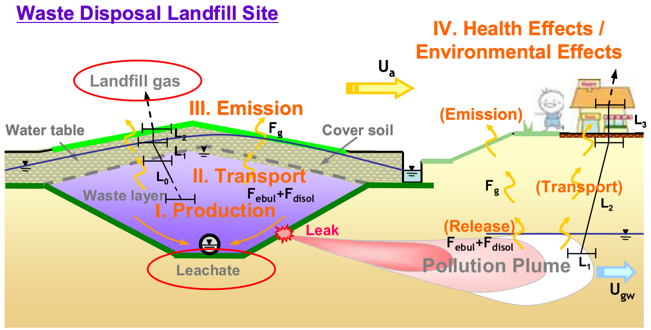
STEP3: Remedial Action

STEP4: Monitoring

Lack of physical, chemical, and biological knowledge, the evaluation is rather empirical. Still we can not provide a scientific/engineering based guidance for a **Risk-Based Corrective Action** at waste landfill sites.

A possible approach

Our final goal is to develop a process-based risk assessment model for various chemicals in the waste-soil-water-air-human system.



Risk Assessment Chain (Step I-IV) for Environmental Impact Gases (EIGs)

F_g: Flux of EIGs in unsaturated zone (gaseous phase)
F_{ebul}: Ebullition flux of EIGs in saturated zone (bubble phase)
F_{disol}: Dissolved EIGs flux in saturated zone (dissolved phase)
$L_{0\sim3}$: Release/Transport/Mixing distance
U_{gw}: Groundwater flux
U_g: EIGs flux in atmosphere

Question 1

Landfill Gas

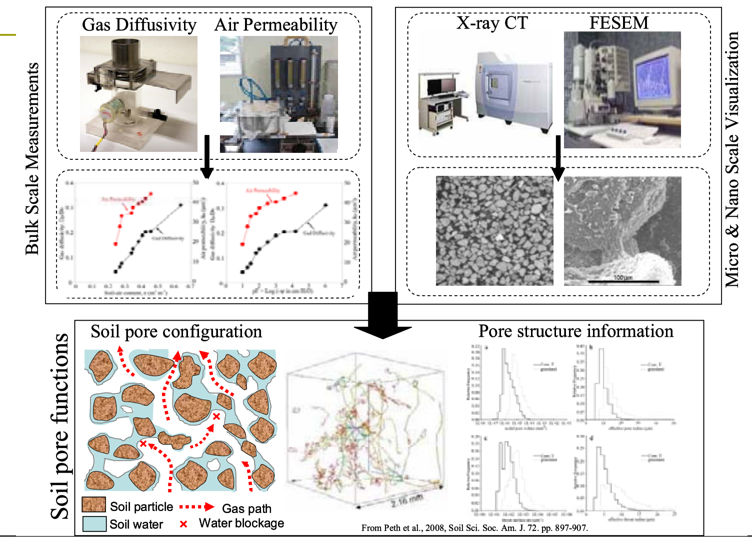
- *Can we estimate gas emission from a landfill site?*
  - **Partially Yes. But, we can not predict seasonal & annual change of gas emission.**
  - **This is because we can not analyze the gas transport process in waste and cover soil layers. We can just measure the gas emission rate on the soil surface at some points and at some times.**

Question 2

Leachate

- *Can we prevent leak of contaminants from surrounding clay layer (or liner) ?*
  - **Probably Yes. If we do not consider the new mobile phase for contaminants in the soil-water system.**
- *What is the new mobile phase?*
  - **That is soil colloid. We know the existence since long long ago but ignore its contribution to transport processes in the soil-water system.**

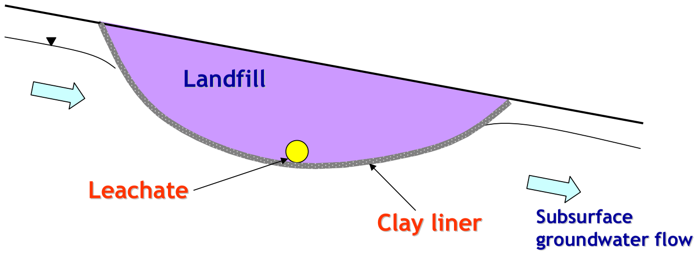
Laboratory Experiments



From Peth et al., 2008, Soil Sci. Soc. Am. J. 72. pp. 897-907.

Emerging Soil Physical Processes and Properties:
Colloid-Facilitated Contaminant Transport in Subsurface

Does contaminant immigrate outside the clay liner?



Heavy metals and other contaminants are highly adsorbed onto soil (clay liner).

- Immobile in soil ?

Subsurface Migration of Heavy Metals at Landfill Sites

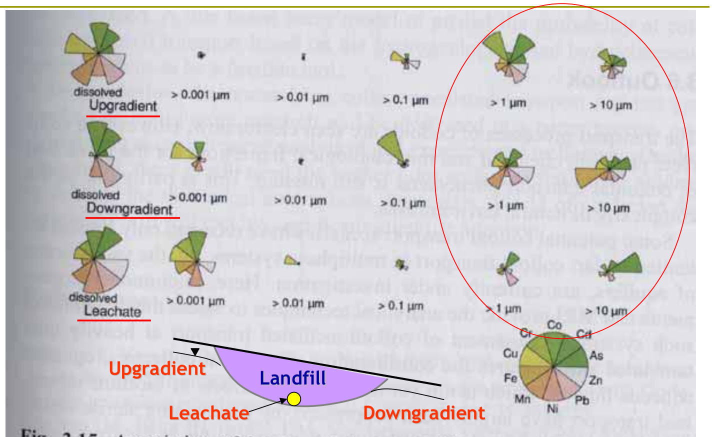


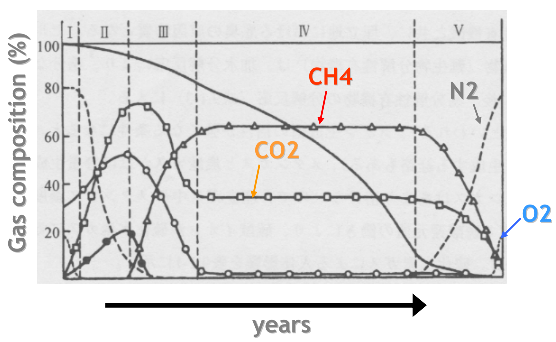
Fig. 3.15. Association of heavy metals to colloids at the Augsburg disposal (Baumann et al. 2006, Copyright with permission from Elsevier).

Emerging Soil Physical Processes and Properties:
Landfill Gas Research

What is Landfill gases?

- **Methane CH4: 40-90%**
- CO2: 20-40%
- VOCs: a few %

Landfill site stabilization

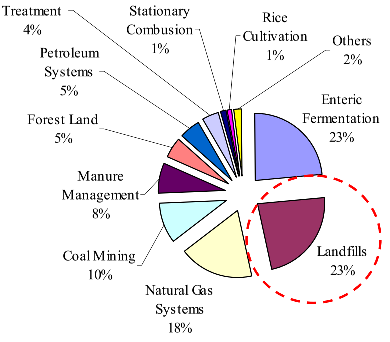


(Source: Ree, 1980)

CH4 Emission at Landfill Sites

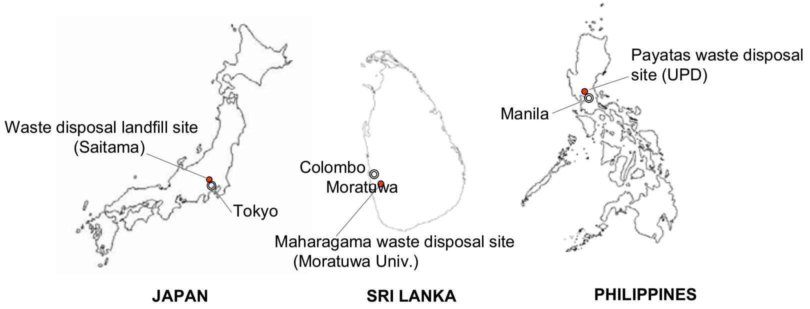
Sources for CH4 emissions in USA (EPA, 2009)

	1990	1995	2000	2005	2006	2007
			Tg CO2 Eq.			
Enteric Fermentation	133.2	143.6	134.4	136	138.2	139
Landfills	149.2	144.3	122.3	127.8	130.4	132.9
Natural Gas Systems	129.6	132.6	130.8	106.3	104.8	104.7
Coal Mining	84.1	67.1	60.5	57.1	58.4	57.6
Manure Management	30.4	34.5	37.9	41.8	41.9	44
Forest Land	4.6	6.1	20.6	14.2		
Petroleum Systems	33.9	32	30.3	28.3		
Wastewater Treatment	23.5	24.8	25.2	24.3		
Stationary Combusion	7.4	7.1	6.6	6.7		
Rice Cultivation	7.1	7.6	7.5	6.8		
Others	13.6	16.1	15	12.4		
Total	616.6	615.8	591.1	561.7		



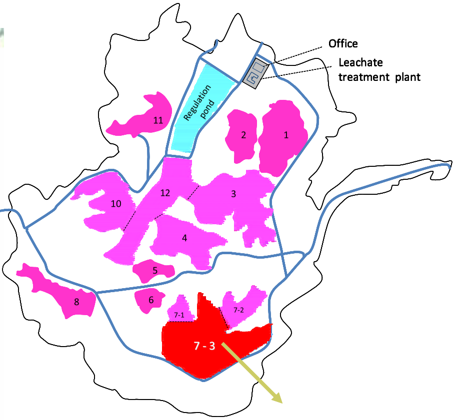
CH_4 Gas Emissions in U.S. (Tg CO_2 Eq.) (EPA, 2009)

Investigation Sites



Yorii landfill Site (Saitama, Japan)

Yorii landfill Site (Saitama, Japan)



- incineration ash
- plastic waste

Area: 50,000 m^2

Capacity: 450,000 m^3

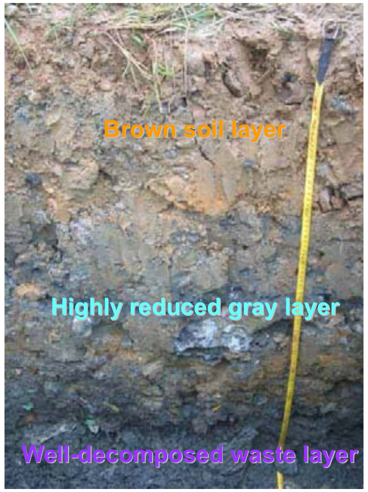
Maharagama landfill site (Sri Lanka)

Payatas landfill site (Quezon, Philippines)

Municipal solid waste: 1,500 ton/day

***Waste slide disaster (July, 2000)**

Payatas landfill site (Quezon, Philippines)

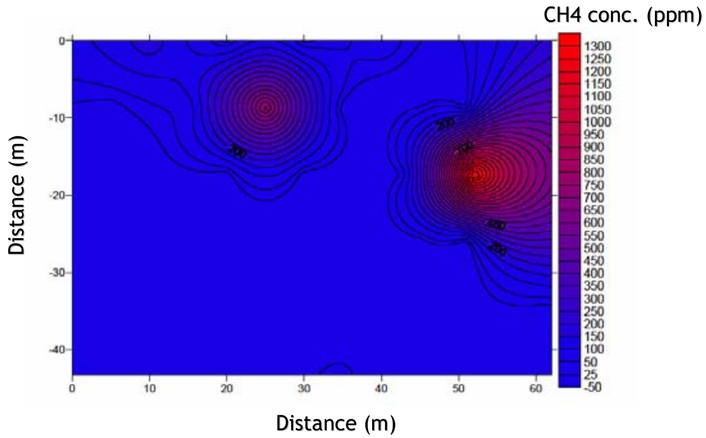


Strategy of Landfill Gas Research:
Transport and Emission

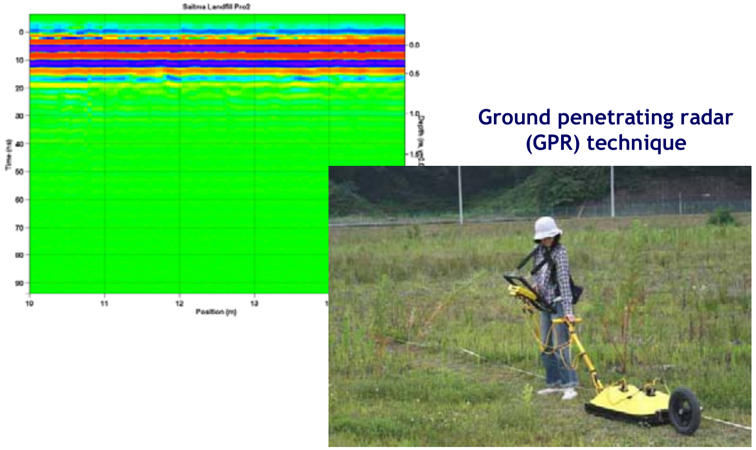
Field monitoring & Laboratory experiment
+ CH4 concentration profile
+ CH4 emission rate
+ Soil moisture
+ Soil hardness
+ Soil temperature
+ Ground water table
+ Gas transport parameters
  + Air permeability
  + Gas diffusion & dispersion coefficients

Modeling & Simulation
+ Predictive models for gas transport parameters
+ Gas flow simulation in final cover soil

CH4 Concentration Counter at Payatas landfill site



Measurement of Perched Water by GPR



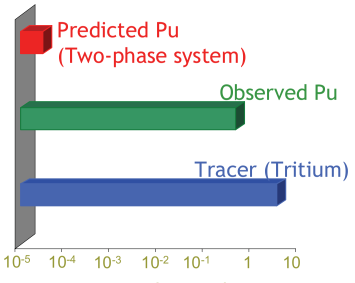
Ground penetrating radar (GPR) technique

Subsurface Migration of Radionuclides (1)

- Transport velocity,

  Observed = 500 m yr^{-1}

  Predicted = 4.2cm yr^{-1}
  (Based on Two-phase system)

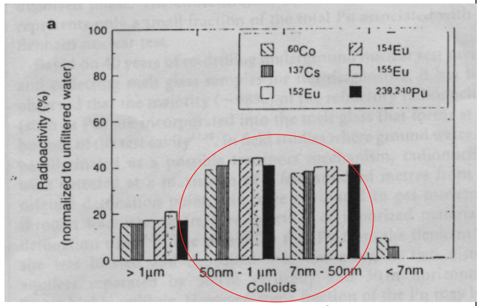


(Penrose et al.,1990)

Is there another phase facilitating contaminant?

Subsurface Migration of Radionuclides (2)

- Nevada nuclear weapon test site

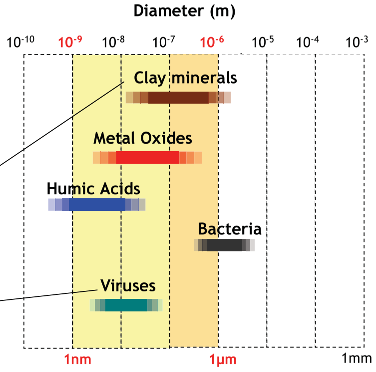


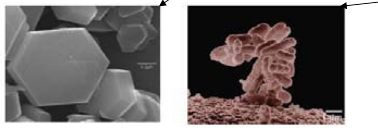
(Kersting et al., 1999)

What is Soil Colloid?

A Colloid is a particle in size between 1μm and 1nm *(Hunter, 2001)*

- Classification of Colloids
  - Clay Particles
  - Metal Oxides } Inorganic colloids
  - Humic Substances
  - Microorganisms } Organic colloids
- Basic Properties of Colloids
  - Very high specific surface area
  - Surface Charge



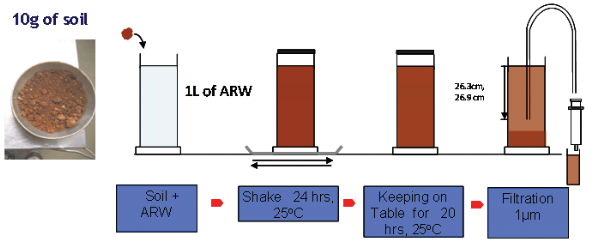


(Pictures from Saiers)

-Size range of colloidal particles-

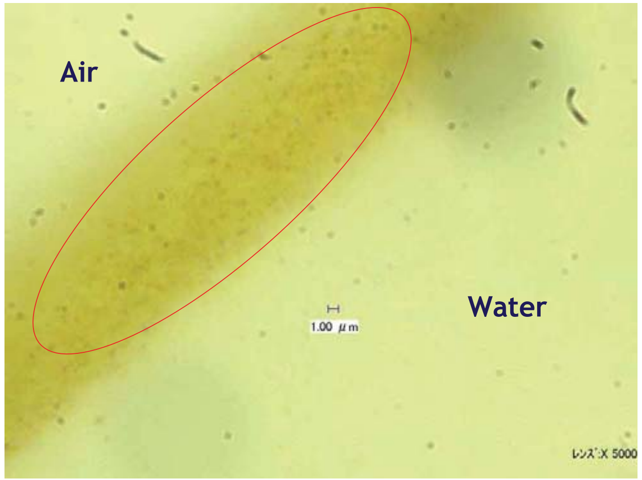
(Kretzschmar et al.,1999)

Extraction of Water Dispersible Colloids



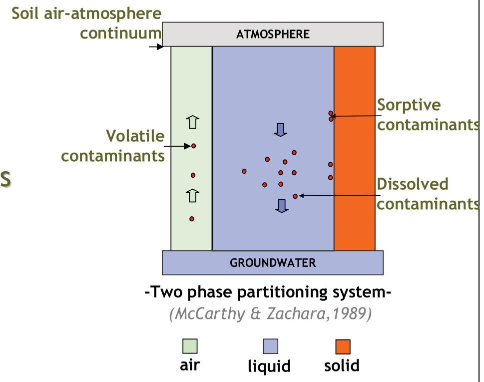
Soil + ARW → Shake 24 hrs, 25°C → Keeping on Table for 20 hrs, 25°C → Filtration 1μm

Soil Colloids



Classic Concept of Contaminant transport in Subsurface

- ➢Early concept of Two-phase partitioning
- ➢Liquid and gaseous
- •Highly sorptive contaminants
- ➢Immobile in soil
- ➢Pose little danger to groundwater

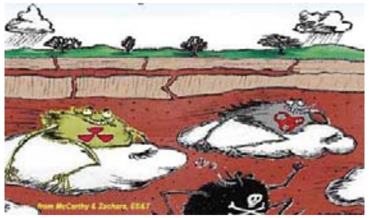


-Two phase partitioning system-
(McCarthy & Zachara,1989)

Are two-phase system predictions always accurate?

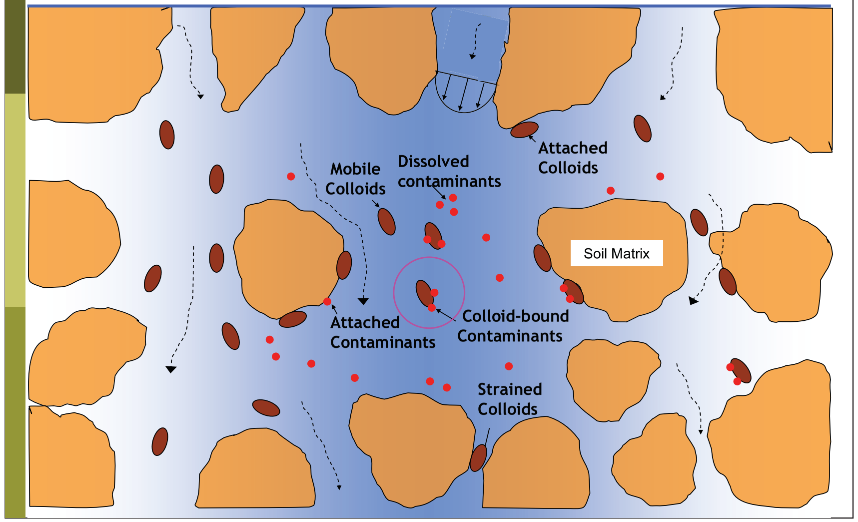
Evidences of Potential Contaminants

- ➢Radionuclides
  - Pu *(Kersting et al., 1999)*
- ➢Heavy Metals
  - Cu,Zn *(Karathanasis, 1999)*
  - Pb *(Karathanasis, 2000)*
  - Cs *(Chen et al.,2005)*
- ➢Pesticides & Fertilizers
  - DDT *(Vinten et al.,1983)*
  - Phosphorous *(de Jonge et al.,2004)*
- ➢Pathogens
  - Bacteria & viruses *(Foppen et al.,2005)*

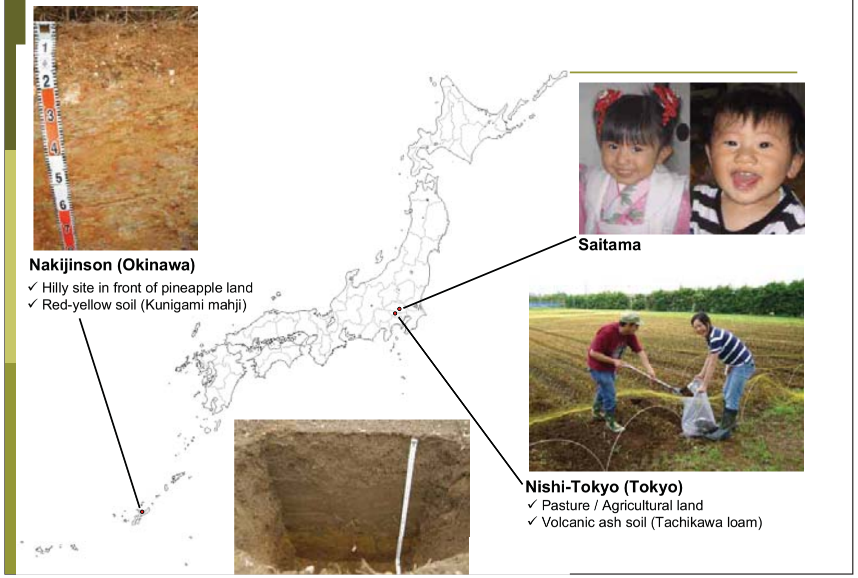


-Colloid-Facilitated Contaminant Transport in Subsurface Soil -
(McCarthy & Zachara,1989)

Colloid-Facilitated Contaminant Transport (CFT)



Soil Materials used for Colloidal Solution



Nakijinson (Okinawa)
- ✓ Hilly site in front of pineapple land
- ✓ Red-yellow soil (Kunigami mahji)

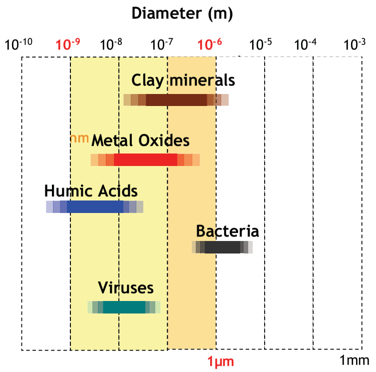
Saitama

Nishi-Tokyo (Tokyo)
- ✓ Pasture / Agricultural land
- ✓ Volcanic ash soil (Tachikawa loam)

What is Soil Colloid?
- Surface Charge -

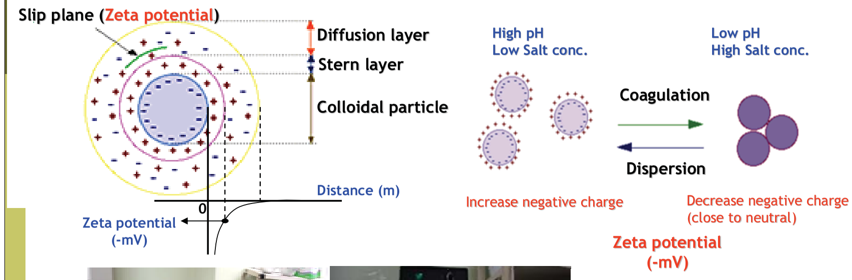
A Colloid is a particle in size between 1μm and 1nm *(Hunter, 2001)*

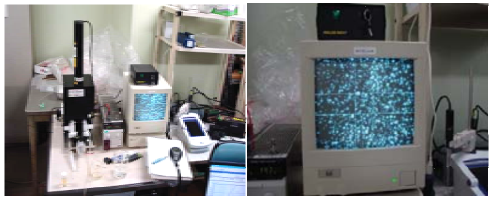
➢Basic Properties of Colloids
- •Very high specific surface area
- •Surface Charge



-Size range of colloidal particles-
(Kretzschmar et al.,1999)

Surface Charge of Colloidal Particle

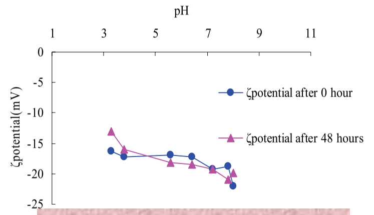




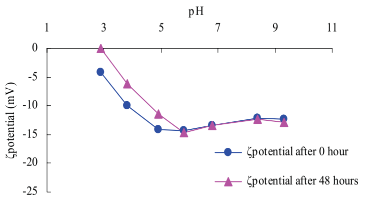
Zeta Potential Meter (Electrophoresis Mobility)

Measured Zeta Potentials

Red Yellow Soil Colloids



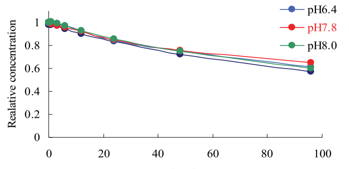
Volcanic Ash Soil Colloids

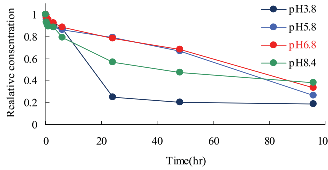


pH independent (permanent) surface charge dominant colloids

tion of pH at

pH dependent surface charge dominant colloids

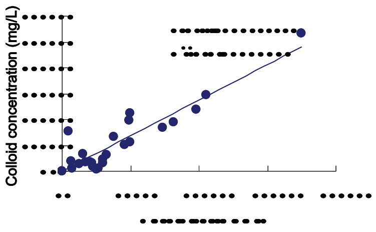


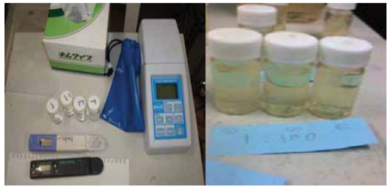


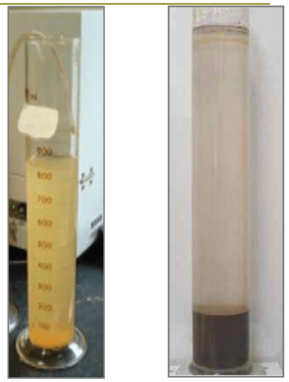
Stability of WDC at different pH

Colloid Concentration

Red-Yellow Soil Colloid



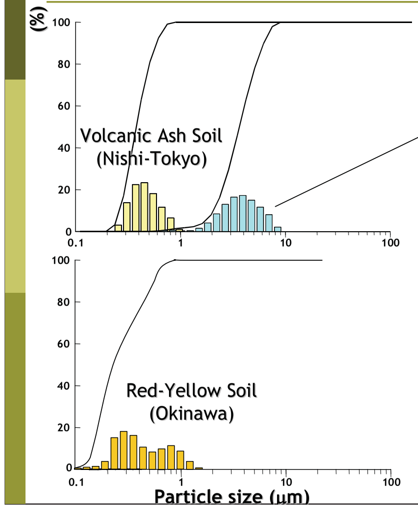

Turbidity Meter

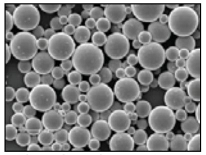

Red-Yellow Soil (Okinawa)

Volcanic Ash Soil (Nishi-Tokyo)

Water dispersible colloidal solution

Particle Size Distribution of Colloid

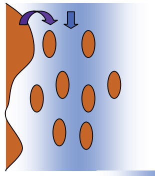



Glass beads (1-10µm)

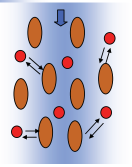

Particle Size Distribution Meter
(Laser scattering and diffraction techniques)

Essential Criteria for Colloid-Facilitated Transport (CFT)

1. Colloids must be present

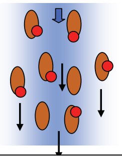

Good!

2. Contaminants must associate with colloids


Poor!!

3. Colloids must be transported through porous media

Very Poor!!!


(Ryan and Elemelech, 1996)

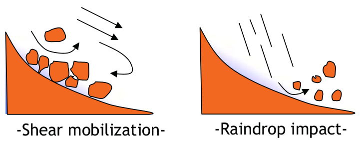
Colloid Mobilization & Transport

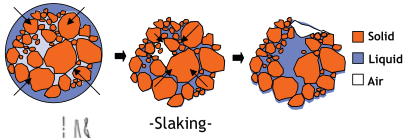
Colloid Mobilization

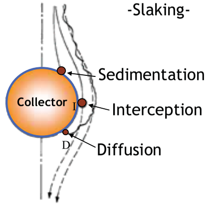
- Physical Perturbations
  - Hydrodynamic shear
  - Raindrop impact
  - Slaking
  - Differential swelling
- Chemical Perturbations
  - Decrease in ionic strength
  - Increase in pH

Colloid Collision with Grains

- Interception
- Sedimentation
- Diffusion





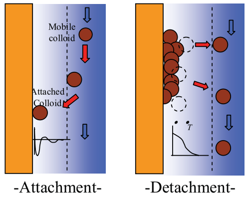


(Yao et al., 1971)

Colloid Deposition in Porous Media

Saturated Media

Attachment and Detachment

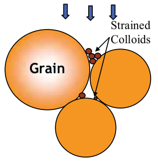


-Attachment- -Detachment-

(Denovio et al., 2004)

- Due to net interaction potential of colloids and surfaces
- Sum of double layer, van der Waals forces
- Effect of Solution chemistry is significant

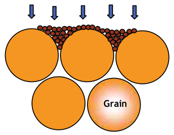
Straining



- Colloids trap at pore throats and grain edges
- Depends on colloid dia.(d_c) and mean grain dia.(d_{50})
- Threshold of straining: d_c/ d_{50} = 0.005

(Bradford et al., 2003)

Mechanical Filtration



- Occurs at the surface of filter media
- Colloids make a surface mat /filter cake on grain surfaces
- Affects the permeability

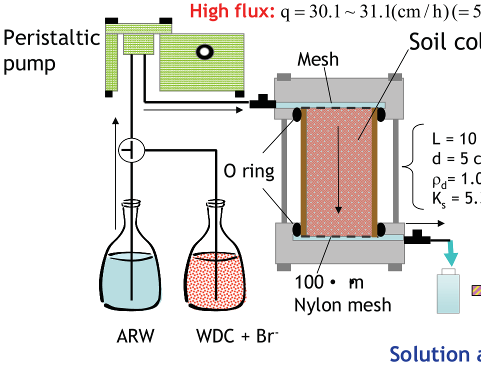
(Bradford et al., 2006)

(Pictures from Saiers)

Laboratory Column Experiments

Low flux: $q = 5.59 \sim 5.66 (cm/h) (= K_s)$

High flux: $q = 30.1 \sim 31.1 (cm/h) (= 5K_s)$

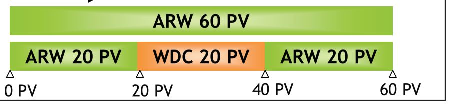


L = 10 cm
d = 5 cm
ρ_d= 1.04 g/cm³
K_s = 5.37 cm/h

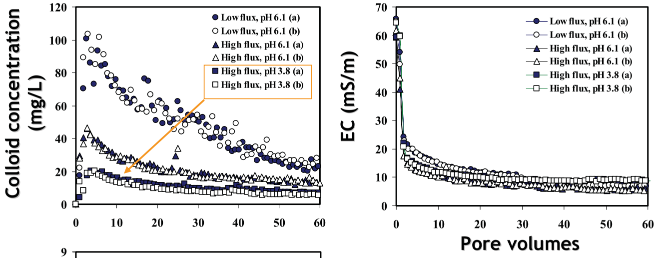
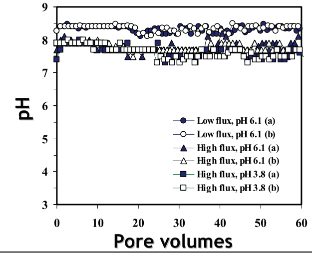
Colloid concentration (Turbidity)
PSD, EC, pH

Natural pH	6.1-6.5
Low pH	3.8

Solution application sequence

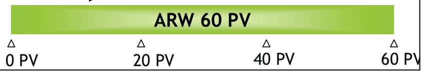


Laboratory Column Experiments
- Results (1) -

Red-Yellow Soil Colloid

Solution application sequence



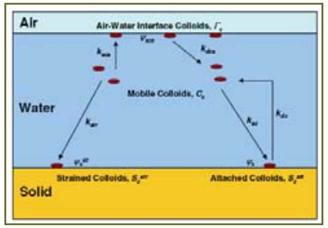
Mathematical Description of Colloid Transport

Convection-Dispersion Equation

$$\frac{\partial \theta_w C_c}{\partial t} + \rho_s \frac{\partial S_c}{\partial t} + \frac{\partial A_{aw}\Gamma_c}{\partial t} = \frac{\partial}{\partial z}\left(\theta_w D_c \frac{\partial C_c}{\partial z}\right) - \frac{\partial q_c C_c}{\partial z}$$

Retention at solid-water interface

Retention at solid-air interface



-Colloid Transport Model-

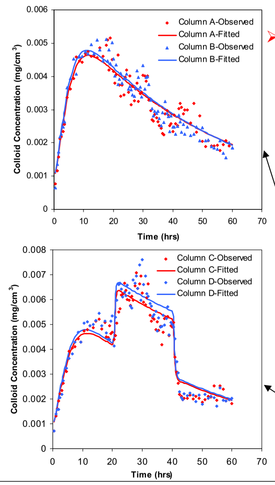
(Šim•nek et al., 2006)

Colloid Retention

$$\rho_s \frac{\partial S_c}{\partial t} = \underbrace{\rho_s \frac{\partial S_c^{att}}{\partial t}}_{\text{Attachment}} + \underbrace{\rho_s \frac{\partial S_c^{str}}{\partial t}}_{\text{Straining}} = \theta_w \underset{\text{Attachment coeff.}}{k_{ac}} C_c - \rho_s \underset{\text{Detachment coeff.}}{k_{dc}} S_c + \theta_w \psi_s \underset{\text{Straining coeff.}}{k_{str}} C_c$$

$$\underbrace{\frac{\partial A_{aw}\Gamma_c}{\partial t}}_{\text{Interface capture}} = \theta_w \psi_{aca} \underset{\text{Attachment coeff.}}{k_{aca}} C_c - A_{aw} \underset{\text{Detachment coeff.}}{k_{dca}} \Gamma_c$$

Laboratory Column Experiments (1)
- Results (2) -



Volcanic Ash Soil Colloid

- ➢Natural Colloid Leaching
  - Two-site(Equilibrium/Kinetic) Model

$$\frac{\partial \theta c}{\partial t} + \rho\frac{\partial s^e}{\partial t} + \rho\frac{\partial s^k}{\partial t} = \frac{\partial}{\partial z}\left(\theta D\frac{\partial c}{\partial z}\right) - \frac{\partial qc}{\partial z}$$

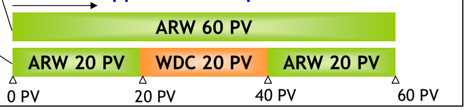
$$s = s^e + s^k \qquad s^e = f_e K_d c$$

$$\rho\frac{\partial s^k}{\partial t} = \alpha_k \rho\left(s_e^k - s^k\right) \qquad s_e^k = (1 - f_e)K_d c$$

- ➢Applied colloid (WDC) transport
  - One-site (Kinetic) Model

$$s = s^k \qquad \rho\frac{\partial s^k}{\partial t} = k_{ac}\theta c - k_{dc}\rho s^k$$

Solution application sequence



Remarks on emerging soil physical processes for environmental risk assessment at waste landfill site

- Landfill gas analysis and colloid-facilitated transport (CFT) of contaminants have been identified as emerging soil physical processes.
- Though significant progresses have been achieved in studies, the knowledge is still evolving.
- … due to poor knowledge and lack of accuracy in measurements and predictions, Landfill gas and CFT did not receive its due position in risk assessment tools.

Acknowledgements

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  Graduate School of Science and Engineering, Saitama University, Japan

GEOTECHNICAL PRACTICES IN HIGHWAY CONSTRUCTION

Dr. Asiri Karunawardena

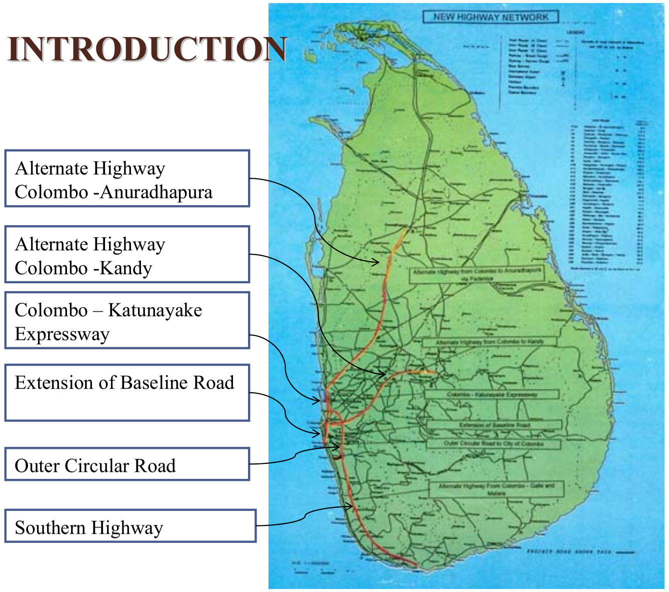
Director – Geotechnical Engineering

NBRO

OUTLINE

- Introduction
- Geotechnical Practices in Low Land Areas
  - Ground Improvement
  - Instrumentation
  - Assessment of the Ground Improvement
- Geotechnical Practices in High Land Areas
  - Design of Cut Slope
  - Lowering the Water Table
  - Monitoring of Slope Stability

INTRODUCTION



Alternate Highway
Colombo -Anuradhapura

Alternate Highway
Colombo -Kandy

Colombo – Katunayake
Expressway

Extension of Baseline Road

Outer Circular Road

Southern Highway

GEOTECHNICAL PRACTICES IN LOW LAND AREAS

Ground Improvement
Instrumentation
Assessment

CLASSIFICATION OF GROUND IMPROVEMENT BASED ON ITS PRINCIPLES

- **Removal and replacement**
  - Softsoil, mostly soft clay or highly organic clay under or near the expected structure is removed and replaced by a good quality foreign materials
- **Densification**
  - Densification of loose granular soils, heterogeneous soils, municipal wastes, liquefiable soils is quite a common practice. The purpose of densification is to increase strength and to reduce settlement of loose granular soils.
    - Vibro-Compaction
    - Heavy Tamping (Dynamic Compaction, Dynamic Consolidation)
    - Sand Compaction Pile Method

CLASSIFICATION OF GROUND IMPROVEMENT CONTD…

- **Consolidation/ Dewatering**
  - When a foundation ground is the cohesive soils with low strength and low permeability, structures constructed on the ground will experience the stability problem and/or the long term unfavorable settlement
    - Surcharge
    - With Vertical drains
    - Vacuum Assisted Preloading
- **Reinforcement**
  - Ground reinforcement consists of creating in-situ a composite reinforced soil system by inserting inclusions in predetermined directions to improve the shear strength characteristics and bearing capacity of the existing ground

---

CLASSIFICATION OF GROUND IMPROVEMENT CONTD…

- **Grouting**
  - Major purpose of grouting technology is to provide increased strength and/or to retard water seepage of soil or rock formation.
- **Admixture stabilization**
  - Admixture stabilization is a technique of mixing chemical additives with soil to improve the consistency, strength, deformation characteristics, and permeability of the soil
- **Thermal stabilization**

PRELOADING

- The aim is to
  - Eliminate 100% primary consolidation
  - Reduce secondary settlement

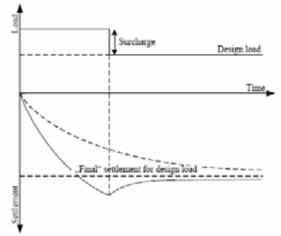


Figure 2: Resulting settlement due to preloading

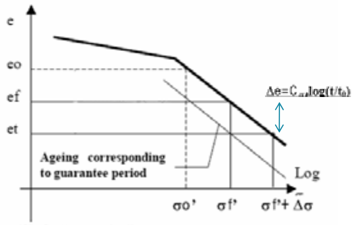
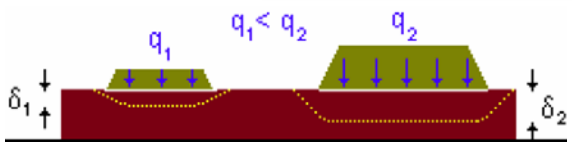


Fig 7 : "pre-ageing" and secondary settlement



VERTICAL DRAINS

- Vertical drains are installed under a surcharge load to accelerate the drainage of impervious soils and thus speed up consolidation
- These drains provide a shorter path for the water to flow through to get away from the soil
- Time to drain clay layers can be reduced from years to a couple of months

VACUUM CONSOLIDATION

- Soft Ground Treatment Methods

Preloading

- Conventional Preloading
- Preloading with vertical drains
- Vacuum preloading

Less time needed for Construction
Reduces the required fill material
Reduces the risk of instability
Reduces the time required for consolidation

- When subsoil is too soft for the fill surcharge to be applied
- When high embankments are constructed on soft soil with high layer thickness

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CONCEPT AND THE THEORY OF VACUUM CONSOLIDATION

Vacuum Preloading method was first introduced by Kjellman (1952)

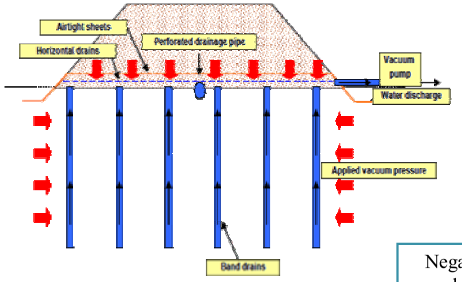

Figure 1: Vacuum system combined with embankment surcharge

- Vertical drains
- Horizontal drain pipes
- Perforated drain pipes
- Drainage layer (granular material)
- Sealing membrane
- Peripheral trench and edge revetment
- Vacuum pump

Negative pressure is created under the membrane by the vacuum pump

This applied negative vacuum pressure is propagated along the vertical drains to the deep subsoil layer

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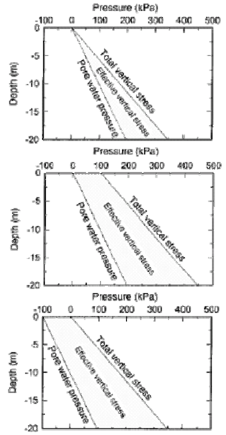
CONCEPT AND THE THEORY OF VACUUM CONSOLIDATION

Vertical Stress Distributions



Initial Condition

$\sigma' = \sigma - u$

Effective stress concept

Conventional Surcharge

Total stress will increase due to an additional load, thus effective stress will increase as well

Vacuum Preload

Total stress remains unchanged, pore pressure is reduced, the effective stress increases

CONCEPT AND THE THEORY OF VACUUM CONSOLIDATION

How vacuum consolidation improves the embankment stability

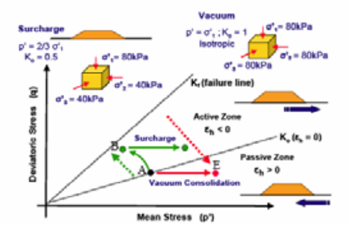
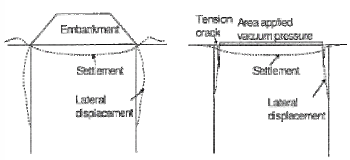


Figure 7 – Stress path for Vacuum Process



- Surcharge outward movement due to antistrophic loading $(\sigma_1 \neq \sigma_3)$
- Vacuum preloading inward movement due to isotropic loading $\sigma_1 = \sigma_3$

The advantage of Vacuum consolidation against surcharge is avoiding instability during construction

---

CONSTRUCTION PROCEDURE

1. Working platform construction

2. Band drain installations

3. Horizontal drain, perforated drain pipes and tank installations

4. Airtight sheet and edge revetment works

CONSTRUCTION PROCEDURE

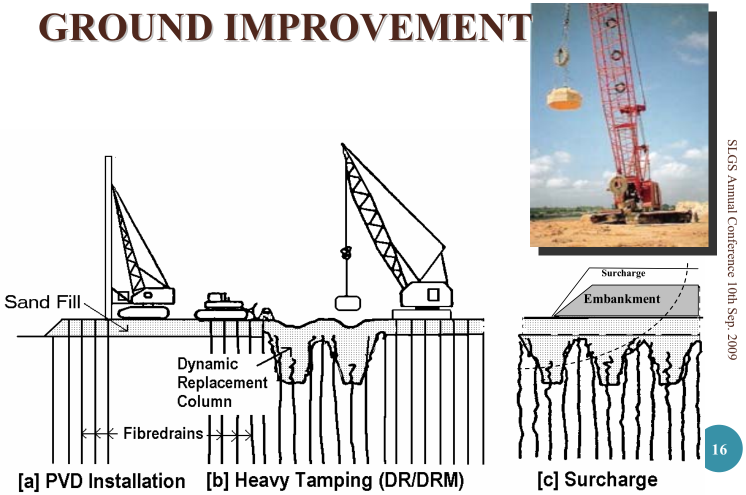
5. Air leak Checking

6. Overview before filling works

7. Filling works

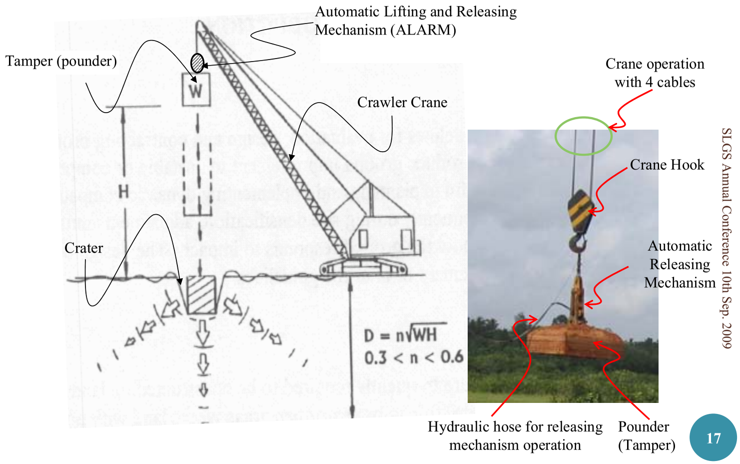
8. Finished stage

HEAVY TAMPING FOR PEATY GROUND IMPROVEMENT



[a] PVD Installation [b] Heavy Tamping (DR/DRM) [c] Surcharge

ACCESSORIES FOR HT



DESIGN CRITERIA (PLAN)

- Selection of the tamper mass and drop height to correspond to the required depth of improvement
- Determination of the applied energy to be used over the project site to result in the desired improvement
- Determination of the grid spacing and number of phases
- Establishing the number of passes

SELECTION OF TAMPER & THE HEIGHT OF DROP

$$D = n\sqrt{WH}$$

D – Depth of loose/soft soil to be improved

W – Weight of the tamper (pounder) in Tons

H – Height of Drop in m

n – A constant ranging 0.3 to 0.6 usually

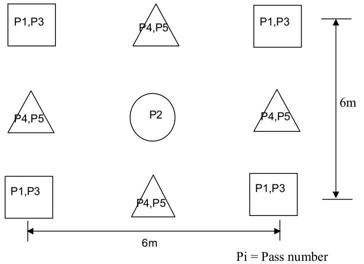
For soft Peaty deposits the value of ***n*** is assumed to be **0.35**

The height of drop is governed by a pounder dimension and crane capacity and boom configuration and hence, is found to be maximum 8m in our operation.

Therefore, the **depth of improvement** we could achieve at the site **with 15T pounder** is about **4m**

Thumb rule is that to raise 15T weight the Crane capacity must be greater than 4×Tamper weight (i.e. Crane capacity >60T)

GRID SPACING, MULTIPLE PASSES (PHASES) AND NUMBER OF DROPS



Pi = Pass number

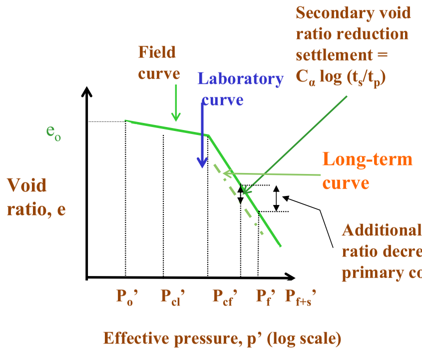
$$\text{Applied energy intensity} = \frac{W \sum_{i=1}^{p} Ni \times Hi}{\text{Representative grid area}}$$

Total Applied Energy = 300tm/m²

Energy status = 300 tm/m²

Phase	# of blows for Drop Height		Energy (tm)
	H =5m	H =8m	
1		12	1440
2		16	1920
3		14	1680
4		12	2880
5		12	2880
			10800

SETTLEMENT COMPUTATION



P_0' = Existing overburden pressure

P_{cl}' = Laboratory critical pressure

P_{cf}' = Field critical pressure

P_f' = P_0' + P = Final effective stress with embankment load only

P = Embankment load

P_{f+s}' = P_0' + P + P_s = Effective stress with embankment load and surcharge

P_s = Surcharge to enforce additional primary consolidation to eliminate secondary void ratio reduction

---

PRIMARY AND SECONDARY COMPRESSION

Primary Compression

- δp = H Cr log(Pf'/P0')/(l+e0)

  for Pf' < Pc'

- δp = H [Cr log(Pc'/P0') + Cc log(Pf'/Pc')]/(l+e0)

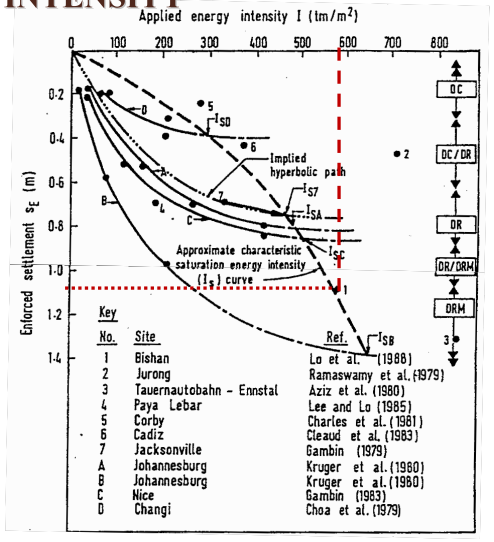
  for Pf' > Pc'

Secondary Compression

- δs = H Cα log(ts/tp)/(l+e0)

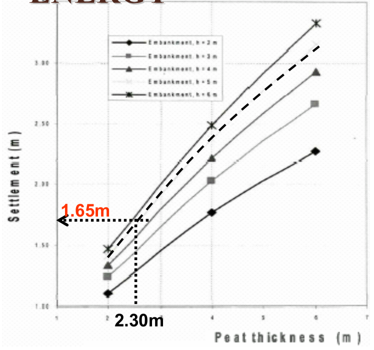
  for Pf' > Pc'

ENERGY INTENSITY

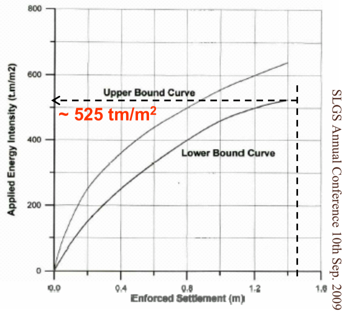


Key No.	Site	Ref.
1	Bishan	Lo et al. (1988)
2	Jurong	Ramaswamy et al. (1979)
3	Tauernautobahn - Ennstal	Aziz et al. (1980)
4	Paya Lebar	Lee and Lo (1985)
5	Corby	Charles et al. (1981)
6	Cadiz	Cleaud et al. (1983)
7	Jacksonville	Gambin (1979)
A	Johannesburg	Kruger et al. (1980)
B	Johannesburg	Kruger et al. (1980)
C	Nice	Gambin (1983)
D	Changi	Choa et al. (1979)

DETERMINATION OF APPLIED ENERGY



Approximate settlement under embankment height in Peat or highly Organic Clay

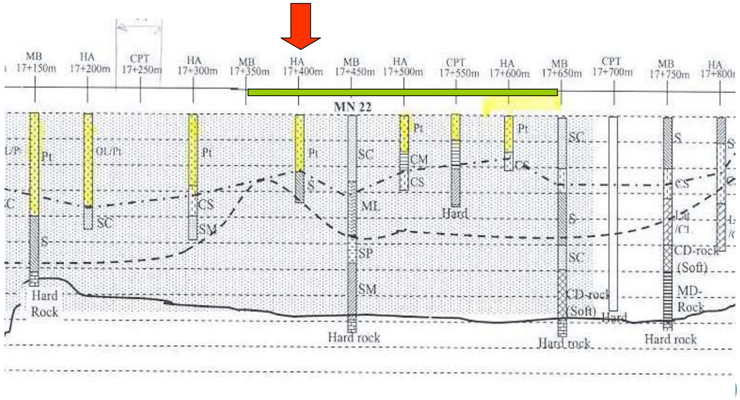


Relation of applied energy and the enforced settlement

HEAVY TAMPING FOR PEATY GROUND IMPROVEMENT

- Subsoil consist of 3m-5m thick very soft peat deposits
- Water Table almost at the surface
- Design Embankment height is around 5m -7m

PEATY CLAY PROFILE – HT SITE



SOIL PROPERTIES

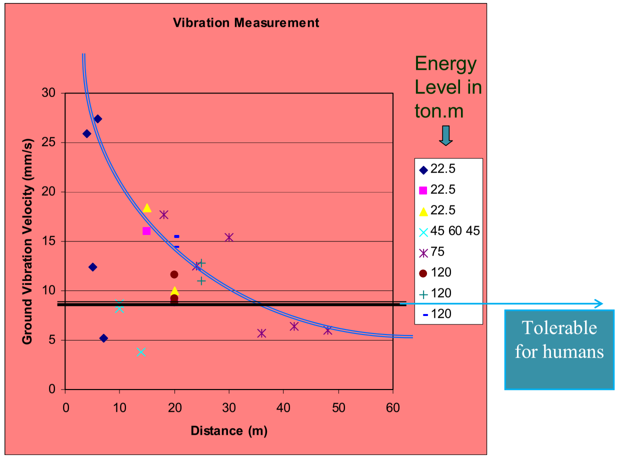
Location	Description	Depth (m)	Init. W (%)	Init. eo	Bulk density (g/cc)	Organic (%)	Comp index, Cc	Pre-consol pressure (kn/m2)	Undr. shear streng. kN.m2
15+800	Peat, black, fibrous -	1.4-2.4	787	13.18 7	0.94	40.4	6.78	17.5	
17+100	Organic silt, peat	1-2	151.7	3.005	1.01		2.455	40	12
17+300	Peat, black, fine fibrous	2.5-3.0	471.8	9.376	0.96	47.0	5.6	32	15

SITE PHOTOS

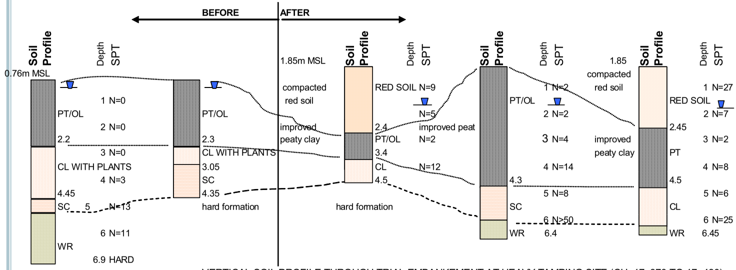
HEAVY TAMPER

SITE PHOTOS

MEASURED GROUND VIBRATION

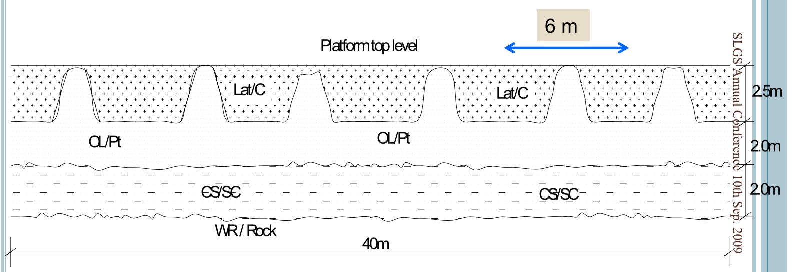


SOIL PROFILE BEFORE AND AFTER



VERTICAL SOIL PROFILE THROUGH TRIAL EMBANKEMENT AT HEAVY TAMPING SITE (CH 17+370 TO 17+430)

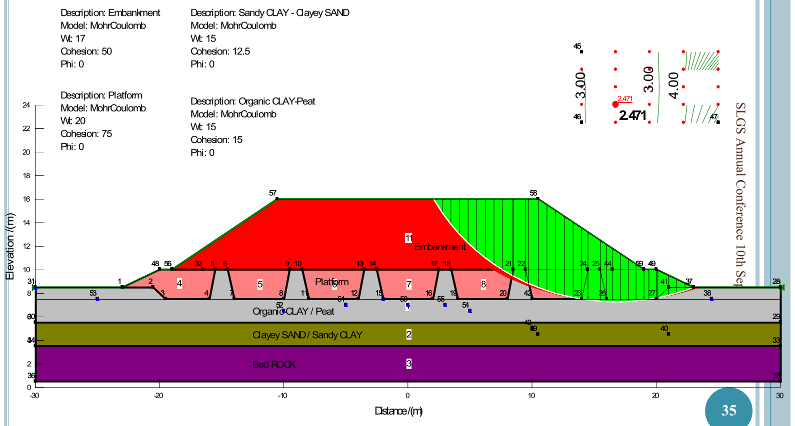
SOIL PROFILE AFTER HT



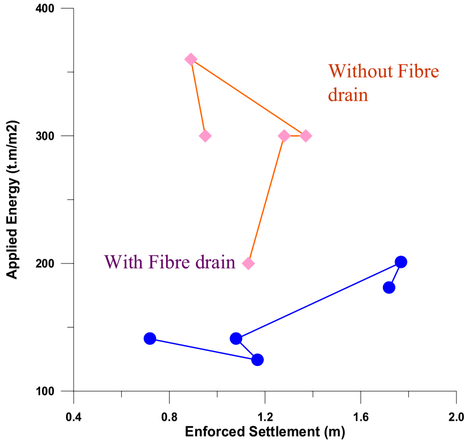
CONSOLIDATION PROPERTIES

Property	Before HT	After HT
Wn (%)	500	225
Void ratio e_0	7.69	3.25
Cr		
Cc	3.36	1.38
Pc	22	57
SPT	0	4-8

STABILITY OF THE EMBANKMENT

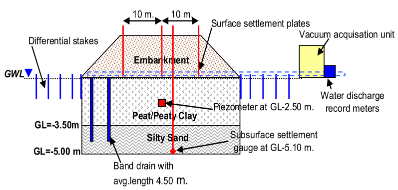


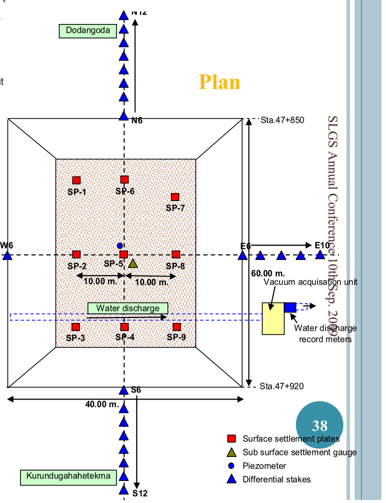
EFFECT OF PORE PRESSURE DISSIPATION



EXCAVATION FOR STRUCTURE AFTER TREATMENT

FIELD MONITORING FOR SOFT GROUND TREATMENT

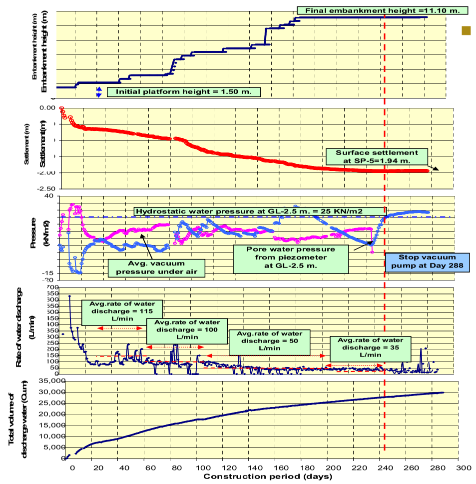




Sections

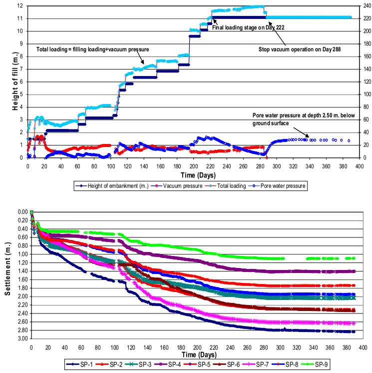
- Settlement Plates
- Differential settlement gauge
- Piezometer
- Displacement stakes
- Vacuum acquisition unit

TYPICAL FIELD MONITORING DATA



- Fill height
- Settlement
- Pore water pressure
- Vacuum pressure (under sheets)
- Rate of water discharge
- Total discharge

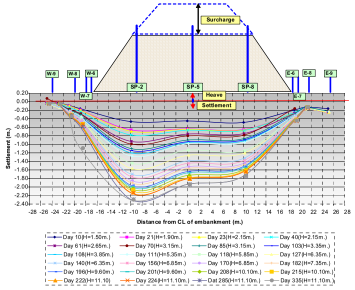
FIELD MONITORING DATA



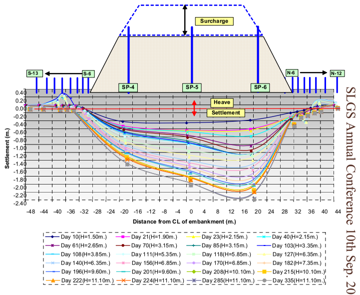
SP.No.	Settlement (m.)	SP.No.	Settlement (m.)	SP.No.	Settlement (m.)
SP-1	2.834	SP-4	1.405	SP-7	2.633
SP-2	2.307	SP-5	1.947	SP-8	1.739
SP-3	2.031	SP-6	2.330	SP-9	1.094

- Time & Settlement Curve

SETTLEMENT PATTERN



- **Transverse Sections**

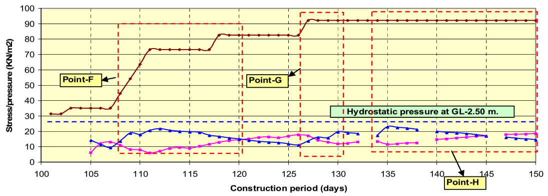


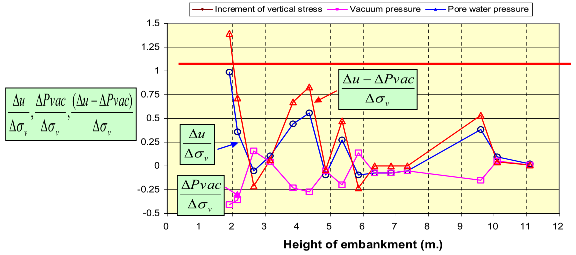
- **Longitudinal Sections**

Non-uniform shape of settlement is due to the varying thickness of peat?

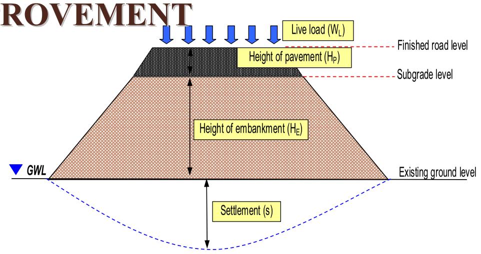
STABILITY CONTROL FOR EMBANKMENT CONSTRUCTION

Monitoring of change in pore water pressure during construction period





EVALUATION OF GROUND IMPROVEMENT



- Check the removable surcharge height
- Design Embankment load (Filling + Settlement)
- Surcharge Load |: Removable surcharge height should be greater than the design surcharge load
- Design surcharge load: Future load …..
  Load that required to reduce the residual settlements as required

---

HOW TO EVALUATE THE PERFORMANCE OF GROUND IMPROVEMENT

1. Assessment of degree of the consolidation from settlement and pore pressure records from field monitoring data
2. The gain in undrained shear strength before and after treatment works
3. Evaluated of the gain in preconsolidation pressure

ASSESSMENT OF DEGREE OF CONSOLIDATION FROM FIELD SETTLEMENT DATA

$$\boxed{\overline{U}\,\% = \frac{S_t}{S_\alpha}}$$

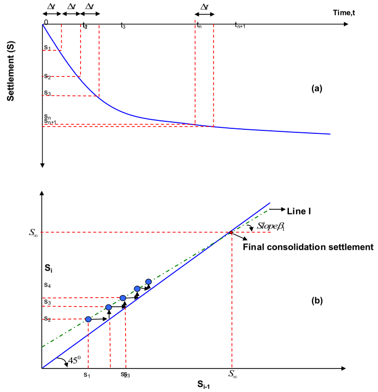
S_t = Settlement at time t

S_α = Ultimate primary consolidation settlement that assessment by Asaoka and Hyperbolic method

$\overline{U}\%$ = Degree of consolidation

ASSESSMENT OF DEGREE OF CONSOLIDATION FROM FIELD SETTLEMENT DATA

Asaoka (1978) Method



Asaoka's graphical plot

ASSESSMENT OF DEGREE OF CONSOLIDATION FROM FIELD SETTLEMENT DATA

SP- Δt 10 Days

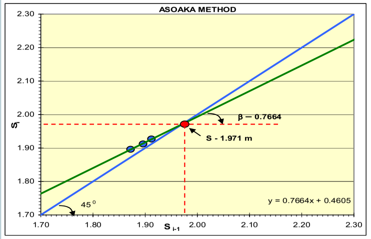
Date	Settlement (m.)	Remarks
8-Dec-07	1.814	Zero day for analysis
18-Dec-07	1.859	S1
28-Dec-07	1.872	S2
7-Jan-08	1.896	S3
17-Jan-08	1.912	S4
27-Jan-08	1.927	S5

Total settlement (m.)	1.971
Actual settlement (m.)	1.927
% of consolidation	97.77

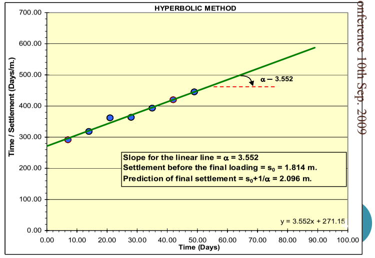
SP- 5

Date	Settlement (m.)	$t-t_0$ Days	$(t-t_0)/s$ days/m	Remarks
8-Dec-07	1.814	0	0.00	Zero day for analysis
15-Dec-07	1.838	7	291.67	
22-Dec-07	1.858	14	318.18	
29-Dec-07	1.872	21	362.07	
5-Jan-08	1.891	28	363.64	
12-Jan-08	1.903	35	393.26	
19-Jan-08	1.914	42	420.00	
26-Jan-08	1.924	49	445.45	

Total settlement (m.)	2.096
Actual settlement (m.)	1.927
% of consolidation	91.94



Asaoka's graphical plot

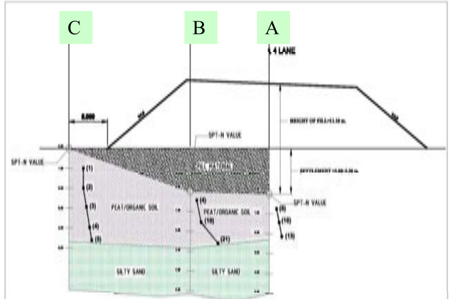


Slope for the linear line = α = 3.552
Settlement before the final loading = s_0 = 1.814 m.
Prediction of final settlement = s_0+1/α = 2.096 m.

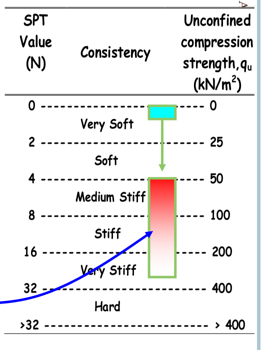
Hyperbolic graphical plot

ASSESSMENT OF GROUND IMPROVEMENT BY FILED & LABORATORY TESTS

SPT - Test

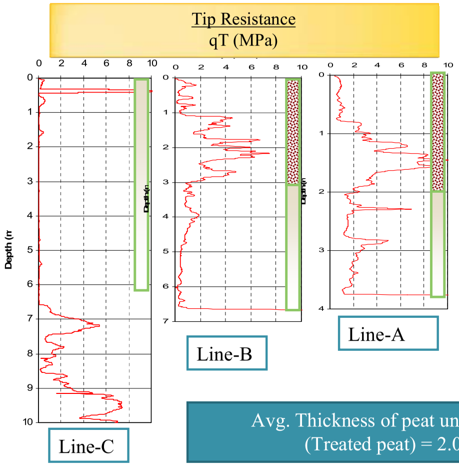


Test Results
Line A – N Value – (5-13)
Line B – N Value – (4-21)
Line C – N Value – (1-5)



Change the consistency of soils
Very soft soil -- > Medium stiff to
Very stiff soil

THE GAIN IN UNDRAINED SHEAR STRENGTH



CPTu - Test

Avg. Tip resistance
Line –A – 1.70 MPa.
Line –B – 1.00 MPa.
Line-C – 0.05 MPa

▨ Fill material

▭ Peat

Avg. Thickness of peat under embankment (Treated peat) = 2.00-3.00 m.

THE GAIN IN UNDRAINED SHEAR STRENGTH

Location		Triaxial test (UU-Test)		Vane shear test		Remarks
Line A	No.1	11.40-12.15 m.[a]	75 kPa.[b]	13.00 m.[a]	211.3 kPa	Treated peat
	No.2	11.00-11.75 m.[a]	130 kPa.[b]			
Line B	No.1	11.00-11.75 m.[a]	50 kPa.[b]	11.50 m.[a]	165.8 kPa	Treated peat
	No.2	12.00-12.80 m.[a]	55 kPa.[b]	12.80 m.[a]	119.1 kPa	
Line C	No.1	2.50-3.50 [c]	22 kPa.[b]	5.10 m.[c]	48.6 kPa	Non-treated peat
				5.35 m.[c]	58.4 kPa	

a - The depth below the top of constructed embankment (Elev + 8.065…..m.)
b - The undrained shear strength at confined stress = 100 kPa
c - The depth below the top of existing platform at toe of constructed embankment (Elev + 1.20 m.)

CHANGE IN WATER CONTENT, VOID RATIO AND PEAT THICKNESS

1. Laboratory

Descriptions	Avg.Initial water content (%)	% change of water content(1)	Avg.initial void ratio	% change of void ratio(1)
Line (A)-SP-5	246	34.6	4.36	21.6
Line (B)-SP-2	139	63	1.895	65.9

2. Calculation

Descriptions	w_n(1) (%)	G	δ (m.)	H (m.)	Δw_n (%)	Final water content (%)	% change of water content
Line (A)-SP-5	376	1.43	1.947	3.10	-236.5	139.5	62.8
Line (B)-SP-2	376	1.43	2.307	4.50	-193.1	182.9	51.3

Change in thickness, water content & void ratio = 50-60%

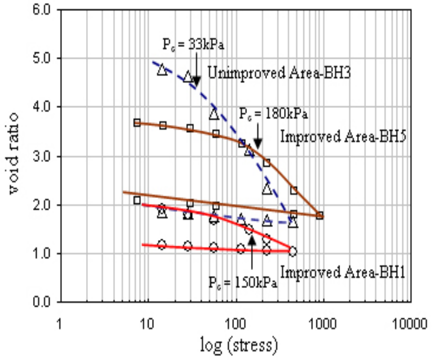
$$\Delta w_n = -\left(w_n + \frac{1}{G}\right)\frac{\delta}{h}$$

(1)– the average initial water content and void ratio from line C used for reference for calculation that we considered as non-treatment peat as shown in item 8.2.2.(%Wn = 376, e_o = 5.56)

Descriptions	Original peat thickness (m.)	Settlement (m.)	% change in thickness of peat
Line (A)-SP-5	3.10	1.947	62.8
Line (B)-SP-2	4.50	2.307	51.2

GAIN IN PRECONSOLIDATION PRESSURE

Laboratory test (Oedometer)



ESTIMATION OF FINAL SETTLEMENT

1. Primary settlement

Remaining primary consolidation settlement

> Can be neglected as Surcharging & Degree of consolidation is greater than 95

-- > Monitoring pore water pressure

2. Secondary compression

> From field monitoring data (Trend line) -- >

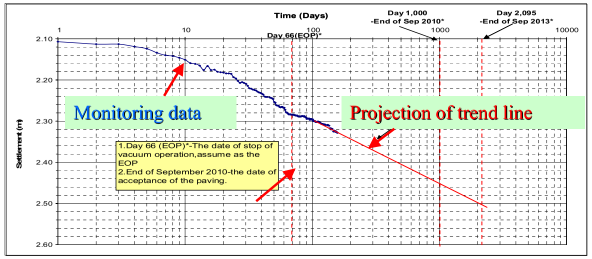
> From theoretical -- > Conservative value -- >

ESTIMATION OF FINAL SETTLEMENT

2. Secondary compression

Using the trend line from monitoring data

Observed Settlment Data and Calculation of Secondary Compression for Settlement Plate SP-06



1.Day 66 (EOP)*-The date of stop of vacuum operation,assume as the EOP
2.End of September 2010-the date of acceptance of the paving.

From monitoring data, the secondary compression Indexs C_α can calculated as follows;

$$C_\alpha = \frac{\text{settlement per log cycle}}{\text{thickness of peat at EOP}}$$

$$C_\alpha = \frac{0.16}{2.5} = 0.064$$

δ_S at September 2010 = 2.450

δ_S at September 2013 = 2.502

Δ = 52 mm

ESTIMATION OF FINAL SETTLEMENT

2. Secondary compression

Using the theoretical formulae with parametric study

> Formulae -- >

$$\rho_s = \sum_{i=1}^{n} \left(\frac{C_\alpha}{1+e_p} H \right)_i \log \frac{t_s}{t_p}$$

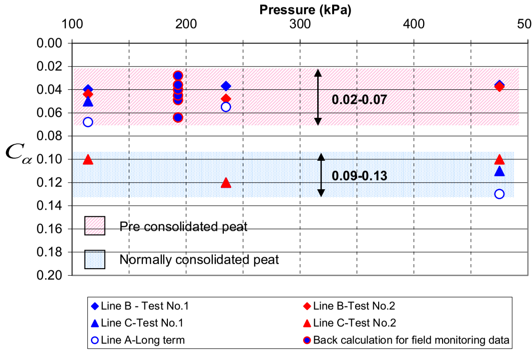
> Parametric study -- > Thickness of peat

-- > Modified secondary compression index.(%)

ESTIMATION OF FINAL SETTLEMENT

2. Secondary compression

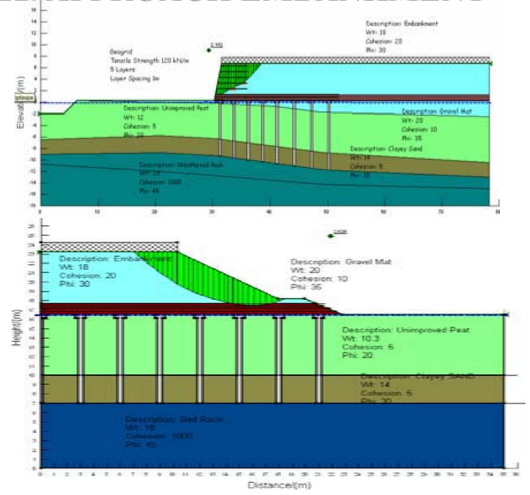
Secondary compression index.



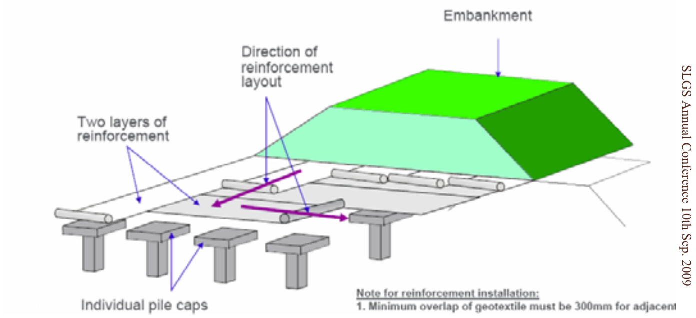
GROUND IMPROVEMENT AT BRIDGE APPROACHES

To avoid any uncomfortable vertical curves and abrupt vertical jolts leading the upward lunges on the bridge caused by relatively large differential settlement of the embankment.

DESIGN OF GEOGRID REINFORCED PILED APPROACH EMBANKMENT



CONSTRUCTION OF GEOGRID REINFORCED PILE EMBANKMENT



Note for reinforcement installation:
1. Minimum overlap of geotextile must be 300mm for adjacent

GEOTECHNICAL PRACTICES IN HIGH LAND AREAS

Design

Construction

Maintenance

DESIGN

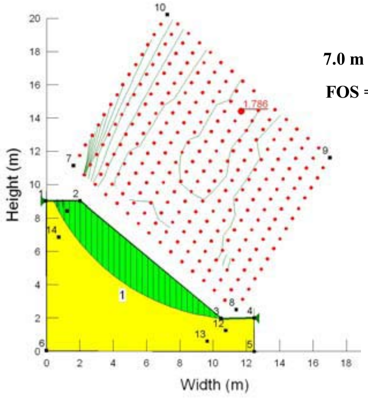
- Design was carried out to ensure the adequate safety factor under normal condition as well for heavy rain period if necessary

During Design Stage	During Excavation Stage
Subsurface Profile	Angle of Under Laying Rock (if any)
Depth of Cut	Condition of Interface Friction Between Soil and Under Laying Rock
Slope Gradient	Seepage Effect
Dip Direction	

STABILITY ANALYSIS

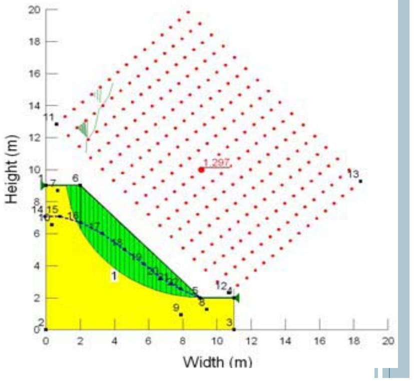
7.0 m Cut without water table effect *(1:1.2)*

FOS = 1.*786*



7.0 m Cut with water table effect *(1:1.2)*

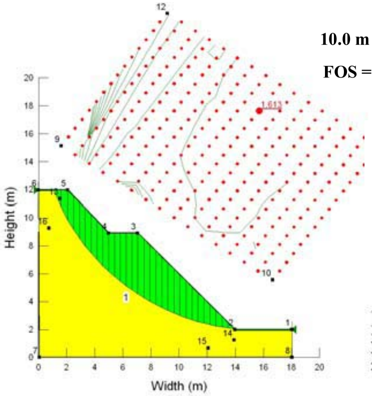
FOS = 1.*297*



STABILITY ANALYSIS

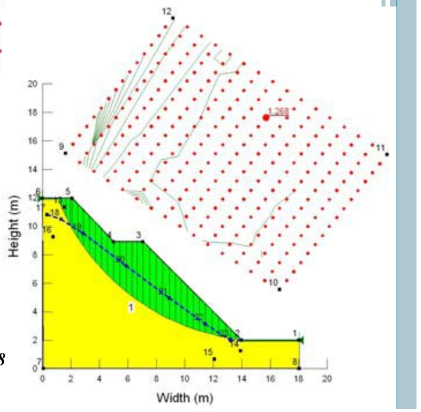
10.0 m Cut without water table effect *(1:1.2)*

FOS = 1.*613*



10.0 m Cut with water table effect *(1:1.2)*

FOS = 1.*268*

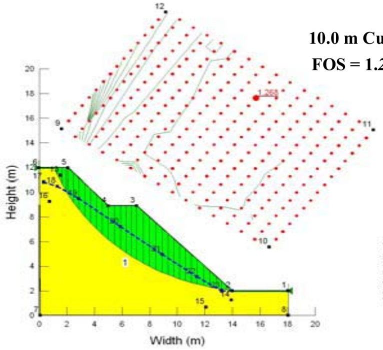


STABILITY ANALYSIS

In the event the water table rises above the failure surfaces, slope geometry should be modified to have a high FOS or some counter measures should be taken to lower the water table (i.e. introduce horizontal drainage system)

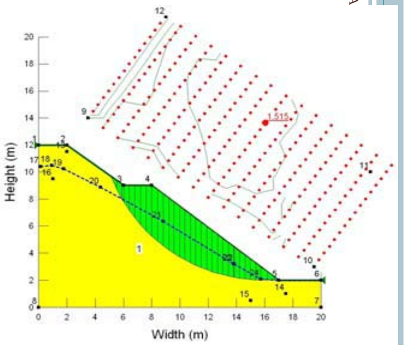
10.0 m Cut with water table effect *(1:1.2)*

FOS = 1.*268*

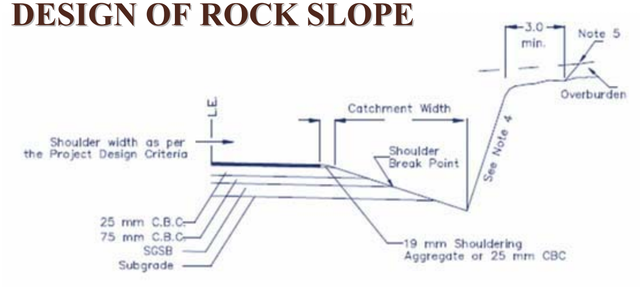


10.0 m Cut with water table effect *(1:1.3)*

FOS = 1.*515*



DESIGN OF ROCK SLOPE



Recommended Catchment Widths

Slope Height (m)	Catchment Width (m)
0 – 8	3
8 – 16	4
16 – 24	5
24 – 32	5
Over 32	5

Reference: Highway Design Manual – British Columbia

CONSTRUCTION

- Correct construction sequence
- If the designed cut profile changes with the actual, redesigning of slope has to be carried out
- Remove all unstable boulders

CUT SLOPES WITH HETEROGENEOUS LAYERS

SEEPAGE

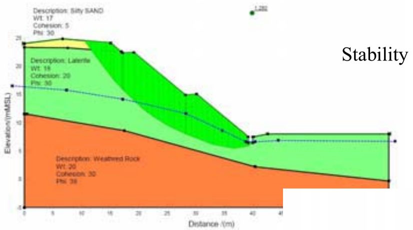
SEALING OF CRACKS

MAINTENANCE

Steps required to maintain the FOS

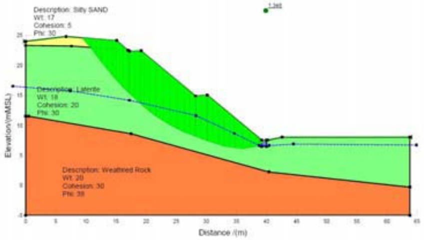
- Derange management (surface /subsurface)
- Surface drainage (cut off drain, berm drain, cascade, surface drain etc)
- Functing of weep hole –subsurface drain
- Turffing, Hydroseeding,
- Frequent site visit- check the slope, check list, any crack have to seal etc..

SUBSURFACE DRAIN CONSTRUCTION



Stability after excavation for Trenches

FOS increases from 1.2-1.3



Stability after completion of subsurf
drains

ACKNOWLEDGEMENTS

- Project Director, Southern Transport Development Project (STDP), RDA
- Deputy Directors (JBIC), Package 1 & 2, STDP, RDA
- Team Leader, Oriental Consultants Limited, STDP
- Project Manager, Taisei Corporation, STDP
- General Manager, Maruyama Industry Co. Ltd
- Director General , National Building Research Organization

THANK YOU!

The Role of Geotechnical Engineers in the Piling Industry

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Abstract

The advantages of an integrated approach for the different stages of the piling process: Site investigation; Preliminary design; Construction; and post construction testing are investigated. The importance of the site investigation data at each stage of the piling process is highlighted and thus the importance of the role of the geotechnical engineers in making correct decisions at the crucial times are discussed. Some important aspects of the construction process are discussed and guidelines are proposed to avoid the defects in piles due to improper construction practices. Certain aspects of post construction testing of piles are investigated and some guidelines are suggested for selecting number of piles for integrity and load tests.

1.0 Introduction

Piles are used to support structural loads due to the inability of the soil strata present at shallow depths to carry the applied loads using shallow foundations. In such subsurface conditions, the piles transfer the structural loads to more strong layers present at the lower levels of the subsurface. As piles highly rely on the capacity mobilized at the lower levels of the ground, the designer should be familiar with the different soil strata present in the subsurface to design an appropriate type of a pile foundation. On the other hand, piles are installed to deeper levels of the ground through very difficult subsurface conditions and the construction process has to be done from the ground surface. The variability of the subsurface and bedrock condition across the site is another factor that has to be considered during both design and construction stages. As it is not possible directly to observe and control the construction process, the construction crew must use some indirect quality control measures during the construction process of piles. Two of the defective piles exposed from excavation are shown in Figure 1. These two piles are from the same site and defects are found at about 3 – 5m depth from the ground surface.

Figure 1 – Defective piles exposed by excavation.

Due to these uncertainties associated with the design and construction of piles, some post construction quality assurance tests are performed. In this context, it is extremely important that well qualified and adequately experienced personnel are involved in all stages of the piling process.

It is often found that the different stages of the piling process are carried out independently and an integrated approach with information from one stage feeding the other stages are required to achieve a successful foundation at an economical cost. Furthermore, as bored and cast in-situ concrete piles are very frequently used in Sri Lanka, the discussion here is based on bored and cast in-situ concrete piles. However, the concerns raised here are equally valid for other types of piles as well.

2.0 Main Steps in the Piling Process

The major stages involved in the piling process can be divided into the following main steps:

i. Site investigation,
ii. Design,
iii. Construction, and
iv. Testing.

The steps given above are not independent to each other, rather heavily interdependent. It is observed that the most of the problems related to piles are due to compartmentalization of the above steps and not using the feedback (or the results) of one step in planning and executing the other steps. The flow chart given in Figure 2 shows the main sub-steps involved in the design stage of the pile foundation. Similarly, the flow chart in Figure 3 illustrates the main sub-steps involved in the construction stage of the piles.

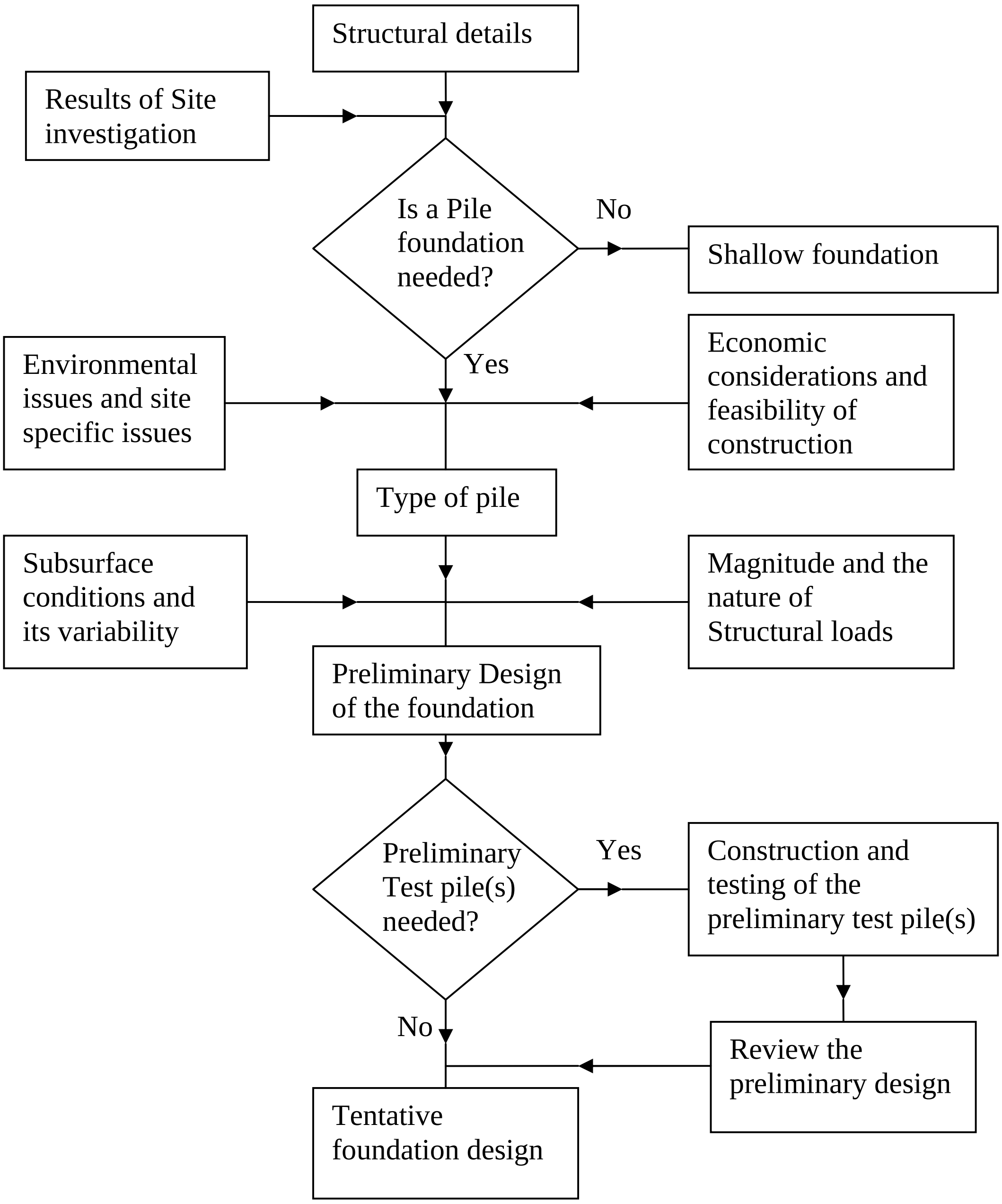


Figure 2 – Flow chart illustrating the main sub-steps in the design stage.

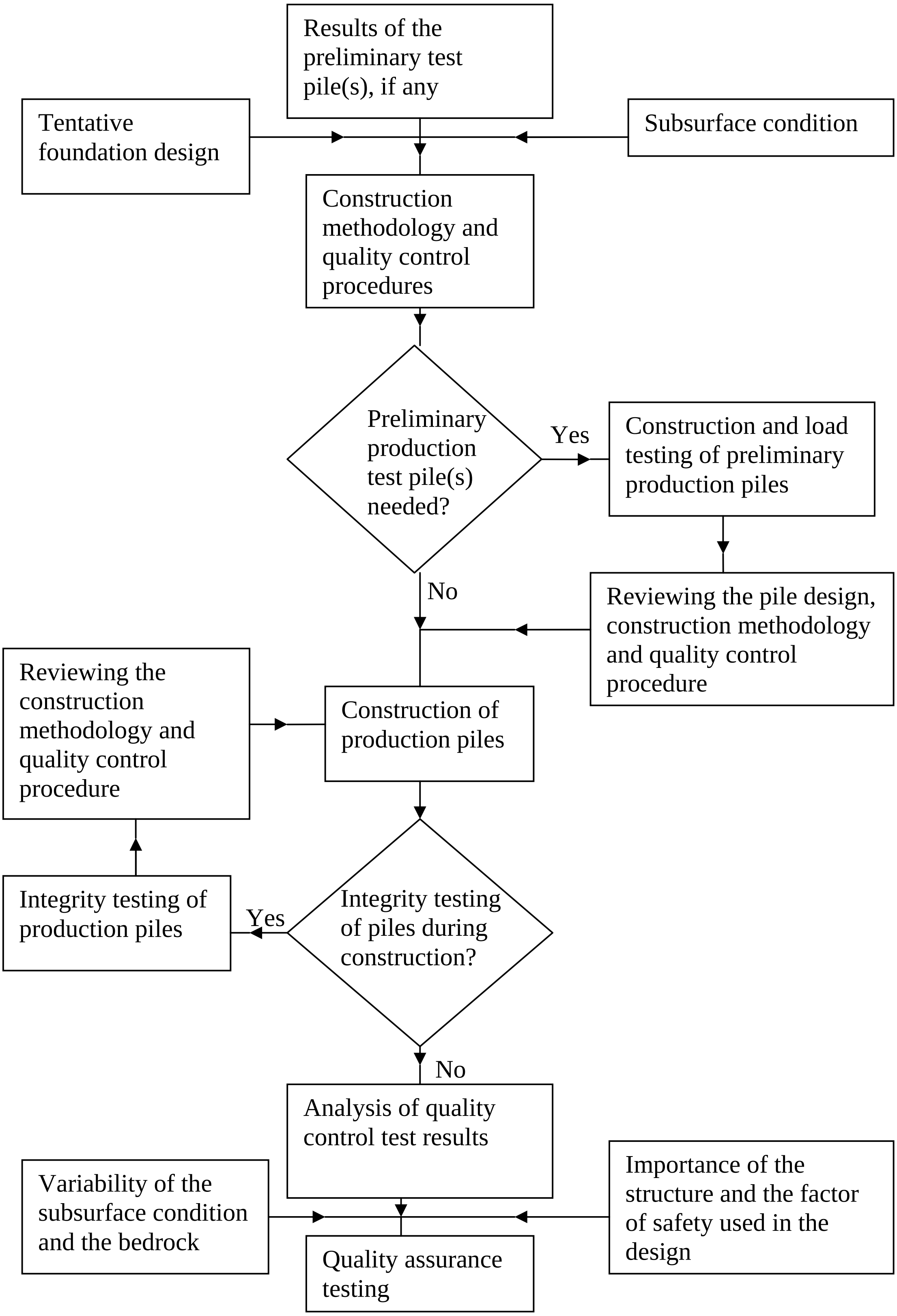


Figure 3 - Flow chart giving the main sub-steps in the construction stage

The flow charts given above show the interdependency of the main steps in the design and construction of pile foundations. It is very clearly demonstrated above the use of the site investigation data in planning and execution of most of the other main steps and sub-steps. The author believes that the engineer(s) involved in the piling related work should be qualified and experienced in the interpretation of the site investigation data. Moreover, they should be capable of arriving at rational decisions, based on the subsurface conditions at the site, in planning and execution of each stage of the piling work. Therefore, engineers involved with the piling work should be suitably qualified and experienced 'Geotechnical Engineers'.

The construction and testing are divided into two stages in the previous categorization. The flow chart showing the sub-steps involved in the construction stage clearly shows that certain amount of integrity and load testing is carried out during the construction stage as well. The testing during construction is carried out as a quality controlling measure rather than for quality assurance purposes. The results of the integrity testing during the construction stage give an indication about the suitability of the construction process adopted. The results of the load testing of preliminary production piles may be used as a measure to gauge the accuracy of the assumptions made in the design process and the suitability of the construction process.

Results of the pile tests during the early stage of the construction can be very effectively used to correct any deficiencies in the tentative pile design and to obtain feedback to improve the construction process and the quality control procedure. On the other hand if defective piles are identified at an early stage of the construction, appropriate remedial measures may be carried out with minimum delay in the construction time. Then, somebody might argue that it is advantages to do all the pile testing during early stage of the construction, i.e. 'earlier the better' concept. But one should consider the following facts before deciding in favor of that concept.

i. The piles that are tested in this manner are the piles which are earliest constructed piles and may not be the the most critical piles in terms of the quality control measures and other concerns. Therefore, cannot be taken as 'proper quality assurance' testing.
ii. Defects that occur between the time period from construction and casting the pile cap (incorporating the piles to the main structure) are not identified if testing is carried out during the early stage of the construction process.

Another important fact to note is the dynamic nature of the piling process. The design at the end of the design stage is termed as the 'Tentative Design' as it may be reviewed based on the additional information gathered from the results obtained from the testing of preliminary production test piles and other information revealed during installation of production piles. It is important to investigate the factors that have to be considered during each stage of the piling process.

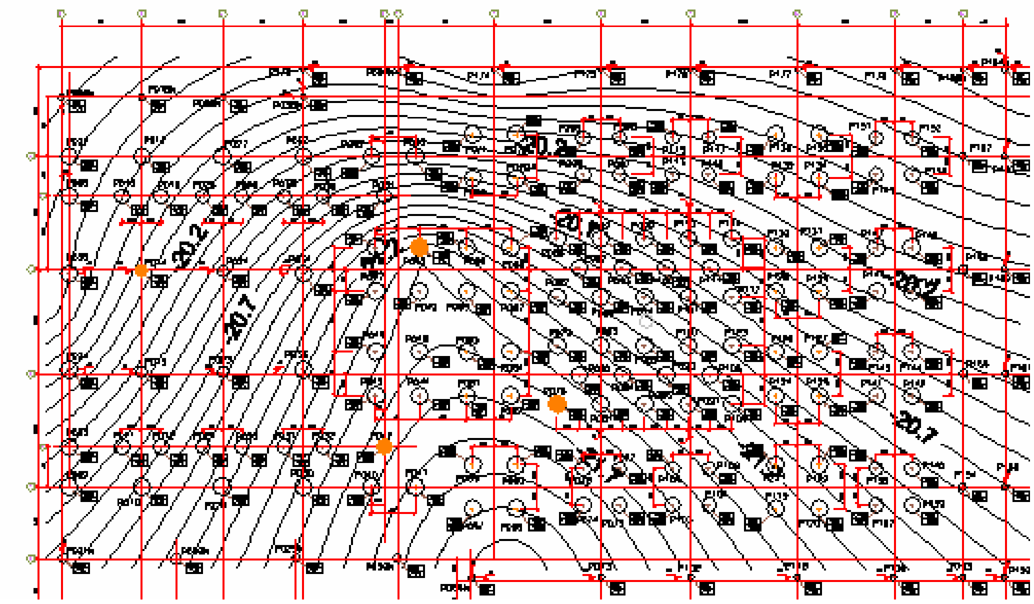
2.1 Site investigation

As shown in the flow charts given in Figures 2 and 3, it is clear that the site investigation provides very vital information regarding the subsurface and bedrock conditions at the site. The information gathered during the site investigation stage is repeatedly used during all the stages of the piling process. Therefore, planning and execution of a suitable site investigation program is a key factor in the construction of a sound pile foundation for a given structure. Unfortunately this aspect is given very low level of attention at present and that has caused most of the problems associated with pile foundations.

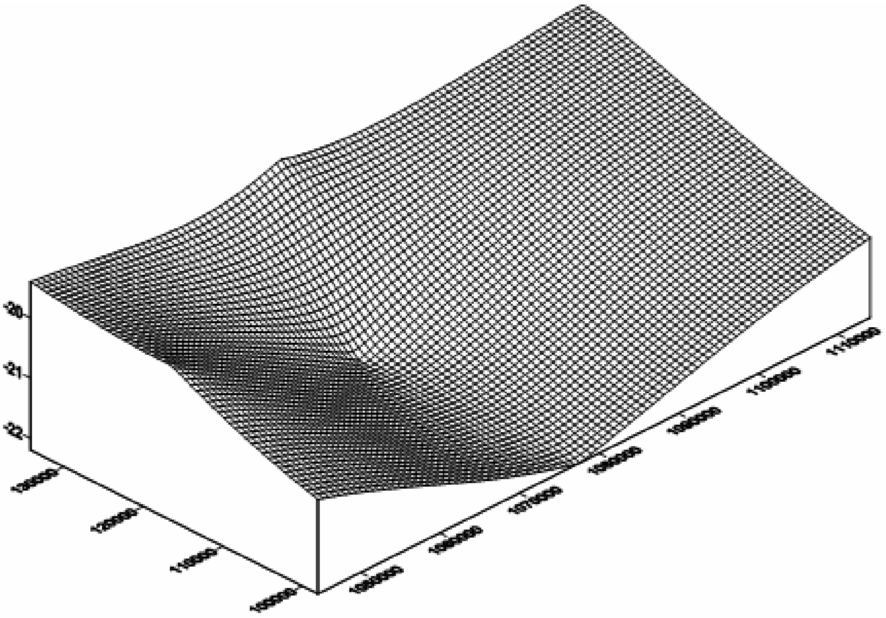
The key factor in the planning a suitable site investigation program is that the results of the site investigation program should show the critical variations in the subsurface and the bedrock profiles. The level of the details of the site investigation program depends also on the magnitude of the superstructure loads that are transferred to the ground. The geotechnical engineer, who is in charge of the site investigation stage, should carry out a thorough preliminary site investigation prior to the actual field investigation to gather information to plan the field investigation. It is true that the type of the foundation is decided based on the results of the site investigation. But the geotechnical engineer should have a certain idea about the probable type of the foundation during the planning stage of the site investigation program. The site investigation program prepared initially should be a flexible one and it should be reviewed based on the information gathered during the site investigation stage itself.

It is sometimes seen that the site investigation program was planned thinking that the probable type of the foundation is a shallow foundation and the field investigation is limited to the subsurface without rock coring. But after the analysis of the structural loads and the subsurface information, it is found that the shallow foundations are not suitable and end bearing bored pile foundations should be used. As rock coring was not done during the early site investigation program, a fresh site investigation program should be carried out to obtain the necessary design parameters. However, if the geotechnical engineer had reviewed the results of the field investigation program while it is in progress, the site investigation program would have been modified to obtain the necessary design parameters.

A frequently asked question is “how many boreholes are needed?” This is a difficult question to answer. There is no mathematical relationship or any other reliable criteria to estimate the number of boreholes. It depends on many factors and among them important factors are: the type of the strata present at the site, variability of the ground conditions across the site, magnitude of the transferred structural loads, and the type of the structure. For an example, consider the following case study. The contour map and a 3D view of the competent bedrock profile shown in Figure 4 is obtained using 6 boreholes drilled at a site for a high-rise (more than 35 storey high) residential building in Colombo, Sri Lanka. Figure 5 shows the contour plan and a 3D view of the competent bedrock profile establish from the pile lengths at the same site.

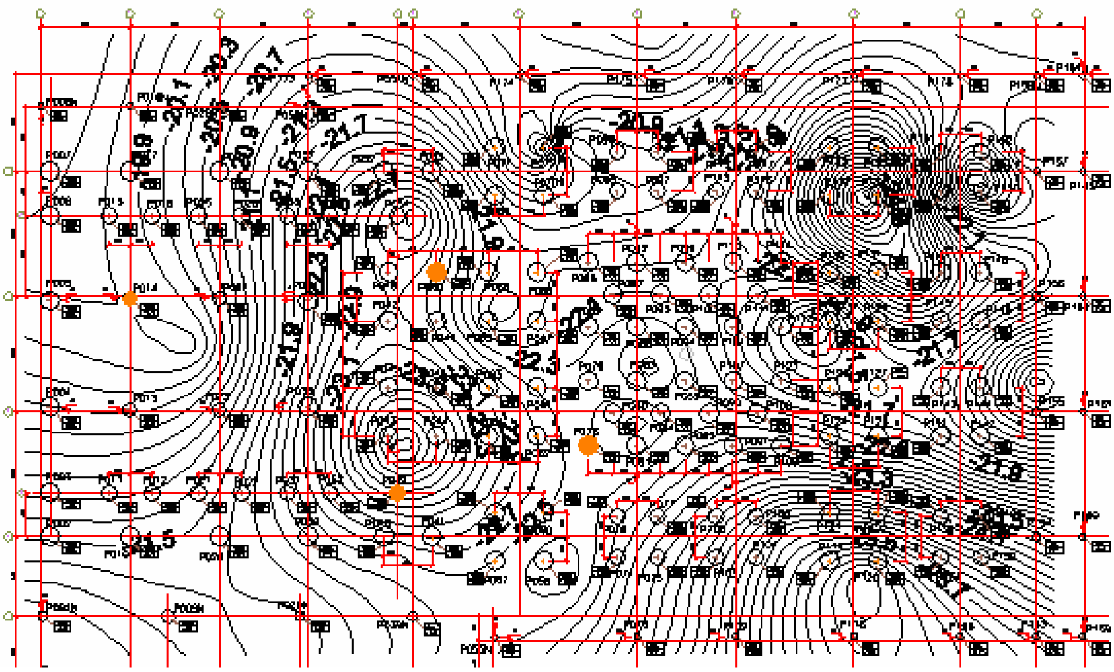


(a) Contour map.

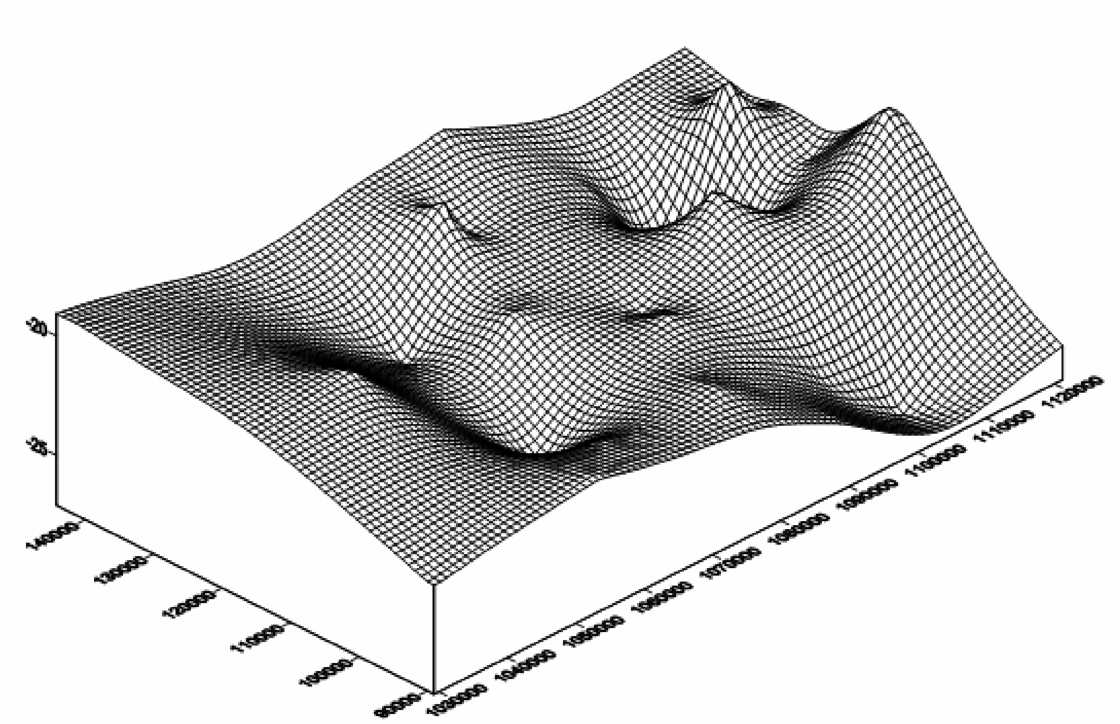


(b) A 3D view.

Figure 4 – Competent bedrock profile establish from 6 boreholes drilled at a high rise residential building site in Colombo, Sri Lanka.



(a) Contour map.



(b) A 3D view.

Figure 5 – Competent bedrock profile established from the pile depths at the same high-rise residential building site in Colombo, Sri Lanka.

Comparison of the bedrock profiles shown in Figures 4 and 5 clearly indicates some of the key local variations in the bedrock that have been missed in the site investigation program. The author has seen more drastic variations in the bedrock profiles that have been missed in the site investigation stage. The main reasons for such shortcomings in the site investigation program are:

i. The fixed nature of the most of the site investigation programs, and
ii. Not reviewing the results of the site investigation data during the field investigation stage.

Most of the site investigation programs specify a certain number of boreholes to be drilled at specified locations and terminate the boreholes after coring the rock certain number of meters irrespective of the bedrock quality. Rock coring is very critical if rock socketed end bearing bored piles are to be used. The author has seen certain site, where minimum rock coring is done to a lower RQD during site investigation stage and piles are specified to be terminated at the bedrock with a higher RQD. This is totally unacceptable and the piling crew is having no reference depth or RQD value to go by. It is also observed that the RQD of the bedrock is highly variable and at some places RQD sometimes decreases with the depth. Therefore, following three facts are emphasized in planning and execution of site investigation programs:

i. Carry out thorough preliminary site investigation before planning the field investigation stage and study the magnitude of the structural loads that are expected from the structure. Prior to the planning of the site investigation, the geotechnical engineer should have an idea of the probable type of the foundation and the range of the capacity expected,
ii. Planning a flexible site investigation site investigation program based on the findings of the above mentioned preliminary site investigation. The planed site investigation should match the type of the probable foundation type and the expected carrying capacity,
iii. Reviewing of the results of the field investigation stage and improve the site investigation to achieve the required objectives.

The results of the site investigation phase forms the basis for the design and construction of the foundation. Unfortunately, the importance given to the site investigation is at a very low level. Inadequate planning and investment in the site investigation program have led to most of the costly foundation remedial measures. Moreover, minimal use of the site investigation results beyond the design stage is another costly mistake of most of the engineers.

2.2 Design of pile foundations

Design of the foundation depends to very large extend on the ground condition at the site and the proposed structure. As shown in the flow chart given in Figure 3, the type of the foundation should be selected after giving due considerations to the following facts:

i. Type of the structure, magnitude of the working loads, nature of the loads that are acting on the foundation,

ii. Subsurface condition, quality of the bedrock, variability of the ground conditions across the site,
iii. Environmental considerations and other site specific issues, and
iv. The other economic and practical considerations.

If it is decided to have a pile foundation, the type of the pile foundation should also be decided based on the above considerations.

The designer should first check whether adequate information is available to carry out the design confidently. The assumptions made should be realistic and backed by sound justifications. For example, it is not reasonable to assume that a RQD of 75% will be achieve at a certain depth when the maximum RQD reported in the site investigation stage is less than 50%. Mostly engineers, who have minimal experience in pile construction work, come up with unrealistic and unachievable designs. If the engineer believes that sufficient site investigation details are not revealed through the site investigation, more investigations should be specified rather than carrying out a substandard design based on insufficient data.

The foundation recommendations given in the site investigation reports should also be considered with caution. The designer should critically look at the recommendations given in the site investigation reports considering the rational behind such recommendations and the 'person' making the recommendation. In the Sri Lankan context, all the structural configurations and other considerations are not conveyed to the party responsible for site investigation and hence, in the opinion of the author, such recommendations should be considered only as a guideline. The actual designer, who is having an overall picture of the structure, is in a better position to accurately assess the requirements and make rational decisions. In using the recommendations given in the site investigation reports, following factors should be considered:

- Recommendations may be based on individual pile not pile groups and certain recommendations may not be valid for pile groups,
- According to some design methods certain design parameters such as, skin fiction and end bearing capacities depend on the size of the pile,
- Certain parameters such as modulus of subgrade reaction depends on the pile diameter and the depth, and
- Some design calculations such as settlement of foundations depends on the size of the foundation.

It is better if the foundation designer extracts basic soil properties from the site investigation reports and carries out a fresh design using basic principles. It is observed that certain key factors such as the possibility of the development of the negative skin friction is overlooked in some of the pile designs. The designer should look into the possibility of the development of consolidation settlement of the surrounding soil and if required, incorporate the negative skin friction in the design. This type of a design is possible only if the pile design is carried out by experienced geotechnical engineers.

The other main consideration is the methodology adopted in the design process. There are various design methods available to estimate the carrying capacity of piles. None of these methods were developed nor the applicability verified for the ground conditions encountered in Sri Lanka and the construction methodology adopted in Sri Lanka.

Mohotti (2009) investigated the accuracy of some of the commonly used methods to estimate the ultimate skin frictional resistance of bored and cast in-situ piles based on a case study.

Ultimate carrying capacities of driven piles are mostly estimated based on the pile driving equations. Thilakasiri et al. (2009a) investigated the accuracy of the capacity estimated from commonly used pile driving equations by comparing with the capacity estimated from the high strain dynamic load testing using PDA® and CAPWAP®. Based on the analysis Thilakasiri et al. (2009a) concluded that the ENR method gave the best capacity prediction out of the methods considered in the study. Moreover, it was shown that the average capacity of the ENR and the Hiley capacities yielded slightly better capacity prediction compared to the measured capacity from signal matching the high strain dynamic test data using CAPWAP®.

2.3 Construction of piles

As previously discussed in section 1.0, pile installation takes place under harsh conditions at deep levels of the ground away from the human eye. The quality of the constructed pile is controlled through some indirect quality control measures. If a thorough site investigation program had been conducted and the variations of the stratification of the ground are well established, planning and execution of the pile construction methodology becomes quite easy. Pile failures very often occur due to faulty construction procedures. If test piles are installed at the site the construction procedure adopted during the construction stage of the test pile(s) provides very valuable feedback information regarding the construction related problems at the site. Therefore, the systematic collection of the information during the construction stage of test piles is of vital importance. The information obtained during the installation of test piles should be analyzed and appropriately used in planning the construction stage of the production piles.

One of the main deficiencies in the Sri Lankan piling industry is the minimal use of the available subsurface information in planning the construction of piles. Mostly the contractors use their typical construction process irrespective of the conditions at the site. The adjustments that are needed according to the site conditions may be minor but may be critical in the final analysis of the soundness of the constructed foundation. Some of the factors to be considered in the construction of bored and cast in-situ piles are:

i. The presence of cohesionless soil layers in the subsurface and the nature of such layers. Due to the high probability of collapsing of the sides in loose to medium sandy soils following construction aspects should be changed accordingly:
   a. Depth of temporary casings,
   b. Frequency of testing of the drilling mud,
   c. Head of the drilling mud in the pile bore,
   d. Cleaning of the pile bore before concreting etc.

ii. The presence of soft organic or sensitive clay layers in the subsurface. In soft organic soil layers bulging of the pile is possible and in soft sensitive clays ‘necking’ is possible.
   a. Use of permanent casing in very soft organic layers,

    b. Use of temporary casings in soft sensitive clay layers
- iii. Type and the nature of the bedrock at the site.
    a. Termination criteria of piles.
- iv. Variability of the subsurface and the bedrock profile across the site,
- v. Amount of information available through site investigation.

In the case of driven piles, some of the concerns are:

- i. Pore pressure buid up during driving, if cohesive soil layers are present in the subsurface,
- ii. Presence of soft layers bellow very hard layers may lead to breakage of piles due to the tensile reflections,
- iii. Presence of obstructions such as small boulders in the subsurface,
- iv. Identification of the end bearing layer (the termination criteria), and
- v. Maintaining the alignment of the piles.

Establishing a termination criteria and modifying it across a variable subsurface conditions is a common problem with both types of piles. The load vs settlement curve of a ‘failed’ rock socketed bored pile during static load testing is shown in Figure 6. The intended test load on the pile is 3750 kN but could not be reached. As it is shown by many researches (Tomlinson, 1994 and Bowles, 1996), the skin friction is mobilized first and subsequently the load is transferred to the pile toe. Moreover, the ultimate skin friction is reached at a relatively low settlement of less than about 10mm.

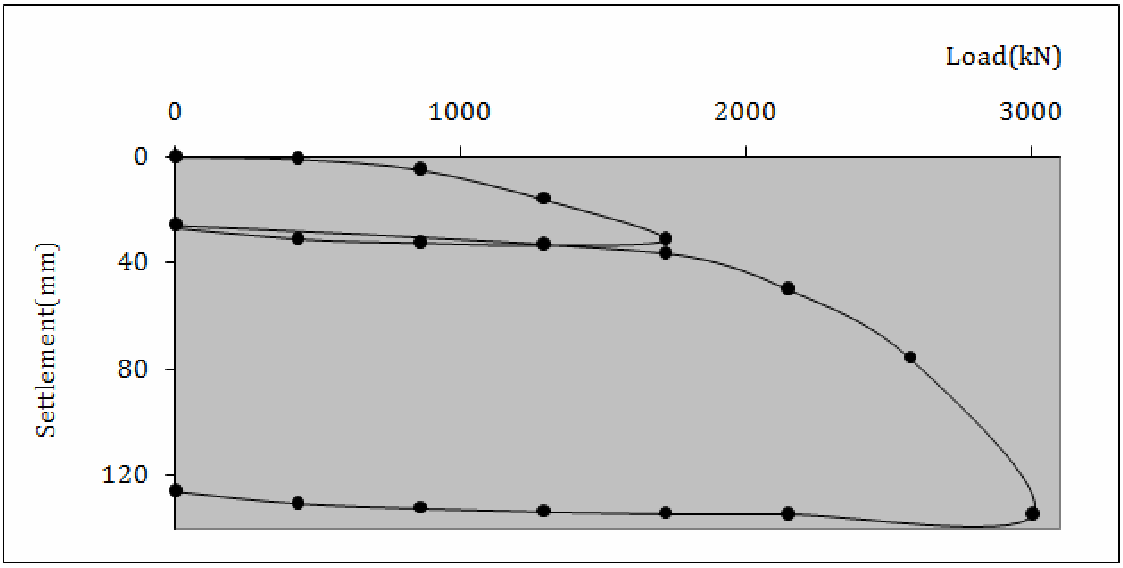


Figure 6 – Load vs settlement curve of a ‘failed’ bored pile.

The load vs settlement behavior of the pile, given in Figure 6, shows a stiff response initially and with the increase in the load the stiffness of the pile response decreases. The pile starts to undergo large deformation as the load is transferred to the pile toe. Clearly this pile is not resting on competent bedrock and the termination of the pile is to be blamed for this.

As illustrated in Figures 4 and 5, the bearing layer profile established through initial site investigation may be sometimes misleading. Local variations may not be identified through few isolated investigation locations. In large projects, secondary site investigation to supplement the initial site investigation may be performed to obtain additional information as required. Therefore, the variation of the layer profiles encountered during the installation of the piles should be mapped and any outliers should be investigated. The installation of the piles should be started at a location close to a borehole location of the site investigation program. As the ground profile at this location is shown, pile can be terminated at the appropriate depth. Piling should progress radially outwards from these locations while mapping the bearing layer profiles.

Investigations of the load vs settlement behaviors of some of the 'failed' bored piles show a different type of failure. For example consider the load settlement curve of a 'failed' large diameter rock socketed bored pile shown in Figure 7. This pile was intended to be loaded to twice the working load. But when the pile was loaded from WL to 1.25WL, it underwent large displacement with very minimal increase in the resistance. Here again as the load is transferred to the pile toe, the pile underwent a large displacement. Due to the sudden nature of the displacement, the pile does not show a gradual yielding type settlement as shown in Figure 6, but shows a sudden settlement most probably due to very weak toe conditions.

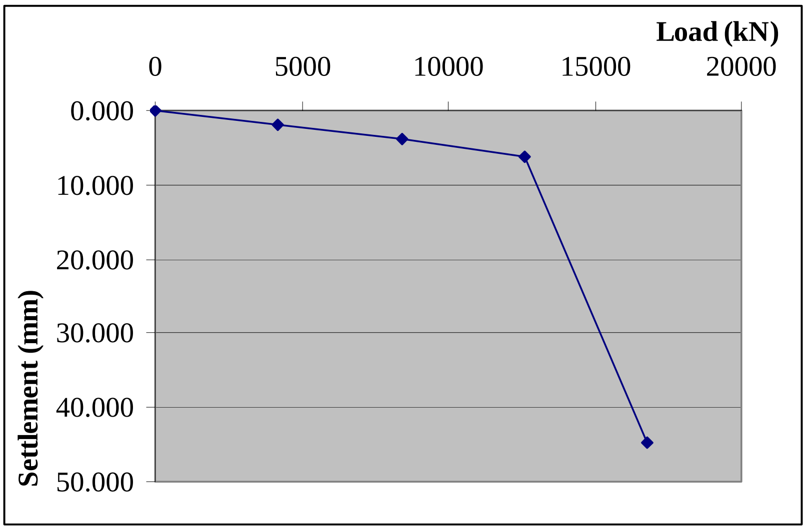


Figure 7 – Load vs settlement behavior of a bored pile due to 'Soft pile toe'.

This type of failure is not due to termination errors rather improper cleaning of the pile bore prior to concreting. This is one of the most common reasons for failure of bored piles. The drilling debris mixed with the drilling mud may settle to the bottom of the pile bore after the drilling process. The accumulation of the debris is more if the subsurface consists of cohesionless soil layers and/or there is a large time gap between end of drilling and concreting. Therefore, it is extremely important to clean the pile bore prior to concreting, as denser material present at the pile bottom may not be replaced by concrete poured into the borehole. The pile bore should be cleaned using circulation of fresh bentonite or air lifting technique and the quality of the drilling mud should be checked prior to concreting. It is sometimes observed that cleaning through the tremie placed only at the center of the pile bore is not sufficient for large diameter piles and cleaning should be carried out at off center locations as well.

Other quality control procedures during the construction stage are also important. Consider defective pile exposed by excavation, as shown in Figure 8. The defect may have been caused by the lack of flow of the concrete out side the tremie pile. The concrete may be designed and mixed according to the specifications. But what matters most is the quality of concrete when it is placed in the borehole. Quality control procedures at the site are extremely important.

Figure 8 A defective pile exposed by excavation.

Some of the important measures that are essential for the construction of defect free pile shaft are:

i. Design and fabrication of the reinforcement cage according to the specifications:
    a. Cover to the reinforcements, and
    b. ‘Effective’ openings for flow of concrete out of the reinforcement cage.
ii. Proper mix design of the concrete to achieve the required properties:
    a. Strength,
    b. Flow characteristics,
    c. Segregation, and
    d. Self compaction.
iii. Quality control testing at the site:
    a. Measurement of the properties of the drilling mud, and
    b. Measurement of the properties of the concrete.
iv. Other quality control measures at the site

a. Quantity of concrete used to construct the pile,
b. Height of the pile above the cut off level,
c. Embedment depth of the tremie pipe, and
d. Withdrawal of the temporary casings.

3.0 Post Construction Testing

There are certain uncertainties associated with the design and construction of piles. Post construction quality assurance is an essential part of the piling process to assess the quality of the piles constructed. As previously discussed in section 2.0, both the accuracy of the assumptions made during the design stage and the suitability of the construction methodology adopted are assessed by post construction testing of piles. There are a few ‘questions’ to be answered related to testing of piles.

i. What are the types of the post construction tests to be used?
ii. How many piles to be tested from different testing methods?
iii. How the piles are selected for testing?
iv. When should these tests be carried out?
v. What methodology is adopted to evaluate the results of post construction testing?

Answers to the above questions are not totally independent. However, answering all the above concerns is not possible in a short paper. Therefore, only one fundamental important aspect, number of piles to be tested from a site, is discussed here. Interested readers are referred to Thilakasiri (2009b) for further details.

3.1 Number of piles to be tested

Mainly two types of post construction tests are conducted: integrity tests and load tests. Often the questions are asked at the conferences and seminars “what percentage of piles should be tested for integrity?” and “what percentage of piles should be load tested from a site?”. One should deeply look into the rational behind post construction testing to answer these questions. The main thinking behind post construction testing is to minimize the probability of pile failure and assure that the constructed foundation and its components are having reasonable factors of safety against failure. It is essentially a probabilistic approach one has to use. However, the defective piles are not randomly distributed across all the sites or even within a site. For example finding a defective pile from a pile with unfavorable ground conditions will be higher than the finding a defective pile from a site having favorable ground conditions. Furthermore, probability of finding a defective pile across the site may also vary depending on the variability of the ground conditions across the site.

All the factors that affect the probability of failure of piles should be considered in arriving at a suitable pile testing program for a given site. Some very important factors to be considered are:

i. The ground conditions at the site; its variability across the site; and the availability of the information of the subsurface,

ii. The importance of the structure and the magnitude of the loads transferred to the ground,
iii. The results of the quality control measures adopted at the site and other relevant information obtained during the installation of piles,
iv. Degree of supervision during piling,
v. Factor of safety used in the design,
vi. Experience of the piling contractor on similar projects under comparable conditions, and
vii. Results of the already tested piles.

The cost of conducting small strain dynamic integrity tests is relatively low and such testing can be easily conducted without much preparation of the pile. On the other hand, load testing piles is expensive and during load testing the pile is subjected to the loading conditions experienced under working conditions. Therefore, the load testing can be considered as the ultimate quality assurance measure one can adopt. It is advisable to decide the number and the piles to be load tested after considering the results of al the other quality control and quality assurance tests including the integrity testing.

3.1.1 Number of piles to be tested for integrity

Williams and Stain (1987) considered the factors that should be considered in selecting number of piles for integrity testing. They suggested a method to select number of piles for integrity tests based on the following two factors:

i. Possibility of allowing certain percentage of defective piles in the foundation without compromising the factor of safety, and
ii. The number of piles at the site.

According to Williams and Stain (1987), if defective pile cannot be allowed (no redundant piles in the foundation), all the piles should be subjected to integrity testing. If defective piles can be allowed without reducing the design factor of safety, they suggested to select the number of piles to be tested based on the number of piles in the site. In this case, Williams and Stain (1987) suggested testing all the piles if the total number of piles is less than 30. On the other hand, if the total number of piles at the site is more than 30, Williams and Stain (1987) suggested testing first 30 piles and subsequently a random sample of not less than 30% of the total number of piles. If any defective piles are found in the sample, Williams and Stain (1987) suggested testing 100% of piles.

Williams and Stain (1987) criteria may be used for deciding the number of piles to be tested. Comparison of the design factor of safety, which generally falls between 2 to 3, should be compared with the factor of safety from the design calculations in arriving at the number of defective piles that can be allowed. However, the ability of the redistribution of the load among piles should also be considered. The author feels that the conditions considered in arriving at the above criteria may be different from the conditions encountered in Sri Lanka. The ground conditions found in Sri Lanka are very much favorable compared to the same found in most of the other parts of the world. If the concerns mentioned above (factors *i* to *viii* listed above) are investigated and found to be positive (or favorable) testing of first 30 piles and testing a selected sample of 30% of the remaining piles is acceptable. However, if any of the piles tested is found to be defective, 100% of the piles should be tested for integrity. On the other hand piles, which are vital to the stability of the structure, must be tested for integrity using appropriate techniques.

3.1.2 Number of piles to be load tested

Load testing of a pile is the highest level of quality assurance testing. Due to the high cost and the time restrictions, it is not possible to test all the piles in a site by load testing. Similar to the integrity testing, it is not reasonable to specify a fixed number of piles or a certain percentage of piles to be load tested from a site. As mentioned in section 2.0, the possibility of installation of preliminary test pile(s) should be considered. Civil Engineers (1978) suggested that at least one preliminary pile should be installed for each major grouping of piles, or at least one pile for each hundred working piles, on a large site. If it is not possible to have a preliminary test pile(s), the use of production pile(s) as preliminary production test pile(s) should be given due considerations. Even though results of load tests are considered as a quality assurance measure of the tested pile, the results of test pile(s), if available during the early stage of the construction program, can be used as a quality control measure to improve the preliminary pile design and the construction methodology, as shown in Figures 2 and 3.

It is always a good practice to subject certain number of piles to load tests to assure that the specifications are met. In the past about 2 to 4% of the piles are subjected to static load tests from a normal site. With the availability of cheaper testing methods such as High Strain Dynamic Test (HSDT), more piles could be tested at a comparable cost. The number of piles to be subjected to load testing should be decided based on the considerations listed in section 3.1. The piles should be selected for load testing following a systematic procedure so that likely problematic piles are subjected to quality assurance tests. Pile construction records and quality control test results may provide vital information in this regards. If a selected pile subjected to load testing fails, the load test results should be interpreted to find the likely cause of failure. In such event, the available records should be studied to identify the piles likely having similar problems. A selected number of such piles should also be subjected to load testing so that the factor of safety of foundation supporting a part of the structure and the overall structure are not compromised

4.0 Conclusions

Interdependent nature of the different stages of the piling process: site investigation; design; construction; and post construction testing are illustrated in this paper and the advantages of an integrated approach in the piling process are highlighted. The information obtained from different stages of the piling process is used to make rational decisions at subsequent stages. It is argued that the site investigation data forms the basis flat form for the piling process. The information of the ground conditions gathered through the preliminary site investigation process is repeatedly used at each stage of the piling process to make crucial decisions. Therefore, it is amply demonstrated that the geotechnical engineers are in an advantages position in make correct decisions at the correct time during piling.

Important aspects during the construction stage are discussed. The importance of termination of piles and cleaning of boreholes for bored piles before concreting are investigated, and some guidelines are suggested to avoid failure due to improper construction practices. Post construction quality assurance testing is also discussed with the factors those should be considered in making crucial decisions related to post construction testing. Some guidelines are suggested to select the number of piles from a site for integrity and load testing.

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