Current Practices in Deep Foundations and Diaphragm Wall Construction in Thailand is published to mark the auspicious occasion of SEAFCO 40th Anniversary. It is a compilation of 48 selected published papers in the international conferences. The case studies and researched works presented in these papers are mainly from the projects carried out by SEAFCO in Bangkok and other parts of Thailand. Published papers authored by SEAFCO’s geotechnical engineers in other major projects and research studies overseas are also included.

In Thailand, according to the authors’ experience as a contractor, development in both construction and design aspects of wet-processed deep-seated bored piles, barrette foundations and diaphragm-walls in past four decades are significant. With recognition of technical and economic advantages of using these high capacity cast-in-place foundations by local practitioners, it is expected that they will be more popular in the future construction industry of Thailand and the region. However, in the authors’ opinion, there is much work to be done with particular focus on constructability issues, concrete technology for wet-processed bored piles and barrette, reliable but cost-effective quality control testing and application of value-engineering in design and construction. Starting from the planning stage, site investigation, design, construction and inspection should be integrated to enable designers, contractors and construction inspectors to participate as a team with a common goal. Appropriate and practical specifications should be established jointly by these parties for local soil conditions and construction methods. Continuing education should be promoted for designers, inspection engineers, and contractors.

SEAFCO is committed to continue its research-minded initiation to link theory and practice in deep foundations. We hope that researched works and findings presented in this book will be useful and serve as a source of reference to all in the field of geotechnical and foundation engineering, particularly those who are involved in the construction industry of Thailand.
Current Practices in Deep Foundations and Diaphragm Wall Construction in Thailand

Editors:

Zaw Zaw Aye
Aung Win Maung
Thayanan Boonyarak
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Nutchachai Prongmanee
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Member of
PREFACE

Seafco Public Company Limited is a deep foundation contractor established in 1974. The company has been involved in numerous foundation and deep excavation projects in Thailand, including large-scale foundation works, infrastructure projects, and more recently the underground structures of Bangkok’s second subway project (MRT Blue Line Extension Project). SEAFCO expanded its operation overseas and successfully took part in the construction of diaphragm walls for Marina Bay Sands in Singapore in 2007. SEAFCO has also established Seafo (Myanmar) Company Limited in late 2013 and completed two projects to date.

Consolidating state-of-the-art technology, sophisticated equipment and engineering professionals, SEAFCO initiated the innovative techniques and solutions in geotechnical and foundation engineering in Thailand and Myanmar. To improve the currently used methods, material, system, design concepts and construction techniques, SEAFCO has spent considerable time and effort on Research and Development activities.

SEAFCO’s R&D team continuously conducts research on bored piles, barrettes, diaphragm walls, associated testing methods, deep excavation and other geotechnical engineering works. New findings and research results have been presented in a number of conferences and seminars organized by various institutions both domestically and internationally. In forty years, R & D Division of SEAFCO has produced over 100 technical papers (both in English and Thai) which were presented and published in international conferences held in various countries. The purpose of this publication is to mark the auspicious occasion of SEAFCO 40th Anniversary. It is a compilation of selected published papers in the international conferences. The case studies and research works presented in these papers are mainly from the projects carried out by SEAFCO in Bangkok and other parts of Thailand. Published papers authored by SEAFCO’s employees in other major projects and research studies overseas are also included.

I would like express my appreciation to the clients, consultants, designers, universities, institutions, the Engineering Institute of Thailand under H.M. the King’s Patronage (EIT) and partners who have been continuously supporting SEAFCO for 40 years. I also would like to acknowledge the effort of SEAFCO’s R&D team for their commitment and hard work in the research works.

I hope that researched works and findings contained in these publications will be useful and serve as a source of reference to all in the field of geotechnical and foundation engineering, particularly those who are involved in the construction industry of Thailand.

Narong Thasnanipan
President & CEO
Seafco Public Company Limited
First of all, I would like to extend my congratulations to SEAFCO for 40 years of successful establishment, not only as a business entity but also as a great contributor for construction industry of Thailand.

SEAFCO is well known for research minded initiatives in foundation engineering. SEAFCO shares outcomes of its R&D works to the engineering society by publishing technical papers, organizing conferences, and delivering lectures in various institutions in both Thailand and overseas. I proudly acknowledge the contributions SEAFCO has been making to strengthen the foundation of civil engineering society in Thailand and beyond.

Since the time I was working on my doctoral research, I have found SEAFCO’s publications in various international conferences very useful to understand practical aspects of deep foundations. SEAFCO puts the innovative theory into practice by equipping itself with knowledge, experience and professionalism. SEAFCO R&D team has put in commendable effort to compile all of their published papers and produced the book *Current Practices in Deep Foundations and Diaphragm Wall Construction in Thailand*. Geotechnical specialists, foundation engineers, structural engineers and practitioners in construction industry will find this book beneficial.

I, as the President of the Engineering Institute of Thailand under H.M. The King’s Patronage (EIT), would like to sincerely thank Seafco Public Company Limited for endless support they have been providing to the various activities of EIT under the leadership of CEO Khun Narong Thasnanipan. In particular, SEAFCO’s great support in a leading role to Thailand Underground & Tunneling Group (TUTG) in winning the bid for Thailand to host the World Tunnel Congress 2012 and successfully organizing the event at Queen Sirikit National Convention Center in Bangkok from 18 to 23 May 2012.

I believe SEAFCO will continue to grow in business with trust and confidence of its stakeholders. I am also confident that SEAFCO will maintain its position as the leading innovators of foundation and geotechnical engineering.

I wish SEAFCO all the success.

Prof. Dr. Suchatvee Suwansawat

President

The Engineering Institute of Thailand
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Barrettes
Strip Piles

When single pile safe working load in excess of 2000 tons is required or conventional bored piling is impractical, barrettes or strip piles are the best alternative. Various dimensions, cross-sections and orientation can be formed and arranged to suit design requirements. SEAFCO barrettes are used in foundations for expressways, MRTA stations and high-rise buildings.
SEAFCO, a specialist in deep foundation and piling work in Thailand since 1974, constructs small diameter and large diameter bored piles to suit the types of designed structures. Small diameter bored piles which cover sizes from 350mm to 600mm have a load capacity of up to 180 tons. The large diameter bored piles larger than 600mm in diameter and up to 2,000mm in diameter have a capacity to carry loads in excess of 1,600 tons. With its solid knowledge and experience in bored piling and collections of up to date equipment SEAFCO can construct large diameter bored piles, as well as barrettes over 60m in depth. SEAFCO also provides design and build services for retaining walls with contiguous piles.

DRILLING EQUIPMENT

The drilling equipment used for bored pile construction are:
- Telescopic kelly equipped with auger.
- Bucket auger on kelly.

Selection of drilling tools depends on:
- Dimension of pile.
- Verticality.
- Site and ground conditions.
- Vibration limit.

Drilling in action for a 1.5m diameter bored pile.
SUBSOIL
Construction method and design of pile foundation are influenced by the subsoil conditions. The subsoil condition should be investigated in order to determine the soil parameters for pile design and selection of pile construction process such as wet or dry process.

DRILLING
The drilling of the borehole for bored piles is performed under the bentonite or polymer based slurry in wet process. A casing with an internal diameter the same as the pile shaft to be made, is installed by high frequency-low amplitude hydraulic vibro-hammer to protect top soil and soft soil layer. An auger drills inside the casing, transporting the soil simultaneously upwards out of the casing. Before reaching the bottom of the casing or water bearing cohesionless soil layer, slurry is introduced into the borehole. Then a bucket auger is used for drilling under slurry. For dry process, an auger or bucket auger is used depending on soil conditions without slurry.

STEEL REINFORCEMENT
When the required depth has been reached and the auger has been removed leaving the casing full of drilling slurry, the reinforcement cage with concrete spacers is placed in position. The reinforcement cage usually has a concrete cover of 75mm and clear spacing of 100mm between bars. Any sediment or loose soils at bottom of the borehole must be removed prior to installation of the reinforcement cage.
CONCRETING

Tremie method is used for concreting operation in wet process. Tremie pipes are lowered into the depth and plug materials are introduced into the tremie pipe to avoid contamination of concrete with slurry prior to concreting. Concrete is generally poured up to 1-2m above the cutoff level of bored piles. The casing is then removed by extraction force with care not to disturb the poured concrete and the pile is completed.

MATERIALS

Concrete: Self-consolidating concrete type
Aggregate: 5-20mm
Cementitious: not less than 380kg/m³
Slump: not less than 150mm and after 4 hours not less than 100mm
Strength: Cylinder strength 240, 280 or 320ksc at 28 days (depending on design requirements)

GROUTING

SEAFCO provides pile base grouting and other grouting services. Pile base grouting with cement milk reduces settlement by compacting the contact area between soil and pile base.

PILE TESTING

Pile testing is carried out for design and construction quality assurance.
- Static or Dynamic load testing with or without instrumentation to determine the load bearing capacity and bearing characteristics.
- Sonic Integrity (seismic) test and sonic core hole logging to inspect the integrity of pile and concrete quality.
- Destructive test, if the pile quality is suspected, a continuous core sample is taken to check the quality.

Table: Typical pile sizes and working load capacities in Bangkok Subsoils.

<table>
<thead>
<tr>
<th>Diameter m</th>
<th>Perimeter m</th>
<th>Section Area m²</th>
<th>Working Load Capacity* ton</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.60</td>
<td>1.88</td>
<td>0.263</td>
<td>80-150</td>
</tr>
<tr>
<td>0.80</td>
<td>2.51</td>
<td>0.503</td>
<td>250-400</td>
</tr>
<tr>
<td>1.00</td>
<td>3.14</td>
<td>0.785</td>
<td>450-600</td>
</tr>
<tr>
<td>1.20</td>
<td>3.77</td>
<td>1.131</td>
<td>650-800</td>
</tr>
<tr>
<td>1.35</td>
<td>4.24</td>
<td>1.431</td>
<td>800-1000</td>
</tr>
<tr>
<td>1.50</td>
<td>4.71</td>
<td>1.767</td>
<td>1050-1200</td>
</tr>
<tr>
<td>1.65</td>
<td>5.18</td>
<td>2.138</td>
<td>1200-1500</td>
</tr>
<tr>
<td>1.80</td>
<td>5.65</td>
<td>2.545</td>
<td>1400-1600</td>
</tr>
</tbody>
</table>

*Note: Working load capacities depend on pile length, subsoil conditions, concrete strength, factor of safety adopted, type of supporting fluid used, etc.
PREFOUNDED STEEL COLUMNS

Staged construction methods such as top down construction methods are adopted for deep basement construction with retaining walls. Steel columns are installed with adequate embedment in single bored piles during piling work to support parts of building structure which has been constructed before excavation reaches the basement formation level. Installation of steel columns accurately in position is done by using SEAFCO adjustable guide frame.

Installing steel reinforcement cage sections for a diaphragm wall panel with pile leg.

BARRETTEES

SEAFCO also provides barrettes (rectangular/strip piles) as an alternative to bored piles to suit the design requirements. Various shapes of foundation elements such as L, T, H, + and overall foundation layout can be formed and arranged with barrettes.

QUALITY CONTROL

Quality control generally covers the following:

- Slurry quality, verticality and dimension of borehole during drilling.
- Borehole base cleaning by slurry recycling under bentonite slurry or using cleaning bucket under polymer slurry.
- Position of reinforcement cage and connection.
- Concrete quality, volume and pouring rate during concreting.

PILE LEGS

SEAFCO also provides bored pile legs under cast in situ diaphragm wall panels as part of foundation system to meet the design requirements. After excavation of a diaphragm wall panel under bentonite slurry, bored piling proceeds. The pile leg and diaphragm wall panel are cast as a single element after installation of reinforcement cage.

General view of bored piling: Siam Paragon Project.
PILE BASE GROUTING IN BANGKOK SUBSOILS

Cast-in-situ bored piles with base grouting are in common use in Bangkok for foundations of heavy structures. Base grouting is usually employed in bored piles to improve pile capacity and performance. The unavoidable problems arising from loosening of soil and sedimentation at the base caused by boring can be minimized by base grouting. The first base grouted bored pile in Bangkok was used in 1985 at the Rama IX cable-stayed bridge across the Chao Phraya River.

Teparaksa, W. (1994) reported that base grouted piles in the second sand layer of Bangkok subsoils have an increment in their performance as below;

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Increment %</th>
</tr>
</thead>
<tbody>
<tr>
<td>In Failure Load</td>
<td>12-24</td>
</tr>
<tr>
<td>In Skin Friction</td>
<td>9-27</td>
</tr>
<tr>
<td>In End Bearing</td>
<td>11-21</td>
</tr>
</tbody>
</table>

The above increments were determined at fully mobilized skin friction 1.59-1.86% and 1.04% of pile diameter for base grouted and non-base grouted piles respectively.

Mechanism of improvement by base grouting involves;
- Replacement of sediments with grout under the pile base.
- Squeezing water out of the sediments.
- Improving the sand stratum around the pile base.
- Prestressing of the pile.
- Increasing skin friction in sand layers.

GROUTING ASSEMBLY

Current base grouting practice uses two U-circuits consisting of 12mm diameter PE tubes attached at the bottom with 12mm GI pipes with perforations wrapped with rubber sleeves (manchettes). They are attached with the reinforcement cage to place the manchettes at the base of the pile.

GROUTING CRITERIA

Grout Pressures: Maximum pressure developed is a single most important parameter. It should be gradually achieved by low injection rates to avoid hydraulic fracturing.

Grout Volumes: Usually total grout take is not a limiting criteria and is used to decide about the staged grouting. Generally, the higher the required maximum pressure to be achieved, the higher will be the grout volume under similar conditions.

Injection Rates: It is not a direct controlling parameter but instead used to achieve maximum pressure in a gradual manner. Injection rates of 5-10 litres/min are proved to be satisfactory.

Residual Pressures: Normally residual pressures of the order of 20-30% of target maximum pressure are desirable.

PERFORMANCE OF BASE GROUTED PILES

Base grouting with tube-a-manchettes technique has been used by SEAFCO in many projects in Bangkok. Improvements achieved have been proved by load tests on both non-grouted and base-grouted bored piles on the same project sites.

![Graph showing load vs. settlement for non-grouted vs. base-grouted piles](image_url)
Generally, total volume of grout at pile base ranged from 500 liters to 4000 liters at single or multiple stage grouting. Grouting pressure ranges from 25 bars to 60 bars depending on the depth of pile base level and relative density of sand layer in the project area.

References:
Object
Contiguous bored pile wall and bored piles for construction of basement and foundation of the Rama Thibodi Hospital.

Project Description
Bored pile wall with temporary braced excavation method for construction of 3 level basements for car parking.

Type of Work
Foundation Piling, Contiguous bored pile wall.

Owner
Rama VI Hospital.

Designer
Arun Chaiser Consulting Engineers Co., Ltd.

Main Contractor
Ch. Karnchang Public Company Limited.

Project Schedule
2005-2006

Construction Method
- Excavation with contiguous pile wall using bottom up construction method to −11.5m.

Construction Details
Foundation bored pile: 7 (Ø0.8mx55m)
121 (Ø1.0mx55m)
163 (Ø1.2mx55m)
32 (Ø1.5mx55m)
Contiguous Pile Wall: 364 (Ø1.0mx20.0m)

Subsoil Conditions
Soft clay: 0.0-11.0m
Medium clay: 11.0-14.0m
Stiff clay: 14.0-28.0m
Dense sand: 28.0-30.0m
Hard clay: 30.0-47.0m
Very dense sand: 47.0-55.0m
**MahaNakhon Deep Foundation Works**

**Project Description**
Barrettes, bored piles and diaphragm walls for a 75 storey high-end Hotel and Residence Tower with 2 level basements, and a retail complex with 4 level basements, located on Narathiwas Road.

**Type of Work**
Foundation piling and diaphragm walling.

**Owner**
Pace Project One Co., Ltd., Pace Project Two Co., Ltd. and Pace Project Three Co., Ltd.

**Project Schedule**
2011-2012

**Construction Details**
Foundation bored pile:
- φ0.80mx57-65m = 59 +11
- φ1.00mx57-65m = 13+14
- φ1.20mx65.0m = 4
- 0.80mx3.0mx65.0m = 1
- 1.20mx3.0mx65.0m = 114

Diaphragm Walls:
- 9,264sqm (0.6m to 0.8m thick x 16.0m to 22.0m)

**Subsoil Conditions**
- Soft clay: 2.0-8.0m
- Medium clay: 8.0-13.0m
- Stiff clay: 13.0-24.0m
- Dense sand: 24.0-38.8m
- Hard clay: 38.8-43.0m
- Very dense sand: 43.0-49.1m
- Hard Clay: 49.1-55.0m
- Very Dense Sand 55.0-67.0m

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No. 144, Prayasuren Road, Bangchan, Khlong Sam Wah, Bangkok 10510, Thailand. Tel. (662) 919-0090 to 97, Fax. (662) 919-0098, 518-3088. Email: seafco@seafco.co.th, Homepage: www.seafco.co.th.
Object
Construction of basement structures of a 42 storey office tower on 9 storey retail podium.

Project Description
Construction of bored pile foundations, diaphragm walls and deep excavation works for 2-3 level basements construction.

Type of Work
Foundation piling, diaphragm walling and substructure works.

Owner
Land and House Property Company Limited.

Designer
Palmer & Turner (Thailand) Limited.

Project Schedule
2008-2009

Construction Method
- Bored piling and diaphragm walling
- Deep excavation to –13.0m with diaphragm walls using bottom up construction method.

Construction Details
Foundation bored pile:  
- Ø0.80mx57m = 22
- Ø1.20mx57m = 65
- Ø1.50mx57m = 183
- Ø1.80mx57m = 25

Diaphragm Walls: 8,172 sq.m. (0.80m thick x 18m deep)

Volume of Excavation: 95,000 cu.m.
Temporary Bracing: 1,987 tons
Temporary Platform: 3,807 sq.m.
Concrete Volume: 24,295 cu.m. (substructure)
Steel Reinforcement: 4,350 tons (substructure)
Basement Floor: 30,909 sq.m.

Subsoil Conditions
Soft clay: 2.5-9.0m
Medium clay: 9.0-15.3m
Stiff to very stiff clay: 15.3-40.0m
Hard clay: 40.0-42.8m
Dense Sand: 42.8-60.0m
Object
Construction of foundation mats, footings, deep basements for car parking and wastewater treatment tanks.

Project Description
Diaphragm wall with 2-3 levels temporary bracing for deep excavation up to –15.90m to construct 3.5 level basements for car parking.

Type of Work
Deep excavation with temporary bracing, Civil Construction.

Owner
B&G Park Company Limited.

Designer
Palmer and Turner (Thailand) Ltd.

Project Schedule
March 2005– November 2006

Construction Method
- Braced excavation to –15.90m with diaphragm wall using bottom up construction method.

Construction Details
Temporary bracing: 1,478 tons
Earth excavation: 54,000 cu.m
Basement floor area: 13,000 sq.m.

Subsoil Conditions
Soft clay: 0.0-14.0 m
Medium clay: 14.0-15.0 m
Stiff clay: 15.0-24.0 m
Hard clay: 24.0-40.0 m
Very dense sand: 40.0 m+

Layout of temporary bracing.

Concrete casting of 3.0m thick foundation mat in progress.

Rebar fixing for foundation mat in progress.
Project Description
Bored piles and diaphragm walls for a 32 storey office tower and shopping complex with 2 to 4 level basements, located on Sukhumvit Road.

Type of Work
Foundation piling and diaphragm walling.

Owner
Bhiraj Buri Co., Ltd (UBC III) & Sukhumvit City Mall Co., Ltd. (EM2)

Project Schedule
2011

Construction Details
Foundation bored pile: φ0.60mx18.5m = 126
Bored Pile Wall φ0.60mx22-23.0m = 433
Bored Pile Wall φ0.80mx18.5m = 393
Foundation bored pile: φ0.80mx60.0m = 158
φ1.00mx60.0m = 30
φ1.20mx60.0m = 27
φ1.50mx60.0m = 308
φ1.80mx60.0m = 140
0.80mx3.00mx60.0m = 4
1.00mx3.00mx60.0m = 22
Diaphragm Walls: 7,057.5 sqm (0.8-1.0m thick x18.0-22.0m)

Subsoil Conditions
Soft clay: 1.5-13.5m
Medium clay: 13.5-16.3m
Stiff clay: 16.3-32.5m
Dense sand: 32.5-34.0m
Hard clay: 34.0-53.3m
Dense Sand: 53.9-58.0m
Object
Construction of a 2 level underground car park under a football field.

Project Description
Construction of bored pile foundations, diaphragm walls and deep excavation works for 2 level basements and football field on top.

Owner
Srinakrin Wiroj University

Architects & Engineers
Krunthep Tanakom Co., Ltd. (Designer)
Plan Consultants Co., Ltd.
Kumjornkit Construction Co., Ltd. (MC)

Project Schedule
2008-2010

Construction Method
- Bored piling and diaphragm walling
- Deep excavation to –7.3m with diaphragm walls using top-down construction method

Construction Details
Foundation bored pile: $0.60\times 18.5 = 508$
Diaphragm walls: $7,664$ sq.m. ($0.60$ m thick $\times 16$ m deep)
Excavation volume: $102,000$ cu.m
Basement floor: $36,000$ sqm
Concrete volume: $39,624$ cu.m
Reinforcement: $4,218$ tons

Subsoil Conditions
Soft clay: $2.0$-$14.5$m
Medium clay: $14.5$-$17.0$m
Stiff clay: $17.0$-$22.0$m
Hard clay: $22.0$-$41.5$m
Sand: $41.5$-$46.0$m
Hard clay: $46.0$-$49.0$m
Dense sand: $49.0$-$60.0$m

Substructure works in progress.
Object
Deep foundation work for wastewater treatment plant.

Project Description
Bored piles and diaphragm walls for construction of deep basements for a wastewater treatment plant in Bangsu, Jatujak area.

Type of Work
Foundation piling, diaphragm walling and soil improvement.

Owner
The Bangkok Metropolitan Administration.

Designer & Engineers
The Bangkok Metropolitan Administration.
CH. Karnchang Public Company Limited (MC)

Project Schedule
2008-2009

Construction Details
Foundation bored pile:
- ø0.80mx57.0m = 971
- ø1.00mx57.0m = 7
- ø1.00mx57.0m = 380
- ø1.50mx57.0m = 46

Diaphragm Walls:
- 2,460 sq.m (1.5m thick x 24.7m deep )
- 10,225 sq.m (1.2m thick x 24.7m deep )

Soil mixing:
- 1,734 cu.m

Subsoil Conditions
Soft clay:
- 2.0-10.0m
Medium clay:
- 10.0-13.5m
Stiff to very stiff clay:
- 13.5-24.0m
Dense sand:
- 24.0-60.0m
Object
Underground car park with disable access, natural light and ventilation for the Central Hospital of Bangkok Metropolitan Authority. Large open space and garden on the roof top.

Project Description
Diaphragm wall with top down excavation method for construction of 4 level basements for car parking and roof top garden.

Type of Work
Foundation Piling, Diaphragm Walling, Deep Excavation, Civil Construction.

Owner & Designer
The Bangkok Metropolitan Administration

Project Schedule
2005-2007

Construction Method
- Excavation with diaphragm Wall using top-down construction method to –13.0m.

Construction Details
- Foundation bored pile: 69 (Ø1.8mx22m)
- Diaphragm Wall: 4,900sq.m. (0.8m thick, 20.0m deep)
- Basement floor area: 11,200sq.m.
- Landscaping: 2,800sq.m.

Subsoil Conditions
- Soft clay: 0.0-11.0m
- Medium clay: 11.0-14.0m
- Stiff clay: 14.0-30.0m
- Dense sand: 30.0-37.0m
- Hard clay: 37.0-42.0m
- Very dense sand: 42.0-50.0m

Cross section of underground car park.
Object
Deep foundation works for a 47 storey luxury condominium.

Project Description
Bored piles, contiguous pile wall, diaphragm walls and soil improvement for construction of deep basement.

Owner
Sathorn Park Co., Ltd. & Grace Ivory Co., Ltd.

Designer
Meinhardt (Thailand) Ltd. (Structural Engineers)

Project Schedule
2007-2008

Construction Details
Foundation bored piles: 1.00mx55.0m = 192
1.20mx55.0m = 238
Contiguous bored piles: 1.00mx22.0m = 83
Diaphragm Walls: 6,352 sq.m. (1.0m thick x 20-22.0m deep)
1,377 sq.m. (1.2m thick x 20-22.0m deep)
Soil mixing: 275 cu.m.

Subsoil Conditions
Soft clay: 2.5-10.0m
Medium clay: 10.0-16.0m
Stiff clay: 16.0-26.0m
Dense sand: 26.0-60.0m
SRINAKRIN UNDERPASS
Civil Works

Project Description
Construction of an underpass for the traffics along Srinakrin Road.

Type of Work
Foundation piling, diaphragm walling and civil works.

Owner
The Bangkok Metropolitan Administration (BMA).

Project Schedule
2009-2011

Construction Details
Foundation bored pile:  φ1.00mx29.0m =  73
0.80mx3.0x25.0m =   3
Diaphragm Walls:  13,672sqm
(0.8m thick x 25.0m)
Structural Concrete: 42,230cu.m
Reinforcement Steel: 5,266tons

Subsoil Conditions
Soft clay:  3.0-15.0m
Medium clay:  15.0-18.0m
Stiff clay:  18.0-19.5m
Hard clay:  19.5-25.0m

Layout of the Project.
GENERAL

Thiam RuamMit (Thailand Cultural Center) Station (S12) is one of 18 underground stations of the Metropolitan Rapid Transit Authority (MRTA) Initial System Project. The station is located on Ratchadapisek Road. The initial system project or Blue Line is the first link in the network and 20km long. The perimeter walls of the station box are made of cast in-situ concrete diaphragm walls 1.0m and 1.2m thick. The south wall of the station is a 1.2m thick diaphragm wall and has openings for three tunnels linking the south and north underground stations to the Depot. A Tunnel Boring Machine (TBM) driving shaft for twin tunnels is also included in the station box on the northern end.

Diaphragm walls are embedded 32-35m deep in sand layer for 24.5m deep excavation work. 63 barrettes (1.2mx3.0m) embedded 44.5-55.0m deep in conjunction with pre-placed steel stanchions at the top are used to facilitate top down construction and to support the station box.

Glass Fibre Polymer Reinforcement was used in diaphragm wall panels at the tunnel opening locations to allow direct breakthrough for TBM.

WORK UNDERTAKEN

BORED PILES:
- Dia. 0.6mx19.5-23.5m: 12 nos.
- Dia. 1.0mx36.0m: 95 nos.
- Dia. 1.2mx36.0m: 9 nos.
- Dia. 1.5mx36.0m: 5 nos.

BARRETTEs:
- 1.2mx3.0m: 63 nos.

STEEL STANCHION:
- 859 tons

DIAPHRAGM WALL:
- 24,338.9 sq.m. (1.0m Thick)
- 1,612.8 sq.m. (1.2m Thick)

TYPE OF WORK: Foundation Piles, Diaphragm Wall and Barrettes
OWNER: The Metropolitan Rapid Transit Authority
MAIN CONTRACTOR: Nishimatsu Construction Co., Ltd. (ION Joint Venture)
DESIGNER: Ove Arup and Partners International Limited
PERIOD: 1998-1999
THIAM RUAM MIT STATION  
Barrettes and Diaphragm Walls

Diaphragm Wall Work

General view of the site (diaphragm wall and barrette work).

Subsoil conditions at the site.

Panel excavation with grab.

A large volume of bentonite slurry supply, storage and treatment facilities for an excavation up to 300 cu.m.

Placing a strip of water-stop in the stop-end plate.

Lowering a reinforcement cage with glass fibre polymer reinforcement soft eye for tunnel opening.

Concrete pouring using double tremie sets for T-Shaped panel.

A wall panel being cast with a water-stop between panel joints.

Exposed diaphragm wall inside (TBM) driving shaft.
A scene of barrette construction activity.

Left – Lowering rebar cage attached with sonic logging access tubes.

Centre – Setting up SEAFCO guide frame for steel stanchion to be installed with great accuracy.

Right – Placing a steel stanchion in position.

Steel stanchions on site, ready to be installed.

Concreting a barrette with pre-placed steel stanchion, using double tremie sets.

Barrette construction in progress – (SEAFCO Guide Frame with extended section to control verticality of stanchion in the foreground)

Verticality of stanchion can also be checked against tall buildings nearby.
Quality Control & Test

Checking trench verticality and dimension with Koden drilling monitoring equipment.

Monitoring and Checking Bentonite Slurry Quality.

Checking concrete quality of barrette with sonic logging method.

Instrumentation

Drilling monitoring record showing good verticality of trench and dimensions.

A jack-out pressure cell to be installed in the reinforcement cage of diaphragm wall.

Attaching a vibrating wire strain gauge in the reinforcement cage of diaphragm wall.

Sonic logging test record showing good quality of concrete.

Testing on barrette capacity with fully instrumented load test.

References


144 Prayasuren Road, Bangchan, Khlong Sam Wah, Bangkok 10510.

Tel. (662) 919 0090-7, Fax. (662) 919 0098
GENERAL

Rangsan Silom Precious (present name is Royal Charoen Krung) building is located at the corner of Silom and Charoen Krung Roads, Bang Rak, Bangkok. It is a 63-storey tower with six basement levels constructed by using top-down method. Foundation bored piles of 1.5m in diameter founded in sand layer at depths up to 60m, support the building. The piles were base-grouted. A 1.02m thick cast in-situ diaphragm wall (toe depth 36.0m below ground level) was used for basement excavation and construction. The maximum excavation depth was about 20.0m. 95 built-up steel stanchions pre-founded in bored piles were used for supporting the basement slabs and parts of superstructure during simultaneous construction of basements and the super structure. Three basement slabs, B1, B3 and B5 with openings were used as lateral supports to the diaphragm wall to allow excavation reach to the required depths.

WORK UNDERTAKEN

BORED PILES: Dia. 1.0mx36.0m 5 nos.
Dia. 1.5mx60.0m 529 nos.
BASE GROUTING: 534 piles
STEEL STANCHIONS: Built-up Sections 95 nos.
DIAPHRAGM WALLS: 13,748 sq.m. (1.02m Thick)
INSTRUMENTATION: 5 Inclinometer Tubes in the wall and 1 Tube in the soil behind the wall.
Lowering a rebar cage section with box-outs for basement slab connections.

Excavation below the basement slab.

Installing a 24.0m-long built-up steel stanchion in the borehole.

Subsoil conditions.

Pile load test results.

Preparation for connections between basement floor and diaphragm wall.

Floors and basements constructed.

References:


144 Prayasuren Road, Bangchan, Khlong Sam Wah, Bangkok 10510. 
Tel. (662) 919 0090-7, Fax. (662) 919 0098
GENERAL

The Din Daeng Underpass is the first of its kind in Bangkok Metropolis to ease the traffic congestion. It is an alternative to flyovers and overpasses. The underpass route extends for 415m with two lanes with one way traffic from the Victory monument. It runs along the middle of the Asoke-Din Daeng Road, and passes under the elevated First Stage Expressway as a cut and cover tunnel. 82m long approach sections at both ends of the underpass were constructed with 0.45m thick reinforced concrete walls. The 252m long middle section of the underpass, which includes portals and cut and cover tunnel, is constructed with 1.0m thick diaphragm walls having toe level 21.0m deep. Cantilever walls and reinforced concrete strut-supported walls are used in the portal sections. Excavation depths for base slab construction varied from 4.9m to 6.5m below the pavement level.

WORK UNDERTAKEN

DIAPHRAGM WALL:
- Thickness 1.0m
- Toe Depth –21.0m
- 10,545 sq.m.

INSTRUMENTATION:
- 8 Inclinometer Tubes in the wall
- 2 Inclinometer Tubes in the ground behind the wall
- 8 Sets of VWS.G. and 2 Vibrating Wire Piezometers

TYPE OF WORK: Diaphragm Wall
OWNER: Bangkok Metropolitan Administration.
MAIN CONTRACTOR: Unique Engineering and Construction Co., Ltd.
DESIGNER: Epsilon Co., Ltd.
PERIOD: 1996
Diaphragm wall panel excavation in the middle of traffics.

Subsoil conditions at the Site.

Diaphragm wall panel excavation with modified grab crane under the existing expressway.

Lowering a rebar cage section.

General view of Din Daeng Underpass.  
Portal section.  
Tunnel section.

References:

GENERAL

The Thammasat Administration Building project site is located on Tha Prachan Campus, Bangkok nearby the Chao Phraya River. The site is surrounded by a historical building and other existing structures. The construction of this building commenced about one year after completion of library building with three basements being separated by an existing building. Bored piles with pile tip in dense sand layer are used for supporting the building. A diaphragm wall of 0.8m in thickness was designed for excavation 9.7m deep with two levels of temporary bracing. The diaphragm wall toe was embedded down to 28.0m to achieve the overall stability of the excavation on the river bank. Various types of instrumentation were installed in the wall and existing buildings to observe the ground movements and response of the buildings during excavation.

WORK UNDERTAKEN

BORED PILES: Dia. 0.8mx48.0m 17 nos. Dia. 1.0mx48.0m 33 nos. Dia. 1.2mx48.0m 5 nos. Dia. 1.5mx48.0m 23 nos.

DIAPHRAGM WALL: 7,742sq.m. (0.8m Thick)

EARTH WORK: 59,592cu.m.

TEMPORARY BRACING: 586.50 tons

INSTRUMENTATION: 6 Inclinometer Tubes, 10 Tiltmeters, 5 Vertical Beam Sensors, VWSGs in one Panel, 4 Earth Pressure Gauges, 20 Surface and Deep Settlement Plates.

TYPE OF WORK: Foundation Piles, Diaphragm Wall and Excavation
OWNER: The Thammasat University
MAIN CONTRACTOR: Ch. Karnchang Public Co., Ltd.
DESIGNER: SJD-3D Co., Ltd.
PERIOD: 1999
Excavation completed to the final depth.

Temporary bracing and working platform.

References:

The Sathorn Complex project is planned for a multipurpose office and shopping complex. It is located at the Sathorn and Rama IV Road intersection. The foundation bored piles and barrettes are embedded 60m deep in sand layer to support the building. A 0.8m thick cast in-situ diaphragm wall 18.0m in depth, with three-level temporary bracing was initially planned for construction of a four-level basement. After completion of diaphragm construction, basement excavation was later modified with four level-bracing to allow excavation down to 15.5m below ground level for construction of five basement levels. For the foundation in the area of the main tower, barrettes were employed together with bored piles to increase load bearing capacity. Bored piles and barrettes were also incorporated as legs in the diaphragm wall to carry the load of building.

**TYPE OF WORK:** Foundation Piles, Diaphragm Wall and Barrettes  
**OWNER:** Quality Houses Public Company Limited  
**MAIN CONTRACTOR:** Kay-Thai Co., Ltd.  
**DESIGNER:** K.C.S. Associates Co., Ltd.  
**PERIOD:** 1995-1996

Foundation and diaphragm wall layout.

**WORK UNDERTAKEN**  
**BORED PILES:** Dia. 1.2mx60.0m 443 nos.  
**BARRETTES:** 0.8mX2.7mX60.0m 50 nos.  
**DIAPHRAGM WALL:** 7,002sq.m. (0.8m Thick)  
**INSTRUMENTATION:** 4 Inclinometer Tubes in the wall

Basement excavation and construction sequence (Schematic).

**PILE LOAD TEST RESULT**

Pile load test result.
Lowering a reinforcement cage attached with inclinometer tube void former.

Subsoil conditions at the site.

Exposed diaphragm wall.

Exposed barrette top sections.

References:
The Rajavej Hospital is located on Phayathai Road, Rajatheewi, Bangkok. It has five basement levels constructed with top-down method for car parking and rooms for utilities. Foundation bored piles of 0.8m, 1.2m and 1.5m in diameter founded in sand layer at depths of 58m, support the building. A 1.0m thick cast in-situ diaphragm wall (toe depth 21.0m below ground level) was used for basement excavation and construction. The maximum excavation depth was about 14.5m. 80 steel stanchions pre-founded in bored piles were used for supporting the basement slabs and parts of superstructure during simultaneous construction of basements and the superstructure. Two basement slabs, B1 and B3 with openings were used as lateral supports for the diaphragm wall to allow excavation to the required depths.

**WORK UNDERTAKEN**

**BORED PILES:**
- Dia. 1.0mx58.0m 5 nos.
- Dia. 1.2mx58.0m 114 nos.
- Dia. 1.5mx58.0m 56 nos.

**STEEL STANCHIONS:**
- 414x405-498x432mm 80 nos.

**DIAPHRAGM WALL:**
- 6,058 sq.m. (1.0m Thick)

**INSTRUMENTATION:**
- 7 Inclinometer Tubes in the wall.

**TYPE OF WORK:** Foundation Piles and Diaphragm Wall

**OWNER:** Rajavej Hospital Co., Ltd.

**SUPERSTRUCTURE CONTRACTOR:** SAE Thailand Co., Ltd.

**DESIGNER:** P & CYGNA Consultants Co., Ltd.

**PERIOD:** 1994-1995
RAJAVEJ HOSPITAL
Bored Piles and Diaphragm Wall

Technical Reference No. 06/99

Diaphragm wall panel excavation.

21m-long reinforcement cage with box-outs for basement slab connections.

Basement slab with temporary openings for excavation work.

Excavation below the basement slab.

Installing a 18.0m-long steel stanchion in the borehole.

Excavation at final depth of 14.5m below ground level.

Subsoil conditions.

References:


144 Prayasuren Road, Bangchan, Khlong Sam Wah, Bangkok 10510.
Tel. (662) 919 0090-7, Fax. (662) 919 0098
Project Description
29 Storey with 3 basements for a high-end residential and commercial building, located on Kabar Aye Pagoda Road, Yangon. Foundation piles consist of bored piles of 1.5m and 1.35m in diameter, barrettes (0.8x2.5m) with pile tip up to ~60m. Diaphragm walls (0.8mx18.0-19.0m) with 2 layers of temporary bracing are used as a soil retaining system for excavation and basement construction work.

Type of Work
Bored piles, barrettes and diaphragm walls work.

Owner
Living Square Co., Ltd.

Project Schedule
2013-2014

Construction Details
Bored Piles: Dia. 1.50mx55-60m = 73 nos.
Dia. 1.35mx55m = 4 nos.
Barrette: 0.8x2.5x60m = 20+28 legs
Diaphragm Wall 0.8x18-19m = 2,972. sq.m
Static Pile Load test = 2 nos.
Inclinometer Installation = 7+2 nos.

Subsoil Conditions
Lateritic clay: 0.0 to 10.0, 20.0m
Alternating lateritic clay and clayey silt: 20.0 to 40.0, 45.0m
Silt to gravelly sand: 40.0, 45.0 to 70.0m

Layout of the Project.

Bored Pile Construction in Progress (Test pile group).

Pilot Pile Load Test in Progress (Loading up to 2,700tons)

Diaphragm Wall and Barrette Pile Construction.
PROCEEDINGS OF THE THIRTEENTH SOUTHEAST ASIAN GEOTECHNICAL CONFERENCE

November 16-20 1998, Taipei, ROC

Barrettes Founded in Bangkok Subsoils, Construction and Performance

N. Thasnanipan, A. W. Maung and P. Tanseng
Seafco Co., Ltd., Bangkok, Thailand

Sponsored by;

Chinese Institute of Civil and Hydraulic Engineering
Southeast Asian Geotechnical Society
Barrettes Founded in Bangkok Subsoils, Construction and Performance

N. THASNANIPAN  SEAFCO Co., Ltd., Bangkok, Thailand
A. W. MAUNG   SEAFCO Co., Ltd., Bangkok, Thailand
P. TANSENG   SEAFCO Co., Ltd., Bangkok, Thailand

SYNOPSIS  Barrettes become not an uncommon foundation element used to support multi-storey buildings, elevated expressways and underground station boxes in Bangkok, Thailand. The demand for barrettes is necessitated mainly by constraints of site conditions, construction method, equipment and extensive bearing capacity requirement, etc. This paper presents construction practice and the performance of barrette constructed in Bangkok metropolis. Trial trenching near a canal for assessment of trench stability and soil deformation are presented. Choice of barrette and common defects found in barrette are also discussed.

INTRODUCTION

Recently published papers indicate that barrettes become a common foundation type used in various soil conditions to carry very high load for skyscrapers. Development of properties and infrastructures such as construction of multi-storied buildings, elevated expressways, subway railway stations, etc. demand deep foundations with large bearing capacity in Bangkok metropolis, Thailand. Bangkok is well known for its subsoils constraints especially, by the presence of thick soft clay on the top, followed by a series of alternating stiff clay or hard clay and dense or medium dense sand layers. Due to not only the subsoil conditions, but also non-availability of adequate construction area and overlapping developments above and underground, barrettes become common to replace the usual large diameter bored cast in-situ piles in circular shape. The earliest barrettes of 0.82mx2.98m-3.0m with toe depth of 50m were constructed in Bangkok around 1985 for International Trade Centre building in conjunction with diaphragm wall for basement construction. The first static load test on a barrette (0.82x2.7x61.8m) in Bangkok was carried out in 1992 for a 55 storey building on Silom Road, Bangkok.

CONSTRUCTION METHOD

Normally a cable hung grab mounted on crawler crane is used for trenching under supporting fluid which is commonly bentonite slurry. Concrete pouring is carried out under bentonite slurry using tremie pipes. Properties of slurry used for trenching in Bangkok subsoil usually comply with DFCP-4 of the Oil Companies Materials Association and the Institution of Civil Engineers’ (ICE) Specification for Piling and Embedded Retaining Walls. The usual bentonite slurry properties specified are 1.10g/ml for density, 30-50sec for viscosity (Marsh’ cone), 8-11.5 for pH and not more than 4% for sand content. Usually 3m to 6m long sections of steel tremie pipe of 25cm in diameter are joined together to form a continuous pipe with required length for concrete pouring under the slurry.

Cast-in-situ reinforced concrete, pre-cast concrete and temporary steel box guide walls appropriate for site conditions are used for trenching. In normal site conditions, guide walls with a minimum depth of 1.2m are commonly used. Base grouting is also used to improve the capacity and performance of barrettes if necessary.

Generally an average rate of excavation about 10-15m²/hr is achieved with mechanical grab in Bangkok subsoil. Usually, excavation rate decreases with increment in excavation depth, having non-linear relationship between excavation time and depth. Figure 1 illustrates excavation time and rate vs. excavation depth for barrettes of 1.50x3.00x55.00m. with no site constraint and idling time.
Trials trench performance

A trial trench (0.8mx2.7mx14m), with above mentioned soil reinforcement close to the canal and high slurry head, was carried out to study the feasibility in construction of barrettes to support the elevated expressway across the Canal. The major concern was stability of 11.0m thick soft clay layer (Fig. 2) which has a very low shear strength less than 15 kPa and natural water content of up to 80%. The planned trench was about 1.5m away from the canal water and four reinforced sheet piles were installed between the trench and the canal prior to excavation. In addition to the reinforced sheet piles, a temporary sheet pile wall was also constructed for protection of slurry spillage into the canal as the local authorities strictly required.

Trench stability

In general site conditions, Bangkok soft clay layer with undrained shear strength value of 15-20 kPa does not pose a particular problem for trench stability during barrette excavation under bentonite slurry. However trenching in the close vicinity of the river or canal/khlong required careful assessment of soil properties, especially sensitivity and natural water content of the soft clay and presence of permeable soil layers connecting the river bed with high pore pressure. Normally, higher guide wall with adequate slurry pressure and reinforcement of the surrounding soft clay by sheet piles placed perpendicular to the alignment of the planned barrette and driven to a stiffer soil layer, overcome the instability of the trench.

Figure 1. Excavation time, rate vs. depth for barrettes 1.5mx3.0m.

Usually 1.0m or more over casting of concrete above designed cutoff level is practiced. Observation of 3 completed projects indicates that concrete over break of barrettes was found to be in a range of 6% to 14% for barrettes with about 2.0m over casting.

Figure 2. Soil movements on opposite faces of trial trench

The trial trench filled with bentonite slurry was open for 48 hour after excavation down to 14m. Lateral movements of trench face along the depth were studied with graphs of the trench profiles plotted by a Koden drilling monitoring equipment stationed at a fix position on the guide wall at every 6 hour interval. Trench profiles plotted were compared and changes were manually measured using the reference scales shown on the graph. The recording accuracy of the equipment is in a range of +/- 0.2% (in this case the accuracy would be within 2-3mm). 20mm to 30mm lateral soil movement inward the trench was observed on the monitoring records on non-reinforced soil side after 24 hours.
and 48 hours respectively in the soft clay layer. For trench face close to the canal, observed movements of the soil being reinforced with sheet piles are generally less than those of non-reinforced soil and some movements are considered induced by the sheet pile wall installed for canal water protection. A profile of trench showing soil movement plotted from the monitoring records is presented in Figure 2.

There is no particular problem associated with trench stability in the sand layers 20 to 45m of typical Bangkok subsoil below the ground level. The drilling monitoring records show no collapse of trench surface along the depth section (Fig. 3).

Figure 3. Trench profile of barrette (1.5mx3.0mx55m) plotted by Koden drilling monitoring equipment

LOAD BEARING CAPACITY OF BARRETTES

Common type of barrettes constructed in Bangkok area is rectangular shape with dimensions ranging from 0.6mx2.7m to 1.0mx2.7m. These barrettes have been mostly constructed incorporated into diaphragm walls for deep basements of high rise buildings. However, individual larger barrettes (1.2mx2.7m and 1.5mx3.0m) are also currently in use to support large column loads as the grabs of large dimensions are currently available in the market. A number of load test on such large barrettes will be carried out in the near future. Maximum bearing capacity of barrettes is usually designed up to allowable concrete stress of 5MPa. Toe depth of barrettes are limited by subsoil conditions and required load capacity with safety factor of 2.0 to 2.5. The toe depth of constructed barrettes ranges from 45.0m to 60.0m. Concrete cube strength of 30MPa to 40MPa are commonly used for barrettes. Minimum reinforcement of 0.5% of the cross sectional area of barrette is usually provided for either full length or partially as per load requirements.

Barrettes can be constructed with flexible layout plan for both vertical and lateral loads. The layout pattern of barrettes can be arranged in a continuous row or column, radial, alternating long and short axis of barrette and a combination of two or more of such patterns. One type of equipment can be used for constructing both barrettes and diaphragm walls in particular projects thus reduces the mobilization cost. In addition to the large bearing capacity requirements, on site difficulties such as limited head room, also demands the barrettes. Piling rigs cannot be utilized under such situations like presence of high voltage power cables overhead, existing overpasses or structures for elevated expressways and planned subway stations.

A summary of observation on barrettes of 19 projects completed, with respect to selection criteria is presented in Table 1.

Table 1. Summary of Barrette Selection

<table>
<thead>
<tr>
<th>Criterion</th>
<th>No. of Project</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Load Capacity</td>
<td>4</td>
<td>incorporated with bored piles</td>
</tr>
<tr>
<td>Minimise Construction Equipment</td>
<td>6</td>
<td>Alternative for bored piles</td>
</tr>
<tr>
<td>Limited Head Room for Excavation</td>
<td>2</td>
<td>Elevated Highways under or above existing structures/power lines</td>
</tr>
<tr>
<td>Combination with Diaphragm Wall</td>
<td>10</td>
<td>As diaphragm wall legs</td>
</tr>
<tr>
<td>Foundation as well as portion of column</td>
<td>3</td>
<td>provision for the future requirement</td>
</tr>
</tbody>
</table>

Since basic shape of the cross section of barrette is rectangular, certain dimensions of barrette can provide a greater perimeter for skin friction than that of the conventional bored pile (in circular shape) with equivalent cross sectional area. Hence, for friction piles, more load carrying capacity per unit volume of concrete can be achieved by a barrette than that of a circular bored pile.

Figure 4 illustrates that typical barrette of 2.7m with various thickness can give 12 to 30% higher friction area than that of circular piles with corresponding equivalent sectional area.
Figure 4. Comparison of shape effect between barrette and circular pile

QUALITY CONTROL

Construction tolerance allowed for barrettes are generally identical to that for bored piles. Normally verticality of 1:100 is allowed in barrette construction in Thailand. Drilling monitoring is employed to check the verticality of the trench on random barrettes or on all barrettes. After construction, integrity testing such as sonic integrity (low strain dynamic test) and sonic logging tests and concrete coring are sometimes applied. Common defects found in barrettes constructed in Bangkok subsoils are presented in Table 2.

Table 2. Common Defects Found in Barrettes

<table>
<thead>
<tr>
<th>Defect found</th>
<th>Possible Causes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete cover</td>
<td>improper cage position, provision of inadequate spacers</td>
</tr>
<tr>
<td>Bentonite slurry</td>
<td>improper tremie concrete pouring, improper desanding and recycling of slurry, inadequate concrete slump</td>
</tr>
<tr>
<td>Quality concrete</td>
<td>inadequate supervision</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>inadequate supervision</td>
</tr>
<tr>
<td>Deviation</td>
<td>improper tremie concrete pouring (with very long tremie pipe embedment in concrete poured) in partially reinforced barrettes</td>
</tr>
</tbody>
</table>

Table 3. Summary of Barrette Load Test

<table>
<thead>
<tr>
<th>Project</th>
<th>Barrette Dimension (m)</th>
<th>Test Load (kN)</th>
<th>Total Settlement (mm)</th>
<th>Permanent Settlement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.82x2.70x</td>
<td>14000</td>
<td>4.83</td>
<td>3.47</td>
</tr>
<tr>
<td></td>
<td>61.80*</td>
<td>28000</td>
<td>12.56</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>35000</td>
<td>17.63</td>
<td>3.47</td>
</tr>
<tr>
<td>B</td>
<td>0.82x2.70x</td>
<td>12000</td>
<td>5.65</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>44.00</td>
<td>24000</td>
<td>34.15</td>
<td>24.72</td>
</tr>
<tr>
<td>C</td>
<td>0.80x2.70x</td>
<td>12900</td>
<td>4.46</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td>55.00</td>
<td>25800</td>
<td>11.01</td>
<td>2.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30140</td>
<td>12.35</td>
<td>2.17</td>
</tr>
<tr>
<td>D</td>
<td>0.80x2.70x</td>
<td>11000</td>
<td>3.07</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>50.00</td>
<td>27500</td>
<td>7.97</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>1.00x2.70x</td>
<td>20555</td>
<td>14.00</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>48.94**</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note:   * with base grouting   ** load test by dynamic load

All the barrettes tested, except the barrette of project B were founded in dense sand layer with embeddment more than 5m into it. The barrette of project B was embedded into about 0.5m in dense sand layer and failed under the maximum test load of 24MN. Base grouting was employed for the barrette of project A. A soil profile and load vs. settlement graph of test barrette of project A are presented in Figures 5 and 6 respectively.

Figure 5. Soil profile at project A site
Barrettes of project E were constructed on both side of water supply canal and one barrette was tested with high strain dynamic load test as no anchor piles were available. A 200kN steel hammer was dropped at a height of 2.6m above the pile cap. A graph of dynamic load test signal analysis and simulated load settlement curve are shown in Figures 7 and 8 respectively.

Load test results listed in Table 3 indicate that all barrettes tested are able to carry the design load with Factor of Safety of 2 or higher. From the available load test results, the end bearing capacity of test barrettes cannot be estimated. However, all barrettes are considered as friction piles, except for project A in which base grouting was used to improve the end bearing of barrette. For test barrette of project A, no permanent settlement was observed under test load of 28MN which is 200% of the design load and base grouting is considered contributed in good performance of barrette.

Barrette load tests with full instrumentation devices such as strain gauges and magnetic extensometers are planned to be carried out for subway stations in the near futures.
performance of barrettes in Bangkok subsoil can then be evaluated with a great accuracy.

CONCLUSION

Construction practice adopted in the foundation industry for barrette installation in Bangkok subsoil has been discussed in this paper. There is no indication of trench instability under bentonite slurry in Bangkok subsoil for barrette construction with proper site preparation. This is evidenced by trial trenching and profiles plotted by drilling monitoring equipment. To achieve a good end bearing capacity for barrette founded in sand, base grouting is recommended. Performance of barrettes has been discussed with available load test results, and found to be satisfactory with load bearing performance.

ACKNOWLEDGEMENT

The authors express their appreciation to the colleagues, especially to Mr. Ganeshan Baskaran and Mr. Muhammad Ashfaq Anwar for their invaluable suggestion and assistance in preparation of this paper.

REFERENCES


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Design, construction and behavior of bored cast-in-situ concrete piles in Bangkok subsoils

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DESIGN, CONSTRUCTION AND BEHAVIOUR OF BORED CAST IN-SITU CONCRETE PILES IN BANGKOK SUB SOIL

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ABSTRACT

This paper briefs design considerations, construction methods and materials, static load test results and pile integrity test results of the cast in-situ bored piles constructed as foundation elements for a wastewater treatment plant. The treatment plant is located in Bangkok, capital of the Kingdom of Thailand and is to serve about 195,000 inhabitants of the city. A total of 402 piles, 296 nos. of 1500 mm diameter and 96 nos. of 1000 mm diameter, were designed to carry the whole structural load of the treatment plant. Maximum working load on individual piles was designed to be 1000 tons and 500 tons for 1500 mm and 1000 mm diameter pile respectively with a safety factor of 2.5. Initially the pile toe depth was designed to be 60m below the ground level, and embedded into hard clay for 1500 mm diameter piles while the smaller diameter pile embedded into second sand layer at a depth of 53m and base was not decided to be grouted. Pile load test results on fully instrumental pilot piles showed the failure well before reaching the designed test load. Piles were re-designed and pile toe depth was set to be 55m, embedded into dense sand layer for both sizes and base grouting was decided to be done for all piles. Proof load test results performed on four working piles(contract piles), two for each size, produced well acceptable results conforming to the design and structural specifications and requirements. However, settlements observed were higher than that observed for other projects in the vicinity. Base Grouting was done using tube - a manchette method. A maximum volume of 500 liter of cement grout (w/c ratio ~ 0.5) or maximum grouting pressure of 40 bars was used as limiting criteria. However, maximum pressure of 40 bars was hardly achieved. From the proof load test results ultimate load capacity was estimated and compared using different methods suggested by previous researchers. Integrity test was decided to be performed on each pile and results available currently are summarized.

KEYWORDS

Bored Piles, Base Grouting, Static Pile Load Test, Integrity Test, Pile Capacity
INTRODUCTION

The city of Bangkok has been emerged as one of heavily populated urban cities in the world. Due to its rapid economic growth, the construction industry has been challenged to develop the infrastructure to cope with the booming economy. As a result, while multi storied private owned buildings become most attractive sky scrapers in this city of Angels, huge portion of the Government budget is swallowed by new projects involving construction of highways and railways, both elevated and at-grade, sub ways, large span bridges and water and wastewater treatment facilities. Construction industry is further challenged by the city’s subsoil condition as it is plagued with a lot of constraints such as, presence of thick soft clay as top strata, land subsidence and high risk of flooding. These constraints compel most structures be relied on pile foundation usually, on large diameter bored cast in-situ piles, embedded into first or second sand layers. Though, the first large bored pile was installed nearly two decades ago in this city and hundreds of thousands of piles have already been constructed, design, construction methods, performances and effectiveness of these piles are seldom studied and rarely reported.

The project site is located at the Rama III road, with east boundary along the Chao Phraya river which is the largest of its kind in the kingdom. The piles are usually of 1500mm and 1000 mm diameters. The maximum design working load on piles are 500 tons and 1000 tons for 1000 mm and 1500 mm piles respectively. Pile toe was designed to be embedded into second sand layer at depth 55m from the ground level. Fig. 1 shows the general layout of the piles. All the piles are cast in situ bored piles under wet process and base grouted.

SUBSOIL CONDITIONS

The geotechnical investigation at the site indicates that a strata of 14 -15m thick soft clay layer occurs in the project area as common in Bangkok sub soil. This soft clay is sensitive and has an-isotropic and time dependent stress strain properties. Below is stiff to very stiff clay layer underlain by hard clay layers. These clay layers extend down to about 42m in depth. However, the subsoil condition below 36m from the ground level in the project area is variable - a sand layer with a thickness of about 13m is present sporadically. In some locations within the project site, a stiff to hard dark gray clay is also found at the depth between 40m and 49m. Generally a thick dense sand layer is located below 50m from the existing ground level. However, at this area alternated layers of dense sand and hard clay of 5m thick also was encountered. Fig. 2 shows the general bore log data observed at the site.

PILE CAPACITY AND DESIGN CONSIDERATIONS

The load capacity of the piles were estimated for the soil failure using the available soil data. Safety Factor of 2.5 was used for both end bearing and shaft bearing capacities considering the negative skin friction as well. The estimated safe working load in tons, using individual three bore hole data are 480, 508 and 583 for 1000 mm piles and 839, 880 and 993 for 1500 mm piles.

The estimated load capacity, with required safety factor, from the soil bore log results is, in many cases, below the designed safe working load. Hence, means of improving pile bearing capacity was considered. The pile bearing capacity can be improved either by extending the pile length further or by grouting the pile or by enlarging the pile base. Usually, enlarging the base for long piles is hardly practiced in Bangkok. Piling contractors rather prefer to extend the length of the shaft and additional required capacity is achieved by the shaft friction. For the bored piles designed at this project site, if an extra bearing capacity of 250 t/m² could be achieved by
base grouting, the grouted piles would be capable of carrying the designed safe working loads with adequate safety factor.

Based on the results from a project just near by the site and past experiences in the vicinity it was proposed that the above mentioned required capacity could be achieved by base grouting the piles with tip founded at about 55 m from the ground.

The option of extending the pile length was given up as the clay layer is found to be at about depth of 60 m and thus possibly higher initial settlement was expected if the piles are embedded into the clay layer. Moreover, the end bearing capacity would be reduced and this reduced end bearing capacity might influence the shaft bearing capacity too. This conclusion was reinforced by the previous pile load test results on a pilot pile, founded at 60m and non base grouted. This non grouted and longer pile of diameter 1500 mm, constructed by some other piling contractor, failed at about 1750 tons, while the designed maximum test load was 2500 tons. The 1000mm diameter pilot pile was, though not plunged to failure, settlement observed was too excessive for the structural requirements. Load settlement characteristics of these pilot test piles have been shown in Fig. 4(a) and Fig. 4(b). Finally the previous piling contractor was disqualified and the new contractor was called upon.

CONSTRUCTION METHODS, MATERIALS AND PROBLEMS

Wet process

The usual wet process using the bentonite slurry was adopted. Temporary casing of length 15m was used to maintain the hole stability in top soft layer. The holes were drilled using augers and then continued using the bucket method. The slurry was introduced upon reaching the first sand layer and maintained all the time higher enough to support the bore. The slurry was monitored continuously for the properties of density, viscosity, pH and sand content.

Construction of bored piles started during the rainy season and therefore the site needed additional requirements to improve the working condition. As the water level was high in the Chaophraya river flood protection was needed. Because of the nature of the Bangkok sub soil and the project site was previously used as urban waste dumping area, the site was very swampy during the rainy season gave a lot of inconveniences in solving problems for drainage of water accumulated. The situation was further aggravated as the excavated soil was not able to be removed on time. All these reduced the productivity rate drastically. However, all the piles were cast within the time frame set according to the contract.

Slurry properties

The limiting values for the bentonite slurry according to the contract specification were 1.04 - 1.20 for density; 30-40s for viscosity (Marsh’s cone); not less than 7 for pH and not more than 3% for sand content.

On average 3.5 % of bentonite concentration was used. Bentonite properties were monitored before concreting for all piles. Arbitrary checking was done during drilling. From the bentonite results, no significant changes were observed. On average, the viscosity observed was 32.8s (Marsh cone); with maximum of 40s and a minimum of 29s. The average values of 1.11g/cc, 8.9, 1.62% were noted for density, pH and sand content respectively. The minimum values are 1.02g/cc, 7, 0.1% and maximum values are 1.11g/cc, 9 and 1.55 respectively. Even though, the sand content was set to be within 3%, at times it was difficult to achieve this. In this cases complete recycling of the slurry was done.

Concreting and Reinforcement

Concrete cube strength of 350 ksc (35 Mpa) was used with minimum cement content of 400 kg/m³. Concreting was done by usual tremie method of 250mm tremie pipe. Concreting was requested to be done within twenty four hours after final excavation. Since a concrete batching plant was established within the project area, concrete supply was usually regular and timely supplied otherwise it would have been a major concern in Bangkok because of its ill fated traffic congestion. Average concreting rate was about 25 - 30 cu.m per hour. After pouring the concrete to the desired level, the temporary casing was withdrawn.

Reinforcement was provided to full length of the pile considering the possible tensile forces during construction period of the structure. Maximum of 0.5% of cross sectional area was used for both sizes of the piles.

All the main bars used for reinforcement are of high strength deformed bars of 4000 ksc (400 Mpa) and round bars of 2400 ksc (240 MPa) were used for spirals. Reinforcement was provided by fabricating in the form of cages as usual and the maximum length of a cage was set to be 12 m. A minimum lap length of forty times the diameter of the bars was used at the joints of the cages.

Inclusions in pile top and concrete level below trimming level

Few cases were observed where the concrete level was below the designed cut off level and/or inclusions in pile top portion. Mainly, this has been contributed by the high un-predicted slumping down of concrete upon extracting the temporary casing. As Fleming et al. (1985) pointed out concrete would normally be retained by cohesive soils with shear strength in excess of 15 kN/m² and this limit is rarely achieved by the top soft clay of Bangkok sub soil.
Final sound concrete level has been a talking point between the main contractor and piling contractor for both cases where the final concrete level is above and below the designed trim level. Usually for the first case the main contractor suffered to trim the long pile heads but in the latter cases the piling contractor was called upon for remedial works. However, it should not be neglected that the possibility of failure to over flush enough concrete at the pile head to ensure that all concrete present below the trim level is of full strength and thus having the inclusions at the pile top.

Verticality and Position Errors

Verticality tolerance was set to be 1 in 100 and no piles have been found to be out of this tolerance. However, four piles (about 1% of the total) have been found to be out of position deviating the specified tolerance of 7.5cm plus the allowance for verticality. However, many of them were found to be under the mat on which the columns were to be erected. Hence, dealing with these erroneous locations was not dramatic and modification of reinforcement for the mat have been done according to the redistributed load.

GROUTING

The effectiveness of base grouted piles in Bangkok sub soil was studied by many previous researches. Teparaksa (1994) suggests base grouting mainly aims to increase soil stiffness beneath the pile base which was affected by the boring process. Also he noted that for these piles the displacement at fully mobilized skin friction is in the order of 1.59 to 1.86 percent of the diameter. Test results in this project too agrees the range suggested.

The PE pipes (0.5") and tube-a manchette were in-corporated into the reinforcement cages and lowered into the bore to facilitate the base grouting. The grouting was done after the concrete is set, usually the base was cracked using a high water pressure not later than twenty four hours, and grout was injected.

![Fig.3 Tube a manchette used for grouting](image)

The grouting process was controlled by either maximum grout volume of 500 liters or a maximum pressure of 40 bars. But, the pressure of 40 bars was rarely achieved and thus usually the grout volume was the limiting factor. Usually the grouting pressure was within the range of 20 - 30 bars.

Injection rate of grout was 10 - 15 liters/ min. Experiments on grout samples collected from three of the bored piles (1500mm dia.) in this site was performed by Anwar (1997). Comparison of base grouted and soft base (non base grouted) piles were studied and summarized in Table 1. It was recommended that low injection rate about 1-2 liters/min would improve the effectiveness of base grouting. However, practicality and sustainability of grout pump, especially for large volume of grout, can hardly justify this requirement. Though the direct comparison of effect of base grouting is not possible as the compared piles are founded in different soil strata other parameters in Table 1 are worth enough to be noted.

<table>
<thead>
<tr>
<th>Table 1  Load distribution and settlement behavior of base grouted and soft base piles.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter(mm)</td>
</tr>
<tr>
<td>1500</td>
</tr>
<tr>
<td>Base Depth (m)</td>
</tr>
<tr>
<td>Design Load Qd (ton)</td>
</tr>
<tr>
<td>Failure Load (ton)</td>
</tr>
<tr>
<td>Load at 20mm total settlement (Qf)</td>
</tr>
<tr>
<td>% increase in Qf</td>
</tr>
<tr>
<td>Portion of Qf carried by shaft (Qs)</td>
</tr>
<tr>
<td>% increase in Qs</td>
</tr>
<tr>
<td>Portion of Qf carried by base (Qb)</td>
</tr>
<tr>
<td>% increase in Qb</td>
</tr>
<tr>
<td>Total settlement at (Qd) (mm)</td>
</tr>
<tr>
<td>Total settlement at 1.5 Qd(mm)</td>
</tr>
</tbody>
</table>

* - Estimated

PILE LOAD TEST AND RESULTS

Four piles, two from each size, were selected to perform the static pile load test. All the test piles and anchor piles are contract piles. The maintained load test was performed in two cycles for each pile with the first cycle having maximum test load of safe design working load. The maximum test load for second cycle was 150 % of the design safe working load. The maximum test load for these both cycles were maintained for 24 hours.

The third cycle was performed as a quick test and each incremental load was maintained for 10 minutes. The maximum test load on this third cycle was 250 % of the safe design working load. The results were satisfactory and complying the design requirements (Max. allowed settlement of 10mm under working load)
Ultimate load interpretation

Predicted ultimate loads were based on graphical methods proposed by Mazurkiewicz (1972), Fuller and Hoy (1970) and Butler and Hoy (1977). It may be appropriate to mention here that ultimate capacity determined from the above mentioned methods generally not yield a load corresponding to a “plunging” failure. The ultimate capacity is defined by the load at which the pile moves rapidly (plunging) without any further load increment. Most of the existing methods for determining the ultimate capacity from pile load test, however, are mostly based on some arbitrary displacement criteria and there is no consensus on the best method of interpreting the failure load. Fellenius (1980) pointed out that preferred method of interpretation often depend on an individual’s experience and he further recommended that several methods be applied and that a preferred method be chosen on the basis of the user’s needs and special conditions of the job.

For base grouted piles in Bangkok sub soil, the predicted failure load based on the above methods well agrees each other (Teparaksa, 1994). For this particular project the variation of predicted loads by these different methods are in the range of 2.5% to 19%. It should be noted that the tests were not performed to failure of piles. As the piles were not tested for failure it was difficult to compare the results with actual failure load.

Table 2 Ultimate load predicted by different methods

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Dia. (mm)</th>
<th>Max. Test. Load (ton)</th>
<th>Ultimate Load (tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP - 1</td>
<td>1000</td>
<td>750</td>
<td>1625</td>
</tr>
<tr>
<td>TP - 2</td>
<td>1000</td>
<td>1250</td>
<td>1700</td>
</tr>
<tr>
<td>TP - 3</td>
<td>1500</td>
<td>1500</td>
<td>3600</td>
</tr>
<tr>
<td>TP - 4</td>
<td>1500</td>
<td>2500</td>
<td>3250</td>
</tr>
</tbody>
</table>

(Note: Piles were not tested to failure)

Summary of pile load test results

Table 3 (a), (b) and (c) summarize the pile load test results and Fig. 5 and Fig. 6 show the corresponding load settlement characteristics.

Table 3 (b) CYCLE - 2

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Dia. (m)</th>
<th>Base depth (m)</th>
<th>Cycle 2 (Maintained Load Test)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Max. Load (ton)</td>
</tr>
<tr>
<td>TP - 1</td>
<td>1000</td>
<td>55</td>
<td>750</td>
</tr>
<tr>
<td>TP - 2</td>
<td>1000</td>
<td>55</td>
<td>1250</td>
</tr>
<tr>
<td>TP - 3</td>
<td>1500</td>
<td>55.3</td>
<td>1500</td>
</tr>
<tr>
<td>TP - 4</td>
<td>1500</td>
<td>55.5</td>
<td>2500</td>
</tr>
</tbody>
</table>

Table 3 (c) CYCLE - 3

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Dia. (m)</th>
<th>Base depth (m)</th>
<th>Cycle 3 (Quick Test)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Max. Load (ton)</td>
</tr>
<tr>
<td>TP - 2</td>
<td>1000</td>
<td>55</td>
<td>1250</td>
</tr>
<tr>
<td>TP - 4</td>
<td>1500</td>
<td>55.5</td>
<td>2500</td>
</tr>
</tbody>
</table>

Load test results on soft base pilot piles

Fig. 4(a) Load test on pilot pile with non base grouted pile (Diam. 1500mm, Toe embeddment at depth 60m below ground)

Fig 4 (b) Load test on pilot pile with non base grouted pile (Diam. 1000mm, Toe embeddment at depth 53m below ground)
Almost all the cracks were indicated on piles adjacent to the sheet pile cofferdam which was used for deep excavation. These piles in many cases were repaired by coring and cement grouted. Physical damages to piles located adjacent to the excavation boundary were a major source of trouble as the sheet piles used for temporary retaining purpose cannot completely retain the soil movement and local soil strains induced could be sufficient enough to cause cracking. Possibility of providing more main reinforcement bars for piles adjacent to the excavation zone to prevent these worse effects would have been worth to have considered.

### Table 4 Interpretations by sonic integrity test

<table>
<thead>
<tr>
<th>Diam. 1500 mm</th>
<th>Diam. 1000mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>nos.</td>
<td>%</td>
</tr>
<tr>
<td>Available test results</td>
<td>300</td>
</tr>
<tr>
<td>Sectional increase (small)</td>
<td>25</td>
</tr>
<tr>
<td>Sectional increase (prominent)</td>
<td>04</td>
</tr>
<tr>
<td>Sectional Decrease (small)</td>
<td>04</td>
</tr>
<tr>
<td>Intermittent sectional variations (1)</td>
<td>10</td>
</tr>
<tr>
<td>Cracks (small) (1)</td>
<td>10</td>
</tr>
<tr>
<td>Cracks (prominent) (2)</td>
<td>03</td>
</tr>
</tbody>
</table>

Notes:
1. These variations were attributed to the sub soil conditions
2. All the cracks have been found on piles adjacent to the excavation supported by steel sheet piles

### CONCLUSION

A case history of cast in-situ concrete bored piles in Bangkok sub soil has been summarized. A total of 402 piles of 1000mm (96 nos.) and 1500mm (306 nos.) have been constructed as foundation elements of a wastewater treatment plant.

Base grouting has been considered to improve the bearing capacity and settlement behavior of bored piles. Maximum pressure of 40 bars or 500 liter of grout volume has been used as limiting criteria. Base grouting with ‘tube-a manchette’ method has been adopted and found to be effective.

Static pile load test results on contract piles and pilot test piles have been summarized. Pilot piles with base non grouted, have shown either high degree of settlement or failed well before the designed maximum test load. Ultimate load capacity has been estimated by different methods and variation of the ultimate capacity found to be within 2.5 to 19 percent.

Pile integrity test was decided to be performed on all piles and commonly indicated signal characteristics have been tabulated for available data.
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REFERENCES


Ng Kim Cheng [1983], “The construction problems and performance of large bored piles in second sand layer”, AIT Master Thesis, Bangkok, Thailand


Deep Foundations on Bored and Auger Piles

BAP III

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Large diameter bored piles in multi-layered soils of Bangkok

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ABSTRACT: Static pile load test results of ten large diameter bored piles founded in the multi-layered soils of Bangkok are discussed in this paper. All piles were instrumented with vibrating wire strain gauges (VWSG) and telltale extensometer rods. Range of shaft friction and end bearing values mobilized in different soil horizons are reported. Progressive type of failure mechanism which is peculiar with the long piles in multi-layered soils, and found to be initiated from the soil layers present at the middle reach of pile shaft and then extending towards the top and bottom layers, is also presented. Brittle type of failure observed in some sand and very stiff to hard clay layers is also discussed. Settlements at design loads calculated from global factor of safety concept and limit load concept are also compared. Additionally, back calculated $\alpha$ and $\beta$ values are also compared with the previously recommended values for similar soils.

1 INTRODUCTION

Bored cast in situ piles are extensively used as a foundation element in Bangkok due to the requirements of huge loads to be transferred and limitations of using driven piles like limited capacities, associated soil movements and some unwanted environmental effects. Diameters of these piles normally fall in the range of 0.80 to 1.50 m and toe depths down to 60 m from ground level are quite common. Soil profiles of plain of Bangkok are always evident of the presence of marine Bangkok Soft Clay (BSC) at the top changing to medium at about 15 to 18 m. First sand layer is usually 5 to 10 m thick and found at 25 m to 30 m depth, below is a series of stiff to hard clay and medium to very dense silty sand layers (Ref. to the Figs. 5 to 14 for typical soil profiles). Actual pore pressure conditions in the upper BSC are hydrostatic from circa 1 m below ground level. Then the hydrostatic conditions changes to piezometric draw down near the bottom level of BSC. Piezometric draw down resulted in increased effective overburden pressure of about 20 ton/m² in the first sand layer and below as shown in Figure 1. The under drainage in the Bangkok soils is attributed to the deep well pumping in the area. According to the latest reports, piezometric draw down conditions have started recovering since last few years due to the control over deep well pumping and shifting of industrial areas away from Bangkok metropolitan.

Figure 1. Variation of effective overburden pressure with depth, (after KERDSUWAN, 1984)

2 METHOD OF CONSTRUCTION

Bentonite slurry in conjunction with rotary bucket is the normal drilling procedure of pile construction when piles are to be founded in the first sand or below. Top 15 to 18 m depth of soft to very soft clay is almost always temporarily cased to assure the stability of borehole. Firstly, auger is used to drill
within the temporary casing followed by rotary bucket with bentonite slurry down to final depth of excavation. Special cleaning bucket or air lift technique is normally applied to clean the borehole base of any congregated sediments before lowering the reinforcement cage which is followed by tremie concreting. Bentonite slurry viscosity is normally maintained within the range of 30 to 50 seconds (Marsh cone viscosity) and construction time, starting from the casing driving till the completion of concreting, usually fall in the range of 10 to 20 hours excluding some accidental delays due to equipment break down, unavailability of ready mixed concrete or similar reasons.

3 DESIGN PARAMETERS

A combination of total stress method for clay layers and effective stress method for sand layers is normally adopted due to its simplicity and limitations regarding unavailability of reliable effective stress parameters. There is no standard criteria of acceptable settlement for these piles, but a limit of 5mm at working loads is normally accepted for high rise buildings.

3.1 Skin friction

Skin friction capacity of clay layers is estimated as $f_s = \alpha \cdot Cu$, with adhesion factor $\alpha$ varying from 0.9 to 0.3 depending upon the undrained shear strength of clay layers. Undrained shear strength $Cu$ of shallow clay layers is usually determined in the laboratory with unconfined compression test while for deeper stiff to hard clay layers $Cu$ is indirectly estimated from Standard Penetration Test (SPT-N) data as $Cu = C_1 \cdot SPT-N$ (ton/m$^2$), with $C_1$ equal to 0.674 and 0.507 for high and low plasticity clays respectively. $C_1$ values mentioned here are based on the statistical analysis carried out by Pitupakorn, 1985. Skin friction capacities for the sand layers are calculated as $f_s = \sigma_v' \cdot K_s \cdot \tan \delta$, with coefficient of horizontal earth pressure $K_s$ equal to 0.7 and $\delta$ equal to $0.75 \phi$. Angle of internal friction $\phi$ is also estimated from SPT-N values by first correcting the $N$ values for overburden correction (Bowels, 1988). Some designers also use an equivalent term of $\beta$ instead of $K_s \cdot \tan \delta$. It must be noted that the empirical parameters discussed here are based on the effective overburden pressure $\sigma_v'$ which is calculated with the assumption of hydrostatic conditions from the ground level, ignoring the piezometric draw down conditions below BSC shown in Figure 1.

3.2 End bearing

Ultimate end bearing capacity for the sand layers is calculated as $N_q \cdot \sigma_v'$ with bearing capacity factor $N_q$ in the range of 5 to 20 depending upon the relative density of the sand layers. A limit of 500 ton/m$^2$ for maximum end bearing is normally used for piles seated in the second sand layer. For clay layers ultimate end bearing is estimated as $N_c \cdot Cu$ with bearing capacity factor $N_c$ equal to 9. Global factor of safety of 2.5 on the accumulated ultimate skin friction and end bearing is normally used to calculate the safe working loads.

Table 1. Principal data of test piles with interpreted failure loads.

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Dimensions (Dia. x Depth)m</th>
<th>Design Load (ton)</th>
<th>Total Mobilized Skin Friction (ton)</th>
<th>Estimated Skin Friction (ton)</th>
<th>Failure Load (ton)</th>
<th>SETTLEMENT(mm) AT QL Limit load at a settlement of 1% of Dia.</th>
<th>Qd1</th>
<th>Qd2 ( = Qd1/2.5 )</th>
<th>Qd2/Qd1</th>
<th>Qd1 ( = 1.5 \times Qd1 )</th>
<th>Qd1 ( = 2.0 \times Qd1 )</th>
<th>Qd1 ( = 1.5 \times Qd1 )</th>
<th>Qd1 ( = 2.0 \times Qd1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>TP-1 φ1.2x57.10</td>
<td>650</td>
<td>1582</td>
<td>1230</td>
<td>1.29</td>
<td>1435</td>
<td>1715</td>
<td>1575 630</td>
<td>0.97</td>
<td>6.8</td>
<td>13.2</td>
<td>35.5</td>
<td>975 650</td>
</tr>
<tr>
<td>2</td>
<td>TP-2 φ1.2x46.25</td>
<td>650</td>
<td>1929</td>
<td>930</td>
<td>2.07</td>
<td>2815</td>
<td>2017</td>
<td>2416 966</td>
<td>1.49</td>
<td>5.5</td>
<td>9.3</td>
<td>15.0</td>
<td>1140 760</td>
</tr>
<tr>
<td>3</td>
<td>TP-3 φ1.0x46.51</td>
<td>500</td>
<td>1118</td>
<td>800</td>
<td>1.40</td>
<td>1477</td>
<td>1544</td>
<td>1557 623</td>
<td>1.25</td>
<td>4.5</td>
<td>8.2</td>
<td>12.3</td>
<td>920 610</td>
</tr>
<tr>
<td>4</td>
<td>TP-4 φ1.0x49.47</td>
<td>450</td>
<td>970</td>
<td>930</td>
<td>1.04</td>
<td>1172</td>
<td>1138</td>
<td>1193 477</td>
<td>1.06</td>
<td>6.3</td>
<td>12.8</td>
<td>40.5</td>
<td>650 430</td>
</tr>
<tr>
<td>5</td>
<td>TP-5 φ1.0x43.00</td>
<td>450</td>
<td>1106</td>
<td>690</td>
<td>1.60</td>
<td>1399</td>
<td>1347</td>
<td>1463 585</td>
<td>1.30</td>
<td>4.5</td>
<td>11.1</td>
<td>34.5</td>
<td>675 450</td>
</tr>
<tr>
<td>6</td>
<td>TP-6 φ1.0x41.00</td>
<td>360</td>
<td>641</td>
<td>620</td>
<td>1.03</td>
<td>759</td>
<td>997</td>
<td>878 351</td>
<td>0.98</td>
<td>3.2</td>
<td>5.0</td>
<td>7.2</td>
<td>830 550</td>
</tr>
<tr>
<td>7</td>
<td>TP-7 φ1.2x43.6</td>
<td>450</td>
<td>1499</td>
<td>850</td>
<td>1.76</td>
<td>891</td>
<td>1396</td>
<td>1141 456</td>
<td>1.01</td>
<td>2.5</td>
<td>4.2</td>
<td>5.4</td>
<td>1150 770</td>
</tr>
<tr>
<td>8</td>
<td>TP-8 φ1.2x43.5</td>
<td>450</td>
<td>519</td>
<td>640</td>
<td>0.81</td>
<td>1012</td>
<td>1380</td>
<td>1057 423</td>
<td>0.94</td>
<td>3.3</td>
<td>5.9</td>
<td>48.5</td>
<td>850 570</td>
</tr>
<tr>
<td>9</td>
<td>TP-9 φ1.0x43.5</td>
<td>400</td>
<td>701</td>
<td>530</td>
<td>1.32</td>
<td>867</td>
<td>946</td>
<td>929 372</td>
<td>0.93</td>
<td>4.0</td>
<td>5.2</td>
<td>36.5</td>
<td>750 500</td>
</tr>
<tr>
<td>10</td>
<td>TP-10 φ1.2x54.00</td>
<td>500</td>
<td>799</td>
<td>1100</td>
<td>0.73</td>
<td></td>
<td></td>
<td>3.5 6.4 16.3</td>
<td>1000 670</td>
<td>5.0</td>
<td>34.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Test piles discussed in this paper were all constructed following the procedure discussed in section 2. All piles were instrumented with VWSG at five to seven different levels to estimate the shaft friction transferred to different soil horizons. One set of telltale extensometers was also used near the tip of each test pile to measure the elastic shortening of the pile shaft and finally pile base movement at different stages of load testing. Static maintained load testing method was used for all piles. Test loads were applied using a system of hydraulic jacks against the reaction frame of steel girders fixed against anchored reaction piles. Normally three cycles of loading are applied with first cycle up to the design load $Q_d$ and maintained for 12 hours followed by second cycle of loading up to 2 times of $Q_d$ and maintained for 24 hours. Third cycle of loading is applied up to 2.5 times $Q_d$ or maximum pre-decided test load. Some times a fourth quick loading cycle up to the maximum test load is also applied and maintained for two hours. All test piles except TP10 were tested to well above the expected failure loads with the concept of sufficiently mobilizing the end bearing.

5 TEST LOAD RESULTS

Test load results with pile dimensions are summarized in Table 1 and Figures 5 to 14. It is quite evident that the safe design loads (Table 1, col.13) calculated from the average of failure loads, interpreted by different available methods (cols.7-11), are in reasonable agreement with the estimated design loads (col.3). But the settlements at the design loads (col.15) have a wide scatter between 2.5mm and 6.8mm. So instead of computing failure loads from methods mentioned in Cols. 7-11, if we use the limit load concept at settlement of 1% of the diameter (col. 18) and then use a partial FOS of 1.5 to calculate design loads (col.19), resulted settlements at this design load (col. 20) will be quite uniform and within the limits of allowable settlement at design loads for high rise buildings. Global FOS against overall ultimate failure will still be in excess of 2.5 against the ultimate failure of the pile which still have not been reached even at large pile head movements (Buttling, 1992). This can also be confirmed from the maximum end bearing values mobilized under maximum test loads i.e. only 25% of the ultimate end bearing is mobilized (ref. Section 5.2).

5.1 Skin friction

Actually mobilized shaft friction capacities (ref. Table 1, col.4) are on the average by 50% higher than the estimated skin friction values with the exception of piles TP8 and TP10 who have 19% and 27% less capacity respectively. It must be noted that TP10 was not tested to failure so ultimate shaft capacity was not yet mobilized.

Unit skin friction values mobilized in different soil layers are given in Table 2. It must be noted that the unit skin friction values reported here are the actual maximum mobilized capacities only. Since piles were not instrumented to measure the corresponding displacements at the interface of each soil horizon, minimum values are not reflecting the ultimate unit friction values.

Table 2. Skin friction values mobilized in different soil horizons and corresponding $\alpha$ values.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>SPT-N</th>
<th>Ave. Cu</th>
<th>Skin Friction</th>
<th>$\alpha$ (Ave.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Mobilized (ton/m²)</td>
<td></td>
</tr>
<tr>
<td>Soft to medium Clay</td>
<td>2-12</td>
<td>4.7</td>
<td>6.0</td>
<td>13.9</td>
</tr>
<tr>
<td>Stiff to very stiff Clay</td>
<td>20-35</td>
<td>14.0</td>
<td>7.5</td>
<td>18.8</td>
</tr>
<tr>
<td>Hard Clay</td>
<td>40-46</td>
<td>21.5</td>
<td>7.3</td>
<td>24.6</td>
</tr>
<tr>
<td>First silty Sand</td>
<td>18-68</td>
<td>-</td>
<td>11.2</td>
<td>19.9</td>
</tr>
<tr>
<td>Second silty Sand</td>
<td>20-100</td>
<td>-</td>
<td>15.6</td>
<td>25.1</td>
</tr>
</tbody>
</table>

Average skin friction values mobilized in the soft to medium clay layers seems to be even higher than average Cu of these layers, most likely reasons for this are: 1) actual diameter in the soft to medium clay layer is slightly higher than the nominal diameter of the pile due to the use of temporary casing; 2) actual Cu of the top few meters of weathered crust is higher than the Cu used in the analysis. Values of $\alpha$ reported in Table 2 are quite conforming to the previous recommendations like Ng, (1983) and Tomlinson (1995). Skin friction values mobilized in the very stiff to hard clay layers are relatively low, since at failure loads considerable portion of the pile head settlement is absorbed by the long pile shaft itself which reduces the relative pile/soil interface movement near the deeper soil layers which results in partial mobilization of skin friction capacity of these soil horizon. In other words, in case of long piles the geotechnical capacity of deeper soil layers may not be fully exploited before they are declared to be failed due to the excessive pile head movement, same is the case for end bearing of these piles. Average values of unit skin friction mobilized in the first sand and second sand layers are 11.2 ton/m² and 15.6 ton/m² respectively which corresponds to a $\beta$ value of 0.45 to 0.55, without considering piezometric draw down discussed in section 1. But if we consider piezometric draw down, corresponding $\beta$ values will be 0.23 to 0.28. This confirms the suggestions made.
by Meyerhof (1976) who concluded that $\beta$ value is also dependent on the length of the pile and can be as low as 0.15 for very long piles. Maximum unit skin friction mobilized in sand layers is also comparable to test results by Reese and O’Neill (1988) who measured a maximum value of 19.15 ton/m$^2$ for sand layers.

Dense sand and very stiff to hard clay layers found to exhibit a brittle type of failure mechanism in majority of the cases, a typical family of curves representing such type of behavior are shown in Figure 2. Residual shaft friction in dense sand layers with increasing butt loads found to be dropped to 50% of the maximum mobilized values in some cases. For very stiff to hard clay layers, such reduction is found to be as high as 60%.

Failure mechanism of different soil horizons with increasing butt loads exhibit a progressive type of behavior for some test piles. A typical, unit skin friction development with increasing butt load (TP10) is shown in Figure 3. In number of cases dense sand and very stiff to hard clay layers found to reach their ultimate unit friction values well before the maximum test loads which shows that pile head displacements required to mobilize ultimate shaft friction in these layers are quite small. An other point to be noted here is that the progressive failure mechanism found to be started from the layers present in the middle reach of the pile shaft and then it extended towards top and bottom. Once the sand and stiff to hard clay layers started yielding most of the loads were shifted to the medium and soft clay layers at the top.

5.2 End bearing

End bearing values mobilized under maximum test loads are given in Table 3. It is quite evident that considerable portion of the total pile head settlement is absorbed by the elastic shortening of the pile shaft. So even at 3 to 3.5 times working loads maximum pile tip movement is less than 7% of the diameter of the pile. Ultimate end bearing values which need a pile base settlement of 20% to 30% of the pile base diameter (Tomlinson, 1995) are still only 25% mobilized.

Maximum mobilized end bearing for piles with tips in the sand layers are plotted in Figure 4. It is clear that the estimated end bearing value of 500 ton/m$^2$ in most of the cases correspond to a base settlement of approximately 7% of the pile diameter and this level of settlement have rarely been achieved even at such a high pile top displacements.

End bearing mobilized in the sand layers for different values of base settlement (ref. Figure 4) match well with the recommendations of Reese and O’Neill (1988) who recommended a value of critical end bearing equal to 5.76*SPT-N (ton/m$^2$) corresponding to a base settlement of 5% of the pile base diameter for sand layers of similar relative density. Since the average value of SPT-N for test piles founded in sand layers is approximately 60, critical end bearing comes out to be 5.76*60 = 350 ton/m$^2$ and from the test piles plotted data Figure 4,
Figure 4: Development of end bearing with base settlement as % of pile diameter.

Critical end bearing corresponding to 5% base settlement is 270 ton/m², a difference between the predicted value of end bearing is due to the reason that end bearing values predicted by Reese and O’Neill (1988) are based on the SPT data with energy ratio (ER) of 0.55, while SPT-N used for the test piles are not corrected for ER.

Back calculated average value of Nc for clay
Figure 8: Test pile TP-4

Figure 9: Test pile TP-5

Figure 10: Test pile TP-6

Figure 11: Test pile TP-7
layers is 8 with 4.5, 6.0 and 12.8 for the three piles seated in clay layers. From Table 3 it can be confirmed that the end bearing values in clay layer also depend upon the base settlement and values of $N_c$ more than 9 can also be mobilized at high base settlement ratios. An other possible reason for the value of $N_c$ more than 9 may be the underestimation of undrained shear strength of clay from correlation with SPT-N discussed in section 3.1.

6 CONCLUSIONS

1. Skin friction and end bearing values of wet process bored piles mobilized in the subsoils of Bangkok are reported and compared with the previous researches made for similar cases.

2. Dense sand and very stiff to hard clay layers often exhibit a brittle type of failure with residual friction capacities as low as 50 to 60% of the peak mobilized values.

3. Failure of long piles in multi-layered soil is dominantly progressive in nature and some soil horizons reach their ultimate friction capacities well before the over all ultimate failure loads.

4. Calculation of ultimate failure loads from the normally available methods and then design

Figure 12: Test pile TP-8

Figure 13: Test pile TP-9

Figure 14: Test pile TP-10
loads by applying a global FOS give widely scattered settlements at design loads and if we calculate design loads from limit load concept, quite uniform settlement at design loads can be achieved.

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REFERENCES


Effect of construction time and bentonite viscosity on shaft capacity of bored piles

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Effect of construction time and bentonite viscosity on shaft capacity of bored piles

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Seafco Co., Ltd., Bangkok, Thailand

ABSTRACT: Effect of construction time and slurry viscosity on the shaft friction capacity of cast in situ bored piles have been studied by various investigators in the past but the extent to which these parameters effect the shaft capacity is still not clear. Major obstacle in this regard is that it is hard to normalize the shaft capacity degradation against other numerous parameters which are influencing simultaneously. Results from eleven pile load tests, constructed with different slurry viscosities and construction times, in the layered alluvial strata of Bangkok are presented in this paper.

It has been concluded that slurry viscosity, do not have significant effect on the shaft load transfer of these piles but considerably reduced with increase in construction time. Trend of reduction in capacity seems to follow an exponential decrease with increase in construction time with major part of degradation within first 24 hours of construction time.

1 INTRODUCTION

Shaft friction capacity of bored piles, especially, in granular soils is greatly affected by the installation procedures, but the sole effect of bentonite slurry (slurry) properties and its exposure time is still unclear. Results from the previous researches on deleterious effects of slurry properties and its contact time on the shaft load transfer of wet process bored piles are a bit anecdotal and to some extent contradictory. Major problem in this regard is that, the effect of either parameter can not be totally isolated from other influencing variables. Due to this reason researches have been found to report almost converse conclusions for the similar type of soils.

In order to assess the effect of slurry viscosity and exposure time on the shaft load transfer, data from eleven instrumented boreed pile load tests is presented in this paper. All test piles were constructed using same type of equipment and procedures, in a similar type of soil conditions. So it is assumed that the influence of other variables would be minimized and the effect of slurry viscosity and exposure time, to some extent, could be estimated.

2 BORED PILES AND SOIL CONDITIONS

Use of bored cast in situ piles in Bangkok, is very common. Diameters of these piles normally fall in the range of 0.80 to 1.50 m and toe depths down to 60 m from ground level are quite usual. Soil profile of plain of Bangkok generally consists of Quaternary alluvial deposits of alternating sand and clay layers as shown in Figure 1, down to the rock face which is reported to be at least 550 m deep in the area (Balasubramanium, 1991).
Below the top few meters of weathered crust, a thick layer of well known “Bangkok Soft Clay” (BSC) changing to medium at about 15 to 18 m, is present. First sand layer is usually 5 to 10 m thick and found at 25 m to 30 m depth, below is a series of stiff to hard clay and medium to very dense silty sand layers. Actual pore pressure conditions in the upper BSC are hydrostatic from approximately 1 m below ground level. Then the hydrostatic conditions changes to piezometric draw down near the bottom level of BSC, Figure 1. Piezometric draw down resulted in increased effective overburden pressure of about 20 ton/m2 in the first sand layer and below. The depressurization of sand layers is attributed to the deep well pumping in the area, Nutalaya (1981).

3 REVIEW OF PREVIOUS WORK

Degradation of perimeter load transfer in permeable soil layers is attributed to the formation of filter cake which is left in place and get sandwiched at soil-pile interface and is not scraped off by fluid concrete. Scouring capability of fluid concrete depends upon its shear strength and that of the filter cake and it is argued that if shear strength of filter cake is more than the fluid concrete, it can not be scoured and left in place resulting in the degradation of load transfer capacity of the shaft. In case of impermeable soils reduction in the shaft load transfer is mainly due to the softening of the soil at the perimeter of the borehole, because the formation of thick filter cake is not possible as slurry can not permeate in to the soil, but some researchers e. g. Veder, C. (1963) observed a thin cake of few millimeters even in clay formations and argued that it may be the result of electrical forces or chemical reaction of bentonite suspension on the wall of the borehole.

O’Neil et al. (1992), reported the effect of slurry properties on two drilled shafts constructed in similar ground conditions (alternating layers of very stiff clay and dense sand). Shafts 1 and 2 were constructed with Marsh cone viscosity of 37 and 49 sec. respectively, time between opening the borehole and completion of concreting for both piles was 5 to 7 hours. Minor difference in load transfer of both shafts was found and authors concluded that the difference in slurry properties used for the construction for both shafts had a little effect on shaft load transfer.

Cernak (1976), performed full-scale load tests on three barrettes in sandy gravels, two barrettes were constructed using slurry with different exposure times, 8 and 97 hours. Load test results indicated a decrease of skin friction capacity of 43 percent and 56 percent, respectively, as compared to the barrette excavated dry and concreted immediately. It must be noted that major part of the reduction i. e. 43 percent took place in the first 8 hours of contact time and only 13 percent (56-43) reduction took place in the rest of 89 hours (97-8), for the second barrette. These findings support the concept that major part of the shaft friction capacity reduction with increase in construction time, took place in first few hours of construction time and further increase in construction time have minor contribution towards the reduction in shaft friction capacity.

Corbette (1975), presented the results of load testing of a diaphragm wall panel excavated in conjunction with bentonite suspension, and construction time of approximately 48 hours. He inferred that the presence of bentonite suspension even for such a long time has not adversely affected the development of skin friction.

Littlechild and Plumbridge (1998), concluded from the pile load tests of BERTS project Bangkok, that shaft friction capacity of bored piles constructed under slurry tends to decrease as the construction time and slurry viscosity increases.

Majano and O’Neil (1993) attempted to model the formation of filter cake in the laboratory with different slurry dosages, differential pressures and exposure times. They stated that perimeter load transfer is a complex function of the physical and chemical characteristics of the slurry and the geomaterials, the roughness of the borehole, the fluid pressures exerted by concrete, the shearing properties of the soil, and possibly the chemistry of the fluid concrete. They attempted to correlate the potential degradation of the soil-pile interface to the thickness and shear strength of the cake formed against the walls, and argued that the shear strength and thickness can not be measured with only one or two parameters like slurry dosage and differential pressure. With a bentonite concentration of 72 kg/m3 and differential pressure of 0.5 psi they achieved a filter cake thickness of 3.14 and 4.5 mm after a contact time of 4 and 24 hours. Authors also compared the laboratory tests with the actual field conditions and proved that due to the presence of enormously high differential pressures in the field in some cases, thicknesses of filter cakes which would be considered detrimental for shaft load transfer, require few hours to develop (They proved that a filter cake of 6 mm thickness would require only 2.3 hours to develop in the field under a differential pressure of 10 psi).

4 METHOD OF CONSTRUCTION

All the eleven piles reported in this paper were constructed, following the internationally accepted
guidelines and specifications. Slurry in conjunction with rotary bucket is used for all piles.

Temporary casing, having length depending upon the thickness of the BSC at the site is driven using a high frequency vibro-hammer. Drilling operation is commenced with an auger down to the base of the temporary casing. Further drilling below the bottom of temporary casing is continued using a drilling bucket by first filling the borehole with slurry. Bentonite, classified as Activated Sodium Bentonite is usually used and slurry properties are controlled within the limits given in Table 1.

Table 1. Slurry specifications followed.

<table>
<thead>
<tr>
<th>Slurry Property</th>
<th>Control Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Viscosity (Marsh cone)</td>
<td>30 - 60</td>
</tr>
<tr>
<td>Density (g/cc)</td>
<td>1.04 - 1.15</td>
</tr>
<tr>
<td>pH</td>
<td>7 - 11</td>
</tr>
<tr>
<td>Sand Content</td>
<td>&lt;4%</td>
</tr>
</tbody>
</table>

Outer diameter of the drilling bucket is slightly less than the nominal size of the pile to provide an annular by-pass for the slurry during lowering and hoisting operations. Required nominal size of the pile is achieved using a cutting edge with sidecutters. Slurry level in the borehole is maintained within 1 to 1.5 m from top of temporary casing. After drilling down to the required tip level of pile, special cleaning bucket and/or air lift technique is normally applied to clean the borehole base of any congregated sediments before lowering the reinforcement cage. Concreting is done using a tremie pipe at the center of the borehole. Total time to carry out this operation, starting from the casing driving till the completion of concreting, usually fall in the range of 10 to 20 hours excluding some accidental delays due to equipment break down, delay in ready mixed concrete due to traffic congestion or similar reasons. In most of the cases whole operation of pile construction is accomplished within 24 hours, but in some cases, due to the local authority regulations which do not permit to operate during night time hours, borehole has to be left exposed for 10 to 12 hours unagitated. In such cases, drilling is intentionally curtailed above the final excavation level and resumes next day. We assume that in such cases, during drilling after a delayed period of 10 to 12 hours, any possible filter cake on the borehole walls is automatically scraped of. Construction times more than 24 hours reported in Table 2 are examples of such cases. Construction time reported in Table 2, col. D is the time elapsed between the start of auger drilling to the finishing of concreting. Longer construction times for the test pile reported are usually due to the time consumed in instrumentation installation with the rebar cage. It must also be noted that the actual contact time for slurry starts, once it is fed to the borehole till the up flowing concrete reaches the bottom level of temporary casing. Since the accumulated time required for concreting above the bottom level of temporary casing and auger drilling inside the temporary casing roughly required one hour, construction times given in Table 2 are supposed to be appropriate for comparison.

5 SHAFT CAPACITY ESTIMATION

A commonly accepted design approach which consists of a combination of total stress method for clay layers and effective stress method for sand layers is adopted to estimate the shaft friction capacity reported in Table 2, col. F. Skin friction capacity of clay layers is estimated as $f_s = \alpha C_u$, with adhesion factor $\alpha$ taken from the curve proposed by Suchada (1989), Figure 2, for Bangkok subsoils. It can be noted that value of $\alpha$ used is quite comparable to the previous recommendations by Tomlinson (1957) and Kulhawy (1984).

![Figure 2. Comparison of Adhesion factor $\alpha$ used for bored piles in Bangkok Subsoils.](image)

Undrained shear strength $C_u$ of shallow clay layers is normally determined in the laboratory with unconfined compression test while for deeper stiff to hard clay layers, $C_u$ is indirectly estimated from Standard Penetration Test (SPT-N), as $C_u = C_1 SPT-N$ (ton/m2), with $C_1$ equal to 0.674 and 0.507 for high and low plasticity clays respectively. $C_1$ values mentioned here are based on the statistical correlations suggested by Pitupakorn, 1985. Skin friction capacities for the sand layers are calculated as $f_s = \sigma v'$. $K_s \tan \delta$ with coefficient of horizontal earth pressure $K_s$ equal to 0.7 and $\delta$ equal to 0.75 $\phi$.  

33
Table 2. Construction parameters and shaft capacities of test piles.

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Pile Dimensions (m)</th>
<th>Bentonite Slurry Viscosity (sec.)</th>
<th>Construction Time (hrs)</th>
<th>Pile Age at the Time of Load Testing (days)</th>
<th>Estimated Shaft Capacity (ton)</th>
<th>Actual Shaft Capacity (ton)</th>
<th>Actual/Estimated Shaft Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-1</td>
<td>φ1.2x57.1</td>
<td>38</td>
<td>43.0</td>
<td>32</td>
<td>1230</td>
<td>1200</td>
<td>0.98</td>
</tr>
<tr>
<td>TP-2</td>
<td>φ1.2x46.3</td>
<td>38</td>
<td>13.1</td>
<td>29</td>
<td>930</td>
<td>1300</td>
<td>1.40</td>
</tr>
<tr>
<td>TP-3</td>
<td>φ1.0x46.5</td>
<td>37</td>
<td>9.8</td>
<td>25</td>
<td>800</td>
<td>1250</td>
<td>1.56</td>
</tr>
<tr>
<td>TP-4</td>
<td>φ1.0x49.5</td>
<td>37</td>
<td>38.7</td>
<td>15</td>
<td>930</td>
<td>750</td>
<td>0.81</td>
</tr>
<tr>
<td>TP-5</td>
<td>φ1.0x43.0</td>
<td>38</td>
<td>26.0</td>
<td>19</td>
<td>690</td>
<td>700</td>
<td>1.01</td>
</tr>
<tr>
<td>TP-6</td>
<td>φ1.0x41.0</td>
<td>45</td>
<td>11.8</td>
<td>32</td>
<td>620</td>
<td>800</td>
<td>1.29</td>
</tr>
<tr>
<td>TP-7</td>
<td>φ1.2x43.6</td>
<td>55</td>
<td>16.8</td>
<td>32</td>
<td>850</td>
<td>1100</td>
<td>1.29</td>
</tr>
<tr>
<td>TP-8</td>
<td>φ1.2x43.5</td>
<td>38</td>
<td>12.3</td>
<td>24</td>
<td>640</td>
<td>750</td>
<td>1.17</td>
</tr>
<tr>
<td>TP-9</td>
<td>φ1.0x43.5</td>
<td>37</td>
<td>11.3</td>
<td>18</td>
<td>530</td>
<td>700</td>
<td>1.32</td>
</tr>
<tr>
<td>TP-10</td>
<td>φ1.2x62.0</td>
<td>41</td>
<td>32.4</td>
<td>27</td>
<td>1534</td>
<td>2000</td>
<td>1.30</td>
</tr>
<tr>
<td>TP-11</td>
<td>φ1.2x54.2</td>
<td>38</td>
<td>30.0</td>
<td>39</td>
<td>1050</td>
<td>1300</td>
<td>1.24</td>
</tr>
</tbody>
</table>

Value of $K_s$ used is in good agreement with the recommended values by different researchers like Fleming (1977), who suggested a value not more than 0.75 to account for the possible degradation due to the formation of filter cake. Angle of internal friction $\phi$ is also estimated from SPT-N values by first correcting the $N$ values for overburden correction (Bowels, 1988). Some designers also use an equivalent term of $\beta$ instead of $K_s \tan \delta$. It must be noted that the empirical parameters discussed here are based on the effective overburden pressure $\sigma'_{\text{v}}$ which is calculated with the assumption of hydrostatic conditions from the ground level, ignoring the piezometric draw down conditions below BSC shown in Figure 1.

6 TEST PILES

Test piles discussed in this paper were all constructed following the procedure discussed in section 4. All piles were instrumented with Vibrating Wire Strain Gauges (VWSG) at five to seven different levels to estimate the shaft friction transferred to different soil horizons. One set of telltale extensometer rods was also used near the tip of each test pile to measure the elastic shortening of the pile shaft and finally pile base movement at different stages of load testing. Static maintained load testing method was used for all piles. Test loads were applied using a system of hydraulic jacks against the reaction frame of steel girders fixed against anchored reaction piles. Normally three cycles of loading are applied with first cycle up to the design load $Q_d$ and maintained for 12 hours followed by second cycle of loading up to 2 times of $Q_d$ and maintained for 24 hours. Third cycle of loading is applied up to 2.5 times $Q_d$ or maximum pre-decided test load. Occasionally, a fourth quick loading cycle up to the maximum test load is also applied and maintained for two hours. All test piles, except TP-2 and TP-10, were tested to well above the expected failure loads with the concept of sufficiently mobilizing the end bearing. Load displacement curves of the test piles are shown in Figure 3 for comparison.

7 ACTUAL SHAFT FRICTION CAPACITIES

Actually mobilized shaft friction capacities are calculated from the VWSG load transfer results at a level where load displacement curves show a maximum curvature and are given in Table 2, col. G.

8 EFFECT OF SLURRY VISCOSITY

Effect of slurry viscosity on the shaft load transfer can best be seen from the plot between the ratio of Actual/Estimated shaft capacity versus slurry viscosity, Figure 4. Over all scatter of data do not suggest any trend. Since the effect of slurry viscosity is difficult to separate, without normalizing the effect of construction time. Test piles TP-6, 7 and 9 have a slight difference in construction time and pile age at load testing but slurry viscosity is significantly different, 45, 55, and 37 sec. respectively but the ratio of Actu./Esti. shaft capacities for these piles are necessarily the same.
5, plot between Actu./Esti. shaft capacity versus construction time. Though the data is widely scattered, best fit curve shows the trend of exponential decrease of shaft load transfer with increase in construction time. Best fit curve for the available data also reveals that the available shaft capacity is more than the estimated capacity for a construction time as long as 40 hours. This confirms the validity of recommendations made by some researchers like Fleming (1977), who suggest to complete the concreting within 24 hours after finishing the drilling, and seems to be more reasonable because slurry is kept continuously agitated during drilling operation and actual exposure time for slurry starts, once the drilling is completed. Test pile TP-4 which exhibited the lowest capacity and has the minimum slurry viscosity (37 sec.), second longest construction time (38.7 hours) and the minimum age (15 days) at the time of load testing, in the group reveals that the shaft capacity reduction is contributed by increased construction time as well as reduced pile age while slurry viscosity have minor effect.

It can also be noted that the major part of the shaft capacity reduction is contributed by the first 24 hours of construction time which normally considered feasible and agreeable by most of the piling contractors under normal circumstances, and further delays beyond 24 hours have a minor addition to the shaft load capacity degradation. These findings are in good agreement with the previous observations discussed in section 3.

Figure 3. Load settlement curves of the test piles.

Figure 4. Effect of slurry viscosity on shaft capacity of bored piles.

which suggest that the sole effect of slurry viscosity on shaft capacity reduction is insignificant. Additionally, TP-1, 4 and 5 which exhibit a maximum reduction in shaft capacity have almost same slurry viscosity but different construction time which confirms that slurry viscosity within the data set available do not directly have any effect on the shaft load transfer of the piles.

9 EFFECT OF CONSTRUCTION TIME

Since ten out of eleven bored piles were constructed with slurry viscosity varying within narrow range effect of construction time can best be seen in Figure
From Figure 5 it can be noted that actually mobilized shaft capacity is equal to the estimated shaft capacity for a construction time of approximately 35 - 40 hours or in other words, the empirical design parameters commonly used to estimate the shaft friction capacities discussed in section 5, are justifiable for a total construction time of longer than 24 hours, especially, to account for the unexpected delays during construction in the downtown areas of Bangkok which are notorious for its traffic congestion and other constraints imposed by local authorities. This also confirms that the allowable time of 24 hours after finishing the drilling operation (Fleming, 1977) is also justifiable. However, with no dispute, shorter the construction time better the shaft load transfer response.

It must therefore be emphasized that the selection of empirical parameters like $\alpha$, $\beta$, $\delta$ and $K_s$, to estimate shaft load capacities at the design stages must also consider the practically possible construction times by suitably incorporating the effects of any possible delays specific to the site and working conditions. Empirical factors mentioned above are not the fundamental properties of the soil-pile system, and need to be accounted for the installation procedures or even the method of pile load testing like Constant Rate of Penetration (CRP) or Maintained Load (ML). Reduction in construction time, generally, lead to better performance of pile shafts but this reduced construction time could never exclude the accidental occurrence of delays. Any potential degradation in load carrying capacity of such piles caused by uncontrollable delays must be considered along with the conceivable trend in reduction under these conditions.

It must also be noted that because of high differential pressures imposed due to depressurized sand layers in Bangkok sub soils (differential pressures of the order of 30 psi will be imposed if slurry level is maintained near the ground level due to piezometric draw down of 20 m in the sand layers), formation of filter cakes which are considered to degrade the shaft load transfer would theoretically take fraction of seconds to develop, if we calculate on the basis of procedure recommended by Majano (1993). This means that the degradation of pile shaft interface due to the formation of filter cake in permeable soil layers is unavoidable within the practical construction time limits.

11 CONCLUSIONS

1. Bentonite slurry viscosity within the data set available do not have significant effect on the shaft load transfer of bored cast in situ piles.
2. Longer construction time reduces the shaft carrying capacity of bored piles constructed under slurry. However, the major portion of degradation occurs within the first 24 hours period, which is unavoidable under normal construction time.

3. Degradation of shaft capacity with increase in construction time beyond 24 hours is not excessive and can be adjusted within the empirical design parameters or factor of safeties chosen at the design stages.

4. Piles constructed within 24 hours showed generally higher shaft capacities than estimated as per normal design practice adopting empirical parameters suggested by different researchers for Bangkok subsoils and can safely be used for slurry viscosity up to 55 sec. and total construction time up to 40 hours.

AKNOWLEDGEMENTS

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REFERENCES

Baker, Jr. C. N. et al. (1970), Load Carrying Capacity Characteristics of Drilled Shafts Constructed with the Aids of Drilling Fluids, Research Report 89-6, Center for Highway Research, The University of Texas at Austin, TX.


Corbett, B. O. et al. (1975), A Load Bearing Diaphragm Wall at Keinsington and Chelsa Town Hall, London, Conf. on Diaphragm Walls and Anchorages, ICE, London.


Nutalaya, P. (1981), Subsidence Studies of the Bangkok Plain, AIT Research Reports.


O’ Neil et al. (1992), Behaviour of Axially Loaded Drilled Shafts in Beaumont Clay, The University of Texas at Austin.

Performance comparison of bored and excavated piles in the layered soils of Bangkok

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Performance comparison of bored and excavated piles in the layered soils of Bangkok

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PERFORMANCE COMPARISON OF BORED AND EXCAVATED PILES, IN THE LAYERED SOILS OF BANGKOK

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ABSTRACT: Installation techniques are considered to effect the performance of cast in situ piles. This paper compares the load transfer characteristics of a fully instrumented bored pile (Dia. 1.5 m) and barrette (1.5 x 3.0 m) having same length equal to 57 m, embedded in identical ground conditions (on same site, 30m apart). Total time consumed to construct the pile and barrette differ considerably, 27 and 75 hours, respectively. No significant difference in load transfer has been observed between the pile and the barrette. It has been concluded that, by observing good engineering practices, any conjectured degradation effects on load capacity of such piles can be eliminated. Additionally, the load applied on barrette was one of the highest test load in the region with a reaction frame “tower” capacity of 6000 tons. Reaction frame and loading system arrangement, to achieve such a high load capacity is also discussed.

1. INTRODUCTION

Barrettes become a common foundation element in combination with bored piles and diaphragm walls in Bangkok (Refer: Thasnanipan et al, 1998-b). Foundations for the BECM tower project located at Rama IX road Bangkok, Thailand, required 560 bored piles of 1.2 m and 1.5 m diameter and 24 number of barrettes having cross section size of 1.5 m x 3.0 m. Foundation plan of the tower is shown in Figure 1. Barrettes were necessary to transfer the very high loads coming from the central 52 story tower’s lift shafts. Piles were approximately 57.0 m long with tips embedded into the second sand layer. Depending upon the load requirements piles were base grouted in the central tower area and at the locations of high column loads. Load testing on working piles was required to verify the design parameters, and it was proposed to test one bored pile of 1.5 m diameter and one barrette with full instrumentation. Since, pile and barrette were constructed using two different techniques and the parameters like construction time, bentonite properties etc, were considerably different, results from the load tests have provided a unique chance to assess the extent of difference in behavior, which have been speculated to effect considerably in the past by some theoreticians.

2. SOIL PROFILE

Soil profile at the site is no different than the typical soil profiles of plain of Bangkok and is shown in Figure 2. Below the top 2 m of weathered crust 11.5 m thick Bangkok Soft Clay is present. First stiff clay starts from 13.5 m depth and extends to 26 m, below is the First sand layer and is about 10 m thick. Below the First sand layer a thick layer of Hard clay which extends down to 54 m, is present, and then starts the second sand layer which extends down to about 80 m where the next hard clay layer starts. Undrained shear strength, Su obtained from unconfined compression test and Standard Penetration Test (SPT) are shown in Figure 2. Ground water table depth at the site generally varies from 1 to 1.5 m, piezometric conditions of the site are also quite similar to the other parts of the Bangkok with piezometric draw down of about 22 m near the bottom of the first stiff clay layer.
3. CONSTRUCTION PROCEDURE

3.1 Bored Pile

Wet process method with rotary drilling bucket was employed. Bentonite slurry conforming to the widely accepted specifications were used as supporting fluid. Top 15 m soft clay was temporarily cased to assure the stability of the borehole. Firstly, auger was used to drill within the temporary casing, followed by rotary bucket with bentonite slurry down to final depth of excavation. Airlift technique was applied to clean the borehole base of any congregated sediments. Before lowering the reinforcement cage special cleaning bucket was used to scrap of the borehole walls and the base. Reinforcement cages were lowered inside the borehole by simultaneously attaching the instrumentation at specified locations. Soon after lowering the rebar cage tremie concreting was started. Properties of the bentonite slurry used are given in Table 1. Polystyrene grains plug was used before the first charge of concrete to avoid the mixing of bentonite with concrete. Time consumed in different construction activities is plotted in Figure 3.

3.2 Barrette

Mechanical rope-grab in conjunction with bentonite slurry was used to excavate the trench. A guide wall cast with inside clear dimensions slightly more than the nominal size of the barrette was used to guide the grab during initial bites. Since time consumed in the preparation of instrumentation was quite long, refer Figure 3, desanding was continuously done to keep the bentonite slurry agitated, which also helps to minimize the growth of filter cake by actually reducing the exposure time.
Figure 2. Comparison of bored pile and barrette, with soil profile at the site.

As another measure, trench was once again occupied by grab to scour the trench walls and to remove, if any, filter cake formed on the walls (Refer: Drilled Shaft Inspector’s Manual, 1989). It is authors opinion that, if due to some unforeseen reasons, reinforcement cage lowering have to be delayed for considerable period of time or for overnight. It is a good practice to use the grab again to scrap the trench walls which eliminates any foreseeable bad effects due to such unexpected delays. After lowering the rebar cage, tremie concreting was done.

Table 1. Comparison of bentonite slurry properties.

<table>
<thead>
<tr>
<th></th>
<th>Pile</th>
<th>Barrette</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Before feeding to the borehole</td>
<td>Before feeding to the trench</td>
</tr>
<tr>
<td></td>
<td>After Recycling &amp; Before concreting (near borehole base)</td>
<td>After Recycling &amp; Before concreting (near trench base)</td>
</tr>
<tr>
<td>Viscosity (sec)</td>
<td>33</td>
<td>36</td>
</tr>
<tr>
<td>Density (g/cc)</td>
<td>1.08</td>
<td>1.10</td>
</tr>
<tr>
<td>pH value</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>Sand Content (%)</td>
<td>0.1</td>
<td>0.8</td>
</tr>
</tbody>
</table>
Figure 3. Comparison of time consumed in different construction activities.

4. INSTRUMENTATION

Both, pile and barrette were instrumented with Vibrating Wire Strain Gauges (VWSGs) and Mechanical Extensometers (ME) at five levels along the pile shaft at the known interface boundaries of different soil layers. Pile has four sets of VWSGs and one set of ME at each level. While the Barrette has six sets of VWSGs and two sets of ME at each level, as depicted in Figure 2.

5. LOAD TEST ARRANGEMENT

Reaction frame anchored against the four working piles was used to load the piles. Reaction frame used to load the barrette was one of the biggest of its kind used in the region with overall height equal to about 8 m as shown in Figures. 4a and 4b. Five numbers of built-up steel girders supported on each side by two 1st level cross girders were used to achieve the required maximum frame capacity of 6000 tons. 1st level crossbeams were supported against the 2nd level cross girders. 2nd level cross girders were then anchored against the surrounding barrettes using anchor blocks at the top. Especially fabricated rigid transfer girders as shown in Figure 5 were used to distribute the tension force coming from the Tie-bars to the dowel bars above the anchor barrette heads. Sixteen number of hydraulic jacks each having 500 ton capacity, were installed between the barrette cap and main girder of the reaction frame. Jacks were arranged in a symmetrical fashion as shown in Figure 4b, to avoid any possible eccentricity of loading. Ball bearings were used between the main girders and hydraulic jacks to keep the line of loading in the true vertical direction.

6. EFFECT OF SHAPE ON SHAFT LOAD TRANSFER

Assessment of shape effect on the shaft load transfer has been attempted by various researchers in the past. Hosoi et al (1994), concluded that the earth pressure acting on the flat surface of a rectangular diaphragm wall panel is larger than that of circular bored piles. Numerical analysis performed by them showed that the earth pressure on the flat surface of a trench with L/B = 1000 (plain strain conditions), where, L and B are the length and width of the trench in plan respectively, is higher than that of circular borehole. In order to assess the effect of different aspect ratios (L/B) on the earth pressure developed around the trench, attempt has been made to model the behavior of trench using finite element computer program. Figure 7 shows the orientation of principal stresses and stress contours developed around the trenches with different L/B ratios. It has been observed that sufficient hoop compression stresses develop around the trench and if L/B < 3. But if L/B is increased above 5, longer sides of the trench start yielding and it can be concluded that under such conditions hoop stresses around the trench become less effective which give rise to the increased
Figure 4a. View of the test frame having maximum capacity of 6000 tons.

Figure 4b. Barrette load test plan.
Figure 5. Stiffening girder used to distribute force to dowel bars.

Figure 6. A view of the instrumented barrette cap with jacks.
Figure 7. Orientation of principal stresses and stress contours around the excavated trenches with different aspect ratios.
earth pressure on flat surface of trenches with large aspect ratios. Although the exact determination of earth pressure developed on the surface of trench depends on the soil properties and the overburden pressure at the level of consideration, it can be concluded that no significant difference in earth pressure is observed between the bore hole (L/B=1) and the barrette (L/B=2).

7. RESULTS OF LOAD TEST

Values of unit skin friction developed at the interface of different soil horizons along the pile and barrette shafts are given in Table 2 and are also plotted in Figure 2. Comparison of unit skin friction values prove that there is no significant difference in shaft load transfer between pile and barrette, even though the construction methodology adopted is different. These findings are in line with the conclusions made by Thasnanipan et al (1998-a). Load settlement curves of the pile and barrette are shown in Figure 8.

Table 2. Comparison of unit skin friction and end bearing mobilized for the test pile and barrette.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soil Type</th>
<th>Mobilized Skin Friction (ton/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Pile At Design Load</td>
</tr>
<tr>
<td>0 - 13.5</td>
<td>Bangkok Soft Clay (CH)</td>
<td>0.20</td>
</tr>
<tr>
<td>13.5 - 25.0</td>
<td>Stiff - V. Stiff Clay (CH)</td>
<td>5.90</td>
</tr>
<tr>
<td>25.0 - 35.0</td>
<td>Med.-Dense Silty Sand (SM-SP)</td>
<td>11.46</td>
</tr>
<tr>
<td>35.0 - 50.0</td>
<td>Hard Silty Clay (CH)</td>
<td>3.09</td>
</tr>
<tr>
<td>50.0 - 55.0</td>
<td>3.5m Hard Clay + 1.5m Clayey Sand (SC)</td>
<td>3.43</td>
</tr>
</tbody>
</table>

Maximum Mobilized End Bearing (Ton/m²) 270 101

Figure 8. Comparison of Load Settlement Curves of Barrette and Pile.
8. CONCLUSIONS

1) No significant difference in shaft load transfer has been observed between the wet process bored pile and barrette, in spite of the presence of considerable difference in construction techniques followed and other relevant parameters like bentonite slurry viscosity and construction time.

2) All foreseeable negative effects attributed to construction parameters can be eliminated by observing good engineering practices.

3) Reaction frame can be designed and erected to apply very high compression loads of the order of 6000 tons for pile load testing.

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REFERENCES


Static Load Testing Report of Barrette Pile for BECM Tower Project, Rama IX Road, Prepared by EDE-STS Engineering Joint Venture, Bangkok, Thailand.

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Base grouting of wet process bored piles in Bangkok subsoil

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Base grouting of wet process bored piles in Bangkok subsoils.

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ABSTRACT: Base grouting techniques are widely used to rectify the soft toe problem of bored piles in Bangkok. Two different techniques, commonly known as flat jack and tube-a-manchette are used. This research study is mainly focused on the tube-a-manchette technique. Six piles were constructed with the provision of PVC-casing inside the shaft to allow the drilling through the pile bases in order to collect the soil samples below the pile tip. Piles were base grouted by varying the controlling parameters such as grout volumes, pressures and injection rates. Soil samples were collected from beneath and at some distance away from the pile tip, and analyzed. It is observed that grout does not penetrate into the surrounding sand of the pile toe even at high pressures. It partially replace and/or precompress the sediments present under the pile tip and most of the grout just rises up the soil/pile interface by replacing the bentonite cake. Recommendations are made to eliminate the soft toe problem more effectively.

1 INTRODUCTION

The method of constructing bored piles under bentonite suspension has become well established in Thailand. It is particularly used for large diameter piles, where powerful rigs can be used with rotary tools to bore through otherwise unstable water bearing strata. It is almost always used with a temporary casing to support the top most Bangkok Soft Clay (BSC), resulting in a hybrid of the casing and bentonite methods.

Construction problems are affecting integrity and performance of such piles. A pile without a sound toe and without a proper seat into the virgin ground should prove disastrous particularly if required to act in point bearing. Even otherwise, such piles could lead to higher orders of detrimental settlements. The known causes of ending up with a defective pile toe, are the loosened materials during boring operation and accumulation of sediments from the bentonite column at the base of the borehole. The technology of flowable concrete and tremie placement are well known to avoid this problem up to certain extent and it is already a standard procedure to use a cleaning bucket or an airlift or both to clear the borehole base of any soft or loose material.

Another solution was found in pressure grouting the base after the completion of the pile. This improves the condition of the soil and pre-stresses the pile. Base grouting is widely used in Thailand to rectify the soft toe problem.

2 BASE GROUTING IN BANGKOK

Teparaksa (1991-1992) reported the construction and performance of base-grouted bored piles in Bangkok subsoils. It was found that base grouting not only helps to improve the end bearing capacity but skin friction as well (ref: Table 1).

Table 1. Improvements made by base grouting (Teparaksa, 1994).

<table>
<thead>
<tr>
<th>Layer of Pile tip</th>
<th>Increase in failure load (%)</th>
<th>Increase in skin friction (%)</th>
<th>Increase in end bearing (%)</th>
<th>Displacement at fully mobilized skin friction (% of Pile Dia.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Sand</td>
<td>26-66</td>
<td>24-66</td>
<td>28-61</td>
<td>0.49 - 0.51, 1.34-1.35</td>
</tr>
<tr>
<td>2nd Clay</td>
<td>27</td>
<td>51</td>
<td>1</td>
<td>0.37, 0.74</td>
</tr>
<tr>
<td>2nd Sand</td>
<td>12-24</td>
<td>9-27</td>
<td>11-21</td>
<td>1.04, 1.59-1.86</td>
</tr>
</tbody>
</table>

NGB: Non Grouted Base, GB: Grouted Base

Pile base grouting was started in Bangkok in 1985, during the construction of Rama IX cable stayed bridge (Morrison, 1987). Here 172 bored piles, 2.0 m dia. and 35 m long seated in the 1st sand were base grouted. Tube-a-manchette technique was used with grout volumes up to 4000 liters and maximum pressures of 60 bar. With the passage of time engineers started applying this technique for the foundations of expressways, high-rise buildings and other heavy structures. In the absence of standard guidelines, the criteria and the controlling parameters changed with time. In current practice, pressures ranging from 20 to 60 bar and grout volumes 500 to 1000 liters are normally applied with Injection Rate (IR) varying from 5 to 36 liters/min.

3 GROUTING PROCEDURE

Two grouting circuits in the form of U-loops with manchettes placed at a level of 5 to 10 cm above the pile tip arranged in a
symmetrical fashion are used (Figure 1). Normal practice is to use PE tubes above the manchettes running along the rebar cage to the top, but if specification requires cross-hole sonic logging, same sonic logging tubes are used to connect the manchettes. Normal practice is to start grouting 24 hours after concreting when high pressure criteria is not to be strictly followed. This is especially suitable for congested piling plans where subsequent manipulation of equipment can damage the circuit tubes. If high pressure is to be maintained and/or sonic logging is required, grouting is started after seven to ten days. First the pile toe is cracked with high-pressure water flow to flush and open the grouting circuit to make way for forthcoming grout and then cement grouting is started. Grouting is stopped when the target grout volume or maximum required pressure is achieved.

4 TESTING METHODOLOGY

In order to investigate the mechanism of grout spreading beneath and in the surrounding soil of pile tip and the effect of different grouting parameters, an attempt was made to collect the soil samples from below the tip of the base grouted piles (Anwar, 1997). PVC-casings were placed in the prototype bored piles as shown in Figure 1 and 2, to allow for the passage of drilling tools down to the pile base. Three piles, 1.0 m in diameter and length in the range 50 to 55 m were constructed with the provision of 7.5 cm diameter PVC-casings at a site in Bangkok metropolitan area. Unfortunately, only one pile out of this group could be drilled because of the failure to lower the drilling tools inside the PVC-casings. For the second time another group of three piles having 1.5 m diameter and length 56 - 57 m were constructed. This time diameter of PVC-casing was increased to 10 cm. Positions of the PVC-casings were staggered with respect to the manchettes as depicted in Figure 1. All the three piles were base grouted following the parameters shown in Table 2. Table 2. Grouting parameters of the test piles.

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Grout Volume (Liters)</th>
<th>Injection Rate (Liters/min)</th>
<th>Maximum Pressure Achieved (bar)</th>
<th>Residual Pressure 2 min after closing the pump (bar)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-1</td>
<td>500</td>
<td>32 - 3*</td>
<td>26 - 16*</td>
<td>14</td>
</tr>
<tr>
<td>TP-2</td>
<td>545</td>
<td>36</td>
<td>36</td>
<td>3</td>
</tr>
<tr>
<td>TP-3</td>
<td>520</td>
<td>8</td>
<td>24</td>
<td>8</td>
</tr>
</tbody>
</table>

* in two phases, refer Figure 4 for clarification.

Portland cement Type I grout with w/c ratio equal to 0.55 was used. Volume of grout was measured from the number of pump strokes and also crosschecked against the volume consumed from the agitator tank. Nominal grout volume used for each pile was 540 liters but a slight difference shown in Table 2 in actual volume of grout measured might be due to the difference in IR used during grouting and at the time of calibration.

All the three piles were grouted with different IR varying from 3 to 36 liters/min. In each case, after injecting target grout volume, grout circuit valve was closed to lock the maximum pressure achieved. Residual pressures two minutes after closing the pump were also measured and are given in Table 2.

After 15 to 20 days of base grouting, diamond bit rotary drilling with double tube sampler was used to core through concrete present below the base of the PVC-casing. Right after reaching the pile tip, Standard Penetration Test (SPT) was started. Steel liners were used inside the split spoon sampler to collect soil samples. All the concrete and soil samples retrieved, carefully examined on site to determine the grout content present. Additionally, in order to determine the lateral spreading of grout into the surrounding soil of pile tip, additional boreholes were drilled 30 cm away from the pile periphery. Washboring was used down to approximately 1 m above the pile tip level and then SPT was employed to collect the samples.

5 CONCRETE CORES RECOVERED

From the concrete cores recovered no trace of grout was found except in case of test pile TP-1 where a 6.5 cm thick grout layer was recovered right below the pile tip as shown in Figure 3.

Below the grout core recovered no traces of grout were found in the sand samples recovered by SPT. Similarly for the rest of two test piles no traces of grout were seen in the soil samples recovered through SPT.

6 DEVELOPMENT OF GROUTING PRESSURE

Development of grout pressure with the increase in grout volume is shown in Figure 4. TP-1 was grouted with a high IR of 32 liters/min for the first 140 liters and then IR reduced to 3 liters/min for the rest of the grouting operation. It must be noted that as the IR is reduced, pressure dropped from 26 to 6 bar and then increased gradually to 16 bar at the end of the grouting. Two minutes after closing the pump, pressure dropped to 14 bars. TP-2 was grouted at a constant and very high IR of 36 liters/min, which corresponds to the maximum available speed of the grout pump. It must also be noted that pressure development
in this case was very quick as it jumped to 15 bars within first few strokes of grout and then rose to 36 bar at about 200 liters. No further increase in pressure was observed till the target volume of grout had been pumped. Pressure abruptly dropped to about 3 bar two minutes after closing the pump. TP-3 was grouted with a constant IR of 8 liters/min. Development of pressure in this case was quite gradual and it approached maximum pressure of 24 bar and dropped to 8 bar 2 min after closing the pump.

Figure 4. Curves showing gradual increase in grout pressure.

7 GROUT INJECTION RATE

7.1 High Injection Rate

It has been observed that IR is a key factor in the process of base grouting. If IR is kept very high, grout pressure rises abruptly but can not be smoothly maintained. A clear fluctuation in pressure is observed during grouting with high IR. Although the apparent maximum pressure indicated by the dial gauge is high at the initial stages but can not be increased further. It seems that at high IR, hydraulic fracturing of the surrounding soil took place which is also evidenced by sharp fluctuations in pressure achieved in this case dissipates very quickly as the grouting operation is terminated. This means that pressure developed in this case mainly consists of the circuit resistance and is not the true pressure induced at the pile/soil interface. Since pressure ceases to increase beyond 400 liters it seems that no further increase in pressure is possible even if large volumes of grout have been injected.

7.2 Low Injection Rates

If IR is kept low, development in pressure took place quite gradually and the pressure fluctuation is minimal. It must also be noted that the pressures development by keeping IR low is quite steady and can also be maintained for certain duration of time, as shown in Figure 4 for TP-1 where a residual pressure of 14 bars has been observed two minutes after closing the pump with a total drop of only 2 bar. It must also be noted that pressure keeps on increasing gradually and higher pressures than that of TP-2 can be achieved with more grout volumes.

8 PENETRATION OF GROUT INTO THE SAND.

From the samples collected, no traces of grout have been observed in the sand present below or surrounding the pile tip. Grain size distribution curves of sand samples collected are shown in Figure 5. It can be seen that the second sand layer falls in the range of medium to fine sand range. Grain size distribution for different types of available cements is also shown in Figure 5.

In order to assess the possibility of permeation grouting of the 2nd sand layer present around the pile tip, Groutability Ratio (GR) of the sand layers was determined. GR is a useful parameter for checking the applicability of grout for use in sand and is given by the formula as follows (Mitchell, 1970):

\[ GR = \frac{D_{15}}{D_{95}} \]  

Where \(D_{15}\) = The particle diameter of the soil to be grouted; and \(D_{95}\) = the particle diameter of the grout 95% of which is finer by weight.

Weaver (1991) summarizes the possibility of grouting a soil for GR ranges as:

- \(GR > 24\), usually;
- \(GR < 19\), not likely; and
- \(GR < 11\), not possible.

GR range for the 2nd sand layer is found to be 1.2 to 4 for the Portland cement type I which means that its not possible for the cement grout to permeate into the sand layers. GR using microfine cements MC-300 (Henn, 1996) shown in Figure 5 comes out to be 12 and 30, so the silty sand layer encountered can be permeation grouted using microfine cements.

9 INSTALLATION EFFECTS

SPT-N values obtained right below the pile tips and 30 cm away from the pile are compared with the values from nearest borehole (10 – 20 m) performed at the time of soil investigation in Figure 6. It is quite clear that the density of the sand surrounding the tip have been increased appreciably, especially, below the tip of the piles.

Figure 5. Grain size distribution curves of samples collected from Second Sand Layer around the pile tip.

Figure 6. Comparison of SPT-N values around the grouted pile tip and the nearest borehole from soil investigation.
10 MANCHETTE BREAKING MODELS

Thickness of concrete cover present below the manchette is very important and determines the flow path of the grout injected. If concrete cover present below the manchette is thicker than the side cover to the outlet points on manchette, it can be suspected that grout directly percolates toward the pile shaft by bursting the side cover. In order to monitor the bursting mechanism of manchettes, 9 model piles 1.2 m diameter and 2.0 m deep were constructed by varying manchette levels above the pile tip (Thasnanipan et al, 1998). Model piles were water burst in a similar way comparable to the normal bored piles. Colored grout was injected to differentiate it from normal pile concrete. Suitable time was given to allow for the grout setting before these piles were exhumed to determine the flow pattern of the injected grout. Some of the unearthed model piles are shown in Figure 7 and 8.

From the analysis of the model piles, it was revealed that: a) full pile base area coverage by grout can be achieved if manchette level is less than 20 cm above the pile tip; b) a partial base area coverage was found up to 30 cm and if; c) manchette level is more than 40 cm above the tip, grout directly percolates towards the pile shaft by bursting the side concrete cover. Presence of grout on the vertical pile/soil interface at the shaft was also confirmed in all cases. Although a direct comparison of grout flow in model piles and prototype piles cannot be made due to the difference in overburden pressures but the upward flow of grout along the pile shaft can be the most likely phenomenon in base grouting. Which, subsequently act as a 'rock socket' and significantly increases the pile stiffness (Francescon, 1992).

11 CONCLUSIONS

From the study carried out following conclusions can be drawn.

1. Most of the grout injected during base grouting rises up along the pile/soil interface, and does not permeate in to the surrounding sand layer to any degree. So the improvement made by base grouting is mainly contributed by increase in skin friction in the form of rock-socket effect which increases the pile stiffness significantly. Since the increase in skin friction depends only upon the volume of grout injected, pressures achieved during grouting are of negligible importance in this regard. Partial replacement and/or precompression of the sediments below the pile tip is possible by following low IR hence, smoothly increasing and maintaining the pressures achieved, if it is desired so.

2. IR is the key factor in the base grouting process, especially if, higher pressures are to be achieved and maintained.

Low IR of the order of 3–5 liters/min are recommended for base grouted piles in silty sand layers of Bangkok.

3. Significant improvement in the density of sand around the pile shaft near the base and especially, below the grouted pile tip has been observed. Although this is also contributed by increased overburden pressure due to concrete, base grouting helps to improve the soil around the pile tip, disturbed during the drilling process.

4. Position of manchettes above the pile base is very important to effectively grout the base of the pile. Care must be exercised to place the manchettes as close to the pile tip as possible but if due to some unforeseen reasons manchettes are lifted up grout can be conjectured to percolate towards the pile base if the concrete cover thickness below the manchettes does not exceed 30 cm.

REFERENCES


Henn, R.W. 1996, Practical guide to grouting of underground structures, ASCE publication, NY.


Morrison, I.M. 1987, Bored piled foundation for Chao Phya river crossing at Wat Sai, Bangkok, 9th SEAGS Conf. Bangkok, pp. 6-207 to 6-218.


Thasnanipan, N. & Umesh, S. 1998, Pile base cracking models with different manchette levels from the pile tip, SEAFCO internal report.

Weaver, K.D. 1991, Dam Foundation Grouting, American Society of Civil Engineers, NY.

Review of the shaft capacity degradation of bored piles constructed with bentonite slurry

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REVIEW OF THE SHAFT CAPACITY DEGRADATION OF BORED PILES CONSTRUCTED WITH BENTONITE SLURRY

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ABSTRACT

Over the last few decades, different researchers have studied the influence of different construction parameters on the shaft load transfer of bored piles constructed with slurries. Still the issue is not fully resolved, different parties involved in the bored piling industry believe on the degree to which these parameters effect the perimeter load transfer. This paper attempts to summarize all the significant works previously done on this subject. It is concluded that the realistic formulation of specifications and selection of design parameters with the consideration of practical constraints at site is very important. Performance of the bored piles is effected by installation procedures, all the ill effects attributed to the construction parameters can be eliminated by following good engineering practices.

INTRODUCTION

It is commonly accepted that the construction parameters like slurry properties, construction time, equipment used, etc., effect the performance of wet process bored cast in-situ piles. But the extent to which these parameters effect the pile capacity is not clearly understood due to the wide variation in the soil conditions and other influencing factors which are simultaneously acting. It has also been recommended in some codes of practice that observing good engineering practices can eliminate all the foreseeable ill effects attributed to these parameters.

CONSTRUCTION PARAMETERS

Among the other various parameters, construction time and slurry properties are particularly considered responsible for the degradation of the pile shaft capacity. In fact the total construction time is not directly related to the pile performance. Critical time, which might contribute to the degradation of the pile, starts after the borehole excavation is completed till the completion of the concrete pouring. Time consumed in the excavation operation is not generally considered to contribute any degradation since slurry is continuously kept agitated during drilling operation. A maximum elapsed time of 24 hours after the completion of the borehole excavation is generally recommended under normal conditions. Slurry properties especially the viscosity is believed to be another important factor related to the quality of the constructed pile, because it determines the degree of replacement by concrete during tremie concreting.

Previous published researches on this issue along with the brief conclusions made are given in the following in a chronological order.
Chaidesson (1961) observed a better load-settlement behavior for one pile drilled with slurry than the other pile drilled by a casing method.
Corbett et al. (1970) presented the results of load testing of test panel excavated under bentonite slurry, excavation took 2 days and on completion the panel was left open overnight, but bentonite was renewed prior to concrete. The authors inferred that the presence of bentonite suspension even for such a long time was not adversely affected the development of skin friction.

Farmer, Buckley & Sliwinski (1970) conducted the laboratory tests on model concrete pile. The results indicated that presence of a bentonite filter cake at the concrete/sand interface has little effect on load transfer. Full scale tests on three deep bored piles constructed under bentonite, indicated that the load transfer was extremely higher than the expected. They concluded that high load transfer level may be obtained from piles constructed under bentonite, this may be due to uneven side-wall configuration.

Reese & Tauma (1972) found that the values of load transfer developed in clayey soils on the sides of the shaft constructed by the slurry displacement method are comparable to those in the shafts constructed in the dry.

Wates & Knight (1975) concluded from their investigation that a bentonite filtercake of a significant thickness would develop in 24 hours. In order to prevent a build up of filtercake, the slurry should be left in place much less than 24 hours.

Cernak (1976) performed full scale load test on three barrettes in sandy gravels. Two of the barrettes were constructed using slurry with different exposure times (8 and 97 hours) and their load test results indicated a decrease of skin friction capacity of 43% and 56% respectively, as compared to the other barrette excavated dry and concreted immediately. It must be noted that the major part of the reduction i.e. 43% took place in the first 8 hours of construction time while only 13% (56-43) reduction took place in the rest of 89 hours (97-8) for the second barrette. These findings support the conclusion of Wates & Knight (1975).

Sliwinski (1977) reported that displacement of suspension of slurry from the sides of the bored hole does not constitute a major problem. The rising column of concrete will displace the fluid suspension because of the considerable difference in unit weight and shear strength of the materials. The author stated that the field and laboratory tests seem to indicate that the influence of some bentonite in the parent soil has an insignificant influence on load support providing that the properties of the slurry are within reasonable limit and the concreting is done within a reasonable short time after the excavation is completed.

Fleming & Sliwinski (1977) reported on 49 filed tests from several countries. The test results suggest that the use of bentonite suspension has no detrimental effect on shaft friction while in granular soil, there is an indication that skin friction at high displacement may be slightly be reduced, though there may be reasons unrelated to the bentonite process. They suggested that concreting should be completed within 24 hours of the completion of boring.

Reese & Tucker (1985) concluded that the capacity of bored piles constructed under slurry could be substantial with proper construction technique. The slurry will be ejected from the excavation and substantial bond will develop at the interface of the concrete and the supporting soil. However, the authors also suggested that the slurry must not be left more than few hours in the excavation without operation of drilling tool; otherwise, a thick filter cake will develop on the side of the excavation.

Reese & O’ Neil (1988) suggested that a solution to the problem of reduced skin friction due to excessive filter cake is to maintain the properties of the slurry within tolerable limits and to place the concrete the same day that the excavation is completed. If for some reason, it is impossible to place the concrete without undue delay, the drilling machine must be re-occupied the excavation to rework the borehole. If the slurry remains for a period of time without agitation, the filter cake can become thick.
Fleming et al. (1992) concluded that the process of forming a pile under bentonite suspension does not materially reduce the shaft friction in both granular soil and cohesive soil. The rising column of concrete with a slump in excess of 175 mm from tremie process will largely remove the filter cake layer on the wall surface of shaft. However, if the hole is left filled with slurry for long period (no agitation action), this will give rise to a significant reduction in shaft friction on completed piles.

O’Neil & Reese (1992) reported the effect of slurry properties on two drilled shafts constructed in similar ground conditions (alternating layer of very stiff clay and dense sand). Shafts 1 and 2 were constructed with Marsh cone viscosity of 37 and 49 sec respectively. Time between opening the borehole and completion of concreting for both piles was 5 to 7 hours. Minor difference in load transfer of both shafts was found and the authors concluded that the difference in slurry properties used for the construction for both shafts had a little effect on shaft load transfer.

Wardle et al. (1992) concluded from four piles tested in London that the axial capacity of driven and jacked piles after one or two months were 14 to 28% greater than those measured during or immediately following installation. CRP tests performed after about 3 years recorded further increase of between 14 to 20% (total became 28 to 48%). Although, it was not possible to record the capacity of the bored pile immediately after installation, the capacity of this pile increases by 47% between 2 months and three years.

Majano & O’Neil (1993) attempted to model the formation of filter cake in the laboratory with different slurry dosages, differential pressures and exposure times. They stated that perimeter load transfer is a complex function of the physical and chemical characteristics of the slurry and the geomaterials, the roughness of the borehole, the fluid pressures exerted by concrete, the shearing properties of the soil, and possibly the chemistry of the fluid concrete. They attempted to correlate the potential degradation of the soil-pile interface to the thickness and shear strength of the cake formed against the walls, and argued that the shear strength and thickness can not be measured with only one or two parameters like slurry dosage and differential pressure. With a bentonite concentration of 72 kg./m³ and differential pressure of 0.5 psi, they achieved a filter cake thickness of 3.14 and 4.5 mm after a contact time of 4 and 24 hours. Authors also compared the laboratory tests with the actual field conditions and proved that due to the presence of enormously high differential pressures in the field in some cases, thickness of filter cakes which would be considered detrimental for shaft load transfer, require few hours to develop (Authors proved that a filter cake of 6 mm. thickness would require only 2.3 hours to develop in the field under a differential pressure of 10 psi).

Hosoi et al. (1994) reported that when the diaphragm wall foundation is cast in slurry, mud cake could not be fully scoured by concrete. So the evaluation of the shaft friction mobilized on the wall foundation cannot overlook the effect of mud cake. They concluded from laboratory tests that the friction resistance between concrete and the sample soil (decomposed granite) through bentonite slurry cake is smaller than the shear strength of the sample soil.

Ho & Lim (1998) reported that the barrette sized 2.8 m x 0.60 m x 47.4 m excavated under bentonite slurry support at Singapore Post Center project with a total construction time of about 110 hours has been tested. The test result showed that substantial shaft friction could be mobilized to carry load satisfactorily with a reasonable factor of safety and within a tolerable displacement of 12 mm at working load.

Littlechild & Plumbridge (1998) presented that the values of actual shaft resistances mobilized for in the five pile load test carried out at BERTS project and compared to the calculated shaft resistances, where calculated shaft resistance is based on the design approach adopted on the project. For cohesive soil $f_s = \alpha \cdot C_u$, $\alpha$ taken as 0.5 for stiff and hard clay
layers and for sand layers shaft resistance of 120 KPa has been determined for all sand layers. Construction time and slurry viscosity on this project was controlled to be less than 24 hours and 35 sec respectively. They concluded that shaft friction capacity of bored piles constructed under bentonite slurry tends to decrease as the construction time and slurry viscosity increases.

Thasninan, Baskaran & Anwar (1998) presented the shaft friction values mobilized from eleven instrumented bored piles constructed under slurry with different construction times and bentonite viscosity from various sites in Bangkok area. They concluded that the slurry viscosity up to 55 sec./Qt do not have significant effect on the shaft load and degradation of shaft capacity with increase in construction time beyond 24 hours is not excessive.

DESIGN CONSIDERATIONS AND PARAMETERS

Selection of design parameters, which suits the local construction practices and procedures, is very important. Design parameters are normally selected based on the previous researches of local institutions/organizations along with some special considerations specific to the sites. Following are the recommendations and suggestions that need to be observed for the estimation of shaft load capacities in Bangkok subsoils.

Sliwinski and Fleeming (1974) suggested that in clay soil, the bentonite has no effect on friction and no reduction of the normal factors for deriving adhesion is justified. There may indeed be a slight increase of adhesion. In granular soils, the bentonite has some effect. The reduction of friction calculated from effective lateral pressure and coefficient of friction should be considered in the region 10-30%.

Oonchittikul (1990) back analyzed a parameter \( \beta = (K_s \cdot \tan \phi) \) from instrumented bored pile load tests conducted with bentonite slurry in Bangkok subsoils by using the effective stresses with both the hydrostatic pore pressure and the declined pore pressure. The resulted \( \beta \) value in the first and second sand layers from effective overburden pressure based on hydrostatic pore pressure falls in the range of 0.4 to 0.65 and 0.2 to 0.4 respectively while \( \beta \) value in the first and second sand layers from effective overburden pressure based on declined pore pressure falls in the range of 0.1 to 0.48 and 0.1 to 0.2 respectively. In both cases \( \beta \) value in second sand layer is less than the value in the first sand layer. The author inferred that the overburden pressure might not fully develop at great depth in sand layer as it has been assumed in the calculations.

Hooley & Brooks (1992) have suggested that if the traditional pile design approach is used, the proper interpretation must be given to the accuracy of the data employed by soil investigation as the potential error in the calculation of ultimate base resistance increases steadily with depth and shaft resistance error increases gradually with depth. The authors reported that it is noticeable that the amount of scatter in the test result from soil investigation samples increases with depth and thus the average line is more difficult to define at greater depths. If the scatter is analyzed statistically, upper and lower bound of average line can be identified on the basis of chance of error 0.01% probability.

Eide & Bellis (1992) extracted data from 13 numbers of loading tests taken to failure from the test results of over 109 tests in Bangkok which are presented by Oonchittikul (1990). The authors found average shaft friction in stiff clay and dense sand in the range 78 to 130 KPa and adopted 100 KPa as design value, which is commonly used for design in Bangkok Subsoil.

Teparaksa (1992) reported the problem from bentonite sedimentation causes a soft base. Unexpectedly, low skin friction can be mobilized at failure (design at yield point) in soft base condition. The author also reported that the static load test results on 1.50 m dia. pile at Pok
Klao bridge project in Bangkok indicates an excessive settlement at 4000 KN load which is caused by deposition of bentonite at the base of the pile. Analysis of load transfer indicates that the skin friction of first stiff clay, first sand, and second stiff clay are equal and about 40 KPa at 4000 KN load. After the base grouting with cement-mortar, the analysis shows the skin friction of this soil is between 40 to 90 KPa at about 5 mm movement. They are in the expected range for each soil layer, which is not affected by grouting.

Meißner et al. (1993) studied the influence on bearing behavior of bored piles when the pile base is approaching a layer of soft soil layer and concluded that a significant punching mechanism is obtained for value of $t/d = 2$, where $t$ is the distant from pile base and the clay layer and, $d$ is the pile diameter. This punching mechanism would decrease significantly both shaft and base resistance.

Tomlinson (1996) suggested that if pile is excavated by grab under water, there is considerable loosening of the soil. This causes a marked reduction in both end bearing and skin friction. Then the design calculation will be calculated on the basis of a low relative density of the granular soil.

Littlechild et al. (1998) reported that the shaft resistance of both the plain and shaft grouted bored piles in sand appears to be mostly independent of the vertical effective stress over the stress range encountered. From the test result of plain bored piles constructed in accordance with a given specification of BERTS project in Bangkok i.e. bentonite viscosity <35 sec and construction time of less than 24 hours, the shaft resistances fall in the range 65 to 170 KPa. In addition, they have determined a design line for shaft resistance of plain bored piles in Bangkok sand of 120 KPa for all ranges of vertical effective stresses.

Thasnanipan et al. (1998) revealed that the end bearing capacity of bored pile where toe is embedded in to the clay layer would be reduced and this end bearing capacity reduction might influence the shaft resistance capacity too. This conclusion was reinforced by the pilot pile load test result in the project located on Rama III road in Bangkok having diameter 1500 mm, toe founded at 60m in clay layer, failed at only about 17500 KN with total settlement of over 90 mm. While the designed maximum test load was 25000 KN compared to the pile load test from contract pile of same diameter toe depth founded in sand layer at depth 55 m and base grouted. Result of test up to 2500 tons with total settlement of only 29.26 mm and elastic recovery was 17.52 mm recorded.

**CODES AND SPECIFICATIONS**

Specifications are prepared for the good of the project. Specs drafted by ignoring the practical site constraints and specific project needs can cause a job to become problem for all parties. For example, some specs have been found to stressing on maximum allowable construction time of 24 hours, which sometimes become impossible to adhere to because of the prevailing traffic congestion in the city centers and local authority regulations in Bangkok, and ultimately instigate the contractor to breach the law in order to comply with such unpractical specs. A summary of the slurry specifications used worldwide is given in Table 1. In the following are given some suggestions related to the excavation under bentonite slurry.

Fleming et al. (1975) suggested that the specifications should set out a cause of action to perform the work, which is reasonable from the contractor’s point of view and gives a reasonable expectation of achieving the desired end product. A specification loses creditability on site if its requirements are outside the range of practical achievement and such a specification can only be detrimental to the whole job. They further suggested that obscure requirements will naturally be the first to be forgotten. Thus, for example, it is better to say “after mixing concrete shall be placed before it lose the specific workability” than to say “concrete shall be placed within X minute(s) from mixing”.

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Littlechild & Plumbridge (1998) reported that the majority of the piling contractors state that a maximum construction time for bored pile constructed to depth 50m under bentonite slurry for BERTS project in Bangkok of less than 24 hours per pile would not be realistic. A maximum construction time of 24 hours and slurry viscosity of less than 35 seconds (use of Bentosund bentonite only) were therefore adopted for the project.

Chodorowski & Duffy (1998) presented the typical control and progress charts for 24 hours limit on construction time of bored piles at BERTS project, Bangkok. Charts prepared by the authors show that during the 24 month period of bored pile construction, there was noticeable percentage of pile constructed longer than 24 hours of approximately 15% which in some months as much as 30%. They reported that longer construction time of more than 24 hours per pile was due to difficulties in securing continuous concrete supply as a result of traffic congestion in the area of different piling sites.

Table 1: Summary of recommended values of slurry properties used for bored piles.

<table>
<thead>
<tr>
<th>Authors</th>
<th>Parameter</th>
<th>Slurry Type</th>
<th>Density (g/cc)</th>
<th>pH</th>
<th>Sand Content</th>
<th>Marsh Funnel Viscosity (sec/Qt)</th>
<th>Plastic Viscosity (cP)</th>
<th>Yield Point (Pa)</th>
<th>10 min. Gel Strength (Pa)</th>
<th>Differential Head (m)</th>
<th>Fluid Loss (30 minute test)</th>
<th>Maximum Contact Time (hr.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ADSC &amp; DFI</td>
<td>Drilled shaft inspector’s manual clause 4.5.1</td>
<td>Bentonite</td>
<td>&lt;1.10</td>
<td>&lt;11.7</td>
<td>&lt;6% (by weight)</td>
<td>&lt;30 to 90</td>
<td>&lt;20</td>
<td>&gt;1</td>
<td>&gt;4 to 10</td>
<td>&gt;1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>FDOT &amp; DFI</td>
<td>Drilled shaft inspector’s manual clause 4.5.1</td>
<td>Bentonite</td>
<td>1.024 to 1.218</td>
<td>10.8 to 11.7</td>
<td>&lt;35% (by weight)</td>
<td>30 to 40</td>
<td>&lt;20 &lt;20</td>
<td>4.2 to 41.8</td>
<td>3.6 to 20</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Majano &amp; others</td>
<td>Drilled shaft inspector’s manual clause 4.5.1</td>
<td>Bentonite</td>
<td>1.03 to 1.195</td>
<td>8 to 11</td>
<td>30% max. (by volume)</td>
<td>28 to 40</td>
<td>&lt;20 &lt;20</td>
<td>1.9 to 10</td>
<td>10 to 20</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Majano &amp; others</td>
<td>Drilled shaft inspector’s manual clause 4.5.1</td>
<td>Bentonite</td>
<td>1.03 to 1.20</td>
<td>8 to 11</td>
<td>&gt;4% (by volume)</td>
<td>28 to 45</td>
<td>&lt;20 &lt;20</td>
<td>2 to 10</td>
<td>10 to 40</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
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<td>Drilled shaft inspector’s manual clause 4.5.1</td>
<td>Bentonite</td>
<td>1.03 to 1.20</td>
<td>8 to 11</td>
<td>&lt;4% (by volume)</td>
<td>28 to 45</td>
<td>&lt;20 &lt;20</td>
<td>1.9 to 10</td>
<td>10 to 40</td>
<td>-</td>
<td>-</td>
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</tr>
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<td>&lt;4% (by volume)</td>
<td>28 to 45</td>
<td>&lt;20 &lt;20</td>
<td>2 to 10</td>
<td>10 to 40</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Majano &amp; others</td>
<td>Drilled shaft inspector’s manual clause 4.5.1</td>
<td>Bentonite</td>
<td>1.02 to 1.13</td>
<td>8 to 11</td>
<td>&lt;4% (by volume)</td>
<td>28 to 45</td>
<td>&lt;20 &lt;20</td>
<td>2 to 10</td>
<td>10 to 40</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
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<td>1.02 to 1.13</td>
<td>8 to 11</td>
<td>&lt;4% (by volume)</td>
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<td>&lt;20 &lt;20</td>
<td>2 to 10</td>
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</tr>
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<td>&lt;4% (by volume)</td>
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<td></td>
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<td>Bentonite</td>
<td>1.02 to 1.13</td>
<td>8 to 11</td>
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<td>28 to 45</td>
<td>&lt;20 &lt;20</td>
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<td>2 to 10</td>
<td>10 to 40</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

* Without agitation and side-wall cleaning

Note: For two rows in each cell (last three columns)
First row: fresh or recycled as supplied to borehole
Second row: prior to concrete pouring

ADSC and DFI drilled shaft inspector’s manual clause 4.5.1, advised that where bentonite slurry remains in the shaft unagitated for 3 or 4 hours, or more than 24 hours even though agitated, a mud cake may develop on the shaft walls which can reduce friction. Unless, the designer has accounted for ‘caking’ and appropriately reduced design friction value or the specification gives other limitation, portion of the shaft or socket designed for side friction should either be excavated and concreted within a 24 hours period (and no more than 3 hours when the slurry is not agitated) or re-reamed or re-scraped to remove the mud.
cake prior to concreting, or some method which is acceptable to the geotechnical engineer and the inspector used to demonstrate that detrimental mud caking has not occurred.

ADSC Specification Clause 2.3.5.2 (g) specified that the drilled pier constructed under bentonite slurry should be concreted and completed the same day that of excavation. If this is not possible, the excavation shall be re-drilled, cleaned and slurry tested before concreting. Shaft friction could be mobilized to carry load satisfactorily with a reasonable factor of safety and within a tolerable displacement of 12 mm at working load.

FHWA guide for drilled shaft specification, stipulates that, that drilling slurry is an effective method of stabilizing drilled shaft excavation until either a casing has been installed or concrete has been placed. Primary concerns connected to slurry use are: the shape of the borehole would be maintained during the excavation and concrete placement; the slurry does not weaken the bond between the concrete and both the natural soil and rebar; all of the slurry is displaced from the borehole by the rising column of fresh concrete; and any sediment carried by the slurry is not deposited in the borehole. The engineer’s concerns regarding the behavior and effectiveness of slurry project, can be satisfied by appropriate specification requirements. These requirements include: specifying a suitable range of slurry properties both prior to and during excavation and prior to concreting; performing slurry inspection test; and construction of pre-production trial shaft by the slurry method.

FULL SCALE TEST TO ASSESS THE EFFECT OF CONSTRUCTION PARAMETERS

Since the degree of effect of different parameters greatly depends upon the soil properties encountered, conclusions made for one type of soil are not directly applicable to other locations with different geology. Static, instrumented load test program on bored and barrette pile conducted at BECM tower project Rama IX road, Bangkok, provided the unique chance to determine the effects of construction parameters on the shaft capacity (Thasnanipan et al., 1999). Bored pile 1.5m diameter and a barrette 1.5x3.0m in cross-section had same lengths of 57.5m founded in the second sand layer and located 30m apart having similar soil embedment conditions. Properties of the bentonite slurry used are given in Table 2.

Table 2. Comparison of bentonite slurry properties for test pile and barrette.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Pile Before feeding to the borehole</th>
<th>Pile After Recycling &amp; Before concreting (near borehole base)</th>
<th>Barrette Before feeding to the trench</th>
<th>Barrette After Recycling &amp; Before concreting (near trench base)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Viscosity (sec)</td>
<td>33</td>
<td>36</td>
<td>36</td>
<td>49</td>
</tr>
<tr>
<td>Density (g/cc)</td>
<td>1.08</td>
<td>1.10</td>
<td>1.10</td>
<td>1.17</td>
</tr>
<tr>
<td>pH value</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>9</td>
</tr>
<tr>
<td>Sand Content (%)</td>
<td>0.1</td>
<td>0.8</td>
<td>1.0</td>
<td>1.1</td>
</tr>
</tbody>
</table>

Total construction time for the bored pile and barrette was 27 and 75 hours, respectively. However, the total elapsed time between the final excavation (re-drilling & cleaning) and concreting for the pile and barrette was 16.25 and 20 hours, respectively. In spite of considerable difference in the total construction time and slurry viscosity, values of unit shaft friction mobilized in different soil layers did not show any difference in the shaft load transfer characteristics. This proves that the total construction time is not a crucial parameter, which effects the shaft load capacity if good engineering practices like re-drilling the borehole is applied.
CONCLUSIONS

Conclusions of various researchers can be summarized as follows:

- Bentonite viscosity up to the normally recommended range i.e. 60 sec., given in Table 1 does not have any significant effect on the skin friction capacity of bored piles. In authors’ point of view, filter loss is the more relevant property of the slurry, which directly effects the formation of the filter cake and finally results in the shaft capacity reduction rather than the viscosity.
- Selection of pile design parameters with due consideration of site constraints is equally important. Adequate strength reduction factors based on the local experiences suggested by local research institutions need to be considered.
- Concreting should be completed within 24 hours after the finishing excavation of borehole. If due to some unavoidable reasons borehole is left filled with slurry beyond 24 hours with out agitation, re-drilling must be carried out to scarp off any filter-cake present on the walls of the borehole.
- Performance of bored piles, to a large extent, depends upon the installation procedures and, the skill of the operating crew, if proper construction procedures have been followed all the negative effects can satisfactorily be eliminated.
- It must be emphasized that the specifications for each project must be tailored to suit the actual site conditions keeping in view the constraints like possible delays in concrete supply due traffic congestion, local authority regulations etc. Among the available control limits for bentonite slurry shown in Table 1, this is authors opinion and experience that AASHTO specs are practically more suitable for pile excavation works in Bangkok subsoils.

REFERENCES


Cernak, B. (1976), The Time Effect of Suspension on the Behavior of Piers, 6th European Conf. on Soil Mechanics and Foundation Engineer, Vienna.


Chodorowski A.R. & Duffy, M.R. (1998), Quality Assurance and Testing on a contract for 16,000 Large Diameter Bored Cast In-Situ Piles. 7th Int. Conf. and Exhibition on Piling and Deep Foundations, Vienna, Austria.


Federation of Piling Specialists (1975), Specifications for Cast-In-Place Piles Formed under Bentonite Suspension, Ground Engineering, 8, No. 2, 50 pp.


Florida Department of Transportation (1988), FDOT Standard Specification for Road and Bridge Construction, Section 455: Foundations of Structures, Tallahassee, FL.


Reese, L.C. & Tauma, F.T. (1972), Load Test of Instrumented Drilled Shafts Constructed by the Slurry Displacement Method. Center for Highway Research. The University of Texas at Austin. U.S.A.


Thasnanipan, N., Teparaksa, W., Maung, A.W. & Baskaran, G. (1998), Design, Construction and Behavior of Bored Cast In-Situ Concrete Piles in Bangkok Sub Soil. Proc. of Fourth Int. Conf. on Case Histories in Geotechnical Engineering, St. Louise, Missouri.


Wates, J.A. & Knight, K. (1975), The Effect of Bentonite in the Skin Friction in Cast in-Place Piles and Diaphragm Walls. Proc. of Sixth Regional Conf. for Africa on Soil Mechanics and Foundation Engineering, Dorhan, South Africa.
Failure mechanism of long bored piles in layered soils of Bangkok

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Seafo Company Limited Bangkok, Thailand
FAILURE MECHANISM OF LONG BORED PILES IN LAYERED SOILS OF BANGKOK

Narong Thasnanipan, Muhammad Ashfaq Anwar, Aung Win Maung
SEAFCO Co., Ltd., Bangkok, Thailand.

ABSTRACT
Simple superposition principle for the estimation of ultimate shaft friction capacity of bored piles sometimes found to be not directly applicable for the piles embedded in the multilayered soils of Bangkok. Because of the early yielding of the stiff clay layers, a localized plunging of the load-settlement curve is often observed. For these layers, although the maximum mobilized skin friction corresponds to the adhesion factor $\alpha$ equal to around 0.35, suggested by different researchers. But due to the movement softening behavior of these layers, residual friction values near the ultimate failure of the piles drops to a corresponding $\alpha$ values of the order of 0.15 or even smaller. It is recommended that due to the progressive nature of failure, lower $\alpha$ values for stiff to hard clay layers should be considered for the estimation of ultimate skin friction capacity under such conditions.

INTRODUCTION
Ultimate shaft friction capacity of piles embedded in the layered soils is calculated as summation of the ultimate shear resistances offered by different soil horizons present along the pile shaft. Load movement curves obtained from instrumented pile load tests have revealed that the very stiff to hard clay layers present within the Bangkok aquifer sands frequently exhibit a well defined movement softening behavior. Peak friction values in these layers have been found to be mobilized at very low pile head movements and well before the ultimate friction values of other layers present along the pile shaft. When rest of the layers reach their ultimate values these layers have already reached their residual friction values, which are as low as 50 percent of the peak friction values. So the simple superposition of ultimate friction values give rise to overestimation of the total shaft friction capacity under these conditions.

SOIL PROFILE
Typical soil profile of plain of Bangkok is shown in Figure 1. Thickness of Bangkok Soft Clay (BSC) at the top varies generally between 15 to 18m. BSC changes to medium stiff consistency before the first stiff clay, which is generally present at 20 to 25 m depth. First sand layer is usually 5 to 10 m thick and found at 25 m to 30 m depth, below is a series of stiff to hard clay and medium to very dense silty sand layers.

PILES EMBEDMENT CONDITIONS
Pile embedment conditions are designed depending upon the actual soil profiles encountered at the sites, as depicted by three locations P-1, P-2 and P-3 in Figure 1. Thasnanipan et al. (1998), presented the instrumented pile load test results of wet process bored piles tested in Bangkok subsoils. The authors have revealed that in case of pile...
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A typical load test showing this type of behavior is shown in Figure 3. It can be seen that the stiff clay layers reach their ultimate friction values well before the sand layers and at the maximum test load stiff clay layers reach their residual friction values. Design parameters like $\alpha$ and $\beta$ suggested by different researchers like Suchada (1995) for the estimation of ultimate skin friction in Bangkok subsoils give a reasonable estimate for the embedment conditions like at P-2 in Figure 1 or where the thickness of 2nd stiff clay layer is relatively low. Exceptions have been observed in case of embedment conditions especially like P-1, where a thick 2nd stiff clay layer is present. Owing to the difference in shaft friction development with pile head movement in the stiff clay layers as shown in Figure 3, direct summation of (simple superposition) ultimate shaft friction capacities in different soil horizons leads to over estimation of shaft capacity of these piles. Due to the brittle nature of failure mechanism in very stiff to hard clay layers peak friction resistance is mobilized at comparatively very little movement at the pile/soil interface. Residual values of friction resistance in these layers drop to 50 percent or even smaller of the peak friction values mobilized.

Figure 3: Skin friction development with the increase in butt load which clearly shows a typical early yielding of stiff clay layers.

On the extreme, from some instrumented pile load test results it has been observed that stiff clay layers reach their peak friction values near the design service loads. Rest of the soil layers need further pile/soil interface movement for the full mobilization of skin friction, and at this stage, stiff clay layers have already arrived at their residual friction values. Now depending upon the relative thickness of stiff clay layers, overall shaft friction capacity mobilization is affected differently. If the thickness of very stiff to hard clay layers is such that their contribution in the total shaft friction capacity is proportionally high, when stiff clay layer yields, load settlement curve shows a localized plunging behavior. The movement of such plunging consists of few millimeters and when the other soil layers share the deficit in

Figure 1: Typical soil profile along a 4.5 km long stretch of an expressway in the plain of Bangkok, with different pile embedment conditions.

embedment conditions like P-1 and P-3, 2nd stiff clay often exhibit a brittle type of failure and peak skin friction values are mobilized well before the sand layers.

A family of t-z curves obtained from the instrumented pile load tests showing this type of failure is presented in Figure 2. The peak friction values in these layers are mobilized at very low pile head movements (10 – 15 mm depending upon the depth of stiff clay layer) and well before the ultimate friction values of other soil layers present along the pile shaft. When rest of the soil layers reach their peak friction values stiff clay layers have reached the residual friction values.

Figure 2: Family of curves obtained from instrumented pile load tests showing typical brittle failure in stiff clay layers.
A typical load test showing this type of behavior is shown in Figure 3. It can be seen that the stiff clay layers reach their ultimate friction values well before the sand layers and at the maximum test load stiff clay layers reach their residual friction values. Design parameters like $\alpha$ and $\beta$ suggested by different researchers like Suchada (1995) for the estimation of ultimate skin friction in Bangkok subsoils give a reasonable estimate for the embedment conditions like at P-2 in Figure 1 or where the thickness of 2nd stiff clay layer is relatively low. Exceptions have been observed in case of embedment conditions especially like P-1, where a thick 2nd stiff clay layer is present. Owing to the difference in shaft friction development with pile head movement in the stiff clay layers as shown in Figure 3, direct summation of (simple superposition) ultimate shaft friction capacities in different soil horizons leads to over estimation of shaft capacity of these piles. Due to the brittle nature of failure mechanism in very stiff to hard clay layers peak friction resistance is mobilized at comparatively very little movement at the pile/soil interface. Residual values of friction resistance in these layers drop to 50 percent or even smaller of the peak friction values mobilized.

![Figure 3: Skin friction development with the increase in butt load which clearly shows a typical early yielding of stiff clay layers.](image)

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Table 1: Comparison of maximum skin friction values mobilized and the values at maximum test load.

<table>
<thead>
<tr>
<th>Clay Layer</th>
<th>Undrained Shear Strength Su (ton/m²)</th>
<th>Mobilized Unit Skin Friction Values (ton/m²)</th>
<th>Max. Mobilized</th>
<th>At Max. Test Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>fs α</td>
<td>fs α</td>
</tr>
<tr>
<td>Soft Clay</td>
<td>1.5</td>
<td>1.7</td>
<td>1.13</td>
<td>1.6</td>
</tr>
<tr>
<td>Medium Clay</td>
<td>4.5</td>
<td>2.2</td>
<td>0.49</td>
<td>1.9</td>
</tr>
<tr>
<td>1st Stiff Clay</td>
<td>13.5</td>
<td>5.8</td>
<td>0.42</td>
<td>5.1</td>
</tr>
<tr>
<td>2nd Stiff Clay</td>
<td>25.5</td>
<td>8.8</td>
<td>0.34</td>
<td>4.2</td>
</tr>
</tbody>
</table>

Peak and residual friction capacity of these layers, once again pile starts taking load followed by this local plunging, as shown in figure 4, till the final plunge exhibiting the failure of all soil layers along the pile shaft reaches. It must be noted that this localized plunging was earlier conjectured to be because of the soft toe of the piles. On the other hand if the contribution of skin friction derived from very stiff to hard clay layers is proportionally low, localized plunging due to earlier failure of very stiff to hard clay layer is not visible. Normal practice to estimate the contribution of skin friction from clay layers is by using $\alpha$-method, with a value of $\alpha$ equal to around 0.3 for very stiff to hard clay layers. Instrumented pile load test results have shown that skin friction values calculated using $\alpha$ value equal to 0.3 corresponds to the peak skin friction in these layers. Owing to the brittle type of failure mechanism in these layers, $\alpha$ values at full mobilization of total shaft friction capacities drop to as low as 0.15 in some cases and the ultimate shaft capacity calculated gives over estimates as compared to the actually mobilized capacities. Skin friction values mobilized in the pile load test shown in Figure 3 are given in Table 1.

![Figure 4: Typical load-settlement curves showing localized plunging before the overall failure of the pile.](image-url)
Peak and residual $\alpha$ values mobilized in the stiff clay layers from Table 1 are plotted in figure 5 along with the suggested curves by different researchers. It can be noted that the peak $\alpha$ value mobilized in 2nd stiff clay layer corresponds well to the value suggested by Stas & Kulhawy (1984). But the residual $\alpha$ value at the maximum test load drops below the curve suggested by Suchada (1989). So the $\alpha$ values proposed in figure 5 for the stiff to hard clay layers gives overestimate of the ultimate shaft friction under these conditions.

![Figure 5: Comparison of adhesion factor $\alpha$, suggested by different researchers with the actual mobilized in the stiff clay layers.](image)

EFFECT OF LOWER END BEARING VALUES ON SKIN FRICTION

Meibner & Van Impe (1993) firstly reported the influence of lower end bearing values on the shaft friction capacity of piles. Laboratory model tests as well as finite element analysis performed by the authors showed that the presence of soft soil within a depth of two to four times pile diameter below the pile tip, not only decrease the end bearing but also the skin friction capacity. Thasnanipan et al. (1998) also revealed that the end bearing capacity of bored pile where toe is embedded in to the clay layer would be reduced and this end bearing capacity reduction might influence the shaft resistance capacity too. This conclusion was reinforced by the pilot pile load test result in the project located on Rama III road in Bangkok where bore pile having diameter 1500 mm, toe founded at 60m in clay layer, failed at only about 1750 tons with total settlement of over 90 mm. While the designed maximum test load was 2500 tons. An other contract pile of same diameter and founded in sand layer at depth 55

<table>
<thead>
<tr>
<th>Clay Layer</th>
<th>Mobilized at Max. Test Load</th>
<th>Skin Friction Values (ton/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft Clay</td>
<td>1.6</td>
<td>$\alpha_{fs} = 1.13$</td>
</tr>
<tr>
<td>Medium Clay</td>
<td>1.9</td>
<td>$\alpha_{fs} = 0.49$</td>
</tr>
<tr>
<td>1st Stiff Clay</td>
<td>5.1</td>
<td>$\alpha_{fs} = 0.42$</td>
</tr>
<tr>
<td>2nd Stiff Clay</td>
<td>4.2</td>
<td>$\alpha_{fs} = 0.34$</td>
</tr>
</tbody>
</table>
m and base grouted was successfully tested up to 2500 tons with total settlement of only 29.26 mm and elastic recovery was 17.52 mm recorded. Even though the second pile was shorter in length than the first, more skin friction was mobilized due to the improved end bearing conditions. A similar decrease in the shaft friction capacity in the sand layers has been observed from the instrumented pile load tests at another project in Bangkok.

CONCLUSIONS

For the piles embedded in the multi layered soils of Bangkok, estimation of ultimate shaft friction capacity needs to consider the brittle type of failure mechanism of stiff to hard clay layers and the $\alpha$ values selected need to be adjusted accordingly. Where the pile tips are embedded in the clay layer, effect of lower values of end bearing on shaft capacity should also be considered.

ACKNOWLEDGEMENTS

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REFERENCES


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BARRETTEs; A VERSATILE FOUNDATION FOR TRANSMISSION LINE TOWERS

Narong Thasnanipan¹, Pornpot Tanseng², Aung W. Maung³ and Muhammad A. Anwar⁴

ABSTRACT

An efficient design of foundations for transmission line towers has always been a challenge for the engineers due to the variety and cyclic nature of the loads. Foundations, especially for the four-legged towers, are subjected to all types of loads (compression, tension, torsion and shear) in different combinations. The cyclic nature of the loads further complicates the situation. Available design parameters proposed by different researchers are also mostly based on the monotonic loading conditions and are not directly applicable for tower foundations. This paper presents the analysis, design and construction practice of the barrettes used for the transmission line towers in Thailand. Adaptability of the barrettes under different site constraints like sensitive underground pipelines is also described.

INTRODUCTION

The 230kV-transmission line project presented in this paper involved construction of 40 transmission towers, 400m apart. The project site was located about 200km, east of Bangkok, Thailand. The transmission line was planned to convey electricity from the power plant to a sub-station. The alignment of 15.6km-long transmission line partially runs along the boundary of an Industrial Estate and thus the foundations of these towers are in close vicinity of existing underground gas pipelines and utilities above ground (Fig.1). Foundation layouts, construction method and the work area were thus restricted by presence of these utilities.

Initially, the structural designer proposed large diameter bored piles of 2.0m in diameter for tower foundations. Construction of such large bored piles would require heavy equipment and plant, and would induce unacceptable vibration level to the existing underground gas pipe lines located within and along the construction area. As an alternative construction method and foundation type to suit the site conditions, barrette construction that utilizes relatively less equipment and induces less vibration than conventional bored pile construction was adopted.

Figure 1. Layout of project site

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² Geotechnical Engineer, Seafco Co., Ltd., Bangkok, Thailand
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⁴ Project Manager, Middle East Foundations Group, Dubai, UAE
Construction of barrettes in a site with limited headroom and constraints and performance of barrettes in Bangkok soil has been reported by Thasnanipan et. al. (1998) and (1999). The past experience indicated the feasibility of this project.

SOIL CONDITIONS

Initially, only a few boreholes were made to investigate subsoil conditions. However as the great distance between towers meant that substantial variations in soil conditions could occur. An extensive soil investigation was later carried out at all tower locations by drilling to verify subsoil condition to optimize barrette design and for excavation feasibility. The general subsoil profile revealed fine to coarse sand at the top, overlaying silty clay (lateritic soil) with varying thickness. The lateritic soil was not present at some locations. Below the sand and lateritic soil was completely decomposed granite at varying depths. Figure 2 shows cross sections of subsoil at tower locations along the transmission line. The groundwater level found also varied from about 0.6m to 9.2m below ground surface.

Figure 2. Soil profile along the transmission line

Figure 3. SPT test results at some tower locations.

SPT N values of soil layers from different boreholes indicated that the depth of competent layer varied significantly (Fig. 3). This meant that penetration depth of barrettes would vary with soil conditions and barrettes need to be designed individually.
BARRETTE DESIGN

The design of barrettes mainly involved (1) layout arrangement and (2) structural and geotechnical capacity of barrettes in connection with load, site and soil conditions and construction practicality. Regarding the layout, a four-barrette group (Fig. 5) was selected to support 38 lattice towers (up to 61.4m high) individually where a large area was available. Otherwise, a cruciform barrette (Fig. 6) was used for monopole towers (51.4m high). Compression, uplift and lateral forces were considered the significant loads to barrettes and they depended on the position of the tower in the line, the natural forces (wind) exerted on towers and cables and the weight of tower and cables. The towers were generally classified according to their positions on the line; end towers, edge towers and towers on a straight line between two other towers. The end towers were longitudinally loaded from one side by the cables and the overturning moment on such towers was large. The edge towers were loaded both vertically and transversally while the towers on straight line mainly carried a vertical load. A summary of estimated loads on the individual barrettes of the towers is presented in Table 1. To achieve load carrying and uplift capacity based on the imposed load and soil conditions, barrette sizes of 1.0x2.7m and 0.8x2.7m with depths from 11.0m to 22.0m were selected.

Table 1. Summary of Loads on individual barrettes

<table>
<thead>
<tr>
<th>Tower Classification</th>
<th>Compression (kN)</th>
<th>Uplift (kN)</th>
<th>Horizontal (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>End Tower</td>
<td>3792</td>
<td>3078</td>
<td>1087</td>
</tr>
<tr>
<td>Edge Tower</td>
<td>3479-3792</td>
<td>2804-3078</td>
<td>898-1087</td>
</tr>
<tr>
<td>Tower on straight line</td>
<td>1370-2039</td>
<td>1032-1565</td>
<td>309-505</td>
</tr>
</tbody>
</table>

Figure 4. Dimensions and type of barrettes for the towers

Figure 5. Typical barrette layout for a transmission tower (Lattice Type)
Generally, a safety factor higher than 2.3 and 1.2 was adopted for compression and uplift load capacity respectively. For uplift capacity, reduced friction capacity (up to 50%) in the cohesionless soil above fixity level was used in calculation since barrettes were subjected to horizontal cyclic loading. Alexander J. Verstraeten. (1987) has described decrease of friction by dynamic loading patterns in details. Finite element method (FEM) was used for calculating bending stesses in the barrettes and prediction of lateral movements. A total of 6 pile types was classified according to imposed loads, pile dimensions and soil conditions, and then analysed by FEM. Reinforcement of barrettes was designed using strength design method. A concrete cover 75mm was used for all barrette types.

The lattice towers were anchored in the barrettes by stubs and cleats which were cast into the extended barrette section after trimming the barrette top. For monopole towers, anchor bolts and steel flanges were used.

Three monopole towers were used where the lattice type tower could not be allocated. Due to the presence of underground gas pile lines and the narrow area available, it was difficult to allocate barrette groups with adequate alignment to resist the maximum bending moment and horizontal force up to 25,840kN-m and 635kN respectively. Cruciform barrettes were then designed to resist such high bending moments and forces acting on the base of the monopole.

Figure 6. Cruciform barrette for a monopole type tower

BARRETTE CONSTRUCTION

A mechanical rope grab was used to excavate the trench under bentonite slurry. The same crane was used for excavation and lifting reinforcement cages as well as construction plant and facilities. A guide wall with inside clear dimensions slightly more than the nominal size of the barrette was used to guide the grab. Circulation of slurry was continuously done to keep the bentonite slurry agitated, to minimize the building up of filter cake on trench wall surfaces. Properties of bentonite slurry were maintained within the specified ranges in wide use. Construction sequences are shown schematically in Figure 7.

Figure 7. Barrette construction sequences (schematic)
Sediment or loose materials at the bottom of the trench were removed and any built-up filter cakes were scraped off by the grab before reinforcement cage installation. Entire barrette length was fully reinforced and up to 24m-long reinforcement cages were fabricated in one complete section, for installation into the trench. The reinforcement cage of the cruciform barrette was as the same shape as the barrette. For the purpose of lifting and handling the cage, temporary stiffeners, lacing and tie bars were necessary. These temporary bars were removed section by section while the cage was lowered into the trench.

After excavation, the trench profile was checked with Koden drilling monitoring equipment. Tremie concreting method was used for casting the barrettes. Since the cutoff level of barrettes was generally at ground level, ready-mixed concrete was poured until all slurry and slime in the trench was completely displaced and fresh concrete could be seen.

Up to 172kg/m³ and 381kg/m³ of SD40 steel bars with maximum steel percentage at barrette top section (about 12.0m) were used to reinforce the group and cruciform barrettes respectively. Ready-mixed concrete with cube strength of 38MPa at 28days was used for casting barrettes.

CONSTRUCTION PROBLEM AND CORRECTION

A trench collapse was reported during desanding operation after trenching was completed for a barrette of lattice tower. A sudden fall of bentonite slurry level by a few meters in the trench was observed and then the trench was found filled with soil up to 10m depth. Mode of failure of the trench was found to be in the form of a localized cavity failure in the loose sand present at 2.0m to 6.0m depth. Failure surface did not extend up to the ground surface and the guide wall was still intact without any distortion or damage. The most probable cause of the trench collapse was due to heavy rainfall during construction. Surface water was seen in the vicinity of the construction area. The rain had raised the groundwater level to near the ground surface and caused a groundwater flow reversal towards the trench, triggering cavity formation in the loose sand stratum. The cavity progressively increased in size, leading to the collapse of the trench during the desanding operation. The collapse trench was backfilled with cement-bentonite mix, and wooden piles were driven into the collapse area outside of the barrette outline. The guide wall was then raised up to 1.5m above ground level. The trench was re-excavated under the slurry head 1.5m above the expected groundwater level and the barrette was completed.

QUALITY CONTROLS

All trenches were checked for verticality and dimensions prior to reinforcement cage installation to avoid any obstruction associated with trench inclination. In particular, all sides of the cruciform trench were checked. If necessary, verticality of trench was improved by careful chiselling with the grab. Profiles of the trench for a cruciform barrette is shown in Figure 10.

As the subsoil conditions varied from one location to another, they were observed during trenching process and verified with the soil conditions assumed in design, especially in the lower section of the barrette. Slurry quality was regularly tested and maintained within the specified ranges. Since barrettes
were highly reinforced, in order to achieve a good flow of concrete, the concrete mix was checked for appropriateness of slump and cohesiveness prior to casting.

Figure 10. Trench profile of a cruciform barrette recorded with Koden equipment - Tower no. 5

Figure 11. Test signal showing sound integrity and toe reflection of a barrette - Tower no. 2

Sonic integrity/seismic test was carried out on all barrettes about 5 days after casting for checking barrette integrity. The test indicated that integrity of all barrettes were sound. Figure 11 shows a signal acquired by sonic integrity test on the cruciform barrette.

DISCUSSION AND CONCLUSION

For barrette piling in loose cohesionless soil, chances of rapid groundwater level rise due to the rain, tide, etc., need to be considered during trenching. If necessary, guide wall with appropriate height above the possible highest groundwater level must be used.

Barrettes are versatile and can be used efficiently in the areas where conventional bored piling cannot be done. Moreover, barrette construction uses less quantity of equipment than bored piling and thus it is suitable for the piling work requiring shifting from one location to another.

Verticality of trench is critical for barrettes with complex cross-sectional shape, such as cruciform, T and H for reinforcement cage installation. A thick concrete cover up to 100mm is thus recommended, in particular for cruciform barrettes for ease of reinforcement cage installation and to maintain the adequate concrete cover, considering trench verticality against high stiffness of rebar cage.

REFERENCES


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Practical installation of stanchions for top-down construction in Bangkok subsoil
Narong Thasnanipan, Aung Win Maung & Zaw Zaw Aye
SEAFCO Co., Ltd., Bangkok, Thailand

27-30 November 2000
Bangkok, Thailand
ABSTRACT
Top-down construction method has been used for construction of deep basement at major projects in Bangkok, Thailand. To be able to apply top-down construction method, structure elements are required to sustain the construction load and to utilize as a part of bracing system. Prefabricated steel columns known as stanchions embedded in bored piles or barrettes are commonly used for this requirement. This paper presents some experiences in the practical installation of the different stanchion types in large diameter deep-seated bored piles and barrettes in Bangkok. The problems encountered during the actual installation of the stanchions and solutions to them are also discussed.

INTRODUCTION
Conventionally, buildings with underground basements are built by bottom-up method where sub-structure and super-structure floors are constructed sequentially from the bottom of the sub-structure or lowest level of basement to the top of the super-structure. Though this conventional method, also called as bottom-up method, is simple in both design and construction, it is not feasible for the gigantic projects with limited construction time and/or with site constraints. Top-down construction method which provides the significant saving of the overall construction time has been adopted for some major projects where time factor is of primary importance. To be able to apply top-down construction method, rigid permanent retaining wall and pre-founded structural columns are required. Diaphragm wall is normally used as basement retaining wall whereas prefabricated steel stanchions embedded in bored or barrette piles are utilized to sustain the construction load as well as to form as a permanent column at later stage. The common length of the stanchions used in Bangkok ranges from 15 to 30m for excavation depth of 12 to 25m. Installation of heavy and lengthy stanchion with stringent positional tolerance in the boreholes or excavated trenches filled with drilling fluid in prevailing subsoil condition of Bangkok is a challenging job for the foundation contractors.

SUBSOIL CONDITION AND PILE CONSTRUCTION METHOD
Bangkok subsoil is featured by alternating clay and sand layers of thick Quaternary deposits. As characteristics and properties of Bangkok subsoil layers have been well documented in many literatures, only a general description of Bangkok subsoil is presented in this article as follows.

Made-Ground consists predominantly of Fill-Materials, Clayey Sand or Silty Clay with some cement block rubble and rock fragments, is commonly found up to 4m depth. Soft to very soft, highly compressible dark gray marine clay lies beneath Made-Ground and in some
ABSTRACT

Top-down construction method has been used for construction of deep basement at major projects in Bangkok, Thailand. To be able to apply top-down construction method, structure elements are required to sustain the construction load and to utilize as a part of bracing system. Prefabricated steel columns known as stanchions embedded in bored piles or barrettes are commonly used for this requirement. This paper presents some experiences in the practical installation of the different stanchion types in large diameter deep-seated bored piles and barrettes in Bangkok. The problems encountered during the actual installation of the stanchions and solutions to them are also discussed.

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Made-Ground consists predominantly of Fill-Materials, Clayey Sand or Silty Clay with some cement block rubble and rock fragments, is commonly found up to 4m depth. Soft to very soft, highly compressible dark gray marine clay lies beneath Made-Ground and in some
areas it lies under weathered crust layers of 2m thick. Depending on the location, this layer is
extended up to 12-18m. About 2m thick Medium Clay layer can be observed between Soft
Clay and underlying Stiff Clay. Generally Stiff Clay layer occurs directly underneath
Medium Clay and its depth goes up to 22m. Below Stiff Clay layer, First Sand layer 5-8m in
thickness can be found. This First Sand layer, however, is absent in some areas. Stiff to Hard
Clay layer underlies First Sand and it is found to be about 5m thick. Second Sand layer
generally occurs at depths between 45 to 65m.

Stanchions are mainly installed in the large diameter deep-seated bored piles and
barrettes founded in first or second sand layer. Bored piles of diameter 1.50m to 1.8m and
rectangular-shape barrettes of 0.80x2.8m to 1.5x3.0m (thickness x width) founding at depth
between 50m and 60m are commonly used to accommodate the stanchions. Wet process
method is always used for construction of these large bored piles and barrettes in Bangkok.
For bored piles, temporary casing of 14-15m in length is used as a support in soft clay layer.
Soil inside the casing is normally excavated by auger applying rotary drilling method and
drilling is continued with the bucket under the slurry from the top of sand layer to the final
depth. A cable-hung-grab mounted on crawler crane is used for excavation of trench with
slurry support in barrette construction. Guide walls having a minimum depth of 1.2m are
commonly used for initial guiding of grab excavation at the top of the trench. Tremie
concreting is necessary for casting both bored piles and barrettes.

APPLICATION OF TOP-DOWN CONSTRUCTION METHOD

Top-down construction method as the name implies, is a construction method, which
builds the permanent structure members of the basement along with the excavation from the
top to the bottom. Top-down method is mainly used for two types of urban structures, tall
buildings with deep basements and underground structures such as car parks, underpasses and
subway stations. For tall buildings with deep basements, a full application of top-down
method enables superstructure to build concurrently with excavation and construction of the
basement giving a significant advantage in reducing the overall construction time. Top-down
method is the most appropriate solution in minimizing the duration and disruption to surface
traffic and other urban activities, for the construction of the underground car parks,
derpasses and subway stations where the structure is located directly underneath the
existing roads.

Application of top-down method offers three main advantages as outlined below.
• It allows early commencement of super-structure construction without the need to
  wait for excavation reaching the bottom level. Overall construction duration can be
  significantly less than that of conventional method thereby saving the cost.
• It does not require temporary bracing system so that time requirement and cost for
temporary works (both material and labor costs) are eliminated, in turn significantly
saving cost.
• Minimize soil movement induced by excavation works which is an important factor
  for some sensitive locations.

STRUCTURAL MEMBERS REQUIRED FOR TOP-DOWN CONSTRUCTION

Design and construction principles for top-down method primarily call for two major
structural elements.
• Columns with sufficient capacity must be pre-founded in bored piles or barrettes to
  sustain the construction load and to utilize as part of bracing system.
- Excavation for basement must be carried out with the support of permanent retaining wall so that basement floor slabs can be utilized as lateral bracing. Diaphragm wall of 0.8m to 1.2m in thickness with sufficient embedment in firm soil layers is commonly used as a retaining wall whereas prefabricated steel columns known as stanchions embedded in either large diameter deep-seated bored piles or barrettes are utilized as structural columns. Figure 1 illustrates the top-down construction method with utilization of stanchions and diaphragm wall.

![Figure 1. Top-down construction with stanchion and diaphragm wall](image)

**TYPES OF STANCHION AND THEIR APPLICATION IN BANGKOK**

Pre-fabricated “H” section steel columns and steel built-up-section columns are mainly used as pre-founded structural columns or stanchions. Pre-cast reinforce concrete columns are very seldom used (Manoharn S. & Aye Z. Z., 1994). For the purpose of convenient reference throughout the paper, the type of stanchion is categorized by size and capacity as presented in the table below.

<table>
<thead>
<tr>
<th>Type of Stanchion</th>
<th>Material &amp; Example Size</th>
<th>General Information</th>
<th>Limitation</th>
</tr>
</thead>
</table>
| Light stanchion   | Steel H-beams 350x350x137kg/m | • For semi top-down construction  
• For temporary decking | Limited capacity of light stanchion does not allow for construction of super structure until completion of basement construction |
| Medium-sized stanchion | Steel H-beams 350x350x390 kg/m | For semi and full top-down construction of shallow to medium deep excavation | Limited number of super structure floors construction |
| Heavy stanchion   | • Steel H-beams 508x457x738kg/m  
• Composite steel columns built up by 2 or more small to medium size H-beams  
• Large section pre-cast RC column (seldom use) | Full top-down construction in deep excavation | Depending on the loading condition, numbers of superstructure floors can be constructed before completing basement excavation |
Embedded length of stanchion usually ranges from 1.5 to 5m depending on the loading condition applied in design and construction.

ALLOWABLE TOLERANCE FOR POSITION OF STANCHIONS

Allowable tolerance for stanchions is usually called by the designer and it is mainly governed by the structural tolerance of the steel as well as required position of the finished columns. In most projects, allowable vertical and horizontal deviation of stanchion are specified as 25 to 50mm whereas verticality is required between 1:200 and 1:400. There are a number of constraints to get highly accurate position of heavy and lengthy stanchions which have to be installed in deep-seated foundation piles constructed by wet process in Bangkok subsoil as described in the following sections.

STANCHION INSTALLATION METHODS

Stanchion installation method is usually selected by the piling contractor who take into consideration three main factors such as installation depth, size of stanchion and size of bored or barrette piles. Though installation details may be different from one contractor to another, stanchion installation can be categorized under two main methods, post-concreting or plunging installation and pre-concreting installation or placing stanchion prior to concreting.

Post-concreting installation or plunging method

In this method, stanchion is installed immediately after completion of bored pile concreting process. General construction sequence involved in this method is demonstrated in Figure 2. Guide frame is used to install the stanchion at the correct position.

Pre-concreting or pre-placing installation method

In this method, stanchion is installed immediately after completion of bored pile concreting process. General construction sequence involved in this method is demonstrated in Figure 2. Guide frame is used to install the stanchion at the correct position.

Figure 2. General construction sequence of pre-concreting installation method

Pre-concreting or pre-placing installation method

In this method, stanchion is installed immediately after completion of drilling and reinforcement lowering prior to concreting process. In some projects stanchion is attached to the last section of reinforcement and installed together with the reinforcement. General construction steps involved in this method are demonstrated in Figure 3.
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Guide Frame for stanchion positioning

Guide frame is used for positioning of stanchion in both installation methods. Effectiveness of guide frame plays one of the important roles in achieving positional accuracy of stanchion. Figure 5 (a,b) shows the guide frame used for pre-concreting installation method in one major project constructed by Seafco Co., Ltd.
The stanchion installation methods used in other parts of the world have been presented in some published papers. Among these, Findlay (1989) reviewed a number of stanchion installation methods used in construction of large diameter bored piles for top-down construction particularly in UK. The author reported that steel columns can be placed with better accuracy by dry process bored piling method than wet process method (with support fluid).

Hollingsworth (1991) demonstrated the installation of stanchions by plunging into concrete with the aid of an adjustable guiding frame called Cemloc (European Patent No. 0302717) for seven levels basement of 25m deep constructed by top-down technique in London.

Ressi di Cervia and Tamaro (1991) briefly explained the stanchion installation method used for construction of an underground garage with top-down technique in Boston, USA. Steel columns were installed prior to concreting in load bearing elements (LBE) or barrette excavated by cable-hung mechanical clamshell buckets. The authors cited that using the same equipment for construction of diaphragm wall and load bearing elements minimized the congestion on site and streamlined the schedule.

Crawley and Stones (1996) presented the installation of 30m long composite steel columns for top-down construction applied for Westminster Station in Central London. Columns were placed with the aid of guide frame followed by concrete pouring. Accuracy of ±25mm at the top of column with verticality of 1:200 was achieved as stated by the authors.

Arz (1989) presented an installation of heavy composite columns fabricated from six steel H-beams encased by concrete (cast during fabrication) for construction of six storey building with five basement using top-down method in Germany. Due to the high accuracy requirement of 25mm, adjustable guide frame equipped with three hydraulically control arms were used. Stanchions were installed prior to concreting as reported by the author.

From the available literatures it is noted that pre-concreting method is more common in use than post-concreting method.

PROBLEMS ENCOUNTERED IN POST-CONCRETING INSTALLATION METHOD

The major problem commonly encountered in this installation method is inability to install the stanchion at the design position due to one or combination of the following causes.

- Inclination of the borehole / trench
- Stanchion can not be inserted up to the required depth as concrete becomes hard due to the premature setting
- During lowering, stanchion is stuck by reinforcement cage and due to the hardening of the concrete, extraction of stanchion becomes impossible for reinstallation

As the installation of the stanchion is often associated with many unforeseen problems, it is likely in many cases that concrete becomes stiff or prematurely set during the installation. In Bangkok, premature setting of concrete is usually found to be attributed by:
- long delivery time due to traffic conjunction
- inappropriate mix
- severe weather
- disruption in concreting / equipment breakdown

PROBLEMS ENCOUNTERED IN POST-CONCRETING INSTALLATION METHOD

Using this method can minimize some problems encountered during installation provided that adequate clearance is available between pile reinforcement and the stanchion for tremie pipes lowering. Inclination of borehole is also important for this installation method. Proper planning in concrete ordering is required to avoid unnecessary waiting of concrete trucks for stanchion placing which may affect the total batching time limitation. As stanchion has to be hung in the position until completion of concrete placement, appropriate and sufficient capacity of the hanging and positioning device should be provided to avoid falling and deviation of stanchion in the borehole. Temporary casing should not be used as a hanging system without additional support for the large stanchion.

FACTORS EFFECTING THE POSITIONAL ACCURACY OF THE STANCHION

It is hardly possible to install a stanchion so that its position and verticality is always as designed. As-built position of stanchion is found to be generally influenced by the installation method. The factors affecting the positional accuracy for two different methods are summarized in the Table 2 and 3.

Table 2. Post-concreting installation method – factors effecting positional accuracy

<table>
<thead>
<tr>
<th>Factors effecting accuracy</th>
<th>Discussion</th>
<th>Recommended measures to improve accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Verticality of piles</td>
<td>Due to the use of fixed guide arm for guiding stanchion against borehole wall, its verticality is solely influenced by the verticality of borehole.</td>
<td>Use adjustable guide</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maintain good horizontal position of drilling rig and verticality of temporary casing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Perform drilling monitor test after completion to check verticality prior to stanchion installation</td>
</tr>
<tr>
<td>Concrete stiffening or hardening during installation</td>
<td>Due to the stiffening or hardening of concrete, stanchion may not be able to place at the right elevation. Using vibro-hammer to force down stanchion can cause stanchion to deviate from the position.</td>
<td>Use concrete with appropriate admixture to achieve longer setting time</td>
</tr>
<tr>
<td>Inappropriate backfill material</td>
<td>Large aggregates backfill can cause locking up casing with stanchion upon casing extraction which create stanchion to deviate</td>
<td>Use appropriate backfill such as sand / fine aggregates.</td>
</tr>
<tr>
<td>Improper backfilling</td>
<td>Back filling from one side of stanchion can cause deviation of stanchion.</td>
<td>Maintain equal distribution of back fill around stanchion</td>
</tr>
<tr>
<td>Improper temporary casing extraction</td>
<td>Stanchion deviation in both horizontal and vertical can be caused by improper temporary casing extraction.</td>
<td>Apply proper equipment and method in casing extraction to suit the site condition and design requirement</td>
</tr>
</tbody>
</table>
Table 3. Pre-concreting installation method – factors effecting positional accuracy

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<tbody>
<tr>
<td>Verticality of piles</td>
<td>In this method stanchion position may be independent from borehole verticality prior to concreting. However due to poorer borehole verticality in comparison with that of stanchion, clearance between stanchion and borehole wall becomes smaller with depth at one side of stanchion. Thus during the tremie pipe lowering and extraction, stanchion is pushed by tremie causing stanchion to move horizontally.</td>
<td>• Maintain good horizontal position of drilling rig and verticality of temporary casing • Perform drilling monitor test after completion of drilling to check verticality prior to stanchion installation and adjust the tremie pipe position according to borehole verticality</td>
</tr>
<tr>
<td>Concrete flows pushing at stanchion during tremie concreting</td>
<td>During pouring of concrete from tremie pipe, force induced by concrete flow tends to move stanchion in both horizontal and vertical direction</td>
<td>• Use appropriate fixing (lock) at top • Use 2 sets of tremie pipes placing at other side of stanchion if possible</td>
</tr>
<tr>
<td>Improper back filling</td>
<td>Same as post-concreting installation method presented in above Table 2</td>
<td>Same as post-concreting installation method presented in above Table 2</td>
</tr>
<tr>
<td>Improper temporary casing extraction</td>
<td>Same as post-concreting installation method presented in above Table 2</td>
<td>Same as post-concreting installation method presented in above Table 2</td>
</tr>
</tbody>
</table>

Skill and experience of the contractor plays a major role in achieving positional accuracy of the stanchion provided that the installation is practical with the design elements such as size of foundation piles in relation to size of stanchion.

Figure 6. View of stanchions embedded in bored piles at final excavation level

STANCHION POSITION EFFECTED BY EXCAVATION

In Bangkok, post-installation movement (horizontal movement) of the stanchion is frequently encountered and it is mainly caused by excavation induced soil displacement particularly at initial stage of excavation where diaphragm wall deflection is characterized by large cantilever rotation within Soft Clay layer. In some projects, deviated stanchions had to be pushed or jacked back to the original position and restrained by means of temporary support until permanent slab was cast.

REVIEW OF THE COMPLETED PROJECTS

The authors were involved in a number of projects constructed by Top-down method in Thailand particularly in Bangkok.
Table 4 shows the summarized technical information of some major projects with different stanchion type, size and installation depth.

### Table 4. Summarized technical information of stanchion application in some major projects in Thailand

<table>
<thead>
<tr>
<th>Project No.</th>
<th>Stanchion Type and Size</th>
<th>Pile Type and Size</th>
<th>Installation Method</th>
<th>Basement Excavation Depth (m)</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Built-up steel column</td>
<td>Bored pile</td>
<td>Pre-concreting</td>
<td>-20.0m</td>
<td>Silom Road Bangkok</td>
</tr>
<tr>
<td></td>
<td>850x850mm, L=24m</td>
<td>φ1.5x 60m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>built-up by 4 H-beams</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>270x248x157kg/m</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>H-section steel column</td>
<td>Bored pile</td>
<td>Post-concreting</td>
<td>-29.5m</td>
<td>Rama IV Road Bangkok</td>
</tr>
<tr>
<td></td>
<td>508x457x 738 kg/m, L=30m*</td>
<td>φ1.8x65m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>H-section steel column</td>
<td>Barrette pile</td>
<td>Pre-concreting</td>
<td>-24m</td>
<td>Ratchadapisek Road Bangkok</td>
</tr>
<tr>
<td></td>
<td>419x407x390kg/m, L=24.7m*</td>
<td>1.2x3.0x47m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>H-section steel column</td>
<td>Bored pile</td>
<td>Pre-concreting</td>
<td>-14.5m</td>
<td>Payathai Road Bangkok</td>
</tr>
<tr>
<td></td>
<td>414x405x232 kg/m, L=18m</td>
<td>φ1.8x65m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Pre-cast RC column</td>
<td>Bored pile</td>
<td>Pre-concreting</td>
<td>-11.5m</td>
<td>Haadyai</td>
</tr>
<tr>
<td></td>
<td>500x1000mm, L=15m</td>
<td>φ1.5x35m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>500x15000mm, L=15m</td>
<td>φ1.8x35m</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: * Top levels of stanchion in Project 2 and 3 are 3m and 1.8m below ground level respectively.

A view of heavy stanchions (built-up sections) exposed after excavating to the first basement level is presented in Figure 7. Pre-concreting installation was used to install the stanchions in this project.

![Figure 7. View of built-up stanchions exposed after excavation to the first level of basement](image)

**COMPARISON OF AS-BUILT POSITIONAL TOLERANCE ACHIEVED BY TWO INSTALLATION METHODS**

Although the as-built position of stanchions installed by two different methods have yet to be statistically analyzed, field survey data of Projects 2 and 3 indicated in above Table 4 suggested that stanchions could be more accurately installed by pre-concreting method. Stanchions with as-built horizontal deviation greater than 100mm installed by two methods (expressed by percentage) are presented in the table below.
Table 5. Comparison of as-built position of stanchions by two installation methods

<table>
<thead>
<tr>
<th>Project</th>
<th>Installation Method</th>
<th>Depth of Measurement</th>
<th>As-built Position of Stanchions with Horizontal Deviation &gt;100mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Post-concreting Method</td>
<td>15.5m B.G.L</td>
<td>20 %</td>
</tr>
<tr>
<td>3</td>
<td>Pre-concreting Method</td>
<td>13.5m B.G.L</td>
<td>4 %</td>
</tr>
</tbody>
</table>

CONCLUSION

Two common stanchion installation methods have been presented with particular emphasis on the common problems encountered by each method and recommended measures to alleviate them. According to the author’s experience, pre-concreting installation method provided fewer problems in practical installation and achieved better positional accuracy. A comparison of as-built position of stanchion between two projects also suggested that stanchions could be more accurately installed by pre-concreting installation method.

The designers such as structural engineers and architects should be aware of the accuracy achievable by practical installation method and take it into consideration at the design stage.

Stanchion installation contractor should select the appropriate method, equipment as well as experienced personnel plus a well-formed plan with the consideration of all potential problems to achieve the successful construction of foundation structure, which is of primary importance for top-down technique.

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<tr>
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<th>As-built Position of Stanchions with Horizontal Deviation &gt;100mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Post-concreting</td>
<td></td>
<td>15.5m B.G.L</td>
<td>20%</td>
</tr>
<tr>
<td>3 Pre-concreting</td>
<td></td>
<td>13.5m B.G.L</td>
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**REFERENCES**


Concrete for wet processed bored piles

Thasnanipan N., Maung A. W., Tanseng P. & Navaneethan T.

**SEAFCO Co., Ltd., Bangkok, Thailand**
CONCRETE FOR WET PROCESS BORED PILES
Thasnanipan Narong, Maung Aung Win, Tanseng Pornpot & Navaneethan Thiruchelvam
SEAFCO Co., Ltd., Bangkok, Thailand.

ABSTRACT
The final quality of wet process bored piles is not only depending on the good construction practice of piling contractors but also on the characteristics of concrete supplied. The concrete characteristics are of the major items that influence the performance of piles after casting. Due to the nature and environment of wet process bored piling, both proper drilling and pouring of concrete would not produce the good quality piles, if the quality of concrete did not meet the basic characteristics required for bored piling process. This paper reviews the basic characteristics of concrete required in wet process bored pile and presents some example problems caused by improper concrete mix. Additionally, the problems caused by improper concrete pouring process and preventive measures are also described.

INTRODUCTION
The quality of piles depends on a good construction process including drilling, reinforcement installation and concrete pouring as well as on good quality of concrete. Quality of concrete has some influence on the workmanship with interrelated performances. For wet-process bored piles, concrete is cast under drilling slurry using tremie pipes. Good quality concrete in bored piling sense means that the properties and characteristics of the concrete are suitable for the process of work and subsequently meet requirements of the finished product. Continuous concrete pouring which is mandatory in piling and it is sometime disrupted by blockage of segregated or prematurely set concrete mix in the tremie pipe. Early setting of concrete after pouring in bored hole can also cause discontinuities in pile by accidental lifting of set concrete during extraction of the temporary casing. Dampness is sometimes found in top section of piles constructed in water-bearing permeable soil layer. The dampness was found to be caused by capillary action of ground water through interconnecting voids formed in the improperly mixed concrete.

Ready mixed concrete for bored piles usually specified as self-compacting concrete and its "compacting factor" is very important for achieving required strength, especially at the top section of pile which usually carries the maximum portion of transferred load. However, the compacting factor is seldom mentioned in piling practice. Besides, no vibration is allowed for tremie concrete while structural concrete is compacted by vibrator during casting. The concrete strength test is usually carried out on the compacted test samples and thus the actual strength of pile can be different from sample strength according to the compacting factor.

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WET PROCESS BORED PILING METHOD

In wet process bored piling, bentonite or other type of suitable drilling slurry is used as drilling fluid to support the borehole during construction. A steel temporary casing is usually
used to case the top weak soil subjected to heavy construction loads. Drilling and reinforcement cage installation and concrete placing are successively executed under drilling slurry. The concrete is poured with tremie pipes, displacing the slurry well above the cutoff level. The temporary casing is then extracted immediately after concreting.

CHARACTERISTICS OF CONCRETE AND CONCRETE MIX

Concrete mix for bored piles is designed according to concrete pouring process and mechanical properties required. Concrete for wet process piles needs to be specially mixed having cohesiveness with high workability (high slump/excellent fluidity) which is not prone to segregation and retain it workability as far as possible throughout the tremie placing operation for the complete pour. Addition to those characteristics, compaction under self-weight, resistance to harsh environment, resistance to leaching, and appropriate strength are essential.

Workability

Excellent fluidity is essential that the concrete has the ability to flow readily through the tremie pipe, to flow laterally through a reinforcement cage and a high lateral stress against the sides of borehole. High workability is best achieved with rounded natural aggregates and natural sand in the mix.

Self Compaction

Compaction under self-weight is essential as vibration of concrete is impractical, except near the surface. The degree of compaction achieved is determined by the density ratio (the ratio of density actually achieved to the density of the same concrete fully compacted). The recommended compacting factor for the required workability of tremie concrete is 0.95 to 0.96 (Xanthakos 1994). Fresh concrete is usually placed through tremie pipes and displaces the slurry by gravity action only. In some cases, lack of self-compaction in the concrete will lead to defects, such as reversed “hanging up”, and “whirls” in the completed pile. If the initial shear of the concrete is very high, the flow is likely to restrain, resulting in bentonite trapped in areas not reached by the concrete (Xanthakos 1994).

Resistance to Segregation

The concrete mix should be cohesive and resistant to segregation, as improperly designed mixes will segregate during placement, resulting in inferior concrete containing honeycombs and high permeable zones within the pile shaft. Concrete that bleeds or disintegrates under the pressures of its own weight can also block the tremie pipe or accept bentonite.

Controlled Setting

The concrete must retain it fluidity thorough the depth of borehole during complete placement of the concrete in the borehole and attain an appropriate strength within a reasonable time after placement. Retarders are used to prevent premature stiffening of some cements or to delay stiffening under difficult placing conditions. The setting time must be checked against the time necessary to complete the placement. The retarders should be used under competent technical advice and after adequate testing.

Table 1. Suggested Concrete Mix for Bored Piles Cast under Bentonite (After Fleming & Slwinski 1977)

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Cement Content</strong></td>
<td>Not less than 400 kg./m³</td>
</tr>
<tr>
<td><strong>Sand Content</strong></td>
<td>30% to 45% of total aggregate weight</td>
</tr>
<tr>
<td><strong>Sand Type</strong></td>
<td>Natural and complying with zone 2 or 3 grading</td>
</tr>
<tr>
<td><strong>Aggregate Type</strong></td>
<td>Natural round stone if possible, 20 mm. max. size</td>
</tr>
<tr>
<td><strong>Water/Cement</strong></td>
<td>Below 0.6</td>
</tr>
<tr>
<td><strong>Slump</strong></td>
<td>175 mm.</td>
</tr>
<tr>
<td><strong>Admixture</strong></td>
<td>Suitable admixture which will improve the workability and extend the period during which such workability is maintained are to be applied.</td>
</tr>
<tr>
<td><strong>Resistance to Segregation</strong></td>
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</tr>
</tbody>
</table>
Resistance to Aggressive conditions

The concrete should have high density and low permeability to resist the possible (chemical and physical) attack of an aggressive sub-subsurface condition. In some instances there is an underground flow of water that can cause a weakening of the concrete after it is placed, and a properly designed mix should be resistant to such flow. However, if the rate of ground water flow is substantial, a permanent casing will be necessary.

Good Mechanical Performance

The mechanical properties of hardened concrete can be satisfied in most instances. However, appropriate tensile strength for the concrete without reinforcement in piles and high level of bending and axial stress must be considered in some cases.

Reese and O’Neil (1988) emphasis that the design of the concrete mix must be given appropriate attention and the design of the mix is dependent strongly on the particular job, and the cement will be selected to be consistent with the design requirement. They observed that bleeding is not a problem for concrete mixes that are properly designed. The trial mix method is usually used in the laboratory. It is necessary to follow-up to see that the materials and proportions used by the batch plant are those of that are recommended. Inspection at the batch plant should include checking the nature, quantity and temperature of the components of the mix, the aggregates, cement, water, admixtures and of the completed mix for conformance with the specifications. For testing at the job site, the organization of the job must be such that time required to perform tests at the job site is kept to a minimum. Excessive job-site testing can lead to harmful effects. No delay in pouring should occur due to field test.

Adding of water to the concrete with very low slump on site to increase the workability can have detriment effect of reducing the strength, compactability and impermeability of the concrete. The results of adding water could be a significant change in the characteristics of mix and the possibility of segregation as the pour is made. Segregation of concrete during pouring can also lead to increase in permeability of concrete, especially at the top section of piles due to upward migration of water in the concrete mix. Adding of water to the concrete on site must not be allowed unless specified.

Suggested concrete mix by Fleming & Sliwinski (1977) for bored piles cast under Bentonite was shown in Table 1 while range of cement content and water-cement ratio in general use for concrete mixes by Bartholomew (1979) is shown in Table 2.

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<table>
<thead>
<tr>
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<tbody>
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<td>Slump</td>
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<tr>
<td>Water/Cement</td>
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</tr>
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<td>Natural and complying with zone 2 or 3 grading</td>
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<td>Sand Content</td>
<td>30% to 45% of total aggregate weight</td>
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<td>Admixture</td>
<td>The use suitable admixture which will improve the workability and extend the period during which such workability is maintained are to be advocated.</td>
</tr>
</tbody>
</table>
Table 2. Range of Cement Content in kg/m³ and Water Cement Ratio in General use for Concrete Mixes (After Bartholomew 1979)

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Conditions</th>
<th>Normal (kg/m³)</th>
<th>Moderately Aggressive (kg/m³)</th>
<th>Highly and Very Aggressive (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Precast</td>
<td></td>
<td>450-475</td>
<td>450-475</td>
<td>450-475</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.4 – 0.5</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>Bored Piles Dry Process</td>
<td></td>
<td>300-450</td>
<td>350-450</td>
<td>380-500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5 – 0.55</td>
<td>0.475 - 0.5</td>
<td>0.45 - 0.5</td>
</tr>
<tr>
<td>Bored Piles with Tremie Process</td>
<td></td>
<td>350-450</td>
<td>350-450</td>
<td>400-500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5 – 0.6</td>
<td>0.475 - 0.5</td>
<td>0.43 - 0.45</td>
</tr>
<tr>
<td>Driven Cast-In-Situ Piles</td>
<td></td>
<td>280-370</td>
<td>330-450</td>
<td>370-500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25 – 0.6</td>
<td>0.3 – 0.55</td>
<td>0.3 - 0.45</td>
</tr>
</tbody>
</table>

Concrete for bored piles compared to that for pre-cast drive piles is the least dense concrete due to pouring and casting process. Bored pile concrete was cast in the aggressive subsurface conditions such as, high salinity, ground water fluctuation or in the vicinity of sea and river. In such environment, the concrete can easily be leached out by ground water. In this case cement content and water cement ratio given in the Table 2 needs to be reviewed considering the site conditions.

Fleming et al (1977) pointed out that the high cement content favored for in-situ pile construction enable the necessary workability mixes to be used with adequate margins of safety against the inevitable variations in strength and workability. It also compensates for some reduction in strength, which may occur on interfaces during displacement.

Concrete is permeable to water to the extent that it has interconnecting void spaces through which water can move. Calcium hydroxide liberated by hydrating cement is water-soluble and may leach out of harden concrete, leaving voids for the ingress of water. Permeability of concrete is governed by amount of cementitious material, water content, aggregate grading, consolidation and curing efficiency.

METHOD OF CONCRETE POURING

The quality of piles depends on a good pouring procedure as well as on good quality of concrete. In wet process, the concrete is usually placed by a steel tremie pipe of 20-25cm in diameter (minimum 6 times of coarse aggregate size).

Prior to charging the tremie pipe with concrete, the bottom of tremie needs to be sealed by a plug of some descriptions may be inserted at the top of tremie before or after the tremie is placed in the bored hole as appropriate. There are two potential problems that are associate with the initial charging of tremie with concrete; the concrete can segregate during placement and the air in tremie will prevent the complete filling of the tremie. These problems can be avoided if the tremie is filled slowly. Faulty initial charging of tremie during concreting can cause entrapment of mud within the concrete.

Excessive initial lifting of tremie can result in possible distribution of leached concrete caused by concrete falling through the slurry. The bottom of tremie must stay well below the top of the column of fresh concrete all the time. Moreover, the tremie must not be lifted and
lowered rapidly to avoid the cause of contamination of concrete with slurry. It is suggested that the tremie pipe must not be lift and lowered rapidly to start or restart the flow of the concrete (Reese & Neil 1988). However Xanthakos (1994) suggested that if the concrete is not deposited easily the triemie pipes may be moved up and down with movement not exceeding 30cm. Moreover, the tremie pipes should not be moved horizontally.

In the area of high ground water level, the concrete must be deposited above the external water table before the casing is withdrawn. The hydrostatic pressure in the concrete column should be greater at all time than the pressure in any column of fluid outside the casing.

**COMMON DEFECTS**

The common defects of piles are cold joints, zone of segregated or contaminated concrete, trapping of bentonite mud and cavities. The first two types of defect result from interruptions in the concrete placement or premature extracting of the tremie pipe either partially or completely above the concrete-slurry interface. Mud trappings are caused by concrete of low workability and impediment to the flow of concrete due to closely spaced bars. Discontinuities or partial separation in the piles at the bottom edge of temporary casing can be caused by accidental lifting of low workability concrete or concrete without controlled setting during casing extraction. If the concrete in the casing is too stiff and has considerable frictional resistance against the casing, a column of concrete can be pulled up with the casing.

Permeability of concrete depends on the capillary porosity, water-cement ratio and degree of hydration. High permeability of concrete can be contributed by presence of capillary pores that are interconnecting voids in the concrete (Figs 1 & 2). The interconnecting voids are caused by bleeding of concrete due to excessive water used in the mix. Bleeding raises the water and air bubbles to top surface of fresh concrete and raises the water-cement ratio of concrete upper part of the forms, thereby reducing the strength and increasing porosity of concrete. Concrete mixed with a water-cement ratio higher than 0.6 can be more permeable. Function of concrete self-compaction can also reduce the permeability and increase the density. In piles, usually the fresh concrete compacts under its own weight, resulting in an increasing density with depth. Besides, concrete in a great depth of pile is generally cured in stable temperature and moisture.

![Figure 1. High permeability of concrete caused by interconnecting voids](image1)

![Figure 2. Low permeability of concrete caused by less interconnecting voids](image2)

Usually one type of concrete mix is used for piling in a particular project in most cases. However, the project in the vicinity of the river, some bored piles are to be installed closer to
the river than the remaining piles and thus the concrete mixes need to be designed accordingly. For the piles installed in the deposition side of the river where sand deposits occur, these piles are usually subjected to be effected by the ground water flow (Fig. 3). If the cement content is not high enough in the concrete mix and bleeding or segregation occurs, dampness or wet patches caused by capillary suction to ground water through the previous tremie location and vertical steels can be found on the pile head (Figs 4 & 5). Figure 6 is a photo of core sample obtained from the pile that exhibits the dampness on top, showing segregated concrete caused by bleeding.

![Figure 3. Capillary flow of ground water in bored pile](image3)

![Figure 4. Capillary suction and hydraulic pressure in concrete](image4)

![Figure 5. Wet pile head caused by capillary flow of ground water through segregated concrete in the pile](image5)

![Figure 6. Core samples showing segregated concrete in the pile](image6)

Figure (7) show the some segregated concrete extracted from the tremie pipe, which was blocked with such concrete. Settlement of solid particles or aggregates in the concrete mix cause bleeding by migration of the water to the top surface of fresh concrete, reducing water-cement ratio of the lower part of the mix. As a result, lower part of the concrete mix stiffens rapidly with aggregates. In such case stiffening concrete can also block the tremie
pipe during concrete pouring (Fig. 8) causing disruption in casting process and thus effects the quality of pile. Equipment for concrete pouring must also be adequate and reliable to avoid any interruption due to a breakdown in continuous tremie concrete pouring operation.

Due to a congested bar arrangement, concrete cannot flow through the bars and as a result concrete cover can be lost. Good arrangement of reinforcing bars is thus necessary. The horizontal spacing of main bars should be at least 10cm and 15cm (minimum 4 times of the maximum size of aggregate) for small and large diameter bars respectively.

RECOMMENDATIONS AND CONCLUSIONS

Foundation designers and concrete suppliers should pay more attention to workability and many other important factors required for cast in-situ tremie concrete than strength alone. It is essential to design the better quality mix than the concrete for other structural works in some aspects, considering the process of work.

For the projects, such as elevated highways, viaducts and bridges, variable soil and ground water conditions can be encountered along the project area. In such cases, concrete mixes need to be designed to suit these conditions and used accordingly in bored pile construction.

An adequate cement content and water cement ratio is necessary to have good impermeability of concrete which is one of factors influencing the durability of concrete, especially for bored piles in water bearing subsoil.

It is concluded that appropriate concrete mix and casting practice is essential in bored piling work to achieve good quality piles.

REFERENCES


RECORD LOAD TEST ON A LARGE BARRETTE AND ITS PERFORMANCE IN THE LAYERED SOILS OF BANGKOK

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Pornpot Tanseng
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Record load test on a large barrette and its performance in the layered soils of Bangkok

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ABSTRACT

This paper presents the load transfer characteristics of fully instrumented barrette of 1.5x3.0m in size seated about 57m below the ground level. The test results were compared with those from the instrumented load test on bored pile of diameter 1.5m with the same length, located 30m away. No significant difference in load transfer has been observed between barrette and pile despite the considerable difference in construction method applied and time consumed.

Keywords: Barrette, static load test, load transfer characteristics

1. INTRODUCTION

Barrettes have been used as deep foundations for various structures in Bangkok for a number of years. The large load carrying capacity achievable by flexible dimension and length of barrettes provides a major advantage in prevailing subsoil condition of Bangkok. In addition to extensive bearing capacity requirement, the demand for barrettes is necessitated mainly by site constraints, applicable construction method and equipment. Barrettes with dimension ranging from 0.80mx2.7m to 1.5mx3.0m for safe working load capacity from 1100 to 2300 ton have been used in some major projects. To assess the performance of large-capacity foundation element in layered subsoil of Bangkok, instrumented load testing is compulsory. This paper presents the static load test results of instrumented barrette tested up to 5290 ton for foundation of a fifty-storey building in Bangkok. The test results, particularly load transfer characteristics, and shaft friction capacity were compared with those of bored pile diameter 1.50m with the same length located 30m away.

2. OVERVIEW OF THE PROJECT

Foundation of the fifty-storey tower called for 560 bored piles of 1.2m and 1.5m diameter and 24 number of barrettes having cross section size of 1.5m x 3.0m. Bored piles and barrettes were seated at approximate depth of 57m in the second sand layer. Barrettes were designed to support the large load for tower’s lift shafts as bored piles were not feasible to utilize in such case. Base grouting was applied for barrettes and bored piles at the locations of high column load mainly in the central tower area. Instrumented static pile load test was proposed for one barrette and one bored pile of diameter 1.5m. The design safe working loads for the base grouted barrettes and bored piles are 2,300 ton and 1,175 ton respectively. Foundation plan of the project is shown in Figure 1.
3. SUBSOIL CONDITION

Soil investigation from five boreholes at different location reveals that subsoil layers along the site are relatively consistent. Similar to other localities in Bangkok a typical subsoil profile at the site is characterized by the alternating layers of clay and sand deposits as soil succession shown in Figure 7. Soft, highly compressible dark gray marine clay lies beneath weathered crust layers of 2m thick and extends up to 13.5m. Stiff Clay layer occurs directly underneath Soft Clay and its depth goes up to 26m. Below Stiff Clay layer, First Sand layer of 10m in thickness can be found. Hard Clay layer underlies First Sand and it is found to be about 12m thick. Second Sand layer occurs at depths between 50 to 72m. Undrained shear strength (Su) obtained from unconfined compression test and Standard Penetration Test (SPT) are shown in Figure 7.

4. CONSTRUCTION METHODS

Both barrette and bored pile were constructed by wet process under bentonite slurry. Bentonite slurry conforming to the widely accepted specification was used. Properties of the bentonite slurry used are given in Table 1. Single stage base grouting was applied 24 hours after concreting for both barrette and bored pile. The detailed procedures utilized in the construction of two different foundation structures are outlined below.

<p>| Table 1. Comparison of bentonite slurry properties |
|------------------------------|---------|---------|------------------------------|---------|</p>
<table>
<thead>
<tr>
<th>Properties</th>
<th>Barrette</th>
<th>Bored Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before feeding to the borehole</td>
<td>Before Recycling &amp; Before Concreting (near trench base)</td>
<td>Before feeding to the borehole</td>
</tr>
<tr>
<td>Density (g/cc)</td>
<td>1.10</td>
<td>1.17</td>
</tr>
<tr>
<td>Viscosity (sec)</td>
<td>36</td>
<td>49</td>
</tr>
<tr>
<td>Sand Content (%)</td>
<td>1.0</td>
<td>1.1</td>
</tr>
<tr>
<td>pH value</td>
<td>8</td>
<td>9</td>
</tr>
</tbody>
</table>

Mechanical rope-grab was used to excavate the trench. A guide wall cast with inside clear dimensions slightly larger than the nominal size of the barrette was used to guide the grab during
initial bites. Since time consumed in the preparation of instrumentation was relatively long, desanding was continuously done to keep the bentonite slurry agitated, which also helps to alleviate the growth of filter cake by minimizing the actual exposure time. As another measure, trench was once again occupied by grab to scrap the trench walls to remove, if any, filter cake formed on the walls. This attempt is in line with the recommendation made by Reese and O’Neill (1988) [1]. It is authors opinion that, if due to some unforeseen reasons, reinforcement cage lowering have to be delayed for considerable period of time, it is a good practice to use the grab again to scrap the trench walls. This measure eliminates any foreseeable negative impacts caused by unexpected delays. After lowering the rebar cage, tremie concreting was done.

Rotary drilling was employed for bored pile excavation. Different from barrette excavation, temporary casing of 15 m length was used as a support in Soft Clay layer for bored pile drilling to assure the stability of the borehole. Firstly, auger was used to drill within the temporary casing, followed by rotary bucket with bentonite slurry down to final depth of excavation. The base of the borehole was cleaned by recycling technique to minimize any congregated sediments. Before lowering the reinforcement cage special cleaning bucket was used to scrap of the borehole walls and the base. Reinforcement cages were then lowered inside the borehole while attaching the instrumentation simultaneously at specified locations. Soon after lowering the rebar cage tremie concreting was commenced. Polystyrene grains plug was used before the first charge of concrete to avoid the mixing of bentonite with concrete. Time consumed in different construction activities is plotted in Figure 2.

Figure 2  Time consumed in different construction activities of barrette and bored pile (after Thasnanipan, 1999 [2])

5. LOAD TEST PROGRAM AND INSTRUMENTATION

5.1 Test Pile Layout

Test pile layout of barrette is presented in Figure 3. With overall height of 8m, reaction frame utilized for static load testing on barrette in this project was claimed to be one of the biggest of its kind in the region. Four barrettes were used as anchoring system. Five numbers of built-up steel girders supported on each side by two 1st level cross beams were used as main beams to achieve the maximum capacity of 6000 tons. First level beams were supported against the second level cross beams. Second level cross beams were anchored against surrounding barrettes using anchor blocks at the top. Specially fabricated rigid transfer girders were used to distribute the tension force coming from the tie-bars to dowel bars above the anchor barrette heads. Sixteen numbers of hydraulic jacks each having 500 ton capacity were placed between the test barrette cap and the main beams of the reaction frame. General view of the barrette load test set up is presented in Figure 4.

Test pile layout of bored pile was similar to those of other static bored pile load tests in Thailand. Steel test frame anchored against four bored piles was used in bored pile load test. Different from the test frame of barrette, only 1 layer of cross beams was required for that of bored pile.
5.2 Monitoring System and Instrumentation

Direct measurement from four dial gauges placed in diametrically opposite positions having equidistance from the test pile axis was used as a main monitoring system of test pile head movement. Precise leveling and piano wire were also utilized as backup for pile head movement measurement. Additional two dial gauges were also used to monitor the lateral movement of the pile. Eight number of SINCO load cells of 500 ton capacity each were installed on top of the hydraulic jacks to evaluate the actual applied load of the first load cycle. Vibrating wire strain gauges (VWSGs) and Mechanical Extensometers (ME) were fixed at five levels along the shafts at the known interface boundaries of different soil layers. At each level, four sets of VWSGs and one set of ME were installed for the pile whereas six sets of VWSGs and two sets of ME were installed for the barrette.
5.3 Load Test

Pile load tests were carried out in accordance with ASTM D 1143-81. Pile and barrette were tested under three and four cycles of loading and unloading respectively.

6. LOAD TEST RESULTS AND INTERPRETATIONS

6.1 Pile Head Movement at Applied Load

Load vs pile head movement graph of barrette and bored pile is illustrated in Figure 5. Measured pile head movements at design load, double of design load and maximum test load are presented in Table 2. As can be seen in the table, both barrette and bored pile experienced only negligible pile head movement at relevant design load.

Table 2  Predicted and measured pile head movement at specific applied load

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Design Load (DL)</th>
<th>Max. Test Load</th>
<th>Measured gross pile head movement at applied load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>At DL</td>
<td>At 2 x DL</td>
<td>At Max. Test Load</td>
</tr>
<tr>
<td>Barrette</td>
<td>2300 ton</td>
<td>5290 ton</td>
<td>-5 mm -12 mm -61 mm</td>
</tr>
<tr>
<td>Bored Pile</td>
<td>1175 ton</td>
<td>2700 ton</td>
<td>-6 mm -19 mm -67 mm</td>
</tr>
</tbody>
</table>

Table 3 shows the estimated ultimate load capacity of barrette and bored pile from the load vs pile head movement using different method.

Table 3  Estimated ultimate capacity of barrette and bored pile

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Parameter</th>
<th>Ultimate capacity ($Q_{ult}$) and pile head movement at $Q_{ult}$ estimated by different method</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Davisson’s</td>
</tr>
<tr>
<td>Barrette</td>
<td>$Q_{ult}$ (ton)</td>
<td>5180 ton</td>
</tr>
<tr>
<td>Pile head movement at $Q_{ult}$</td>
<td>18 mm</td>
<td>18 mm</td>
</tr>
<tr>
<td>Bored Pile</td>
<td>$Q_{ult}$ (ton)</td>
<td>2675 ton</td>
</tr>
<tr>
<td>Pile head movement at $Q_{ult}$</td>
<td>30 mm</td>
<td>38 mm</td>
</tr>
</tbody>
</table>

Figure 5  Pile head movement at the last load cycle (maximum loading) of barrette and bored pile
6.2 Mobilized Skin Friction vs Pile Head Movement

The graphs showing the ratio of mobilized skin friction to maximum mobilized skin friction against ratio of pile head movement to pile diameter (s/D) are presented in Figure 6a and 6b respectively. Symbol ‘D’ shown in Figure 6a represents the barrette dimension in equivalent diameter. According to the figures, in general, skin friction mobilized to the maximum values at pile head movement of 0.6% and 1.6% of equivalent shaft diameter for barrette and diameter of bored pile respectively.

6.3 Load Transfer Characteristics of Barrette and Bored Pile

Load transfer curves along the shaft of barrette and bored pile at various applied load are shown in Figure 7. The values of unit skin friction developed at the different soil layers along the shaft of barrette and bored pile in comparison with those of calculated ultimate unit skin friction are demonstrated in Figure 8. This comparison suggests that mobilized skin frictions of barrette and bored pile are higher than those of calculated values using empirical formulas. It proves the findings of various researchers on shaft friction improvement of base grouted piles (e.g., Teparaksa et al. 1999, [3]). It is also evident that overall shaft resistance of both pile was not fully developed at design load.
6.4 Effect of Construction Time on Shaft Capacity Reduction

Excessive construction time is one of the main parameters claimed to be responsible for shaft friction capacity reduction of drilled shaft foundation constructed with wet process under bentonite slurry as reported by various researchers. Total construction time of barrette and pile were 75 hours and 27 hours respectively. Though total construction time consumed for test barrette was almost 3 times more than that of bored pile, there is no significant difference in shaft load transfer between them. The measures adopted in construction of barrette to minimize the excessive filter cake formation along the sand layers by proper desanding (continuous agitation), and retrenching with grab are considered the main reasons contributed to this achievement. Comparison of developed unit skin friction values of barrette and pile proves that difference in utilized bentonite viscosity as shown in Table 1 does not have significant effect on the shaft capacity. These findings are in line with conclusions made by Thasnanipan et al (1999) [4].

6.5 Effect of Shape on Shaft Load Transfer

Hosoi et al (1994) [5] concluded, from the results of numerical analysis that earth pressure acting on the flat surface of diaphragm wall panel is larger than that of circular bored pile. Thasnanipan et al (1999) [2] attempted to assess the effect of different aspect ratios (L/B) on the earth pressure developed around the trenches by using finite element program and reported that no significant difference in earth pressure was observed between the borehole (L/B=1) and the barrette (L/B=2). Further research is necessary to evaluate this finding. No significant difference is found between the maximum mobilized skin friction against displacement of rectangular-shape barrette and circular-shape bored pile according to Figure 6a and 6b respectively. Displacements at maximum mobilized skin frictions of barrette and bored pile fall within the range (0.5 % - 2 % of shaft diameter) reported by Reese (1978) [6].
7. CONCLUSIONS

(1) Elastic deformation of both barrette and bored pile at relevant design load was found to be negligible. Shaft resistance of barrette and bored pile was not fully mobilized at relevant design load. These observations suggested that the selected size and length of barrette and bored pile for specified design loads are sufficient for acting as friction piles.

(2) Mobilized shaft frictions of barrette and bored pile at maximum test load particularly in the first sand layer are considerably higher than calculated values. Shaft friction capacity improvement was considered to be contributed by base grouting which is in line with the findings reported by various researchers.

(3) Despite the considerable difference in construction method applied and construction time consumed (75 and 27 hours for barrette and bored pile respectively), similar characteristics of shaft load transfer were observed between barrette and bored pile. This achievement indicated that adopted measures (proper desanding and scraping of trench wall by grab prior to reinforcement lowering) to alleviate the excessive growth of bentonite filter cake on the barrette wall proved to be effective.

(4) Static load test up to 5290 ton conducted on barrette set the record as the highest load ever tested for a single cast-in-situ deep foundation in Thailand.

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REFERENCES

[1] Reese L.C., & O'Neill M. W., Drilled Shafts : Construction Procedures and Design Methods, ADSC: The International Association of Foundation Drilling, Dallas, Texas, USA, 1988


Performance of Wet-Process Bored Piles Constructed with Polymer-Based Slurry in Bangkok Subsoil

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Abstract

Conventionally, medium to large bored piles deep-seated in the multi-layered soils of Bangkok were constructed by rotary drilling under wet process or slurry replacement method, using bentonite slurry as support fluid in boring process. As an alternative to bentonite slurry, polymer-based slurry has been increasingly used in Bangkok recently. Polymer-based slurry having environmentally friendly properties, is found to be a good solution in minimizing the problems associated with the disposal of contaminated soils as well as the site pollution. This paper presents the performance of medium to large diameter bored piles constructed with polymer-based slurry in Bangkok. The static pile load test results of polymer-based slurry bored piles are compared with the predicted values using bentonite piles parameters. The measured frictional resistance of pile shaft from instrumented pile load tests are also reported. Shaft resistance of bored piles constructed with polymer-based slurry is found to be better than that of bored piles constructed with bentonite slurry.

Introduction

Medium to large diameter bored piles with length ranging from 24 m to 60 m have been used as foundations of the various structures in Thailand for over 3 decades. Rotary drilling technique coupled with wet process method using bentonite slurry as support fluid is commonly applied to construct these piles. As bentonite slurry had proven acceptable to make qualified bored piles in the past, most of them were constructed only by using this mineral based slurry. Recently, polymer-based slurry has been increasingly used in Bangkok as an alternative supporting fluid in bored pile drilling. In the initial stage, the primary reason for using this alternative slurry was to minimize the problems associated with using classic bentonite such as site pollution and the disposal of excavated soils which are usually contaminated. Construction records and static pile load test conducted on bored piles constructed with polymer-based slurry proved that there is no adverse effect on stability of borehole during drilling as well as capacity of the pile from using this alternative supporting fluid in Bangkok subsoil. Dry anionic partially hydrolyzed polyacrylamide (PHPA) polymer powder premixed with fresh water and a small percentage of bentonite was used in the construction of bored piles presented in this paper.
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Subsoil Profile and Existing Pore Pressure Condition

Subsoil profile is relatively consistent at different localities in Bangkok. A typical subsoil profile is characterized by the alternating layers of clay and sand deposits as shown in Figure 1 and is briefly described below.

Weathered crust of 2 m thick is commonly found as the top layer. In urban areas of Bangkok, this layer is covered by filled material. Soft to very soft, highly compressible dark gray marine clay lies beneath weathered crust. Depending on the location, this layer extends up to 12-18 m. About 2 m thick medium clay layer can be observed between soft clay and underlying stiff clay. Generally stiff Clay layer occurs directly underneath medium clay and its depth goes up to 22 m. Below stiff clay layer, first sand layer 5-8 m in thickness can be found. This first sand layer, however, is absent in some areas. Stiff to hard clay layer underlies first sand and it is found to be about 5 m thick. Second sand layer generally occurs at depths between 45 to 65 m.

Existing pore water pressure conditions in upper part of Bangkok soft clay are hydrostatic from nearly 1m below the ground level. Then the hydrostatic condition changed to piezometric drawdown near bottom level of Bangkok soft clay as shown in Figure 1. Excessive ground water abstraction from the deep aquifers in the past is responsible for this significant drawdown of piezometric pressure (Natalaya, 1981, Thasmanipan et al., 1998).

Wet-Process Bored Pile Construction Method

In Bangkok, medium to large diameter (0.6 m to 1.8 m diameter) bored piles with depth ranging from 24 m to over 60 m are constructed by wet process due to the prevailing subsoil condition and limited application of dry process. Wet process method as its...
name implies makes the pile under wet condition by using drilling slurry. Temporary casing of 14-15 m in length is also used as a support in soft clay layer. Drilling is commonly commenced by dry process with auger applying rotary drilling action in the soft clay layer and stiff clay layer. Before reaching to the first sand layer slurry is fed to the borehole and drilling is continued with the bucket to the final depth. A cleaning bucket is commonly used to clean the bottom of the borehole prior to reinforcement cage installation. Tremie concreting is necessary for the piles installed under wet process.

**Polymer-Based Drilling Slurry**

The primary function of drilling slurry is to prevent the borehole instability as deep-seated bored piles are required to install through water bearing and caving soils. Bentonite slurry was mainly used in the past as it adequately fulfilled the requirement. Polymer-based slurry has become increasingly popular as an alternative supporting fluid in recent years. The main reason for switching to use polymer-based slurry is to minimize the problems associated with environmental issues caused by the use of bentonite slurry.

In the initial stage of using polymer-based slurry, the major concerns were the potential negative impact on stability of the borehole and the shaft friction capacity as this alternative slurry was not yet accustomed to the bored piling industry in Bangkok. Trial boreholes conducted in the initial stage of introducing polymer-based slurry revealed that borehole is well stabilized if the right dosage is used. Dry anionic partially hydrolyzed polyacrylamide (PHPA) polymer powder premixed with fresh water and a small percentage of bentonite is mainly used for long bored piles in Bangkok. First, a small dosage of bentonite powder is premixed with fresh water prior to polymer powder introduction. The predetermined dosage of dry polymer is then added and mixed with the bentonite slurry. Premixed polymer-based slurry is stored in the steel tanks before supplying to the bored pile drilling. The reason for adding bentonite for long bored piles is to minimize the fluid loss which was observed from the trial boreholes drilled only with pure polymer slurry. Depending on the thickness of sand layers present at site and the length of pile, addition of bentonite may not be necessary. Close monitoring was made by sonic caliper measurement (KODEN drilling monitoring equipment) to check the stability of the borehole. The results of the sonic caliper measurement proved that borehole can be well stabilized with the use of polymer-based slurry at total construction time prior to concreting even over 24 hours. According to the cross-hole sonic logging or sonic coring tests conducted on bored piles, there is no evidence that degraded concrete remains along the pile shaft due to the use of polymer-base slurry.

Load tests were carried out to confirm the capacity of bored piles constructed with the alternative drilling slurry. As most of the test piles constructed with polymer-based slurry gave smaller settlement values, particularly at maximum test load than those obtained from piles constructed with bentonite slurry, confidence in using polymer-based slurry was raised among the designers and the contractors.

**Review of Shaft Friction Capacities Affected by Drilling Slurry**

Shaft capacity degradation of wet-process bored piles may be attributed to three main factors: reduction of effective stress at borehole interface, properties of slurry and...
excessive filter cake formation. Reduction of effective stress at borehole interface caused by installation process of wet-process bored pile is theoretically explicable and understandable but to the authors’ knowledge, detailed study on the issue has not been well documented.

Properties of bentonite influence on the shaft capacity degradation has been studied by many researchers. Thasanipan et al. (1998) concluded from the case study conducted on 11 bored piles in Bangkok subsoil that viscosity of bentonite slurry not more than 55 sec/qt (Marsh Funnel Viscosity) does not have significant effect on the shaft load transfer.

A number of researchers reported the reduction of shaft capacity caused by excessive formation of the filter cake on borehole wall of bored piles constructed with bentonite slurry. It is mainly contributed by long construction time without slurry agitation as cited by them.

Few researches are available for the effect of polymer-based slurry on shaft friction capacity of bored pile. Ata and O’Neill (1998) concluded that unit side shear resistances in both clay and sand layers of polymer test piles are higher than those of bentonite piles. Bustamante and Gianselli (1998) carried out the comprehensive study on the performance of polymer bored piles. The authors reported that no reduction of shaft friction was observed from load test results of polymer bored piles in comparison with bentonite bored piles.

Littlechild and Plumbridge (1998) concluded from test bored piles conducted in Bangkok, that the shaft friction resistance of bored piles constructed under polymer slurry shows no significance change with construction times up to 42 hours. Based on the test results of bored piles they also reported a slight reduction in shaft resistance with decreasing polymer viscosity.

**Design Parameters for Bored Piles**

Following empirical formula are normally adopted to estimate the ultimate skin friction capacity of bored piles.

Total stress approach is used to calculate the shaft friction capacity of clay layers.

\[ f_s = \alpha \cdot C_u \]

where,

- \( \alpha \) = adhesion factor
- \( C_u \) = undrained shear strength

Undrained shear strength, \( C_u \) is usually obtained directly from laboratory test results for soft clay layer and indirectly estimated from SPT “N” values for deeper stiff to hard clay layers.

Effective stress approach is applied to calculate the shaft friction capacity of sand layers.

\[ f_s = \beta \cdot \sigma' \]

where,

- \( \sigma' \) = effective overburden pressure
- \( \beta \) = shaft friction factor = \( K_s \cdot \tan \delta \)
- \( K_s \) = coefficient of horizontal earth pressure
- \( \delta \) = coefficient derived from internal friction angle \( \phi \) of sand layer

---

**Table 1. Technical data of polymer-based slurry used for test piles**

<table>
<thead>
<tr>
<th>ID</th>
<th>Dia. x Depth</th>
<th>Polymer (kg/m^3)</th>
<th>Bentonite (kg/m^3)</th>
<th>Mixed Ratio</th>
<th>Total Time Prior to Concreting (Hrs : Mins)</th>
<th>Density (g/ml)</th>
<th>Viscosity*</th>
<th>pH**</th>
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<tbody>
<tr>
<td>TP-1</td>
<td>1.00 x 49</td>
<td>0.50</td>
<td>12</td>
<td>17</td>
<td>20</td>
<td>1.03</td>
<td>45</td>
<td>1.03</td>
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<tr>
<td>TP-2</td>
<td>0.80 x 50</td>
<td>0.50</td>
<td>12</td>
<td>12</td>
<td>20</td>
<td>1.03</td>
<td>45</td>
<td>1.03</td>
</tr>
<tr>
<td>TP-3</td>
<td>1.20 x 51</td>
<td>0.50</td>
<td>10</td>
<td>14</td>
<td>00</td>
<td>1.01</td>
<td>44</td>
<td>1.01</td>
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<tr>
<td>TP-4</td>
<td>1.80 x 62</td>
<td>0.50</td>
<td>10</td>
<td>23</td>
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<td>1.04</td>
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<tr>
<td>TP-5</td>
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<td>0.50</td>
<td>12</td>
<td>15</td>
<td>30</td>
<td>1.04</td>
<td>44</td>
<td>1.04</td>
</tr>
<tr>
<td>TP-6</td>
<td>1.00 x 51</td>
<td>0.22</td>
<td>5</td>
<td>7</td>
<td>00</td>
<td>1.02</td>
<td>36</td>
<td>1.02</td>
</tr>
<tr>
<td>TP-7</td>
<td>1.00 x 50</td>
<td>0.50</td>
<td>12</td>
<td>12</td>
<td>20</td>
<td>1.03</td>
<td>45</td>
<td>1.03</td>
</tr>
<tr>
<td>TP-8</td>
<td>1.20 x 51</td>
<td>0.50</td>
<td>10</td>
<td>14</td>
<td>00</td>
<td>1.01</td>
<td>44</td>
<td>1.01</td>
</tr>
<tr>
<td>TP-9</td>
<td>0.80 x 43</td>
<td>0.50</td>
<td>12</td>
<td>8</td>
<td>00</td>
<td>1.03</td>
<td>45</td>
<td>1.03</td>
</tr>
<tr>
<td>TP-10</td>
<td>0.60 x 26</td>
<td>0.50</td>
<td>5</td>
<td>2</td>
<td>30</td>
<td>1.02</td>
<td>43</td>
<td>1.02</td>
</tr>
<tr>
<td>TP-11</td>
<td>0.60 x 24</td>
<td>0.70</td>
<td>-</td>
<td>0</td>
<td>45</td>
<td>1.02</td>
<td>42</td>
<td>1.02</td>
</tr>
</tbody>
</table>
Effective overburden pressure, $\sigma_v'$ at existing piezometric drawn down condition is usually used to calculate shaft friction capacity of sand layers. The design lines of $\alpha$ and $\beta$ of the previous research data which were developed from the instrumented load test results of bored piles constructed with bentonite slurry are commonly used in estimation of pile capacity. Hence, it is important to justify whether these design lines are applicable for bored piles constructed with polymer-based slurry. Assessment is made in the later section of this paper to justify this issue.

**Load Test on Polymer-Based Slurry Bored Piles**

Static pile load test on eleven test piles constructed by wet process with polymer-based slurry are presented in this paper. Out of these, three piles were instrumented with vibrating wire strain gauges and mechanical extensometers at interface of different soil layers. All test piles except TP-8 were tested to minimum 2.5 times of pre-defined design load, which is commonly estimated in the preliminary design stage.

**Slurry Properties and Construction Records of Test Piles.** Table 1 shows the properties of polymer-based slurry and construction records of test piles. It is to be noted that all these test piles were constructed by the same contractor. Hence it is assumed that influence of other variables such as equipment as well as construction procedure used in drilling, slurry mixing and quality control measures applied for all piles would be minimized in evaluation of the load test results.

Table 1. Technical data of polymer-based slurry used for test piles

<table>
<thead>
<tr>
<th>Pile ID</th>
<th>Pile Dimension Dia. x Depth (m)</th>
<th>Mixed Ratio per m³ of water</th>
<th>Total Time Prior to Concreting (Hrs : Mins)</th>
<th>Properties of Slurry Prior to Concreting</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Polymer (kg)</td>
<td>Bentonite (kg)</td>
<td>Density (g/ml)</td>
</tr>
<tr>
<td>TP-1</td>
<td>0.80 x 49</td>
<td>0.50</td>
<td>12</td>
<td>17 : 20</td>
</tr>
<tr>
<td>TP-2</td>
<td>0.80 x 50</td>
<td>0.50</td>
<td>12</td>
<td>12 : 20</td>
</tr>
<tr>
<td>TP-3</td>
<td>1.20 x 51</td>
<td>0.50</td>
<td>10</td>
<td>14 : 00</td>
</tr>
<tr>
<td>TP-4</td>
<td>1.80 x 62</td>
<td>0.50</td>
<td>10</td>
<td>23 : 00</td>
</tr>
<tr>
<td>TP-5</td>
<td>1.50 x 54</td>
<td>0.50</td>
<td>12</td>
<td>15 : 30</td>
</tr>
<tr>
<td>TP-6</td>
<td>1.00 x 51</td>
<td>0.22</td>
<td>5</td>
<td>7 : 00</td>
</tr>
<tr>
<td>TP-7</td>
<td>1.00 x 30</td>
<td>0.50</td>
<td>5</td>
<td>3 : 45</td>
</tr>
<tr>
<td>TP-8</td>
<td>0.80 x 41</td>
<td>0.50</td>
<td>10</td>
<td>6 : 55</td>
</tr>
<tr>
<td>TP-9</td>
<td>0.80 x 43</td>
<td>0.50</td>
<td>12</td>
<td>8 : 00</td>
</tr>
<tr>
<td>TP-10</td>
<td>0.60 x 26</td>
<td>0.50</td>
<td>5</td>
<td>2 : 30</td>
</tr>
<tr>
<td>TP-11</td>
<td>0.60 x 24</td>
<td>0.70</td>
<td>-</td>
<td>0 : 45</td>
</tr>
</tbody>
</table>

Note: *Marsh Funnel Viscosity, **API RP 13-B
Prediction vs Performance of Test Piles. As none of the test piles except TP-5 was tested to failure load, no attempt is made to determine the capacity of the piles from the failure load criteria for further assessment. Instead the performance of test piles is evaluated from load-settlement characteristics as described below.

As large quantities of tested-to-failure instrumented load test results of bored piles constructed with bentonite slurry in Bangkok are available, load-settlement behavior of this pile type can be accurately predicted using back-analyzed data. Load-settlement behavior of all test piles constructed with polymer-based slurry presented in this paper were predicted by taking the parameters obtained from back-analyzed data of bentonite bored piles.

Predicted load-settlement curves are compared with those of actual test results as shown in Figure 2 to 12. It is to be noted that the term “gross settlement” indicated in the graphs represents the total pile head movement at applied load. The objective of these comparisons is to assess from the actual load test results, whether or not use of polymer-based slurry in bored pile construction affects the capacity of bored piles.

The summary of predicted and measured load-settlement data is presented in Table 2. At the design load, measured pile head movement of most of the piles are within 5 mm as can be seen in the table. It is also evident that movement measured from the field load test is smaller than that of prediction for most of the test piles, and is more clear at the applied load ranging from 1.5 to 3 times of pre-defined design load. This finding pointed out that better capacity of bored pile constructed in Bangkok can be achieved with use of polymer-based slurry. The pattern of the load-settlement curves obtained from the filed tests suggested that majority of the applied load is resisted by the shaft friction. Therefore, it is reasonable to conclude that overall shaft frictional resistance of bored pile is improved with the use of polymer-based slurry.

![Figure 2. Load-Settlement curves of TP-1 (Dia. 0.80 x 49 m) : Prediction vs. Measurement.](image)
Figure 3. Load-Settlement curves of TP-2 (Dia. 0.80 x 50 m) : Prediction vs. Measurement.

Figure 4. Load Settlement curves of TP-3 (Dia. 1.20 x 51 m) : Prediction vs. Measurement.

Figure 5. Load-Settlement curves of TP-4 (Dia. 1.80x62 m) : Prediction vs. Measurement.
Figure 6. Load-Settlement curves of TP-5 (Dia. 1.50x54 m) : Prediction vs. Measurement.

Figure 7. Load-Settlement curves of TP-6 (Dia. 1.00x51 m) : Prediction vs. Measurement.

Figure 8. Load-Settlement curves of TP-7 (Dia. 1.00x30 m) : Prediction vs. Measurement.
Figure 9. Load-Settlement curves of TP-8 (Dia. 0.80x41 m) : Prediction vs. Measurement.

Figure 10. Load-Settlement Curves of TP-9 (Dia. 0.80x43 m) : Prediction vs. Measurement

Figure 11. Load-Settlement curves of TP-10 (Dia. 0.60x26 m) : Prediction vs. Measurement
Figure 12. Load-Settlement curves of TP-11 (Dia. 0.60x24 m): Prediction vs. Measurement

Table 2. Summary of predicted and measured gross settlement

<table>
<thead>
<tr>
<th>Pile ID</th>
<th>Pile Dimension Dia. x Depth (m)</th>
<th>Design Load, DL (kN)</th>
<th>Maximum Test Load (kN)</th>
<th>Gross Settlement at Applied Load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Prediction</td>
</tr>
<tr>
<td>TP-1</td>
<td>0.80 x 49</td>
<td>3300</td>
<td>9900</td>
<td>5.50</td>
</tr>
<tr>
<td>TP-2</td>
<td>0.80 x 50</td>
<td>3300</td>
<td>9900</td>
<td>6.50</td>
</tr>
<tr>
<td>TP-3</td>
<td>1.20 x 51</td>
<td>7200</td>
<td>18000</td>
<td>6.62</td>
</tr>
<tr>
<td>TP-4</td>
<td>1.80 x 62</td>
<td>11000</td>
<td>33000</td>
<td>5.62</td>
</tr>
<tr>
<td>TP-5</td>
<td>1.50 x 54</td>
<td>12000</td>
<td>32000</td>
<td>9.31</td>
</tr>
<tr>
<td>TP-6</td>
<td>1.00 x 51</td>
<td>4500</td>
<td>11250</td>
<td>5.50</td>
</tr>
<tr>
<td>TP-7</td>
<td>1.00 x 30</td>
<td>2600</td>
<td>6500</td>
<td>3.50</td>
</tr>
<tr>
<td>TP-8</td>
<td>0.80 x 41</td>
<td>3300</td>
<td>9900</td>
<td>5.72</td>
</tr>
<tr>
<td>TP-9</td>
<td>0.80 x 43</td>
<td>3300</td>
<td>6600</td>
<td>6.20</td>
</tr>
<tr>
<td>TP-10</td>
<td>0.60 x 26</td>
<td>1000</td>
<td>2500</td>
<td>2.56</td>
</tr>
<tr>
<td>TP-11</td>
<td>0.60 x 24</td>
<td>800</td>
<td>2000</td>
<td>2.59</td>
</tr>
</tbody>
</table>

Instrumented Pile Load Test. Shaft frictional resistance of polymer-based bored piles in sand layers were of major interest to the authors as it is considered that drilling slurry has less influence on clay layers according to the previous studies on bentonite bored piles.

Three instrumented pile load tests were available at the time of this paper preparation. Vibrating wire strain gauges (VWSG) and mechanical extensometers (ME) were installed at the major soil boundaries for all 3 piles. It is to be noted that the lowest level of strain gauges and extensometer of test piles TP-1, TP-2 and TP-3 are located at 1.5, 1.0 and 1.0 m above the relevant pile tip level respectively.
Load distribution curves along the test pile shaft at various applied load are shown in Figure 13 to 15. The values of unit skin friction developed by the maximum test load at the different soil layers along the shaft of test piles in comparison with those of ultimate unit skin friction calculated using the formula shown in the earlier section are also illustrated in these figures.

Figure 13. Load distribution curves and unit skin friction of TP-1 (Dia. 0.80x49 m).

Figure 14. Load distribution curves and unit skin friction of TP-2 (Dia. 0.80x50 m).
conducted on long bored piles in multi-layered soil of Bangkok, shaft frictions of deeper head movement of different soil layers obtained from test pile TP-1. As can be seen in constructed with polymer-based slurry. Figure 16 illustrates the shaft friction versus pile the excessive pile head movement. This finding may also be valid for bored piles soil layers were not fully mobilized though the piles were declared to have failed due to Thasnanipan et al. (1998), based on the various instrumented static pile load test against pile displacement is also influenced by the pile length. According to procedure and speed, etc. (Hirayama, 1990). Moreover, shaft friction development construction methods of bored piles, time from construction to a loading test, loading development against pile displacement is also influenced by the pile length. According to Displacements required to mobilize ultimate shaft friction of piles are 0.5 - 2 % of the shaft diameter (Reese, 1978). As the total pile head displacements of TP-1, TP-2 and TP-3 are 2.8, 3.9 and 1.6 % of the shaft diameter respectively, it is likely that the shaft friction of these piles were fully mobilized at relevant maximum test load. However, the pile displacement value required to reach peak shaft resistance depends upon many conditions of the pile-soil interface and the stress strain behavior of surrounding soils: soil characteristics, stresses of the ground surrounding the pile shaft, construction methods of bored piles, time from construction to a loading test, loading procedure and speed, etc. (Hirayama, 1990). Moreover, shaft friction development against pile displacement is also influenced by the pile length. According to Thasnanipan et al. (1998), based on the various instrumented static pile load test conducted on long bored piles in multi-layered soil of Bangkok, shaft frictions of deeper soil layers were not fully mobilized though the piles were declared to have failed due to the excessive pile head movement. This finding may also be valid for bored piles constructed with polymer-based slurry. Figure 16 illustrates the shaft friction versus pile head movement of different soil layers obtained from test pile TP-1. As can be seen in the figure, at the maximum test load where pile head movement reached 2.8 % of the

Figure 15. Load distribution curves and unit skin friction of TP-3 (Dia. 1.20x51 m).

A summarized comparison between measured total shaft friction resistance at maximum test load and that of theoretically calculated values of 3 test piles is presented in Table 3.

Table 3. Calculated and measured shaft resistance

<table>
<thead>
<tr>
<th>Pile ID</th>
<th>Pile Dimension Dia. x Depth (m)</th>
<th>Design Load (kN)</th>
<th>Maximum Test Load (kN)</th>
<th>Total Shaft Resistance of Pile (kN) Calculated</th>
<th>Measured at Maximum Test Load</th>
<th>Measured / Calculated</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-1</td>
<td>0.80 x 49</td>
<td>3300</td>
<td>9900</td>
<td>6900</td>
<td>9710</td>
<td>1.41</td>
</tr>
<tr>
<td>TP-2</td>
<td>0.80 x 50</td>
<td>3300</td>
<td>9900</td>
<td>6200</td>
<td>9670</td>
<td>1.56</td>
</tr>
<tr>
<td>TP-3</td>
<td>1.20 x 51</td>
<td>7200</td>
<td>18000</td>
<td>10820</td>
<td>17060</td>
<td>1.58</td>
</tr>
</tbody>
</table>

Figure 16. Unit skin friction versus pile head movement of different soil layers (TP-1). Figure 17 shows the values of pseudo-β factors for bentonite bored piles in the first and second sand as Medium dense sand Medium dense sand Stiff to very stiff clay Soft clay Very dense sand

Displacements required to mobilize ultimate shaft friction of piles are 0.5 - 2 % of the shaft diameter (Reese, 1978). As the total pile head displacements of TP-1, TP-2 and TP-3 are 2.8, 3.9 and 1.6 % of the shaft diameter respectively, it is likely that the shaft friction of these piles were fully mobilized at relevant maximum test load. However, the pile displacement value required to reach peak shaft resistance depends upon many conditions of the pile-soil interface and the stress strain behavior of surrounding soils: soil characteristics, stresses of the ground surrounding the pile shaft, construction methods of bored piles, time from construction to a loading test, loading procedure and speed, etc. (Hirayama, 1990). Moreover, shaft friction development against pile displacement is also influenced by the pile length. According to Thasnanipan et al. (1998), based on the various instrumented static pile load test conducted on long bored piles in multi-layered soil of Bangkok, shaft frictions of deeper soil layers were not fully mobilized though the piles were declared to have failed due to the excessive pile head movement. This finding may also be valid for bored piles constructed with polymer-based slurry. Figure 16 illustrates the shaft friction versus pile head movement of different soil layers obtained from test pile TP-1. As can be seen in the figure, at the maximum test load where pile head movement reached 2.8 % of the

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<th>Total Shaft Resistance of Pile (kN) Calculated</th>
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<tbody>
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<td>3300</td>
<td>9900</td>
<td>6900</td>
<td>9710</td>
<td>1.41</td>
</tr>
<tr>
<td>TP-2</td>
<td>0.80 x 50</td>
<td>3300</td>
<td>9900</td>
<td>6200</td>
<td>9670</td>
<td>1.56</td>
</tr>
<tr>
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<td>7200</td>
<td>18000</td>
<td>10820</td>
<td>17060</td>
<td>1.58</td>
</tr>
</tbody>
</table>

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<tr>
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<th>Maximum Test Load (kN)</th>
<th>Total Shaft Resistance of Pile (kN) Calculated</th>
<th>Measured at Maximum Test Load</th>
<th>Measured / Calculated</th>
</tr>
</thead>
<tbody>
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<td>3300</td>
<td>9900</td>
<td>6900</td>
<td>9710</td>
<td>1.41</td>
</tr>
<tr>
<td>TP-2</td>
<td>0.80 x 50</td>
<td>3300</td>
<td>9900</td>
<td>6200</td>
<td>9670</td>
<td>1.56</td>
</tr>
<tr>
<td>TP-3</td>
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<td>7200</td>
<td>18000</td>
<td>10820</td>
<td>17060</td>
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<td>3300</td>
<td>9900</td>
<td>6900</td>
<td>9710</td>
<td>1.41</td>
</tr>
<tr>
<td>TP-2</td>
<td>0.80 x 50</td>
<td>3300</td>
<td>9900</td>
<td>6200</td>
<td>9670</td>
<td>1.56</td>
</tr>
<tr>
<td>TP-3</td>
<td>1.20 x 51</td>
<td>7200</td>
<td>18000</td>
<td>10820</td>
<td>17060</td>
<td>1.58</td>
</tr>
</tbody>
</table>
Shaft friction factors \( \beta \) of the first and second sand layer of TP-1, TP-2 and TP-3 are back calculated from the load transfer values of relevant layers obtained from the VWSG data. However, the back-calculated \( \beta \) factors (named as pseudo-\( \beta \) for purpose of convenient reference in this paper) particularly for the second sand layer are the values at applied maximum test loads and not at ultimate test loads. In other words, it is likely that the back-calculated \( \beta \) factors for the second sand layer are the values at partially mobilized shaft friction of this layer. The back-calculation of \( \beta \) value is mainly aimed to get an idea on the magnitude of \( \beta \) factors for polymer-based bored piles from the currently available data in comparison with the design values commonly used for bentonite bored piles constructed in Bangkok.

Figure 17 shows the values of pseudo-\( \beta \) of polymer-based slurry piles obtained from 3 instrumented pile load tests plotted on the graph of \( \beta \) values derived from the full-scale static pile load tests of bentonite bored piles in Bangkok subsoil from the previous research works (Submaneewong, 1999). Submaneewong (1999) recommended the design line of \( \beta \) factors for bentonite bored piles in the first and second sand as indicated in Figure 17. It is evident that pseudo-\( \beta \) for the first sand layer of all 3 test piles constructed under polymer-based slurry are significantly higher than those of design values. The pseudo-\( \beta \) values of the second sand layer are also slightly higher than those of design line. It is most likely that \( \beta \) values of second sand layers would be higher than pseudo-\( \beta \) values indicated in Figure 17 if the piles were tested to higher loads or to failure, since shaft friction at this layer might not have been fully mobilized at applied maximum test load as described in above paragraphs. These findings suggested that polymer-based slurry bored piles produced better shaft friction resistance in sand layers than that of bentonite bored piles. Further research is needed to determine the \( \beta \) values of the first and second sand layers at fully mobilized shaft friction resistance.

![Figure 16. Unit skin friction versus pile head movement of different soil layers (TP-1).](image-url)
Figure 17. Back-calculated $\beta$ values (pseudo-$\beta$ factor) of polymer bored piles at maximum test load plotted on design line of bentonite bored piles constructed in Bangkok subsoil.

Conclusions

(1) Performance of bored piles constructed with polymer-based slurry in Bangkok subsoil have been presented using the results of eleven static pile load tests.
(2) The measurement of borehole profile by sonic caliper and construction records proved that there is no adverse effect of using polymer-based slurry in Bangkok subsoil as far as the stability of borehole is concerned.
(3) Pile head settlements at applied loads obtained from the field load test results of all test piles constructed with polymer-based slurry are smaller than those of prediction using bentonite piles parameters.
(4) Overall shaft resistance of bored piles constructed with polymer-based slurry is higher than that of theoretically calculated values in the order of 1.5.
(5) The back-calculated $\beta$ factors of current polymer-based slurry test bored piles are higher than those of design values commonly used in Bangkok for bentonite bored piles.
(6) Above findings suggested that higher shaft friction resistance can be obtained if pile is constructed with polymer-based slurry.
(7) Further research is needed to establish the design parameters for bored piles constructed with polymer-based slurry in Bangkok subsoil.

Acknowledgement

The authors wish to express their appreciation to the project directors and engineers of SEAFCO Co., Ltd. for providing the bored pile construction records. Thanks are also
extended to EDE Co., Ltd. for providing the load test data. The authors also wish to thank Miss. Siriphen Chaiyapak (former office engineer, SEAFCO Co., Ltd.) and Mr. Artorn Kantawateera (trainee student, Khonkaen University) for their assistance in preparing this paper.

References


Abstract
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Introduction
In the area of foundation engineering, Bangkok, the capital of Thailand, is well known for its weak soils. As such, most of the structures are supported by piled foundations. Construction of heavy structures such as skyscrapers, elevated expressways, overpass bridges, underground car parks and subway stations commonly require the high capacity foundations. Bored cast-in-place piles are mainly used to fulfill this requirement. However, in some projects bored piles are not feasible due to the site constraints, applicable construction method and/or extensive bearing capacity requirements. In such cases, the use of barrette foundations would make a suitable alternative. Barrettes with dimension ranging from 0.80 m x 2.7 m to 1.5 m x 3.0 m for safe working load capacity from 11,000 to 23,000 kN have been used in some major projects. To assess the performance of large-capacity foundation elements in layered subsoil of Bangkok, instrumented load testing is compulsory. This paper presents the static load test results of an instrumented barrette tested up to 52,900 kN for the foundation of a fifty-storey building in Bangkok. The test results, particularly load transfer characteristics, and shaft friction capacity were compared with those of bored pile diameter 1.50 m with the same length located 30 m away.
Barrette of Over 50,000 kN Ultimate Capacity  
Constructed in the Multi-Layered Soil of Bangkok

Narong Thasnanipan*, Zaw Zaw Aye**, and Wanchai Teparaksa, Ph. D.***

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Abstract

A static load test carried out on barrette having cross sectional size of 1.5 m x 3.0 m seated at 57 m below ground level is presented in this paper. Over 50,000 kN load was applied to evaluate the performance of this fully instrumented barrette. The load applied on barrette with a reaction frame “tower” capacity of 60,000 kN was claimed to be one of the highest test loads in the region. The test results were compared with those from the instrumented load test on bored pile of diameter 1.5 m with the same length, being located 30 m away. Since bored pile and barrette were constructed by different method and the parameters such as construction time, pile shape, slurry properties etc, were considerably different, results from the load tests have provided a unique opportunity to assess the extent of difference in behavior.

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Project Summary

The project is located in the prime area of Bangkok on Rama IX road. The design of fifty-storey office complex called for the high-capacity piled foundation: 560 bored piles of 1.2 m and 1.5 m diameter and 24 barrettes having cross section size of 1.5 m x 3.0 m. Bored piles and barrettes were seated at an approximate depth of 57 m in the sand layer. Barrettes were designed to support the large load for tower’s elevator shafts as bored piles were not feasible in this case. Base grouting was applied for barrettes and bored piles at the locations of high column load mainly in the central tower area. Instrumented static pile load tests were carried out for one barrette and one 1.5 m diameter bored pile. The design safe working loads for the base grouted barrettes and bored piles are 23,000 kN and 11,750 kN respectively. The barrette load test was conducted on July 8, 1998 with maximum applied load of 52,900 kN.

Subsoil Profile and Ground Water Condition

Similar to other localities in Bangkok a typical subsoil profile at the site is characterized by the alternating layers of clay and sand deposits as shown in Figure 1. Undrained shear strength (Su) from unconfined compression tests and standard penetration tests (SPT) are indicated with depth in Figure 1. It is to be noted that unconfined compression tests were conducted only for soft clay layer whereas SPT tests were carried out for all the layers below soft clay. Soil investigation from five boreholes at different location reveals that subsoil layers along the site are relatively consistent. The soil layers present at site are briefly described below.

Figure 1. Soil profile at site and the typical ground water condition of Bangkok
Soft, highly compressible dark gray marine clay lies beneath a two meter thick weathered crust and extends up to 13.5 m. A stiff clay layer occurs directly underneath soft clay and its depth goes up to 26 m. Below the stiff clay layer, 10 m thick medium to dense sand layer also known as the first sand layer can be found. Hard clay underlies the first sand and it is found to be about 16 m thick. Dense to very dense sand (the second sand layer) occurs at depths between 51 to 72 m. Within the second sand layer, a thin layer of hard clay is found.

Figure 1 depicts the present typical piezometric profile of Bangkok. As can be seen in the figure, existing pore water pressure conditions in upper part of Bangkok soft clay are hydrostatic from nearly 1m below the ground level and then the hydrostatic condition changed to piezometric drawdown below the bottom level of Bangkok soft clay. Excessive ground water abstraction from the deep aquifers in the past is responsible for this significant drawdown of piezometric pressure (Natalaya, 1981, Thasnanipan et al., 1998).

**Installation Method and Process**

Both the barrette and bored pile were constructed by wet process under bentonite slurry. Bentonite slurry conformed to the locally accepted specification (AASHTO 1992) was used. The actual properties of the bentonite slurry used are given in Table 1 along with those specified by the project. Single stage base grouting was applied 24 hours after concreting for both barrette and bored pile. The detailed procedures utilized in the construction of two different foundation structures are outlined below.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Project Specified Values</th>
<th>Measured Actual Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Barrette Before feeding to the borehole</td>
</tr>
<tr>
<td>Density (g/cc)</td>
<td>1.03-1.20</td>
<td>1.10</td>
</tr>
<tr>
<td>Marsh Funnel Viscosity (sec/qt)</td>
<td>28-45</td>
<td>36</td>
</tr>
<tr>
<td>API Sand Content (%)</td>
<td>&lt; 4.0</td>
<td>1.0</td>
</tr>
<tr>
<td>pH value</td>
<td>8-12</td>
<td>8</td>
</tr>
</tbody>
</table>

A mechanical cable-suspended grab was used for barrette construction. Excavation of the trench was carried out by the cyclic process of lifting and lowering of the grab under gravity and tangential force of the clamshell operated by cables. A guide wall with inside clear dimensions slightly larger than the nominal size of the barrette was used to guide the grab during initial bites. Bentonite slurry was introduced to the trench as soon as initial excavation commenced. The excavation was continued under...
the bentonite slurry to the final depth. Since time consumed in the preparation of instrumentation was relatively long, circulation of the bentonite slurry was continuously done to keep the slurry agitated, which also helped to alleviate the growth of filter cake by minimizing the actual exposure time. Prior to lowering the reinforcement cage the trench was once again occupied by grab to scrape the trench walls to remove, if any, filter cake formed on the walls. This attempt is in line with the recommendation made by Reese and O’Neill (1988). It is authors opinion that, if due to some unforeseen reasons, reinforcement cage lowering have to be delayed for considerable period of time, it is a good practice to use the grab again to scrape the trench walls. This measure eliminates any foreseeable negative impacts caused by unexpected delays. After lowering the rebar cage, tremie concreting was done.

Hydraulic rotary drilling was employed for bored pile excavation. Different from barrette excavation, temporary casing of 15 m length was used as a support in Soft Clay layer for bored pile drilling to assure the stability of the borehole. Firstly, an auger was used to drill within the temporary casing and to the bottom of the stiff clay layer. Bentonite slurry was then supplied to the borehole and drilling was continued with a bucket down to final depth of excavation. Before lowering the reinforcement cage a special cleaning bucket was used to scrape the borehole walls and to clean the base. Reinforcement cages were then lowered inside the borehole and concreting was carried out by tremie method. Time consumed in different construction activities of test barrette and bored pile is shown in Table 2. Equipment used in the excavation of barrette and bored pile are shown in Figure 3.

Table 2. Time consumed in different construction activities of test barrette and bored pile

<table>
<thead>
<tr>
<th>Activities</th>
<th>Time Consumed (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Barrette</td>
</tr>
<tr>
<td>Drilling</td>
<td>19</td>
</tr>
<tr>
<td>Recycling and Desanding</td>
<td>36</td>
</tr>
<tr>
<td>Reinforcement Cage Lowering</td>
<td>10</td>
</tr>
<tr>
<td>Delay</td>
<td>2.5</td>
</tr>
<tr>
<td>Concreting</td>
<td>7.5</td>
</tr>
<tr>
<td>Total Time Consumed</td>
<td>75</td>
</tr>
</tbody>
</table>

Pile Toe Grouting Process

Pile toe grouting was conducted on both test piles. A special grouting pipe system (a grouting circuit) in the form of U-loop consisting of 0.5” PE pipes connected with 0.5” steel pipe at the bottom was used. The steel pipe horizontally placed at the bottom of the U-loop is perforated and wrapped by a rubber sleeve "Manchette". Three grouting circuits (3 loops) were used for barrette whereas two grouting circuits (2 loops) were employed for bored pile. The grouting circuits were attached to the reinforcement cage.

Twenty-four hours after concreting, the pile toe was cracked by pressurized water to flush and open the grouting circuit so as to make a way for forth coming grout. Cement grouting was commenced soon after completion of the water cracking. The preset grouting criteria was to stop the grout injection when the grout volume reached to
the maximum target volume (1500 and 750 liter for barrette and bored pile respectively) or the maximum target grouting pressure (40 kN/m$^2$) was achieved, whichever came first. The grouting for test barrette and bored pile was stopped by the volume criteria since the maximum grouting pressure achieved for both piles was only 20 kN/m$^2$.

The pile toe grouting was conducted on both test piles. A special grouting pipe system (a grouting circuit) in the form of U-loop consisting of 0.5" PE pipes connected with 0.5" steel pipe at the bottom was used. The steel pipe horizontally placed at the bottom of the U-loop is perforated and wrapped by a rubber sleeve "Manchette". Three grouting circuits (3 loops) were used for barrette whereas two grouting circuits (2 loops) were employed for bored pile. The grouting circuits were attached to the reinforcement cage. Twenty-four hours after concreting, the pile toe was cracked by pressurized water to flush and open the grouting circuit so as to make a way for forthcoming grout. Cement grouting was commenced soon after completion of the water cracking. The preset grouting criteria was to stop the grout injection when the grout volume reached to

![Figure 2. General configuration of test barrette and bored pile](image)

![Figure 3. Excavation and drilling tools of barrette and bored pile](image)
Major Differences of Two Test Piles

Basic installation parameters of the test barrette and bored pile are considerably different as shown in Table 3. The general configuration of test barrette and bored pile with soil profile at site is illustrated in Figure 2. The influence of the major differences indicated in Table 3 on capacity of piles are discussed in later sections of this paper.

Table 3. Major differences in installation process of barrette and bored pile

<table>
<thead>
<tr>
<th>Test Pile Type</th>
<th>Equipment and Drilling Process</th>
<th>Major Differences</th>
<th>Shape</th>
<th>Properties of Bentonite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barrette</td>
<td>Mechanical grab with excavation under gravity and grab action</td>
<td>Construction Method / Use of Temporary Casing</td>
<td>None</td>
<td>Rectangular</td>
</tr>
<tr>
<td>Bored Pile</td>
<td>Hydraulic rotary piling rig with rotary drilling action</td>
<td>Total Construction Time</td>
<td>75 hours</td>
<td>Different as indicated in Table 1</td>
</tr>
<tr>
<td></td>
<td>15 m Steel Casing</td>
<td></td>
<td>Circular</td>
<td>Different as indicated in Table 1</td>
</tr>
</tbody>
</table>

Load Test Program

Pile load tests were carried out in accordance with ASTM D 1143-81 Item 5.1 “Standard Loading Procedure”. The pile and barrette were tested under three and four cycles of loading and unloading respectively.

Test Frame Configuration. Test barrette and bored pile layout is presented in Figure 4. General view of the barrette and bored pile test frame set up is presented in Figure 5. With overall height of 8 m, reaction frame utilized for static load testing on barrette in this project was claimed to be one of the biggest of its kind in the region. Four barrettes were used as anchoring system. Four numbers of built-up steel girders supported on each side by two 1st level cross beams were used as main beams to achieve the maximum capacity of 60,000 kN. First level beams were supported against the second level cross beams. Second level cross beams were anchored against surrounding barrettes using anchor blocks at the top. Specially fabricated transfer-girders were used to distribute the tension force transferred from the tie-bars to dowel bars above the anchor barrette heads. Sixteen hydraulic jacks each having 5,000 kN capacity were placed between the test barrette cap and the main beams of the reaction frame.

Test pile layout of bored pile was similar to those of other static bored pile load tests in Thailand as it was considered that preset maximum test load would not require special test frame set up. Steel test frame anchored against four bored piles was used in bored pile load test. Different from the test frame configuration of the barrette, only 1 layer of cross beams was required for that of bored pile.
Instrumentation and Monitoring System. Vibrating wire strain gauges (VWSGs) and Mechanical Extensometers (ME) were fixed at five levels along the shafts at the known interface boundaries of different soil layers. At each level, four sets of VWSGs and one set of ME were installed for the pile whereas six sets of VWSGs and two sets of ME were installed for the barrette.

Direct measurement from four dial gauges placed in diametrically opposite positions having equidistance from the test pile axis was used as a main monitoring system of test pile head movement. Precise leveling and piano wire were also utilized as backup for pile head movement measurement. Additional two dial gauges were also
Performance of Toe-Grouted Large Diameter Bored Pile in Sand-Gravel Formation of Chiang Mai City, Northern Thailand

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Sponsored by
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Performance of diameter 1.5 m toe-grouted bored pile founded at depth 44 m in sand-gravel formation of Chiang Mai City is presented in this paper. Toe grouting was applied by using tube-a-manchette method. Effectiveness of toe grouting was observed from the standard penetration tests conducted immediately below pile toe through grouting pipes at two stages, prior to grouting and three weeks after completion of grouting. Improvement of pile end resistance by toe-grouting is demonstrated with a series of back analyses carried out by load-transfer method employing hyperbolic transfer function. Construction experience of large diameter deep-seated bored piles in multi-layered subsoil and practical aspects of toe-grouting are also briefly discussed.

INTRODUCTION

Chiang Mai founded 800 years ago is now known to be the tourist city of the northern Thailand. It is located about 700 km north of Bangkok at latitude 18.75 N and longitude 99 E. Bored piles have been used as foundations of heavy structures in Chiang Mai. Most of the bored piles constructed in Chiang Mai are of small to medium diameter short bored piles: diameter ranging from 0.60 to 0.80 m with toe depths of about 20 m. Large diameter deep-seated bored piles were rarely constructed in the city as high-capacity foundations were not in great demand in the past. Hence, behavior and performance of large diameter deep-seated bored piles in this city have not yet been studied. In some projects, design of bridge pier called for a single pile per column concept to facilitate the available space. Large diameter deep-seated bored piles are necessary in such cases. Construction experience of diameter 1.5 m toe-grouted bored piles founded at depth 44 m in sand–gravel formation of Chiang Mai City is presented in this paper. A total of 33 bored piles were constructed for bridge foundation of elevated interchange. The instrumented static pile load test was carried out up to 25,000 KN on one of the working piles as a proof-loading test, which was claimed to be the highest static pile load test ever tested in the city of Chiang Mai.

SITE GEOLOGY AND SUBSOIL PROFILE

The project is located in the inner part of the lowland floodplain area. The site subsurface geology is characterized by thick alluvial deposits composed of alternating series of sand and clay layers overlaying gravel beds. A relatively thin layer of gravel formation is observed between sand and clay layer at depth about 14-17 m.

Figure 2 illustrates the generalized soil profile along the site derived from 5 boring logs with indication of SPT-N values obtained from each borehole. As can be seen in the figure a large variation in soil profile along the site is observed. Above 35 m depth, the thickness and occurrence of strata radically varies from one borehole to another. However, below this depth the top and bottom boundary elevation of stratum 11 (sand and gravel mixture formation) is relatively uniform across the site. Pile toe was designed to be founded in this layer. The simplified description of soil stratum along with SPT-N values and undrained shear strength, S_u (for clay layers) are presented in Table 1. Ground water level observed from the soil investigation boreholes was about 1 m below the existing ground.
Figure 1  Geological map of Chiang Mai and adjacent areas (modified from Intrasuta, 1983)

Figure 2  Generalized soil profile along the project (the numbers denoted behind the borehole are SPT-N values)
Table 1 Summarized description of soil stratum

<table>
<thead>
<tr>
<th>Stratum No.</th>
<th>Description</th>
<th>Unit Weight (KN/m³)</th>
<th>SPT &quot;N&quot;</th>
<th>S₀₉₀ (KN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Clayey sand (brownish grey)</td>
<td>18.0</td>
<td>4 to 16</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>Medium to very stiff silty clay</td>
<td>19.0</td>
<td>8 to 18</td>
<td>90-120</td>
</tr>
<tr>
<td>3</td>
<td>Loose sand</td>
<td>18.5</td>
<td>3 to 5</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>Soft to medium stiff silty clay</td>
<td>18.0</td>
<td>3 to 7</td>
<td>35-60</td>
</tr>
<tr>
<td>5</td>
<td>Medium to very dense sand</td>
<td>19.5</td>
<td>15 to 46</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>Gravel</td>
<td>21.0</td>
<td>21 to 68</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>Very stiff clay</td>
<td>20.0</td>
<td>21 to 38</td>
<td>150-200</td>
</tr>
<tr>
<td>8</td>
<td>Medium to dense clayey sand</td>
<td>21.0</td>
<td>26 to 40</td>
<td>-</td>
</tr>
<tr>
<td>9</td>
<td>Very stiff to hard silty clay</td>
<td>22.0</td>
<td>22 to 69</td>
<td>&gt; 200</td>
</tr>
<tr>
<td>10</td>
<td>Clayey sand</td>
<td>22.0</td>
<td>26 to 49</td>
<td>-</td>
</tr>
<tr>
<td>11</td>
<td>Very dense sand-gravel formation</td>
<td>22.5</td>
<td>40 to 90</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>Very dense gravel</td>
<td>-</td>
<td>50 / 0&quot;</td>
<td>-</td>
</tr>
</tbody>
</table>

Bored Pile Construction Method

All the contract bored piles including the test pile were constructed by rotary drilling under wet process or slurry-replacement method, using bentonite slurry as support fluid in boring process. Depending on the presence of loose material, temporary steel casings of length ranging from 3 to 11 m were used to stabilize the borehole in top soil layer. After installing the temporary casing by hydraulic vibrator (Vibro-hammer) soil inside the casing was drilled out by an auger. The borehole was filled with bentonite slurry prior to advancing the drilling below the casing. Drilling bucket was then used to excavate the soil under the slurry. Upon reaching to 15-19 m depth, verticality of the borehole was checked by Koden drilling monitor (sonic caliper method). Koden test at this depth also helped to assess the stability of borehole in gravel layer (stratum 6). After the careful examination of the borehole, verticality monitoring results, drilling was continued to the final depth. Additional Koden tests were performed to ensure the verticality of the borehole at regular interval before reaching to the final depth if excessive inclination was found in the first test.

Since the sand content in the slurry after drilling to the final depth was higher than that of specified (API 4%), recycling and desanding were carried out to screen out the excessive sand in the bentonite and to maintain the properties of slurry. Up to 1m³ of sand in the slurry was removed by the recycling process. This represented about 0.5 m height of sediment at the base of pile. After completion of recycling, a final Koden test was conducted. Figure 3 shows the borehole profile obtained from the final Koden test conducted on test pile. Pile base was then cleaned by cleaning bucket to remove the accumulated loose material deposited on the base of the pile during drilling process. After the cleaning, the depth of the pile was measured and recorded. Reinforcement steel cages were then installed section by section in the borehole. Four sonic logging access tubes (internal diameter 54 mm black steel pipes) complete with tube-a-manchette were fixed in the reinforcement cage as all bored piles were required to have sonic logging test. The sonic logging tubes were also utilized as toe-grouting tubes. The arrangement and configuration of grouting circuits are presented in the later section. Once the reinforcement cage installation was done, tremie pipes complete with air lifting accessories were lowered and air lifting operation commenced. Air lift was continuously perform until the arrival of the first concrete truck to minimize the accumulation of sediments at the pile base if not entirely eliminated. The depth of the pile was re-measured to ensure that the base of the pile was reasonably cleaned. Hundred percent cleaning or removal of undesired sediments was most unlikely particularly for the long bored piles in the presence of thick sand formation and layers of gravel. Concreting was done by tremie method. All the contract piles including the test and anchor piles were constructed by the procedure described above. The properties of bentonite slurry measured from the test pile at different stages are shown in Table 2.

Figure 3  Borehole profile from Koden Test (soil profile shown in the figure is based on the record of a visual inspection of soil from pile drilling)

<table>
<thead>
<tr>
<th>Stage</th>
<th>Density (g/ml)</th>
<th>Marsh Funnel Viscosity (sec/qt)</th>
<th>API Sand Content (%)</th>
<th>pH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before drilling</td>
<td>1.05</td>
<td>31</td>
<td>0</td>
<td>8</td>
</tr>
<tr>
<td>After recycling</td>
<td>1.07</td>
<td>33</td>
<td>0.5</td>
<td>8</td>
</tr>
<tr>
<td>After cage lowering</td>
<td>1.10</td>
<td>30</td>
<td>1</td>
<td>8</td>
</tr>
</tbody>
</table>
SONIC LOGGING TEST

The construction specification called for the integrity assessment of all the piles by sonic logging test. Sonic logging test was normally carried out at a minimum of 7 days after completion of pile concreting. Figure 4 shows the sonic logging result of the lower part of the test pile tested 7 days after pile concreting.

Figure 4  Sonic logging test profile of lower part of the test pile (depth indicates in the figure are measured from the top of sonic tubes)

PILE TOE-GROUTING

Original Specification

The original project specification called for the use of drill-and-grout method for toe-grouting. Drill-and-grout method involves coring through the pile base up to 20 cm into the soil beneath the pile base. Coring is to be carried out by drilling tools using sonic tubes as access to the base of the pile. After completion of drilling, sonic tubes are to be cleaned and filled with grout followed by grout injection from top of the tubes. This method was considered inappropriate as it is likely that uplift pressure at pile toe level blow out sand gravel mixture into the sonic tubes which in turn would cause the undesired consequences; blockage of the sonic tubes and progressive collapse of the soil below pile base.

Alternative Method (Tube-a-manchette Method)

The piling contractor therefore proposed tube-a-manchette technique as an alternative toe-grouting method. In this method a system of grouting consisted of two U-shape loops formed by two pairs of inside diameter 54 mm main vertical steel pipes (sonic tubes) attached with horizontal steel pipe of inside diameter 40 mm at bottom of each pair. The horizontal steel pipes were perforated and wrapped with rubber sleeves (manchettes). The bottom of the sonic tubes were securely closed with a steel cap. The position arrangement and layout of grouting circuit is shown in Figure 5. The grouting circuits completed with manchette fabricated together with the lowest section of sonic tubes were fixed to the reinforcement cage and installed prior to concreting as described in the earlier section. In the initial stage of piling, manchette opening and water cracking was done 24 hours after casting of piles and cement-grout injection was performed after 7 days or more upon completion of sonic logging test. However, with this process difficulty was found in cement grout filling process as grout circuit was blocked by intrusion of sand and very small gravels into the pipes through perforations since manchettes were broken out during earlier water cracking.

Figure 5  Grout circuit arrangement and layout (spiral reinforcement is not shown in the figure for clarity)

Modification of Grouting Process

Therefore it was decided to change the sequence by performing water cracking immediately prior to the actual cement grouting. To facilitate this change, the level of manchette was modified by placing just above pile tip level (in the initial stage the manchettes were placed about 10 to 20 cm above the pile tip level). After completion of the sonic logging test, manchette opening and water cracking was carried out followed by the filling of grout in the grout tubes. Each grout loop was filled with cement grout from the top by using one pipe as inlet and another as outlet. Cement grout was injected at each loop with low pressure from inlet pipe until the cement grout comes out from the outlet pipe. Once both outlet pipes were observed with discharge of cement grout, two outlet grout pipes were connected to the grout pump and pressurized grout injection started. Toe-grouting was performed through all 4 pipes simultaneously. Grouting was stopped when the injection pressure reached to 20 bar. The target grouting pressure was set at 20 bar (2,000 KN/m²) for two main reasons; (i) to avoid excessive hydrofracturing of surrounding soil as pile base was embedded in highly permeable sand-gravel formation, (ii) the main purpose of the toe-grouting in this project was to improve the soft toe condition of the pile. Compacting the remaining loose materials at the base of the pile and compressing the distressed soil immediately below the pile base with the target pressure was considered sufficient to improve the soft toe condition. Total volume of grout required per
piles was generally 350-500 liter, but up to 750 liter was consumed for a few piles. No pile uplift was observed during pressure grouting.

**ASSESSMENT OF TOE-GROUTING BY SPT**

In order to evaluate the effect of toe grouting, standard penetration tests (SPT) were conducted immediately below pile toe through sonic tubes at two stages, prior to grouting and after completion of grouting. The tests were performed on one working pile adjacent to the test pile. Toe-grouting was carried out from one loop so that the other loop (two sonic tubes) can be used for the access of SPT sampler. The first SPT was conducted 7 days after casting of the pile whereas the second test was performed 3 weeks after toe-grouting. Prior to conducting the SPT at both stages, a steel cap at the bottom of the sonic tubes were drilled through by diamond core bit coupled with core barrel. Figure 6 shows the SPT-N values obtained from two different stages in comparison with that of original obtained from nearest borehole (BH-4). As can be seen in the figure, it is clear that the density of the soil below pile toe has been increased appreciably from soft toe condition by pile-toe grouting. After completion of second SPT, an additional toe-grouting was carried out from two sonic tubes which were used for the access of SPT sampler as described earlier.

As grout injection was applied under relatively low pressure, upward migration of grout along the pile shaft was unlikely. Following are the reasons.

- Toe-grouting was carried out minimum 7 days after pile concreting. Time gap between concreting and grouting would have allowed some positive improvement of bentonite skin (if any) along the pile shaft.
- Sediments of approximately 5 cm were recorded for most of the piles prior to concreting though intensive air lifting was carried out. Possibly loose gravels were also present at pile base since removal of relatively heavy material by air lift is unlikely. Hence, it is most likely that the base of the pile was loose and weaker than not only the surrounding soil but also pile / soil interface along the lower portion of pile shaft.
- At 20 bar, a relatively low pressure, the grout would first compact, compress and permeate the weakest zone around the applied area (pile base as described above). Grout would also bounded and cemented loose sand and gravel at pile base.

Cement grout mixed with sand and some gravels were observed from the first 5 cm of the collected sample followed by pure sand-gravel mixture during standard penetration test at pile base, which favored the assumed mechanism described above. More investigations are required to establish the exact mechanism of grout at pile toe. Simplified version of the most likely geometry of the pile base at different construction stages is demonstrated in Figure 7.

**MECHANISM OF TOE-GROUTING**

Teparaksa et al. (1999) investigated the penetration of grout into the sand layer from the toe-grouting of bored piles in Bangkok. Toe-grouting was performed 24 hours after concreting with maximum pressure 40 bar for grouting the base of large diameter deep-seated bored piles in medium to fine sand layer. From the series of SPT and soil sampling performed below the pile toe, the authors concluded that grout did not permeate into the sand layer at pile base. Upward flow of grout along the pile / soil interface is the most likely phenomenon of toe-grouting in Bangkok as stated by them.

In this case study, likely mechanism of toe-grouting under maximum pressure 20 bar in sand-gravel formation are;

- Compaction of loose material at the pile base
- Bounding and cementing the loose gravel at pile base
- Compressing of distressed soil at pile base

As can be seen in the figure, it is clear that the density of the soil below pile toe has been increased appreciably from soft toe condition by pile-toe grouting. After completion of second SPT, an additional toe-grouting was carried out from two sonic tubes which were used for the access of SPT sampler as described earlier.
STATIC PILE LOAD TEST

Static pile load test (Maintain Load) was commenced 36 and 29 days after concreting and toe-grouting respectively. Additional 4 bored piles constructed without toe-grouting were used as anchor piles. Vibrating wire strain gauges (VWSGs) and Mechanical Extensometers (ME) were fixed at five levels along the shafts at the known interface boundaries of different soil layers. At each level, two sets of VWSGs and one set of ME were installed. The lowest level of instrument was set at 70 cm above the pile toe level. The pile was tested to maximum 25,000 KN, which is 2 times of the design load (12,500 KN).

Load vs. Pile Displacement

The total applied load vs. pile head displacement of the test pile is shown in Figure 8. Pile shaft and end resistance in relation to the pile head displacement are also included in the figure.

Load Distribution Mechanism

Load distribution along pile shaft at various applied loads derived from the strain gauge data is shown in Figure 9.

Load transferred to the pile toe at maximum test load was about 5500 KN (accounted about 22 percent of the total applied load) as can be seen in Figure 9. At maximum applied load, displacement at pile tip measured by extensometer was 8.29 mm (0.5 % of pile diameter).

LOAD-DISPLACEMENT ANALYSIS OF THE PILE

Load-displacement behavior of pile was analyzed using load-transfer method employing hyperbolic transfer function proposed by Hirayama (1990). A computer program written based on this method was used to analyze the load-displacement behavior of the pile. Figure 10 illustrates the hyperbolic stress-displacement curves of bored cast-in-place piles presented by Hirayama (1990). It should be noted that end resistance “qult” indicated in Figure 10 represents physical ultimate end resistance which is related to pile tip displacement in hyperbolic function. Conventional definitions of ultimate end bearing capacity for bored piles therefore do not correspond to the “qult” indicates in this paper.

One-dimensional finite element model constructed by elastic bar elements used in the method proposed by Hirayama is shown in Figure 11. The detailed discussion of the background principle and procedure to determine the required parameters in using this method can be seen in the published paper of Hirayama (1990). The major advantage of the method is that input parameters required can be determined from the results of routine in-situ tests such as SPT and CPT. The predicted load-displacement curves agreed well with those of the actual measurement according to the authors’ experience in Bangkok subsoil.
Two preliminary analyses were carried out. The input soil parameters required in the analyses were established based on the recommendation of Hirayama (1990) as shown in Table 3 with the following key assumptions:

- Same shaft resistance values (derived from SPT-N values of original site investigation) for both cases.
- Preliminary Case 1: No pile toe improvement so that normal toe condition exists – lower factor (100) is used in ultimate end resistance derivation.
- Preliminary Case 2: Toe-grouting improves pile toe so that stiffer toe condition than case 1 – higher factor (300) is used in ultimate end resistance derivation.

Table 3  Factors used in estimation of shaft and end resistance from SPT-N values for preliminary analyses

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Soil Type</th>
<th>Case 1</th>
<th>Case 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate shaft Resistance, f_{ut} (KPa)</td>
<td>Sand and gravel</td>
<td>5N (&lt; 200 KPa)</td>
<td>10N (&lt;150 KPa)</td>
</tr>
<tr>
<td>Ultimate end Resistance, q_{ut} (KPa)</td>
<td>gravel</td>
<td>100N</td>
<td>300N</td>
</tr>
<tr>
<td>Assigned pile toe condition</td>
<td>Normal toe</td>
<td>Stiff toe</td>
<td></td>
</tr>
</tbody>
</table>

Note: Normal toe condition in above table represents non-grouted or plain pile toe condition.

Back Analysis Based on Pile Load Test Results

Fixing the ultimate shaft resistance derived from strain gauges data obtained from static pile load test, a series of trail runs were made to back calculate the ultimate end resistance of the test pile. Table 4 shows the input data of shaft and end resistance used in the preliminary and back analyses.

<table>
<thead>
<tr>
<th>Input data in analyses</th>
<th>Preliminary analysis</th>
<th>Back analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Case 2</td>
<td>Case 1</td>
</tr>
<tr>
<td>$f_{ut}$ (KPa)</td>
<td>$q_{ut}$ (KPa)</td>
<td>$f_{ut}$ (KPa)</td>
</tr>
<tr>
<td>0-4 Sand</td>
<td>7</td>
<td>35</td>
</tr>
<tr>
<td>4-12 Clay</td>
<td>7</td>
<td>70</td>
</tr>
<tr>
<td>12-14 Sand</td>
<td>18</td>
<td>90</td>
</tr>
<tr>
<td>14-16 Gravel</td>
<td>20</td>
<td>100</td>
</tr>
<tr>
<td>16-18 Sand</td>
<td>30</td>
<td>150</td>
</tr>
<tr>
<td>18-22 Clay</td>
<td>24</td>
<td>150</td>
</tr>
<tr>
<td>22-24 Sand</td>
<td>60</td>
<td>200</td>
</tr>
<tr>
<td>24-30 Clay</td>
<td>50</td>
<td>150</td>
</tr>
<tr>
<td>30-32 Sand</td>
<td>44</td>
<td>150</td>
</tr>
<tr>
<td>32-36 Clay</td>
<td>40</td>
<td>150</td>
</tr>
<tr>
<td>36-42 Clay</td>
<td>40</td>
<td>150</td>
</tr>
<tr>
<td>42-44 Sand-gravel</td>
<td>85</td>
<td>200</td>
</tr>
<tr>
<td>Assumed displacement required to fully mobilize shaft resistance</td>
<td>0.2 % of pile diameter</td>
<td>0.2 % of pile diameter</td>
</tr>
<tr>
<td>Assumed displacement required to fully mobilize end resistance</td>
<td>6 % of pile diameter</td>
<td>5 % of pile diameter</td>
</tr>
<tr>
<td>Pile toe condition</td>
<td>Normal toe</td>
<td>Stiff toe</td>
</tr>
</tbody>
</table>

Note: SPT-N indicate in the table are average values of relevant soil layers.

The load-displacement curves of preliminary and back analyses along with that of actual measurement from static pile load test are depicted in Figure 12.

Assessment of Load-displacement Behavior

Reviewing the analyses results shown in Figure 12 in conjunction with Table 4, load-displacement characteristics of the test pile may be evaluated as follows. Same pattern and magnitude of pile head displacement as the actual measurement can be observed for preliminary Case 1 before reaching to 18,000 KN applied load. However, displacement increases abruptly over 18,000 KN and turns to the
plunging pattern at 23,000 KN, which suggests that the pile would experience plunging failure before reaching to the maximum test load of 25,000 KN, if the normal toe condition for non-grouted or plain bored pile assumed in Case 1 existed. Load-displacement pattern of Case 2 is found to be very similar to that of the actual measurement. The factor (300N) adopted in the derivation of end resistance shown in Table 3 is therefore considered reasonable.

With the use of maximum shaft friction values obtained from the load test results and end resistance value of 40 MPa as input parameters, the load-displacement curve of back analysis best fits the actual measurement which means that the selected value of end resistance in back analysis fairly represents that of actual magnitude. In other words, the measured load-displacement curve strongly suggests that end resistance of the pile is markedly high; detrimental soft toe condition does not exist at pile base.

To examine the influence of end resistance on the load-displacement mechanism, a series of additional back analyses were carried out using different values of end resistance with the fixed shaft resistance values (obtained from the load test results) as presented in Figure 13. It is evident from Figure 13 that load-displacement behavior is not excessively sensitive to variation for end resistance values in the range of 30 to 60 MPa. For end resistance less than 15 MPa, displacement increases significantly and load-displacement curve turns to a plunging failure pattern, which is more significant for the case of 10 MPa as can be seen in the figure. In summary, it should be pointed out that test pile would have experienced plunging type of failure if it had seated on the normal pile toe condition (non-grouted or plain pile). It is therefore considered that pile end resistance was effectively improved by the toe-grouting method employed.

From Figure 13, it is reasonable to conclude that the pile end resistance is in the range of 30 to 40 MPa. The load-displacement curves of end resistance 30 and 40 MPa suggest that the higher load up to 37,000 KN can be applied on pile without reaching to the plunging failure.

CONCLUSIONS

The practical aspects of toe-grouting in sand-gravel formation of Chiang Mai City has been presented. Direct observation from SPT conducted at pile base before and after toe-grouting clearly confirms the improvement of pile base by tube-a-manchette toe-grouting technique employed in the project. Target pressure, 20 bar proved to be sufficient for toe-grouting in sand-gravel formation.

A practical method for examining the load against pile displacement behavior is illustrated by a series of load-displacement curves with various pile end resistance values. Load-displacement analyses with the application of hyperbolic load-transfer method provided an effective tool in assessment and evaluation of pile toe condition. In addition to SPT conducted at pile base, improvement of pile end resistance by toe-grouting is demonstrated by back analyses results. Without applying the toe-grouting, test pile is likely to experience plunging failure before reaching to the maximum test load. Toe-grouting in this case could therefore be considered as extra safety measure to ensure the sufficient capacity of the pile.

The back analyzed load-displacement curves also demonstrate that the pile might be able to resist the load up to 37,000 KN without reaching to the plunging failure. Therefore, the specified design load 12,500 KN might have been underestimated for the toe-grouted bored pile in sand-gravel formation. Instead of conducting a proof-loading test on the working pile, static pile load test should have been performed on the pilot or trial pile with the higher load up to failure to determine the ultimate capacity of the pile and to optimize the pile design which might provide some cost-saving for the project.

REFERENCES


APPLICATION OF POLYMER-BASED SLURRY FOR WET-PROCESS BORED PILES CONSTRUCTION IN MULTI-LAYERED SOIL OF BANGKOK

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Project Manager¹, Project Director², Geotechnical Engineer³
SEAFCO Co., Ltd., Thailand
1. Introduction

Drilling slurry or supporting fluid plays a major role in construction of wet-processed medium to large diameter deep-seated bored piles in multi-layered soil of Bangkok. Rotary drilling technique coupled with wet-process method using Bentonite slurry as support fluid was commonly applied to construct these piles for over two decades. As bentonite slurry had proven acceptable to make qualified bored piles in the past, most of them were constructed only using this mineral based slurry. Recently, Polymer-based slurry (PBS) has been increasingly popular in Bangkok as an alternative supporting fluid in wet-process bored pile drilling. The primary reason, in the initial stage, for using this alternative slurry was to minimize the problems associated with using classic Bentonite such as site pollution and the disposal of excavated soils which are usually contaminated. This paper presents the application of polymer-based slurry for construction of wet-process bored piles in multi-layered soil of Bangkok.

2. Wet-Process Bored Pile Construction Method

Wet process method as its name implies makes the pile under wet condition by using drilling slurry as supporting fluid for borehole stability. Depending on the subsoil condition, temporary casing of 14-18m in length is also used as a support in soft clay layer. Drilling is commenced by dry process with auger applying rotary drilling action in the soft clay layer and stiff clay layer. Before reaching to the first sand layer, slurry is fed to the borehole and drilling is continued with the bucket to the final depth. A cleaning bucket or airlift is used to clean the bottom of the pile prior to reinforcement cage installation. Concreting is carried out by tremie method under the slurry. Slurry from borehole is normally pumped back to the storage tank during concreting and reused for next piles after treating to get specified properties. Depending on the level of design pile cut-off, bored piles are normally overcast about 1 to 3m above cut-off level to avoid presence of contaminated concrete within the design pile length.
3. Basic Principles of Different Drilling Slurries

It is important that practicing engineers involved in bored pile construction understand the basic principles on reaction of soil layers with different drilling slurries so that the correct procedures and methods can be applied in operation to avoid the detrimental effect of wet-process bored piles. Bentonite and polymer slurry react differently with the soil layers. Pure bentonite slurry stabilized the borehole wall by its density and formation of a membrane or filtercake particularly in permeable layers. Bentonite slurry is capable of maintaining the soil particles in suspension. Therefore, soil particles in bentonite slurry column stay in suspension for long hours and slowly settled toward the borehole bottom. Formation of filtercake reduces the slurry filtration into the permeable layers, thick first and second sand layers typically encountered in Bangkok.

Unlike pure bentonite slurry, polymer-based slurry does not suspend sand especially coarse grain particles. Hence, in polymer slurry, sand particles (mainly from cuttings during drilling) tend to fall out of suspension and settle on the base of borehole significantly sooner than that of pure bentonite as evident from field observation. In Bangkok, depending on subsoil profile and composition of polymer-based slurry, 0.1m to 1m of sediment is commonly observed on base of borehole in 2 to 8 hours. The sedimentation of sand particles is found to be stable with time.

Polymer molecules tend to wrap around clay and silt particles and produce small agglomerated structure. By its own weight, the resulting agglomerated structures tend to settle out of suspension slowly and accumulate mushy sediments on the firstly settled sand sediments at the bottom of the borehole. Some of the agglomerated particles also tend to float temporarily on the surface of polymer slurry stay in the suspension. The polymer molecules then proceed to attach themselves to larger and appear as a bulky material that some observers have termed “oatmeal” (O’Neill and Reese, 1999 [4]). In Bangkok as small percentage of bentonite is added in polymer-based slurry, similar to polymer-clay system (Majano and O’Neill, 1993 [3]), it can be assumed that a thin polymer-bentonite layer is formed in the soil-slurry interface of permeable layers. This thin membrane of polymer-based slurry is capable of reducing the high filtration in sand layers of Bangkok. However, overall volume of fluid loss for polymer-based bored piles is found to be higher than those of bentonite bored piles.

4. Practical Application of Polymer-Based Drilling Slurry In Bangkok

In Bangkok, the primary reason for switching from classic pure bentonite slurry to polymer-based slurry in most cases was to minimize the problems associated with environmental issues caused by the use of bentonite slurry. In the initial stage of using polymer-based slurry, the major concerns were the potential negative impact on stability of the borehole and the shaft friction capacity as this alternative slurry was not yet accustomed to the bored piling industry in Bangkok. However, no adverse effects from the use of polymer-based slurry have been experienced to date and numbers of projects have been successfully completed with this environmental friendly drilling slurry.

4.1 Mixed-Ratio and Procedure of Mixing

Dry anionic partially hydrolyzed polyacrylamide (PHPA) polymer powder premixed with fresh water and a small percentage of bentonite is mainly used for long bored piles in Bangkok. First, a small dosage of bentonite powder is premixed with fresh water (bentonite 5-15kg per cubic meter of water) and leave in the storage tank for minimum 12 hours. The predetermined dosage of dry polymer (typically polymer powder 0.5kg per cubic meter of premixed bentonite slurry) is then added and mixed to get so called polymer-based slurry. O’Neill and Reese (1999 [4]) cited that vigorous mixing of polymers supplied in the dry form should be avoided, since polymer chains can be broken down and the polymer slurry rendered ineffective. Polymer-based slurry is stored in the steel tanks before supplying to the bored pile drilling. The reason for adding bentonite for long bored piles is to minimize the fluid loss (high filtration of slurry into formation soils), which was observed from the trial boreholes drilled only with pure polymer slurry. Depending on the thickness and permeability of sand layers present at site and the embedded length of the pile in sand layers, addition of bentonite may not be necessary. No specific method or formula is available to determine the required best composition of polymer-bentonite ratios. Therefore, at each site, different mixed-ratios should be made on trial in the initial stage to determine the most appropriate ratio. Table 1a and 1b present the mixed composition of polymer-based slurry of bored piles constructed at 10 different locations. According to the load-settlement curves obtained from static pile load tests, observed settlements of all piles were less than those of predicted values using bentonite pile parameters.
Table 1a. Technical data of polymer-based slurry bored piles at different locations in Bangkok

<table>
<thead>
<tr>
<th>Description</th>
<th>Pile ID</th>
<th>T1</th>
<th>T2</th>
<th>T3</th>
<th>T4</th>
<th>T5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile Diameter (m)</td>
<td>0.80</td>
<td>0.80</td>
<td>1.20</td>
<td>1.80</td>
<td>1.50</td>
<td></td>
</tr>
<tr>
<td>Pile Length (m)</td>
<td>49</td>
<td>50</td>
<td>51</td>
<td>62</td>
<td>54</td>
<td></td>
</tr>
<tr>
<td>Mixed Ratio per 1m³ (kg)</td>
<td>Polymer</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>Bentonite</td>
<td>12</td>
<td>12</td>
<td>10</td>
<td>10</td>
<td>12</td>
</tr>
<tr>
<td>Total standing time prior to concreting (Hr : Mins)</td>
<td>17:20</td>
<td>12:20</td>
<td>14:00</td>
<td>23:00</td>
<td>15:30</td>
<td></td>
</tr>
<tr>
<td>Density (g/ml)</td>
<td>1.06</td>
<td>1.03</td>
<td>1.01</td>
<td>1.04</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>Viscosity (sec/qt)</td>
<td>40</td>
<td>45</td>
<td>44</td>
<td>51</td>
<td>44</td>
<td></td>
</tr>
<tr>
<td>pH</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Sand content (%)</td>
<td>0</td>
<td>0.5</td>
<td>0</td>
<td>0</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Approx. Pile embedded in sand layers (m)</td>
<td>First sand</td>
<td>12</td>
<td>8</td>
<td>7</td>
<td>20</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Second sand</td>
<td>7</td>
<td>9</td>
<td>13</td>
<td>18</td>
<td>13</td>
</tr>
</tbody>
</table>

Table 1b. Technical data of polymer-based slurry bored piles at different locations in Bangkok

<table>
<thead>
<tr>
<th>Description</th>
<th>Pile ID</th>
<th>T6</th>
<th>T7</th>
<th>T8</th>
<th>T9</th>
<th>T10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile Diameter (m)</td>
<td>1.00</td>
<td>1.00</td>
<td>0.80</td>
<td>0.60</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>Pile Length (m)</td>
<td>51</td>
<td>30</td>
<td>41</td>
<td>26</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>Mixed Ratio per 1m³ (kg)</td>
<td>Polymer</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>Bentonite</td>
<td>5</td>
<td>5</td>
<td>10</td>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>Total standing time (after drilling completion) prior to concreting (Hr : Mins)</td>
<td>7:00</td>
<td>3:45</td>
<td>6:55</td>
<td>2:30</td>
<td>0:45</td>
<td></td>
</tr>
<tr>
<td>Density (g/ml)</td>
<td>1.02</td>
<td>1.03</td>
<td>1.01</td>
<td>1.02</td>
<td>1.02</td>
<td></td>
</tr>
<tr>
<td>Viscosity* (sec/qt)</td>
<td>36</td>
<td>45</td>
<td>44</td>
<td>43</td>
<td>42</td>
<td></td>
</tr>
<tr>
<td>pH</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Sand content (%)</td>
<td>0</td>
<td>0</td>
<td>0.1</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Approx. Pile embedded in sand layers (m)</td>
<td>First sand</td>
<td>9</td>
<td>12</td>
<td>12</td>
<td>8</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Second sand</td>
<td>15</td>
<td>-</td>
<td>11</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

* Marsh Funnel Viscosity

4.2 Borehole Stability

Ata & O'Neill (1998, [2]) reported that polymer slurry when introduced with the right dosage and prior to the borehole reaching the piezometric level, maintained the stability of the boreholes for more than 18 hours without significant change of the borehole diameter at any depth. In Bangkok subsoil, as temporary casing of about 15m length is installed to protect the collapse of soft clay layer, polymer-based slurry is commonly introduced only after drilling reaches to stiff clay layer. However, the top level of slurry is required to maintain about 1m below the existing ground at all times once borehole excavation reaches to the first sand layer. The results of the sonic caliper measurement conducted on hundreds of piles proved that borehole can be well stabilized with the use of polymer-based slurry at total construction time prior to concreting even over 72 hours. Figure 1 shows the borehole profile test conducted after 24 hours leaving the hole filled with polymer-based slurry without any agitation.

4.3 Properties of Polymer-Based Slurry

It is important that properties of slurry used are within the applicable ranges to avoid the detrimental effect on the performance of bored piles. O’Neill & Reese (1999, [5]) cited that there is no perfect set of slurry specifications that can be used on every job, but specifications should be tailored to fit the requirements of a particular job at a particular location, where possible. Today, though use of polymer slurry is widely acceptable in bored piling industry, information on the desirable properties is still extremely limited. Table 2 shows the ranges of properties of polymer slurries specified by the International Association of Foundation Drilling Standards and Specification Committee (ADSC, 1999 [1]), those successfully used on several projects as reported by O’Neill and Reese (1999, [4]) from the work of Majano et al. (1994) and the properties commonly applied in Bangkok subsoil.
Table 2 Properties of polymer slurries specified by ADSC and Polymer-based slurry used in Bangkok

<table>
<thead>
<tr>
<th>Property</th>
<th>ADSC (at 20°C)</th>
<th>Majano et al., 1994</th>
<th>Common Practice in Bangkok (PBS***)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Emulsified PHPA</td>
<td>1.02 Maximum</td>
<td>0.995 to 1.01</td>
<td>0.995 to 1.01</td>
</tr>
<tr>
<td>Dry PHPA (Vinyl)</td>
<td></td>
<td></td>
<td>1.05 Maximum</td>
</tr>
<tr>
<td>Density (g/cc)</td>
<td>1.02</td>
<td>0.99 to 1.01</td>
<td>0.995 to 1.01</td>
</tr>
<tr>
<td>Viscosity (sec/qt)</td>
<td>40 – 90*</td>
<td>33 - 45</td>
<td>50 - 120</td>
</tr>
<tr>
<td>pH</td>
<td>7 – 12</td>
<td>8 – 11.7</td>
<td>8 – 11.7</td>
</tr>
<tr>
<td>API Sand Content</td>
<td>1.0 Maximum</td>
<td>0 to 1</td>
<td>0 to 1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Less than 1%</td>
</tr>
</tbody>
</table>

*or as recommended by manufacturer and approved by geotechnical engineer, ** Polymer-based slurry

4.4 Integrity of Polymer-Based Bored Piles

According to sonic integrity and sonic logging or sonic coring tests conducted on large numbers of polymer-based bored piles in Bangkok, there is no evidence that integrity of the pile is affected by the use of polymer-based slurry.

4.5 Effective Base Cleaning for Polymer-Based Bored Piles in Bangkok

O’Neill & Reese (1999 [4]) pointed out that vigorous mechanical agitation such as pumping by centrifugal pumps can shear the polymer chains severely. As soil-polymer reaction system is largely depending on the polymer chemical chain structure, excessive breaking of it would affect the performance of the slurry. Therefore, high turbulent means of base cleaning methods such as recycling by centrifugal pumps should not be performed for the polymer-based bored piles. Field experience shows that base cleaning by air lift method cannot effectively clean thick sediments at the base of the polymer-based bored pile. In Bangkok, the effective base cleaning for long bored piles may be achieved by the following procedure.

1. Carry out drilling to 1 to 2m above the final design depth under polymer-based slurry for the first bored pile
2. Leave the borehole for 2 to 8 hours (depending on the rate and stabilization trend of sedimentation observed from trail borehole at each site)
3. Move the drilling rig to commence the drilling of second bored pile
4. Drilling rig come back to the first bored pile and carry out final drilling for (remaining 1 to 2m) with the cleaning bucket
5. Immediate installation of reinforcement cage and concrete pouring

Table 3 Summarized comparison of bentonite and polymer-based slurry in application

<table>
<thead>
<tr>
<th>Description</th>
<th>Bentonite Slurry</th>
<th>Polymer-based slurry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixing method</td>
<td>Add bentonite powder in water and mixed by</td>
<td>First add bentonite powder in water and mixed similar to normal bentonite mixing</td>
</tr>
<tr>
<td></td>
<td>• Electric mixer (or)</td>
<td>• Adding polymer powder in premixed bentonite slurry and mix with electric mixer</td>
</tr>
<tr>
<td></td>
<td>• Water jet mixer (using submersible pumps in tank and mix with jetting action)</td>
<td>• Vigorous mixing of polymer powder must be avoided</td>
</tr>
<tr>
<td>Main function in borehole stability</td>
<td>Unit weight (density) and filtercake effect</td>
<td>• Unit weight (density) and binding of soil particle by polymer strand</td>
</tr>
</tbody>
</table>
|                              | • Possible forming of very thin filter cake due to addition of bentonite | • Possible forming of |}

5. Review of Shaft Friction Capacities Affected by Drilling Slurry

Properties of drilling slurries influence on the shaft capacity has been studied by many researchers. A number of researchers reported the reduction of shaft capacity caused by excessive formation of the filter cake on borehole wall of bored piles constructed
with bentonite slurry. It is mainly contributed by long exposure time without slurry agitation as cited by them.

Few researches are available on the comparison of shaft friction capacity between bored piles constructed with bentonite and polymer. Ata and O’Neill (1998 [2]) reported that developed shaft friction factors $\beta$ (for sand) and $\alpha$ (for clay) of bored piles constructed with polymer were higher than those recommended by USA-FHWA.

6. Performance of Polymer-Based Slurry Bored Piles in Bangkok

Instrumented static pile load test results of three test piles constructed by wet-process with polymer-based slurry in Bangkok are presented in this paper.

6.1 Slurry Properties and Construction Records of Test Piles

Table 1a shows the properties of polymer-based slurry and construction records of test piles (T1, T2, and T3). It is to be noted that all these test piles were constructed by the same contractor. Hence it is assumed that influence of other variables such as equipment as well as construction procedure used in drilling, slurry mixing and quality control measures applied for all piles would be minimized in evaluation of the load test results.

6.2 Instrumented Pile Load Test Results

Shaft frictional resistance of polymer-based bored piles in sand layers were of major interest to the authors as it is considered that drilling slurry has less influence on clay layers according to the previous studies on bentonite bored piles.

Load distribution curves along the test pile shafts at various applied loads of 3 test piles are illustrated in Figure 2, 3 and 4. The values of average unit skin friction developed by the maximum test load at the different soil layers along the shaft of test piles in comparison with those of ultimate unit skin friction calculated using the empirical formula are also illustrated in these figures. As can be seen in the figures, developed average unit shaft friction in sand layers are significantly higher than those of theoretically calculated values using normally used formula and parameters applied for bentonite bored piles in Bangkok. This finding suggested that polymer-based slurry bored piles produced better shaft friction resistance in sand layers than that of bentonite bored piles.
Thasnanipan et al. (2002, [5]) reported that load-settlement behavior of eleven polymer-based bored piles constructed in Bangkok subsoil are better than that of prediction using bentonite bored piles parameters.

7. Main Factors on Shaft Friction Capacity Improvement of Polymer-based Bored Piles

Shaft friction capacity improvement of bored piles constructed under polymer-based bored piles in comparison with those of bentonite bored piles is considered to be contributed by the following factors:

- No thick mudcake or filtercake is formed by polymer-based slurry
- Polymer strands tend to bind the soil particle in the formation which generates the drag forces and cohesion (Unlike plate-shaped bentonite, polymer molecules are hair-shaped strands)

8. Conclusion

(1) The practical application of polymer-based slurry in construction of wet-process bored piles in Bangkok subsoil has been presented.

(2) The measurement of borehole profile by sonic caliper and construction records proved that there is no adverse effect of using polymer-based slurry in Bangkok subsoil as far as the stability of borehole is concerned.

(3) Overall shaft resistances of bored piles constructed with polymer-based slurry are higher than that of theoretically calculated values using bentonite bored pile parameters in the order of 1.5.

(4) The developed average unit shaft frictions in sand layers of 3 instrumented bored piles constructed under polymer-based slurry are found to be higher than those of ultimate values calculated by empirical formula using bentonite piles parameters. This finding suggested that higher shaft friction resistance could be obtained if pile is constructed with polymer-based slurry.

(5) Design parameters currently used for bored piles in Bangkok were developed from bentonite bored piles. These parameters are found to be conservative for polymer-based bored piles. Further research is needed to confidently establish the design parameters for bored piles constructed with polymer-based slurry in Bangkok subsoil.

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For Further Details

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References


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Development in construction of bored pile and barrette for infrastructure projects in Thailand, A Country Report

Narong Thasnanipan, Kamol Singtogaw, Zaw Zaw Aye and Sumate Pravesvararat
DEVELOPMENT IN CONSTRUCTION OF BORED PILE AND BARRETTE FOR INFRASTRUCTURE PROJECTS IN THAILAND, A COUNTRY REPORT

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ABSTRACT

This paper presents the authors’ experiences in development of bored piles and barrettes for infrastructure projects in Thailand with particular focused on construction issues. Some literatures related to wet-processed bored piles and barrettes in Thailand published throughout the past 30 years are also summarized together with recent research works.

KEYWORDS: bored pile, barrette, deep foundations, bentonite, polymer-based slurry

INTRODUCTION

Cast-in-place deep foundations, particularly wet-processed deep-seated large diameter bored piles and barrettes have been used for foundations of various infrastructure projects in Thailand for over two decades. The versatility of the construction method and the high-load capacity which in turn offered the constructibility and cost-saving are the main factors contributed to the increasing use of deep-seated large diameter bored piles and barrettes. In the initial stage there were a number of questions on design and construction aspects of these foundation systems particularly in Bangkok subsoil. With the passage of time, the construction equipment, installation techniques and testing of deep foundation elements have been developed. Large numbers of instrumented full-scale static pile load tests were conducted throughout 1990’s which provided better understanding on behavior of these deep-seated foundations. Researches focused on the design parameters and methods were produced based on these test results. The design parameters were so well established that wet-process cast-in-place foundations became regarded as reliable foundations for not only in the property sectors but also for practicing engineers involved in the infrastructure projects. This paper presents the development of wet-processed bored piles and barrettes in Thailand. The information contained in the paper are mainly from the infrastructure developments built in Bangkok and adjacent areas.

CONSTRUCTION METHOD

Large diameter deep-seated wet-processed bored piles can be constructed by 2 methods, reverse-circulation and rotary drilling method. As construction procedure of historic reverse-circulation method has been well documented in many literatures and it is less frequently utilized nowadays in Thailand, only a rotary-drilling method is briefly presented in this article as follows.

In rotary drilling method, a temporary casing of appropriate length (12 to 18m in Bangkok depending on the thickness of soft clay) with required diameter (internal diameter not less than that of design bored pile diameter) is first installed to ensure the stability of the borehole in the top soft or loose soil layers. In some projects where the vibration is strictly
limited, standard length of casing with oscillator or short temporary casing of 5 to 8 m is pushed down in combination with pre-boring process. Drilling is commenced by auger to drill out the soil within the temporary casing and up to the top of the first water-bearing sand layer or bottom of the casing in case of using short casing method. Drilling slurry or supporting fluid is then supplied to the borehole and drilling is continued with a bucket down to the design final depth of the pile. Before lowering the reinforcement cage, a special cleaning bucket is used to clean the base of borehole. If bentonite slurry is used, recycling method by air-lift or pump is applied as base cleaning process. Reinforcement cages are then lowered into the borehole and concreting is carried out by tremie method.

A mechanical or hydraulic cable-suspended grab is commonly used for barrette construction. Excavation of the trench is carried out by the cyclic-process of lifting and lowering of the grab under gravity and tangential force of the clamshell operated by cables. Different from bored pile construction, a guide wall of depth 1 to 1.5 m with inside clear dimensions slightly larger than the nominal size of the barrette is used to guide the grab during initial bites. Bentonite slurry is introduced to the trench as soon as initial excavation commenced. The excavation is continued under the bentonite slurry to the final depth. After recycling of slurry and lowering the rebar cage, concreting is done by tremie method.

**OVERVIEW OF APPLICATION**

**Bored pile**

The first wet-process large diameter bored pile was constructed for Pinklao Bridge in Bangkok 30 years ago. Using reverse circulation method, diameter 1.50m bored pile was installed up to 45m in the second sand layer. Three major bridges were constructed by bored pile of reverse circulation method in Bangkok from early 1970 to 1980. The summarized information of these bridges are tabulated below.

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Year of Construction</th>
<th>Construction Method</th>
<th>Diameter (m)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pinklao Bridge</td>
<td>1971</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>45</td>
</tr>
<tr>
<td>Sathorn Bridge</td>
<td>1979</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>46</td>
</tr>
<tr>
<td>New Memorial Bridge</td>
<td>1982</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>49</td>
</tr>
</tbody>
</table>

The first wet-processed bored pile of rotary-drilling method down to first sand layer was constructed in Bangkok in late 1970 for high-rise building project, Royal Orchid Hotel. Since then bored piles constructed by rotary-drilling method have been extensively used for foundations of various heavy structures such as high-rise buildings, elevated expressways, overpass-bridges, underground car park buildings, waste-water treatment plants and most recently underground train stations of Bangkok first subway project. By mid 1980, bored pile became the foundation of choice for heavy structures particularly in urban area of Bangkok. The versatility of the construction method and the high-load capacity which in turn offered the constructibility and cost-saving are the main factors contributed to the increasing use of deep-seated large diameter bored piles and barrettes. Most of the early wet-process bored piles (1980’s) in Bangkok were constructed up to 50 m depth. Diameter of bored piles
commonly constructed in early days were ranging from 0.6 to 1.5 m. **Table 2** below summarizes the information of early major high-rise building projects in Bangkok.

**Table 2** First high-rise building projects constructed by large diameter wet-processed bored piles in Thailand (from 1979 to 1983)

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Construction Method</th>
<th>Diameter (m)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Royal Orchid Hotel (1979)</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.00</td>
<td>33.0</td>
</tr>
<tr>
<td>Taiping Tower (1980)</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>32.0</td>
</tr>
<tr>
<td>River City Hotel (1982)</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 – 1.00</td>
<td>27.5</td>
</tr>
<tr>
<td>Asoke Tower (1983)</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>50.0</td>
</tr>
<tr>
<td>Time Square Building (1983)</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>50.0</td>
</tr>
</tbody>
</table>

**Barrette**

In some projects where bored piles were not feasible due to the site constraints, applicable construction method and/or extensive bearing capacity requirements, the use of barrette foundations would make a suitable alternative. Barrettes with dimension ranging from 0.80 m x 2.7 m to 1.5 m x 3.0 m for safe working load capacity from 11,000 to 23,000 kN have been used in some major projects. The first barrette in Bangkok was believed to be constructed in late 1970 for the foundation of Bangkok Bank Head Office Building at Silom road with size of 0.6-0.8 x 2.5m and toe depth of 33m in late 1970. Barrettes can be constructed with flexible layout plan for both vertical and lateral loads. The layout pattern of barrettes can be arranged in a continuous row or column, radial, alternating long and short axis of barrette and a combination of two or more of such patterns. One type of equipment can be used for constructing both barrettes and diaphragm walls in particular projects thus reduces the mobilization cost for additional equipment. In addition to the large bearing capacity requirements, on site difficulties such as limited head room where piling rigs cannot be utilized, under such situations like presence of overhead high voltage power cables, existing overpasses or structures for elevated expressways and planned subway stations, also demands the barrettes. A summary of observation on barrettes of completed 26 projects completed with respect to selection criteria is presented in **Table 3**.

**Table 3** Summary of Barrette Selection

<table>
<thead>
<tr>
<th>Criterion</th>
<th>No. of Project</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Load Capacity</td>
<td>4</td>
<td>Incorporated with bored piles</td>
</tr>
<tr>
<td>Minimize Construction</td>
<td>6</td>
<td>Alternative for bored piles</td>
</tr>
<tr>
<td>Equipment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Limited Head Room for</td>
<td>3</td>
<td>Under existing structures such as bridges,</td>
</tr>
<tr>
<td>Excavation</td>
<td></td>
<td>elevated expressway and power lines</td>
</tr>
<tr>
<td>Combination with Diaphragm</td>
<td>10</td>
<td>As diaphragm wall legs</td>
</tr>
<tr>
<td>Wall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation as well as</td>
<td>3</td>
<td>Provision for the future requirement</td>
</tr>
<tr>
<td>portion of column</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Static load test up to 5290 ton conducted on barrette set the record as the highest load ever tested for a single cast-in-situ deep foundation in Thailand (Thasnanipan et. al. 2002a)

PROBLEMS OF WET-PROCESSED BORED PILES AND BARRETTES IN EARLY DAYS

Basic but extensive problems were experienced in early stages of bored pile construction in Thailand as summarized below.

- Limited availability and capacity of equipment
- Lack of skills in operation of equipment (particularly drilling rig)
- Adverse effect due to a slow rate of drilling (e.g. excessive formation of filter-cake by using bentonite slurry)
- Limited knowledge and less advance technique in control of bentonite slurry
- Limited experience in construction method and related negative impact
- Quality of concrete for tremie concreting method
- Lack of experienced engineers and foremen
- Improper construction and quality control specification and guidelines for deep-seated piles in local soil condition
- Improper design for constructibility

DEVELOPMENT IN CONSTRUCTION

Over the past 3 decades, along with the development of wet-processed piling technology world-wide, equipment, construction method, design as well as better understanding and realization of construction impact on the performance have been significantly improved in Thailand. Table 4 summarizes the areas of improvement in bored pile and barrette construction and main factors contributed to these.

Research works presenting some of these developments were published both locally and internationally. The design, construction and behavior of bored cast in-situ concrete piles in Bangkok Subsoil was presented by Thasnanipan et. al., (1998a). The construction and performance of barrettes in Bangkok Subsoil was also reported by Thasnanipan et. al., (1998b). Effect of construction time and bentonite viscosity on shaft friction of bored piles was also pointed out by Thasnanipan et al., (1998c). Teparaksa et. al., (1999) published the base grouting of wet process bored piles in Bangkok subsoil which demonstrated the method of base grouting and reported the grout spreading mechanism for long bored piles and performance of base grouted bored piles.

**Fig. 1** (a) Guide wall for cruciform barrette (b) View of cruciform barrette after installation of reinforcement
Construction of cruciform barrette (shown in Figure 1) for monopole type high-voltage power transmission line was described in the work of Thasnanipan et. al., 2000a. Above-mentioned research works reflect the development history of bored pile and barrettes in Thailand and offered useful information to the local construction industry and perhaps to the international deep foundation engineering society.

Table 4 Summary of development in bored pile and barrette construction

<table>
<thead>
<tr>
<th>Area of development / improvement</th>
<th>In the past (Early 1980’s)</th>
<th>At present</th>
<th>Main factor contributed to development</th>
</tr>
</thead>
<tbody>
<tr>
<td>Speed of construction</td>
<td>Minimum 3 days required to complete diameter 1.5m tip 50m bored pile</td>
<td>Less than 1 day to complete diameter 1.5m tip 50m bored pile</td>
<td>Better equipment, operating skills as well as improved-knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Pile size and depth</td>
<td>Bored pile Maximum diameter 2.0m and common depth 25-50m for bored pile. Barrette Limited in size and depth</td>
<td>Bored Pile Maximum Diameter 2.0m and common depth 25-60m Barrette Various sizes &amp; depth over 60m</td>
<td>Better equipment, operating skills as well as improved-knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Base grouting</td>
<td>Not available</td>
<td>Available</td>
<td>Equipment availability and advance technology</td>
</tr>
<tr>
<td>Application of polymer-based slurry (for bored pile only)</td>
<td>Not available</td>
<td>Extensively use</td>
<td>Material availability and research works</td>
</tr>
<tr>
<td>Construction impact on quality and performance</td>
<td>Not well understood</td>
<td>Improving</td>
<td>Experience from past projects and research works</td>
</tr>
<tr>
<td>Quality control in construction process</td>
<td>Not well established and systematic</td>
<td>Well established and systematic</td>
<td>Experience from past projects and research works</td>
</tr>
<tr>
<td>Quality control test method and interpretation</td>
<td>Less methods available and limited knowledge in interpretation</td>
<td>Better equipment available and better knowledge in interpretation</td>
<td>Advance equipment, experience from past projects and research works</td>
</tr>
</tbody>
</table>

Experience from past projects and extensive research works provided better understanding on construction impact on quality and performance of these wet-processed deep foundations. Marked difference between outcome quality of bored piles constructed in 1980s and 1990s can be observed by significant less defects found in the latter. It can also be observed from load test results that bored piles constructed in late 1990 have higher capacity than those of 1980s as shown in Table 5.
Table 5 Load test results of bored piles constructed in 1970 to 1980 and 1990s

<table>
<thead>
<tr>
<th>Year of construction</th>
<th>Project Name</th>
<th>Pile Dimension (Dia. &amp; Depth)</th>
<th>Design Load (ton)</th>
<th>Test Load (ton)</th>
<th>Total settlement at max. test load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1971</td>
<td>Pinklao bridge</td>
<td>1.5m x 45m</td>
<td>410</td>
<td>820</td>
<td>5</td>
</tr>
<tr>
<td>1995</td>
<td>Central Plaza Pinklao</td>
<td>1.2m x 45m</td>
<td>700</td>
<td>1400</td>
<td>27</td>
</tr>
<tr>
<td>1980</td>
<td>Taiping tower</td>
<td>1.0m x 32m</td>
<td>300</td>
<td>1125</td>
<td>118</td>
</tr>
<tr>
<td>1990</td>
<td>High-rise building at Ekamai Rd.</td>
<td>1.0m x 32m</td>
<td>390</td>
<td>975</td>
<td>23</td>
</tr>
</tbody>
</table>

Fig. 2 Progress of piling work for the Rama VIII main bridge

Bored piles have been extensively used for foundations of the majority of the elevated expressways since 1991. Thousands of base-grouted bored piles were constructed for these infrastructure projects including Second Stage Expressway (1991), Don Muang Tollway Extension (1997), Bangna-Bang Pli-Bangprakong Expressaway (1998), and Wat Nakorn-In project (2001). Over 700 bored piles of diameter ranging from 0.8m to 1.5m with depth from 35m to 54m were constructed (1999-2000) by polymer-based slurry for the foundation of Rama VIII Bridge, one of the initiatives of his majesty the King Bhumibol Adulyadej. Progress of piling works at the bank of Chao Phraya river for the main bridge of Rama VIII Bridge is depicted in Figure 2.

The availability of more reliable and powerful equipment and tools for drilling makes possible to construct the rock-socket bored piles. Figure 3 shows the drilling rig equipped with rock-auger used for construction of highway bridge across Mool River in 2001, in the north-eastern part of Thailand, where drilling was carried out through weathered-sandstone by powerful rotary drilling rig with core barrel and rock-auger.

Fig. 3 Rock-socket bored pile construction for Mool River Project - Ubon Ratchathani Province
DEVELOPMENT IN DESIGN CONCEPT AND PARAMETERS

In the initial stage of introducing bored piles in Thailand, the design concepts and parameters were mainly based on the available literatures from the research works carried out in other parts of the world such as Tomlinson (1957), Skempton (1959), Broms (1966), Bowles (1968), Meyerhof (1976) etc. The work of Chiruppapa (1968) was believed to be the first research data available for design parameters of bored pile in Bangkok soft clay. The author conducted the research based on 6 dry-processed small diameter bored piles with load cells installed at the pile toe in AIT campus in Phatumthani. Though the piles were simply constructed by dry-processed with casing method, this early-stage research study provided some important information such as load-settlement behavior, adhesion factor ($\alpha$) and bearing capacity factor (N) of bored pile in Bangkok soft clay.

With the passage of time, design method and parameters for local subsoil were improved as a result of research works carried out in 1980s. Differences between the behavior of driven piles and bored piles were well realized from these researches. (Ng, 1983) presented the load distribution characteristics of wet-processed bored piles founded in both first and second sand layers of Bangkok based on the instrumented (strain gauges) pile load test results. (Chiewcharnsilp, 1988) reported the shaft friction factor $\beta$ values of sand layers in Bangkok based on the instrumented load test results. In addition to the researches previously available (Chiruppapa, 1968; Suwanakul, 1969; Promboon, 1981; Ng, 1983; Chiewcharnsilp, 1988) Suchada (1989) determined the shaft friction factor (adhesion factor, $\alpha$) of Bangkok clay layers based on 11 bored piles of diameter ranging from 0.50 to 0.80m with the embedded length varying between 21.50m and 46m. It should be noted that the design parameters obtained from the research works of 1980s were mainly based on the estimation of shaft friction loads from plain static pile load test results with numbers of assumption since instrumented pile load test results were limited.

With the peak of construction-boom, large numbers of instrumented full-scale static load tests on bored piles were conducted throughout 1990’s which provided better understanding on behavior of these deep-seated foundations. Researches focused on the design parameters and methods were produced based on these test results. The design parameters were more accurate and well established that wet-process cast-in-place foundations became regarded as reliable foundations in construction industry of Thailand. With more confidence on soil parameters and behavior of these deep foundations, the designers designed to carry significantly higher loads for bored piles and barrettes constructed in late 1990 than those of 1980s. (Thasnanipan et. al., 1999) reported the failure mechanism of long bored piles in layered soil of Bangkok. The authors cited that for the bored piles embedded in the multi layered soils of Bangkok, estimation of ultimate shaft friction capacity needs to consider the brittle type of failure mechanism of stiff to hard clay layers and the $\alpha$ values selected need to be adjusted accordingly. Peak and residual $\alpha$ values mobilized in the stiff clay layers analyzed by the authors were plotted as shown in Figure 4 along with the suggested curves by different researchers. It can be seen from the figure that the residual $\alpha$ value of 2nd stiff clay layer (undrained shear strength values of 25 ton/m^2) at the maximum test load drops below the curve suggested by (Suchada, 1989). So the $\alpha$ values proposed in Figure 4 for the stiff to hard clay layers gives overestimate of the ultimate shaft friction under these conditions.

Introduction of polymer-based slurry for wet-process bored piles marked a major breakthrough for both construction and design engineers. Thasnanipan et al.,(2002b), reported that bored piles constructed with polymer-based slurry have higher capacity than those constructed with bentonite slurry. Figure 5 shows the shaft friction factors $\beta$ of sand layers for polymer-based bored piles in comparison with the design line of bentonite bored piles.
Fig. 4 Comparison of adhesion factor $\alpha$, suggested by different researchers with the actual mobilized in the stiff clay layers (after Thansnanipan et al. 1999). 

Fig. 5 Back-calculated $\beta$ values of polymer bored piles at maximum test load plotted on design line of bentonite bored piles constructed in Bangkok subsoil (after Thansnanipan et al. 2002b).

DEVELOPMENT IN QUALITY CONTROL TESTING

Improvement of testing equipment and powerful computer facilities are the key factors contributed to the development in quality control testing. Interpretation skills relevant to local soil condition and construction method of these tests were significantly improved in local industry. For instance, in early 1980, interpretation of the sonic integrity (seismic test) test results were needed to send to the specialists abroad which made the testing cost more expensive. Significant cost-saving were achieved in some major projects of late 1990, as more practical and precise interpretation were made to verify and establish the acceptance criteria in proving the quality of suspected piles with anomalies.
Koden drilling monitoring system

In Thailand, prior to the availability of Koden testing equipment, borehole verticality was checked by mechanical type equipment such as plumb float and adjustable globe. However, the reliability of these methods was doubtful. Koden drilling monitor system was believed to be first used in Thailand in 1980. The verticality of slurry-filled borehole can be monitored rapidly from the electronic plot of continuous profile by Koden equipment. This system is very useful for the verticality-control of long barrettes - unlike bored piles, guiding by temporary casing is not available for barrette drilling.

Sonic integrity test

Sonic integrity test, also known as seismic test is the most common method of integrity testing for both driven and bored piles in Thailand. Sonic integrity test is usually selected for both quality check (control test) and retrospective investigation. It is the cheapest in terms of cost and the simplest in terms of testing process. The main advantage of this test is that since no particular preparation is necessary during the pile construction phase it is more flexible to select which pile is to be tested. However, interpretation of sonic integrity testing needs considerable experience and knowledge in testing, subsoil condition and construction method. In many projects it is a part of the contractual requirement to conduct sonic integrity test. Minimum 10 % to maximum 100 % of production piles are commonly tested. It is also a reasonably acceptable method for applying as a retrospective investigation in determining integrity of the pile. The signal characteristics and their interpretations of sonic integrity test on piles founded in Bangkok subsoil were reported in details by Thasnasipan et al. (1998d).

Cross-hole sonic logging test

Sonic logging test is relatively expensive. It is mainly employed as a pre-planned site quality control testing. The major advantage of this method is that test can be carried out shortly after the pile construction. Hence, rectification measures can be implemented if the pile is detected with defect while the foundation contractor is on site. However, this method is generally not applicable if pile integrity is in question due to post-construction activities as access tubes are usually grouted after completion of the test. The first sonic logging test was believed to be conducted in 1982 for the wet-processed bored piles of Memorial Bridge Project where bad concrete zones were detected at depth about 20m and 1m (Ng, 1983). Use of Sonic logging test for checking pile integrity has increased in Thailand in recent years. The results from sonic logging test conducted on model piles in Bangkok helped to extend the knowledge of the signal characteristics and interpretation (Thasnanipan et. al., 2000b).

High strain dynamic load test

High strain dynamic integrity test has become a well-accepted method especially for evaluating the pile capacity in today’s foundation industry of Thailand and it is applied for both driven and bored piles. A large number of related technical papers and case histories of the test have been published and it is a part of standards and specifications such as ASTM D4945-89 (Standard Method for High-strain Dynamic Testing of Piles). Thasnanipan et. al. 2000b, reported the application of dynamic load testing in Thailand.

CONCLUDING REMARKS

Development of bored pile and barrettes in Thailand has been presented. According to the authors’ experience as a contractor, development in both construction and design aspects of these deep foundations in past decades are significant. In authors’ opinion, however, there are many works to be done to improve further with particular focus on constructibility issues, concrete technology for wet-processed bored piles and barrette, reliable but cost-effective
quality control testing and value-engineering. Starting from planning stage, site investigation, design, construction and inspection should be integrated such that designers, contractors and construction inspectors participating as a team with common goal. Appropriate and practical specification should be established jointly by these parties for local soil condition and construction method. Practical acceptance criteria should be developed to verify bored piles or barrettes with suspected anomalies. Continuing education should be promoted for both designers, inspection engineers and contractors. The Geotechnical Chapter of the Engineering Institute of Thailand under the royal patronage of his majesty the King Bhumibol Adulyadej, has started to establish the standard code of practice and guidelines for wet-processed bored piles which will serve as a yardstick for the deep foundation industry upon completion in near future.

REFERENCES

Thasnanipan N., Maung A. W. & Navaneethan T., Z. Z. Aye (2000b), Non-Destructive Integrity Testing on Piles Founded in Bangkok Subsoil, 6th International Conference on Application of Stress-Wave Theory to Piles, September 2000, Sao Paulo, Brazil
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Behaviour of Polymer-based Slurry for Deep-seated Bored Piles in Multi-layered Soil of Bangkok

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SEAFCO Co., Ltd. Bangkok, Thailand
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SEAFCO Co., Ltd. Bangkok, Thailand

ABSTRACT: This paper focuses the behaviour of polymer-based slurry both in the practical construction of deep-seated bored piles and in the laboratory. The behaviour of polymer slurry with different mixed-ratios is studied in the laboratory by means of model tests. Test models were constructed with the sand samples obtained from 25 to 40m depth, First and Second Sand layers of Bangkok. The model tests showed that significant strength improvement can be achieved in slurry-sand interface if polymer slurry (polymer 0.05% by weight) is mixed with a small percentage of bentonite (1% by weight). The model tests also proved that filtration of slurry in permeable layers can be reduced by adding a small percentage of bentonite in polymer slurry.

1 INTRODUCTION

In Bangkok, deep-seated large diameter bored piles of depths ranging from 35 to 60m and diameter of 800mm to 1800mm are commonly used for foundation of heavy structures such as high rise buildings, elevated expressways, flyovers or overpass bridges and recently for underground subway stations. Conventionally, these deep-seated bored piles in the multi-layered soil of Bangkok were constructed by rotary drilling under wet process or slurry replacement method, using pure bentonite slurry as supporting fluid in boring process. As an alternative to bentonite slurry, polymer-based slurry has been increasingly used in Bangkok recently. Polymer-based slurry with environmentally friendly properties, is found to be a good solution in minimizing the problems associated with the disposal of contaminated soil as well as the site pollution. This paper focuses the behaviour of polymer-based slurry both in the practical construction of deep-seated bored piles and in the laboratory.

2 GENERAL DESCRIPTION OF BANGKOK SUBSOIL

Subsoil profile is relatively consistent at different localities in Bangkok. A typical subsoil profile is characterized by the alternating layers of clay and sand deposits as shown in Figure 1.

Figure 1. Typical subsoil profile and piezometric profile of Bangkok (Thasnanipan et. al. 2002)

3 WET-PROCESS BORED PILE CONSTRUCTION METHOD

Wet process method as its name implies makes the pile under wet condition by using drilling slurry as supporting fluid for borehole stability. Depending on the subsoil condition, temporary casing of 14-18m in length is also used as a support in soft clay layer. Drilling is commenced by dry process with auger applying rotary drilling action in the soft clay layer and stiff clay layer. Before reaching to the first sand
layer, slurry is fed to the borehole and drilling is continued with the bucket to the final depth. A cleaning bucket or airlift is used to clean the bottom of the pile prior to reinforcement cage installation. Concreting is carried out by tremie method under the slurry. Slurry from borehole is normally pumped back to the storage tank during concreting and reused for next piles after treating to get specified standard properties. Depending on the level of design pile cut-off, bored piles are normally overcast about 1 to 3m above cut-off level to avoid presence of contaminated concrete within the design pile length.

4 BEHAVIOUR OF SLURRIES IN OVERVIEW

Bentonite and polymer slurries react differently with the soil layers. Pure bentonite slurry stabilized the borehole wall by its density and formation of a membrane or filtercake particularly in permeable layers. Bentonite slurry is capable of maintaining the soil particles in suspension. Therefore, soil particles in bentonite slurry column stay in suspension for long hours and slowly settled towards the borehole bottom. Formation of filtercake reduces the slurry filtration into the permeable layers, thick First and Second sand layers typically encountered in Bangkok as shown in figure 1.

Unlike pure bentonite slurry, polymer-based slurry does not suspend sand especially coarse grain particles. Hence, in polymer slurry, sand particles (mainly from cuttings during drilling) tend to fall out of suspension and settle on the base of borehole significantly sooner than that of pure bentonite as evident from field observation. In Bangkok, depending on subsoil profile and composition of polymer-based slurry, 0.1m to 1m of sediment is commonly observed on base of borehole in 2 to 8 hours. The sand sedimentation process under polymer-based slurry is found to be stable with time. Polymer molecules tend to wrap around clay and silt particles and produce small agglomerated structure. By its own weight, the resulting agglomerated structures tend to settle out of suspension slowly and accumulate mushy sediments on the firstly settled sand sediments at the bottom of the borehole. Some of the agglomerated particles also tend to float temporarily on the surface of polymer slurry stay in the suspension. The polymer molecules then proceed to attach themselves to larger and appear as a bulky material that some observers have termed “oatmeal” (O’Neill and Reese, 1999). In Bangkok as small percentage of bentonite is added in polymer-based slurry, similar to polymer-clay system Majano and O’Neill, (1993), it can be assumed that a thin polymer-bentonite layer is formed in the soil-slurry interface of permeable layers. This thin membrane of polymer-based slurry is capable of reducing the high filtration in sand layers of Bangkok. However, overall volume of fluid loss for polymer-based bored piles is found to be higher than that of bentonite bored piles according to the field observation of deep-seated bored pile construction in Bangkok. Figure 2 (a) and (b) demonstrates the generalized behaviour of bentonite and polymer slurry in borehole respectively.

5 BEHAVIOUR OF SLURRIES IN LABORATORY

Laboratory study was conducted to investigate the followings properties of Bangkok sand layers in different types of slurries (Boonyarak, 2002).

- Filtration through the sand layers
- Shear strength of slurry-sand interface (by pocket penetrometer)
- Friction test for slurry-sand interface

5.1 Sand specimen preparation

The procedure involved in preparation of sand specimen is outlined below.
Sand samples were collected from the actual bored piles drilling – excavated soil of Bangkok First and Second sand layers from depth about 25m and 40m below existing ground level respectively.

Basic properties (minimum and maximum density, grain size distribution, hydraulic conductivity) of collected sand were tested and recorded. Determined the natural density of sand.

Sand samples were then mixed with water to obtain water content of 10%.

Poured sand into the testing mold.

Compacted the sand in the mold to the desired natural density determined above

5.2 Slurry preparation and physical Tests

Slurries were mixed by propeller-type mixer following the procedure outlined below.

- Mixed bentonite powder with clean water until blended. Leave the slurry for few hours and remix.
- Mixed polymer powder with clean water until all powder dissolved.
- Mixed premixed bentonite and polymer slurry together (except for pure bentonite and polymer slurry)
- Tested basic properties of the slurry

The results of the physical tests of different slurry types from the works of Boonyarak (2002) are presented in Table 1.

<table>
<thead>
<tr>
<th>Slurry Type</th>
<th>Polymer (%)</th>
<th>Bentonite (%)</th>
<th>Slurry Density (g/cm³)</th>
<th>Marsh Funnel Viscosity (sec)</th>
<th>pH</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.05</td>
<td>1</td>
<td>1.007</td>
<td>41.8</td>
<td>9.16</td>
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<tr>
<td>2</td>
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<td>1.5</td>
<td>1.012</td>
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<tr>
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<td>9.14</td>
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<td>9.28</td>
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<td>1.019</td>
<td>120.4</td>
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<td>60.3</td>
<td>7.81</td>
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<tr>
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<td>1.008</td>
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<td>9.18</td>
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<tr>
<td>15</td>
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<td>2</td>
<td>1.020</td>
<td>160.3</td>
<td>9.32</td>
</tr>
<tr>
<td>16</td>
<td>0</td>
<td>5</td>
<td>1.050</td>
<td>70</td>
<td>9.12</td>
</tr>
</tbody>
</table>

5.3 Filtration tests

In order to study the laboratory behaviour with regard to the filtration rate of different slurries in sand formation, series of filtration tests were performed. The procedure involved in the test is outlined in the following section. The test apparatus and arrangement used were similar to that of standard API filter press as shown in Figure 3. The applied pressure was predefined for the laboratory filtration test in simulation of actual in-situ bored pile drilling condition – the differential fluid pressure between the slurry and pore pressure of formation soil at specified level, taking piezometric draw-down condition of Bangkok sand layers (as shown in Figure 1) into consideration.

- Poured the slurry into the test chamber on prepared sand specimen as described above
- Applied predefined pressure through the pressure inlet.
- The slurry was flowed through the sand medium by applied pressure depositing solids in suspension at the slurry-sand interface (filter cake formation)
- The filtrate was collected in a graduated cylinder
- The cumulative filtrate volume was recorded at regular interval until the target test time (60 minutes)
- Released air pressure and pumped out the slurry from the chamber (sample was then ready for penetrometer test)

Figure 3. Picture and schematic of filtration test

The filtration test results of Bangkok First Sand layer is presented in Figure 4. An influence of filter cake is evident in Figure 4 for pure bentonite slurry (Bentonite 5%) - the filtrate volume was very low and became constant within a relatively shorter period. For the pure polymer slurry, filtration volume was high and no constant rate of filtration was observed until the target testing time. This is apparently due to the absence of filter cake formation. In case of polymer-based slurry (polymer 0.05% with...
bentonite 1%), filtration volume was significantly reduced in comparison with pure polymer slurry as can be observed in the Figure. This finding is consistent with the field application in actual bored pile construction for deep-seated bored piles in Bangkok.

The relationship between shear strength of interface and dosage of polymer in slurry is depicted in Figure 6. Based on the regions in which the shear strength values fall, the test results can be categorized in 3 groups – interface of pure bentonite slurry, pure polymer slurry and polymer-bentonite slurry. The results indicate that interface of polymer-bentonite slurry is significantly stronger than that of pure bentonite and polymer. The pictures showing the condition of interface produced by pure polymer slurry and polymer-bentonite slurry can be seen in Figure 7 (a) and (b) respectively. Stronger surface condition of polymer-bentonite interface is evident from these pictures.

5.4 Shear strength of slurry-soil interface

Shear strength of slurry-sand interface formed by different types of slurries was tested by means of pocket penetrometer. These tests were conducted immediately after completion of filtration test described above. Picture and schematic of pocket penetrometer test are shown in Figure 5.

Figure 4. Filtration of different slurry types through Bangkok First Sand sample.

5.5 Friction tests

Though it is almost impossible to model the field condition of shaft friction resistance influence by slurry, attempt was made to study the frictional resistance of slurry-sand interface in the lab. A series of friction tests were conducted using the apparatus shown in Figure 8 following the procedure outlined below.

- After completion of pocket penetrometer test the sample was cut in such a way that less disturbed slurry-sand interface was obtained.
- Cement-mortar plate was prepared few days prior to the test
- Set up the sample in such a way that slurry-sand interface was placed (upside-down position) directly on the top of cement-mortar as depicted in Figure 9 (a).
Applied a range of normal and shear loads as shown in schematic of Figure 9.
Recorded loads and displacement at regular interval until friction load reached maximum value or displacement reached 25% strain.
The test was also conducted on pure sand sample directly placed on the cement-mortar following above procedure to make a rough comparison of frictional resistance between slurry-sand interface and pure sand against mortar plate.

Frictional resistance of slurry-sand interface of polymer-bentonite (polymer 0.1% + bentonite 1.5%) is slightly higher than that of pure bentonite (5% bentonite).
The difference of frictional resistance of slurry-sand interface between polymer-bentonite (polymer 0.1% + bentonite 1.5%) and pure bentonite (5% bentonite) was far less than expected. Likely factor contributed to this result is that the contact time between the bentonite slurry and sand sample was not long enough to develop an excessively thick filter cake.

The relationship between normal stress and maximum friction stress of different samples are plotted and presented in Figure 10. Following points can be drawn from this Figure.

- Frictional resistance of slurry-soil interface of both pure bentonite (5% bentonite) and polymer-bentonite (polymer 0.1% + bentonite 1.5%) were less than that of sand-mortar – suggested that frictional resistance was reduced by application of slurry.

6 FIELD APPLICATION AND BEHAVIOUR OF POLYMER-BASED SLURRY IN BANGKOK

In Bangkok, the primary reason for switching from classic pure bentonite slurry to polymer-based slurry in most cases was to minimize the problems associated with environmental issues caused by the use of bentonite slurry. In the initial stage of using polymer-based slurry, the major concerns were the potential negative impact on stability of the borehole and the shaft friction capacity as this alternative slurry was not yet accustomed to the bored piling industry in Bangkok. However, no adverse effects from the use of polymer-based slurry have been experienced to date and numbers of projects have been successfully completed with this environmentally friendly drilling slurry.

6.1 Properties of polymer-based slurry

It is important that properties of slurry used are within the applicable ranges to avoid the detrimental effect on the performance of bored piles. O’Neill & Reese (1999) cited that there is no perfect set of slurry specifications that can be used on every job, but specifications should be tailored to fit the re-
requirements of a particular job at a particular location, where possible. Today, though use of polymer slurry is widely acceptable in bored piling industry, information on the desirable properties is still extremely limited. Table 2 shows the ranges of properties of polymer slurries specified by the International Association of Foundation Drilling Standards and Specification Committee (ADSC, 1999), those successfully used on several projects as reported by O’Neill and Reese (1999) from the work of Majano et al. (1994) and the properties commonly applied for bored piles in Bangkok subsoil.

Table 2. Properties of polymer slurries specified by ADSC and Polymer-based slurry used in Bangkok

<table>
<thead>
<tr>
<th>Property</th>
<th>ADSC (at 20°C)</th>
<th>Majano et al., 1994</th>
<th>Common Practice in Bangkok (PBS**)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>1.02</td>
<td>0.995 to 1.01</td>
<td>Maximum</td>
</tr>
<tr>
<td>Viscosity</td>
<td>40 – 90*</td>
<td>33 - 45</td>
<td>50 - 120</td>
</tr>
<tr>
<td>pH</td>
<td>7 - 12</td>
<td>8 – 11.7</td>
<td>8 – 11</td>
</tr>
<tr>
<td>API Sand</td>
<td>1.0</td>
<td>0 to 1</td>
<td>0 to 1</td>
</tr>
</tbody>
</table>

*or as recommended by manufaturer and approved by geotechnical engineer, ** Polymer-based slurry

6.2 Mixed-ratio and procedure of mixing

Dry anionic partially hydrolyzed polyacrylamide (HPA) polymer powder premixed with fresh water and a small percentage of bentonite is mainly used for long bored piles in Bangkok. First, a small dosage of bentonite powder is premixed with fresh water (bentonite 5-15kg per cubic meter of water) and leave in the storage tank for minimum 12 hours. The predetermined dosage of dry polymer (typically polymer powder 0.5kg per cubic meter of premixed bentonite slurry) is then added and mixed to get so called polymer-based slurry.

O’Neill and Reese (1999) cited that vigorous mixing of polymers supplied in the dry form should be avoided, since polymer chains can be broken down and the polymer slurry rendered ineffective. Polymer-based slurry is stored in the steel tanks before supplying to the bored pile drilling. The reason for adding bentonite for long bored piles is to minimize the fluid loss (high filtration of slurry into formation soils), which was observed from the trial boreholes drilled only with pure polymer slurry. Depending on the thickness and permeability of sand layers present at site and the embedded length of the pile in sand layers, addition of bentonite may not be necessary. No specific method or formula is available to determine the required best composition of polymer-bentonite ratios. Therefore, at each site, different mixed-ratios should be made on trial in the initial stage to determine the most appropriate ratio. Table 3 presents the mixed composition of polymer-based slurry of bored piles constructed at 5 different locations in Bangkok. According to the load-settlement curves obtained from static pile load tests, observed settlements of all piles were less than those of predicted values using bentonite pile parameters Thasnanipan et al. (2002).

Table 3. Technical data of polymer-based slurry constructed at different localities in Bangkok

<table>
<thead>
<tr>
<th>Description</th>
<th>Pile ID</th>
<th>Pile Diameter (m)</th>
<th>Pile Length (m)</th>
<th>Mixed Ratio per 1m³ slurry (% in weight)</th>
<th>Pile Diameter (m)</th>
<th>Pile Length (m)</th>
<th>Mixed Ratio per 1m³ slurry (% in weight)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>T2</td>
<td>T3</td>
<td>T4</td>
<td>T5</td>
<td>T1</td>
<td>T2</td>
<td>T3</td>
</tr>
<tr>
<td>Density (g/ml)</td>
<td>1.06</td>
<td>1.03</td>
<td>1.01</td>
<td>1.04</td>
<td>1.04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Viscosity (sec/qt)</td>
<td>40</td>
<td>45</td>
<td>44</td>
<td>41</td>
<td>41</td>
<td></td>
<td></td>
</tr>
<tr>
<td>pH</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand content (%)</td>
<td>0.5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* polymer, **Bentonite

6.3 Effective base cleaning for polymer-based bored piles in Bangkok

Since bentonite and polymer slurry react differently with the soil layers as described in section 4 above, base cleaning method for two different slurries should be selected based on their behaviour. O’Neill & Reese (1999) pointed out that vigorous mechanical agitation such as pumping by centrifugal pumps can shear the polymer chains severely. As soil-polymer reaction system is largely depending on the polymer chemical chain structure, excessive breaking of it would affect the performance of the slurry. Therefore, high turbulent means of base cleaning methods such as recycling by centrifugal pumps should not be performed for the polymer-based bored piles. Field experience shows that base cleaning by air lift method cannot effectively clean thick
sediments at the base of the polymer-based bored pile since the sediments deposited at the base of the pile are comparatively dense for bored piles constructed with polymer-based slurry. According to the field observation on polymer-based bored piles, rate of sedimentation at pile base was generally found to be stabilized few hours after stopping the drilling operation. Figure 11 presents the thickness of the sediments at base of polymer-based bored piles recorded at regular interval. The records shown in Figure 11 were made at the same site in Bangkok for diameter 1000mm bored piles of about 55m in length. The soil profile at the site is similar to that of Figure 1, a typical subsoil profile of Bangkok. This sedimentation behaviour of polymer-based bored pile should be taken account in base cleaning method and process.

The effective base cleaning for long bored piles in Bangkok, may be achieved by the following procedure.
• Carry out drilling to 1 to 2m above the final design depth under polymer-based slurry for the first bored pile
• Leave the borehole for 2 to 8 hours (depending on the rate and stabilization trend of sedimentation observed from trail borehole at each site)
• Move the drilling rig to commence the drilling of second bored pile
• Drilling rig come back to the first bored pile and carry out final drilling for (remaining 1 to 2m) with the cleaning bucket
• Immediate installation of reinforcement cage and concrete pouring

6.4 Shaft friction resistance of polymer-based bored piles

Ata and O’Neill (1998) concluded that unit shaft resistances of test piles constructed with polymer slurry in the USA are higher than those of bentonite piles. Similar finding was reported by Thasnanipan et. al. (2002) from the results of instrumented test pile in Bangkok.

Load distribution curves along the pile shaft at various applied loads of instrumented test pile (T3 of Table 3) constructed by polymer-based slurry is illustrated in Figure 12. The values of average unit skin friction developed by the maximum test load at the different soil layers along the shaft of test piles in comparison with those of ultimate unit skin friction calculated using the empirical formula are also illustrated in these figures. As can be seen in the figure, developed average unit shaft friction in sand layers are significantly higher than those of theoretically calculated values using normally used formula and parameters applied for bentonite bored piles in Bangkok. This finding suggested that polymer-based slurry bored piles produced better shaft friction resistance in sand layers than that of bentonite bored piles.

7 DISCUSSION ON LAB TEST RESULTS AND FILED PERFORMANCE

Polymer-based slurry used in construction of bored piles in Bangkok is commonly composed of 0.05% and 1% (by weight per 1m³ slurry) of polymer and bentonite respectively. The performance of bored piles constructed with this composition was proved to be significantly better than that of bored piles constructed with pure bentonite slurry. Using pure polymer in the field, construction of deep-seated bored piles in multi-layered soil of Bangkok is found to be impractical particularly at the site where thick sand layer presence due to the excessive loss of slurry into the formation. Table 4 presents the findings of laboratory tests in comparison with field observation. Though perfect comparison of laboratory findings and field observation is not possible, Table
4 provides some useful summary on the behaviour of slurry in the laboratory and of practical application in the field.

### Table 4. Summarized comparison of parameters between lab findings and field observation

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Laboratory test</th>
<th>Field observation</th>
<th>Discussion notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Filtration</td>
<td>Polymer (0.05%) + bentonite (1%) significantly reduces filtrate volume than pure polymer</td>
<td>Polymer (0.05%) + bentonite (1%) significantly reduces filtrate volume (slurry loss)</td>
<td>Laboratory results agree well with field observation</td>
</tr>
<tr>
<td>Shear strength of slurry-sand interface</td>
<td>Slurry produced by polymer mixed with small percentage of bentonite has significant higher strength than pure polymer or bentonite</td>
<td>Slurry produced by polymer mixed with small percentage of bentonite has significant higher shaft friction capacity than that of pure bentonite</td>
<td>Indirect comparison of lab shear strength vs. actual shaft friction measured from instrumented pile load test agrees well</td>
</tr>
<tr>
<td>Frictional resistance of slurry-sand interface</td>
<td>Slurry produced by polymer mixed with small percentage of bentonite has significant higher shaft friction capacity than that of pure bentonite</td>
<td>Slurry produced by polymer mixed with small percentage of bentonite has significant higher shaft friction capacity than that of pure bentonite</td>
<td>Reduction of frictional resistance is not significant in laboratory. In lab, contact time between bentonite slurry and sand sample was not long enough to develop an excessively thick filter cake</td>
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</table>

8 CONCLUSION

Behaviour of polymer-based slurries in both laboratory and field have been presented. The laboratory findings of this research work provided backup and useful information for the actual construction of bored piles in the field. The laboratory model tests reveal that significant strength improvement can be achieved in slurry-sand interface if polymer slurry (polymer 0.05% by weight) is mixed with a small percentage of bentonite (1% by weight). This mix composition significantly reduced the filtrate volume in the laboratory test in comparison with that of pure polymer. Similar dosage of bentonite is commonly added in preparing polymer-based slurry for construction of deep-seated bored piles in Bangkok which is proved to be performing better than classic pure bentonite slurry in both construction and loading capacity aspects.

### REFERENCES


Boonyarak T., Behaviour of Polymer Slurry for Wet-process bored Piles Construction in Bangkok Subsoil, Master Thesis (in Thai), Chulalongkorn University, 2002

Balkema : Rotterdam,

Majano R. E. and O’Neill M. W. 1993. Effect of mineral and polymer slurries on perimeter load transfer in drilled shafts, A Report to ADSC, University of Houston, Dept. of Civil Engineering, UHCE.


Effectiveness of Toe-Grouting for Deep-Seated Bored Piles in Bangkok Subsoil

Narong Thasnanipan, Zaw Zaw Aye, Chanchai Submaneewong
Effectiveness of Toe-Grouting for Deep-Seated Bored Piles in Bangkok Subsoil

Narong Thasnanipan1, Zaw Zaw Aye2, Chanchai Submaneewong3

ABSTRACT:
A comprehensive study on the effectiveness of two different toe-grouting methods, known as tube-â-man chette and drill-and-grout, commonly applied for deep-seated bored piles in Bangkok is presented in this paper. Mobilized unit end bearing of bored piles constructed by two toe-grouting methods are compared. The problems encountered in practical application of different methods for large diameter deep seated bored piles are highlighted. Despite considerable extra cost, various construction problems encountered and risks involved in successful execution of grouting, drill-and-grout method does not offer particular advantage over tube-â-manchette method as far as pile capacity improvement is concerned.

INTRODUCTION
In Bangkok, due to the prevailing subsoil and groundwater conditions, all the deep-seated bored piles (depth ranging from 25 to over 60 m) are constructed by wet-processed or slurry displacement method. Although the wet-processed method is most suitable and its application has been well established in Bangkok subsoil, the method itself cannot avoid undesired effects especially loosening and upheaval of soil at pile base as well as accumulated suspension of loose materials or sediments from drilling operation. These effects naturally may create a loose or soft pile toe causing negative impact on the pile capacity or load vs. pile head movement behavior. One of the special measures to improve the soft pile toe is to apply the post-grouting at pile toe. Two different methods, known as tube-â-manchette and drill-and-grout are commonly applied in Bangkok. A comprehensive study on the effectiveness of two toe-grouting techniques applied for deep-seated bored piles in Bangkok is presented in this paper.

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Narong Thasnanipan¹, Zaw Zaw Aye², Chanchai Submaneewong³

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INTRODUCTION

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SUBSOIL PROFILE AND EXISTING PIEZOMETRIC PROFILE

Subsoil profile is relatively consistent at different localities in Bangkok. A typical subsoil profile is characterized by the alternating layers of clay and sand deposits as shown in Figure 1. Pore water pressure profile illustrating the current piezometric drawdown condition of Bangkok is also shown in Figure 1.

![Figure 1. Typical soil profile of Bangkok with piezometric drawdown condition (Thasnanipan, 2002)](image)

WET-PROCESS BORED PILE CONSTRUCTION METHOD

In Bangkok, medium to large diameter (0.6 m to 1.8 m diameter) bored piles with depth ranging from 24 m to over 60 m are constructed by wet process due to the prevailing subsoil and ground water condition. Temporary casing of 14-18 m in length is typically used as a support in soft clay layer. Drilling is commenced by dry process with auger applying rotary drilling action in the soft clay layer and stiff clay layer. Before reaching to the first sand layer, supporting fluid or slurry is fed to the borehole and drilling is continued with the bucket to the final depth. Depending on the type of slurry used, either a cleaning bucket or airlift method is commonly applied to clean the bottom of the borehole prior to reinforcement cage installation. Concreting is done by tremie method for wet-processed bored piles.

TOE GROUTING METHODS IN BANGKOK

Two grouting methods known as tube-à-manchette and drill-and-grout method used in some major projects where the authors were involved are presented in this paper. In Bangkok, both toe-grouting methods are mainly applied for the piles with the tip embedded in the sand layer though there were few cases where it was applied in clay layer.
Tube-à-manchette Toe-grouting Method (TAM Method)

This grouting method has been widely used in other parts of the world according to the available published papers. A review of past and present toe-grouting methods used throughout the world has been reported by Mullins et. al. (2000). In Bangkok, the first toe-grouting for large diameter bored piles diameter 2.0m seated at depth 35m in the first sand layer was carried out by tube-a-manchette method in 1985 for the construction of Rama IX cable stayed bridge (Morrison, 1987). Since then, tube-a-manchette method has been commonly used for toe-grouting of wet-processed bored piles in Bangkok. It is estimated that over 5,000 bored piles have been constructed using this toe-grouting method in Bangkok.

Grout pipes configuration for tube-à-manchette method. In normal practice of toe-grouting by this method in Bangkok, a system of grouting circuit consists of two U-shape loops formed by two pairs of PE pipes with manchette placed at 5 to 10 cm above pile tip. The manchette is formed by the perforated steel pipe wrapped with rubber sleeves as shown in Figure 2. The manchette allows the grouting material to flow out of the pipe from the perforations but prevents return of grouting material when the grout injection is stopped. PE tubes are fixed along the rebar cages to the top. In some projects where the specification required cross-hole sonic logging tests, same access tubes for the sonic test apparatus are used in lieu of PE pipes.

![Grouting Circuit Configuration](image)

**Figure 2. Position and layout configuration of grouting circuit in Tube-à-manchette method**
**Grout Mixture Composition and Mixing Process.** The typical cement grout consists of cement and water with the ratio of 0.5-0.6 by weight. Bentonite or other reagent can also be added but trial mix should be made to find out the characteristics of the grout. Cement and water are added at right proportion in a high turbulence mixer and mixed about 2 to 4 minutes. Cement mixture is then conveyed to the agitator. The grout must be agitated continuously or at regular interval during the period between the time of grout mixing and the time it enters the pump. Usually, first grout injection should be started within 10 to 15 minutes after mixing the first batch to minimize the stiffening of the cement grout during grouting operation. It is essential that grouting team understands the required properties such as viscosity, density, setting time of grout are met to effectively grout the toe of the pile. Grouting operation should be completed within the initial setting time of the grout. Mixing rate, storage capacity and grouting rate should be well planned to complete the grouting work within the time limit.

**Grouting procedure.** The process involved in grouting by tube-a-manchettes method is illustrated in Figure 3. First, the grout pipes are flushed with water to ensure that the grouting loops are clear from any blockage. The pile toe is then cracked with relatively high pressure water flow, 12 to 24 hours after concreting, to open the manchettes and to make way for forthcoming grout. The commencement of cement grouting depending on the preset pressure criteria. If specified maximum pressure is low (less than 20 bar / 2000 kN/m²), grouting is commenced immediately after the pile toe cracking. Grouting is normally commenced 7 to 10 days after concreting if higher maximum control pressure is specified to avoid damage to pile by high pressure grouting. Grouting is stopped when the specified volume or pressure is achieved. In current practice, pressures ranging from 20 to 60 bar (2000 to 6000 kN/m²) and grout volumes 500 to 1000 liters are normally applied.

**Figure 3. Grouting process in Tube-à-manchette method**

Drill-and-grout Method (DAG Method)

Toe-grouting by this method is in fact not frequently used in Bangkok. It is to the author’s knowledge, not commonly used in the current practice of toe-grouting of bored piles world wide. To date, less than 500 bored piles have been completed in Bangkok by this method.

Grout pipes configuration for drill-and-grout method. Depending on the size of bored piles, three to four black steel pipes used for sonic test access tubes are mostly utilized as grouting pipes in this method. Steel pipes are equally spaced and fixed around inside perimeter of reinforcement cage as shown in Figure 4. The bottom of steel pipes are securely closed with steel cap to prevent intrusion of foreign material during installation and concreting.

Grout Mixture Composition and Mixing Process. Cement to water ratio of 0.6-0.7 by weight is normally used in drill-and-grout method. Cement and water are filled at right proportion and mixed about 2 to 4 minutes in a high turbulence mixer. Bentonite is then added at 0.5 % of cement in weight and mixed for another 2 minutes and the readily mixed grout is conveyed to the storage tank equipped with agitator. The control factors such as viscosity, density and setting time are also equally important as mentioned in the tube-à-manchette method.

Grouting procedure. The procedure for drill-and-grout method presented in this paper is for diameter 1.50m bored pile with 4 sonic access tubes. The sonic logging test is performed minimum 7 days after casting of the pile. Coring is then carried out through the pile base to the depth 20 cm below the pile toe. First, drilling rod attached with core-barrel is lowered into the water-filled sonic access tubes (steel pipes) and drilled through the bottom steel cap and the concrete at pile base. Drill rod is then withdrawn to change core-barrel with the soil sampler and lower again to drill the soil beneath the pile base about 20 cm. It is important that sonic tube is completely filled with water during the drilling process to avoid excessive blow-in of soil into the tubes. After completion of toe coring and sampling, the grout pipes are
flushed in a sequence to ensure that flow path exists between the tubes. To do this, after initial cleaning, two pipes (inlet pipes) are connected to the pressure pump and low pressure up to 10 bar (10 kN/m²) is applied using clean water. The other two pipes (outlet pipes) are filled with water and are left open. As the pressure from the pump is gradually increased, water will start to flow from one or more of the other two open pipes. Once the flow is observed from the first outlet pipe, it is closed off and the water flushing continued until water flow is observed from the second pipe. The process may be repeated in like sequence from each of the grout pipes until satisfactory flow is observed from each pipe. Once the flushing of the preinstalled grout pipes are completed, water-supply hoses are disconnected from 2 inlet grout pipes and connected with the grout-supply hoses. Two outlet pipes are then opened and cement grout is filled from two inlet pipes with low pressure until the grout discharge from other 2 outlet pipes.

![Grouting process in Drill-and-grout method](image)

**Figure 5. Grouting process in Drill-and-grout method**

Initial discharged cement grout is normally contaminated with the in-situ sand so that it is required to check the purity of outcoming grout. Once clean cement grout is flowed out, 2 inlet pipes are immediately connected with the grout-supply hoses and pressurized grouting commences. Grouting is started with the grout injection rate (IR) between 25 to 30 liter per minute. Grout pressure development is closely monitored from the pressure gauges installed at the top of each pipe. If the grouting pressure does not reach to 20 bar (2000 kN/m²) after injecting the 300 to 500 liter of grout, the injection rate is reduced to 12-15 liter per minute. Otherwise grouting is continued until the control criteria is achieved. Figure 5 illustrates the working procedure of pile toe grouting by drill-and-grout method used in Bangkok subsoil for 3 major projects where the authors involved.

**Grout Spreading Mechanism**

*Tube-à-manchette method.* Teparaksa et al. (1999) investigated the penetration of grout into the sand layer from the toe-grouting of bored piles in Bangkok by tube-à-manchette method. Toe-grouting was performed 24 hours after concreting with
maximum pressure 40 bar (4000 kN/m²) for grouting the base of large diameter bored piles constructed by wet-processed method under bentonite slurry seated in sand layer. From the series of SPT and soil sampling performed below the pile toe and immediate vicinity of the shaft, the authors concluded that grout did not permeate into the sand layer at pile base. Upward migration of grout along the pile shaft is most likely in tube-à-manchette method, as grout injection was applied under relatively high pressure within 24 hours after concreting of the piles, plane of weakness was formed at pile-soil interface which is most likely to be contributed by the following conditions.

- Stress relaxation from the drilling process was not yet fully restored along the shaft as time gap between pile drilling and grouting was relatively short
- Presence of bentonite filter-cake along the shaft

**Drill-and-grout method.** Grout spreading mechanism of drill-and-grout method are discussed in relation to the steps involved in grouting operation (illustrated in Figure 5) as summarized below.

- **Step 1:** drilling 4 boreholes of diameter 48 mm up to 20 cm below pile toe (using water-filled sonic tubes as access pipes to the bottom of the pile)
- **Step 2:** water flushing the sonic tubes to remove the sand inside the tubes and to form flow-paths between the tubes
- **Step 3:** filling the sonic tubes with the grout
- **Step 4:** pressurized grouting until pressure reaches 6,000 kN/m² and maintain for 5 minutes

Step 1 creates 4 boreholes in the soil underneath the pile base. These boreholes become larger as pocket of sand between them is washed out during Step 2 and created a cavity or relatively large opening beneath the pile base prior to the commencement of Step 3. This cavity is filled by grout in Step 3 which eventually formed a mass grout bulb underneath the pile base as illustrated in Figure 5. Pressurized grouting in Step 4 thus compacted the grouted mass which in turn densified the grout-soil interface. Basically grout did not enter soil pores but remained in a compacted homogeneous mass. Upward migration of grout along the pile shaft is considered unlikely since toe-grouting was carried out minimum 14 days after pile concreting so that time gap between concreting and grouting would have allowed some positive improvement of pile shaft condition (influence of stress relaxation and bentonite filter cake).

**PERFORMANCE OF TOE-GROUTED BORED PILES**

**Typical Load Transfer Mechanism of Toe-Grouted Bored Piles in Bangkok**

Table 1 shows the measured mobilized shaft resistance of bored piles with different toe grouting method at specific load in comparison with design working load and calculated ultimate values using empirical formula. The details of the empirical formula commonly used in Bangkok subsoil has been presented by
Thasnanipan et al. (2002). As can be seen in the table under design working load, shaft friction carried the large portion of the load (over 90% of applied load). Only the small percentage of the total load was transferred to the pile tip. Therefore, even with the toe-grouting application, the end bearing component of bored piles is still under-utilized and thus remaining unmobilized end bearing capacity serves only as a reserve that forms the additional factor of safety.

Table 1. Summary of mobilized shaft resistance at specific load in comparison with design working load and calculated ultimate value

<table>
<thead>
<tr>
<th>Test Pile No.</th>
<th>Pile Dia. x L (m)</th>
<th>Grouting method</th>
<th>Design Working Load, DL (kN)</th>
<th>Total shaft resistance (KN)</th>
<th>Qfm / DL</th>
<th>Ratio of shaft resistance (Qfm2 / Qfu)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TT-1</td>
<td>1.2 x 44.6</td>
<td>TAM</td>
<td>5000</td>
<td>6900</td>
<td>4480</td>
<td>8560</td>
</tr>
<tr>
<td>TT-2</td>
<td>1.2 x 53.8</td>
<td>TAM</td>
<td>6500</td>
<td>9500</td>
<td>6120</td>
<td>10830</td>
</tr>
<tr>
<td>TT-3</td>
<td>1.2 x 46.0</td>
<td>TAM</td>
<td>5000</td>
<td>7900</td>
<td>4730</td>
<td>9250</td>
</tr>
<tr>
<td>TD-1</td>
<td>1.50 x 50.0</td>
<td>DAG</td>
<td>13000</td>
<td>14600</td>
<td>12200</td>
<td>22160</td>
</tr>
<tr>
<td>TD-2</td>
<td>1.50 x 56.5</td>
<td>DAG</td>
<td>13000</td>
<td>16500</td>
<td>12590</td>
<td>24270</td>
</tr>
<tr>
<td>TD-3</td>
<td>1.50 x 52.0</td>
<td>DAG</td>
<td>13000</td>
<td>16400</td>
<td>12570</td>
<td>24410</td>
</tr>
</tbody>
</table>

Mobilized Unit End Bearing

Figure 6 shows the mobilized unit end bearing vs. pile head movement of TAM and DAG grouted bored piles. In general, as can be seen in the figure, mobilized unit end bearing at specific pile head movement of piles constructed with tube-à-manchet grouting method (TAM) is higher than those of piles constructed with drill-and-grout method (DAG). It is also evident from Figure 6 that large pile head movement is required to mobilize the end bearing despite toe-grouting was applied for all piles. It should however to be noted that DAG grouted bored piles of Figure 6 (TD1, TD2 and TD3) were not tested to failure.

Figure 6. Mobilized unit end bearing of TAM and DAG grouted bored piles

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Though it is not advisable to simply draw the conclusion on particular toe-grouting method effects on performance of piles solely by mobilized unit end bearing criteria from the available limited test results, the data provided in this research study however could be taken as preliminary guidance to assess the effectiveness of the different toe-grouting methods.

According to the recent full-scale instrumented static pile load tests carried out in late 2002 and early 2003 on 13 drill-and-grout bored piles of diameter 1.50m seated at depth over 50m (a complete set of data is not available while preparing this paper), developed end bearing values of these piles were found to be scattered or inconsistent despite the fact that grouting pressure of all piles were the same - reached 60 bars (6,000 KN/m²). Therefore in authors’ opinion a careful justification is to be made to design the deep-seated bored piles in Bangkok subsoil with high end bearing values regardless of high pressure achievement in toe-grouting application.

**Load vs. Pile Head Movement Comparison**

Figure 7 shows the Load vs. Pile Head Movement of ungrouted, grouted piles of tube-à-manchette and drill-and-grout methods. Table 2 summarized and compared the load test results of the test piles of Figure 7.

![Figure 7. Load vs. pile head movement of ungrouted and toe-grouted piles](image)

Following conclusions can be drawn from Figure 7 and Table 2.

- Limit loads at 20mm pile head movement are improved (14% and 21% for Project A and B respectively) by tube-a-manchette toe grouting method (TAM) in comparison with ungrouted piles.
- No significant difference of load-settlement behavior is found for ungrouted piles in comparison with TAM grouted and DAG grouted piles at design load and 2 times of design load.
Table 2. Summarized comparison of load test results

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter (m)</td>
<td>1500</td>
<td>1500</td>
<td>1500</td>
<td>1500</td>
<td>1500</td>
<td>1500</td>
</tr>
<tr>
<td>Length (m)</td>
<td>60.00</td>
<td>60.50</td>
<td>63.70</td>
<td>63.70</td>
<td>1500</td>
<td>1500</td>
</tr>
<tr>
<td>Qd (KN)</td>
<td>10,000</td>
<td>10,000</td>
<td>10,000</td>
<td>10,000</td>
<td>13,000</td>
<td>13,000</td>
</tr>
<tr>
<td>QL (at 20mm total settlement)</td>
<td>26,500</td>
<td>23,200</td>
<td>34,280</td>
<td>21,750</td>
<td>25,500</td>
<td>26,000*</td>
</tr>
<tr>
<td>% increase in Qd at Qd</td>
<td>14.20</td>
<td>21.57</td>
<td>N.A</td>
<td>N.A</td>
<td>6.10</td>
<td>5.80</td>
</tr>
<tr>
<td>Total pile head movement at 1.5 Qd</td>
<td>6.50</td>
<td>6.50</td>
<td>7.20</td>
<td>6.00</td>
<td>11.90</td>
<td>11.00</td>
</tr>
<tr>
<td>Total pile head movement at 2 Qd</td>
<td>10.50</td>
<td>12.00</td>
<td>10.00</td>
<td>9.70</td>
<td>21.50</td>
<td>17.50</td>
</tr>
</tbody>
</table>

Note: * At 17.50mm pile head movement

Therefore, despite the various construction problems encountered and risks involved in successful execution of grouting, drill-and-grout method does not offer particular advantages over tube-à-manchette method.

EFFECTIVENESS OF THE DIFFERENT METHOD IN PRACTICAL APPLICATION

Problems Encountered in Practical Application of Different Method

Table 3 summarizes the most critical problems encountered in practical application of different toe-grouting methods used in Bangkok. In summary, it is obvious that higher risks are involved in the drill-and-grout method. The most critical risk of this method for the long piles is to maintain a good verticality of the grout access tubes.

Table 3. Summaries of the problems encountered in practical application of different methods

<table>
<thead>
<tr>
<th>Toe Grouting Method</th>
<th>Problems Encountered and Risks Involved to Complete Toe-grouting</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tube-à-manchette</td>
<td>Unsuccessful or ineffective cracking of manchette, Possible but rarely occur, Not required</td>
</tr>
<tr>
<td>Drill-and-grout</td>
<td>High verticality is extremely important. If access tube is inclined drilling rod can not be lowered to the bottom of the pipe for toe coring, High risk. If coring is unsuccessful, toe grouting can’t be performed.</td>
</tr>
</tbody>
</table>

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Impact Caused by Different Toe-grouting Method on Bored Pile Construction

The procedure involved in preparation and execution of two methods of toe-grouting poses different impact on bored pile construction. Since verticality is the main governing criteria in the successful application of grouting by drill-and-grout method as explained in above section the major impact caused in this method is due to the need of significant extra time in the following activities to produce the good verticality of the pile itself and the grout access tubes.

- Drilling
- Extensive borehole monitoring for verticality by sonic caliper method
- Grout pipe installation

Due to the excessive time consumption in the above mentioned operations in construction of the piles to further carry out the toe-grouting by drill-and-grout method, to produce 1 bored pile of diameter 1.50 m seated over 50m below ground, could take 2 to 3 days excluding time consumes for grouting work. For normal bored pile of same size and length, it normally takes less than 24 hours to produce 1 pile. Therefore considerable additional cost would incur for applying drill-and-grout method due to the requirement of significant extra construction time.

Assessment on the Effective Toe-grouting Method in Practical Construction

Assessment on Constructibility. Constructibility is one of the most important issues for most construction projects. If the construction method is impractical or if it is involved various problems and high risk in actual execution, it should not be employed in the first place. Careful justification on assessment of the constructibility is essential in selection of the toe-grouting method and in establishing the technical criteria. The party who is responsible for drawing the specification such as project consultant or designer should take the following points into consideration.

- The potential risks involve in the practical execution and consequential impact in minimizing the risk imposed by the method itself (e.g. delay in construction, extra cost)
- Additional cost involved for toe-grouting against saving from higher bearing capacity achivable by toe-grouting
- Clear and practical acceptance and rejection criteria in case the problems are encountered in actual execution

Assessment on the Performance of Pile. As presented in the earlier section, deep-seated large diameter bored piles in Bangkok are mainly supported by shaft friction and very small percentage of the load is transferred to pile toe under working load and double of working load. Therefore, even with the toe-grouting application, the end bearing component of bored piles is still under-utilized. Thus remaining unmobilized end bearing capacity serves only as a reserve that forms the additional factor of safety.
CONCLUSION

Based on the field application and measured data of instrumented pile load tests, following conclusions can be drawn.

(1) Under design working load for large diameter deep-seated bored piles with toe grouting in Bangkok subsoil, the large portion of load (over 90% of applied load) is carried by the shaft friction. Only the small percentage of the total load is transferred to the pile tip. No significant difference of load-settlement behavior is found for non-grouted piles against TAM grouted and DAG grouted piles at design load and 2 times of design load. Therefore, for deep-seated bored piles in Bangkok subsoil, even with the toe grouting application, the end bearing component of bored piles could be still under-utilized. Thus the remaining unmobilized end bearing capacity serves only as a reserve which forms the additional factor of safety.

(2) The drill-and-grout method has higher risks than the tube-à-manchette method in successfully constructing the toe grouted bored pile. Considerable additional cost would incur in applying drill-and-grout method due to the requirement of significant extra construction time to complete a qualified pile.

(3) Despite the considerable extra cost, various construction problems encountered and risks involved in successful execution of grouting, drill-and-grout method does not offer particular advantage over tube-à-manchette method as far as pile capacity improvement is concerned.

(4) The potential risks involve in the practical execution and consequential impact in minimizing the risk imposed by the method itself should be taken into account in selecting the effective toe-grouting method.

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REFERENCES


CURRENT PRACTICE AND FUTURE TRENDS
OF DEEP FOUNDATIONS IN BANGKOK, THAILAND

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ABSTRACT

This paper presents the authors’ experiences in development of deep foundations, bored piles and barrettes in particular in Thailand. Some literatures related to wet-processed bored piles and barrettes in Thailand published throughout the past 30 years are also summarized together with recent research works.

KEYWORDS: bored pile, barrette, deep foundations, bentonite, polymer-based slurry

INTRODUCTION

Cast-in-place deep foundations, particularly wet-processed deep-seated large diameter bored piles and barrettes have been used for foundations of various infrastructure projects in Thailand for over two decades. The versatility of the construction method and the high-load capacity which in turn offered the constructibility and cost-saving are the main factors contributed to the increasing use of deep-seated large diameter bored piles and barrettes. In the initial stage there were a number of questions on design and construction aspects of these foundation systems particularly in Bangkok subsoil. With the passage of time, the construction equipment, installation techniques and testing of deep foundation elements have been developed. Large numbers of instrumented full-scale static pile load tests were conducted throughout the 1990’s which provided better understanding on behavior of these deep-seated foundations. Researches focused on the design parameters and methods were produced based on these test results. The design parameters were so well established that wet-process cast-in-place foundations became regarded as reliable foundations for not only in the property sectors but also for practicing engineers involved in the infrastructure projects. This paper presents the development of wet-processed bored piles and barrettes in Thailand. The information contained in the paper are mainly from the infrastructure developments built in Bangkok and adjacent areas.

CONSTRUCTION METHOD

Large diameter deep-seated wet-processed bored piles can be constructed by 2 methods, reverse-circulation and rotary drilling method. As construction procedure of historic reverse-circulation method has been well documented in many literatures and it is less frequently utilized nowadays in Thailand, only a rotary-drilling method is briefly presented in this article as follows.

In rotary drilling method, a temporary casing of appropriate length (12 to 18m in Bangkok depending on the thickness of soft clay) with required diameter (internal diameter not less than that of design bored pile diameter) is first installed to ensure the stability of the borehole in the top soft or loose soil layers. In some projects where the vibration is strictly limited, standard length of casing with oscillator or short temporary casing of 5 to 8 m is
pushed down in combination with pre-boring process. Drilling is commenced by auger to drill out the soil within the temporary casing and up to the top of the first water-bearing sand layer or bottom of the casing in case of using short casing method. Drilling slurry or supporting fluid is then supplied to the borehole and drilling is continued with a bucket down to the design final depth of the pile. Before lowering the reinforcement cage, a special cleaning bucket is used to clean the base of borehole. If bentonite slurry is used, recycling method by air-lift or pump is applied as base cleaning process. Reinforcement cages are then lowered into the borehole and concreting is carried out by tremie method.

A mechanical or hydraulic cable-suspended grab is commonly used for barrette construction. Excavation of the trench is carried out by the cyclic-process of lifting and lowering of the grab under gravity and tangential force of the clamshell operated by cables. Different from bored pile construction, a guide wall of depth 1 to 1.5 m with inside clear dimensions slightly larger than the nominal size of the barrette is used to guide the grab during initial bites. Bentonite slurry is introduced to the trench as soon as initial excavation commenced. The excavation is continued under the bentonite slurry to the final depth. After recycling of slurry and lowering the rebar cage, concreting is done by tremie method.

**OVERVIEW OF APPLICATION**

**Bored pile**

The first wet-process large diameter bored pile was constructed for Pinklao Bridge in Bangkok 30 years ago. Using reverse circulation method, diameter 1.50m bored pile was installed up to 45m in the second sand layer. Three major bridges were constructed by bored pile of reverse circulation method in Bangkok from early 1970 to 1980. The summarized information of these bridges are tabulated below.

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Year of Construction</th>
<th>Construction Method</th>
<th>Diameter (m)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pinklao Bridge</td>
<td>1971</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>45</td>
</tr>
<tr>
<td>Sathorn Bridge</td>
<td>1979</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>46</td>
</tr>
<tr>
<td>New Memorial Bridge</td>
<td>1982</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>49</td>
</tr>
</tbody>
</table>

The first wet-processed bored pile of rotary-drilling method down to first sand layer was constructed in Bangkok in late 1970 for high-rise building project, Royal Orchid Hotel. Since then bored piles constructed by rotary-drilling method have been extensively used for foundations of various heavy structures such as high-rise buildings, elevated expressways, overpass-bridges, underground car park buildings, waste-water treatment plants and most recently underground train stations of Bangkok first subway project. By mid 1980, bored pile became the foundation of choice for heavy structures particularly in urban area of Bangkok. The versatility of the construction method and the high-load capacity which in turn offered the constructibility and cost-saving are the main factors contributed to the increasing use of deep-seated large diameter bored piles and barrettes. Most of the early wet-process bored piles (1980’s) in Bangkok were constructed up to 50 m depth. Diameter of bored piles commonly constructed in early days were ranging from 0.6 to 1.5 m. **Table 2** below summarizes the information of early major high-rise building projects in Bangkok.
<table>
<thead>
<tr>
<th>Project Name</th>
<th>Construction Method</th>
<th>Diameter (m)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Royal Orchid Hotel (1979)</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.00</td>
<td>33.0</td>
</tr>
<tr>
<td>Taiping Tower (1980)</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>32.0</td>
</tr>
<tr>
<td>River City Hotel (1982)</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 – 1.00</td>
<td>27.5</td>
</tr>
<tr>
<td>Asoke Tower (1983)</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>50.0</td>
</tr>
<tr>
<td>Time Square Building (1983)</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>50.0</td>
</tr>
</tbody>
</table>

**Barrette**

In some projects where bored piles were not feasible due to the site constraints, applicable construction method and/or extensive bearing capacity requirements, the use of barrette foundations would make a suitable alternative. Barrettes with dimension ranging from 0.80 m x 2.7 m to 1.5 m x 3.0 m for safe working load capacity from 11,000 to 23,000 kN have been used in some major projects.

The first barrette in Bangkok was believed to be constructed in late 1970 for the foundation of Bangkok Bank Head Office Building at Silom road with size of 0.6-0.8 x 2.5m and toe depth of 33m in late 1970. Barrettes can be constructed with flexible layout plan for both vertical and lateral loads. The layout pattern of barrettes can be arranged in a continuous row or column, radial, alternating long and short axis of barrette and a combination of two or more of such patterns. One type of equipment can be used for constructing both barrettes and diaphragm walls in particular projects thus reduces the mobilization cost for additional equipment. In addition to the large bearing capacity requirements, on site difficulties such as limited head room where piling rigs cannot be utilized, under such situations like presence of overhead high voltage power cables, existing overpasses or structures for elevated expressways and planned subway stations, also demands the barrettes. A summary of observation on barrettes of completed 26 projects completed with respect to selection criteria is presented in **Table 3**.

**Table 3** Summary of Barrette Selection

<table>
<thead>
<tr>
<th>Criterion</th>
<th>No. of Project</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Load Capacity</td>
<td>4</td>
<td>Incorporated with bored piles</td>
</tr>
<tr>
<td>Minimize Construction Equipment</td>
<td>6</td>
<td>Alternative for bored piles</td>
</tr>
<tr>
<td>Limited Head Room for Excavation</td>
<td>3</td>
<td>Under existing structures such as bridges, elevated expressway and power lines</td>
</tr>
<tr>
<td>Combination with Diaphragm Wall</td>
<td>10</td>
<td>As diaphragm wall legs</td>
</tr>
<tr>
<td>Foundation as well as portion of column</td>
<td>3</td>
<td>Provision for the future requirement</td>
</tr>
</tbody>
</table>

Static load test up to 5290 ton conducted on barrette set the record as the highest load ever tested for a single cast-in-situ deep foundation in Thailand (Thasnanipan et. al. 2002a)
Basic but extensive problems were experienced in early stages of bored pile construction in Thailand as summarized below.

- Limited availability and capacity of equipment
- Lack of skills in operation of equipment (particularly drilling rig)
- Adverse effect due to a slow rate of drilling (e.g. excessive formation of filter-cake by using bentonite slurry)
- Limited knowledge and less advance technique in control of bentonite slurry
- Limited experience in construction method and related negative impact
- Quality of concrete for tremie concreting method
- Lack of experienced engineers and foremen
- Improper construction and quality control specification and guidelines for deep-seated piles in local soil condition
- Improper design for constructibility

DEVELOPMENT IN CONSTRUCTION

Over the past 3 decades, along with the development of wet-processed piling technology world-wide, equipment, construction method, design as well as better understanding and realization of construction impact on the performance have been significantly improved in Thailand. Table 4 summarizes the areas of improvement in bored pile and barrette construction and main factors contributed to these.

Research works presenting some of these developments were published both locally and internationally. The design, construction and behavior of bored cast in-situ concrete piles in Bangkok Subsoil was presented by Thasnanipan et. al., (1998a). The construction and performance of barrettes in Bangkok Subsoil was also reported by Thasnanipan et. al., (1998b). Effect of construction time and bentonite viscosity on shaft friction of bored piles was also pointed out by Thasnanipan et al., (1998c). Teparaksa et. al., (1999) published the base grouting of wet process bored piles in Bangkok subsoil which demonstrated the method of base grouting and reported the grout spreading mechanism for long bored piles and performance of base grouted bored piles.

![Fig. 1](image1.png)  
(a) Guide wall for cruciform barrette  
(b) View of cruciform barrette after installation of reinforcement

Construction of cruciform barrette (shown in Figure 1) for monopole type high-voltage power transmission line was described in the work of Thasnanipan et. al., 2000a.
Above-mentioned research works reflect the development history of bored pile and barrettes in Thailand and offered useful information to the local construction industry and perhaps to the international deep foundation engineering society.

**Table 4** Summary of development in bored pile and barrette construction

<table>
<thead>
<tr>
<th>Area of development / improvement</th>
<th>In the past (Early 1980’s)</th>
<th>At present</th>
<th>Main factor contributed to development</th>
</tr>
</thead>
<tbody>
<tr>
<td>Speed of construction</td>
<td>Minimum 3 days required to complete diameter 1.5m tip 50m bored pile</td>
<td>Less than 1 day to complete diameter 1.5m tip 50m bored pile</td>
<td>Better equipment, operating skills as well as improved-knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Pile size and depth</td>
<td><strong>Bored pile</strong> Maximum diameter 2.0m and common depth 25-50m for bored pile. <strong>Barrette</strong> Limited in size and depth</td>
<td><strong>Bored Pile</strong> Maximum Diameter 2.0m and common depth 25-60m <strong>Barrette</strong> Various sizes &amp; depth over 60m</td>
<td>Better equipment, operating skills as well as improved-knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Base grouting</td>
<td>Not available</td>
<td>Available</td>
<td>Equipment availability and advance technology</td>
</tr>
<tr>
<td>Application of polymer-based slurry (for bored pile only)</td>
<td>Not available</td>
<td>Extensively use</td>
<td>Material availability and research works</td>
</tr>
<tr>
<td>Construction impact on quality and performance</td>
<td>Not well understood</td>
<td>Improving</td>
<td>Experience from past projects and research works</td>
</tr>
<tr>
<td>Quality control in construction process</td>
<td>Not well established and systematic</td>
<td>Well established and systematic</td>
<td>Experience from past projects and research works</td>
</tr>
<tr>
<td>Quality control test method and interpretation</td>
<td>Less methods available and limited knowledge in interpretation</td>
<td>Better equipment available and better knowledge in interpretation</td>
<td>Advance equipment, experience from past projects and research works</td>
</tr>
</tbody>
</table>

Experience from past projects and extensive research works provided better understanding on construction impact on quality and performance of these wet-processed deep foundations. Marked difference between outcome quality of bored piles constructed in 1980s and 1990s can be observed by significant less defects found in the latter. It can also be observed from load test results that bored piles constructed in late 1990 have higher capacity than those of 1980s as shown in **Table 5**.

**Table 5** Load test results of bored piles constructed in 1970 to 1980 and 1990s
Bored piles have been extensively used for foundations of the majority of the elevated expressways since 1991. Thousands of base-grouted bored piles were constructed for these infrastructure projects including Second Stage Expressway (1991), Don Muang Tollway Extension (1997), Bangna-Bang Pli-Bangprakong Expressway (1998), and Wat Nakorn-In project (2001). Over 700 bored piles of diameter ranging from 0.8m to 1.5m with depth from 35m to 54m were constructed (1999-2000) by polymer-based slurry for the foundation of Rama VIII Bridge, one of the initiatives of his majesty the King Bhumibol Adulyadej. Progress of piling works at the bank of Chao Phraya river for the main bridge of Rama VIII Bridge is depicted in Figure 2.

The availability of more reliable and powerful equipment and tools for drilling makes possible to construct the rock-socket bored piles. Figure 3 shows the drilling rig equipped with rock-auger used for construction of highway bridge across Mool River in 2001, in the north-eastern part of Thailand, where drilling was carried out through weathered-sandstone by powerful rotary drilling rig with core barrel and rock-auger.

<table>
<thead>
<tr>
<th>Year of construction</th>
<th>Project Name</th>
<th>Pile Dimension (Dia. &amp; Depth)</th>
<th>Design Load (ton)</th>
<th>Test Load (ton)</th>
<th>Total settlement at max. test load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1971</td>
<td>Pinklao bridge</td>
<td>1.5m x 45m</td>
<td>410</td>
<td>820</td>
<td>5</td>
</tr>
<tr>
<td>1995</td>
<td>Central Plaza Pinklao</td>
<td>1.2m x 45m</td>
<td>700</td>
<td>1400</td>
<td>27</td>
</tr>
<tr>
<td>1980</td>
<td>Taiping tower</td>
<td>1.0m x 32m</td>
<td>300</td>
<td>1125</td>
<td>118</td>
</tr>
<tr>
<td>1990</td>
<td>High-rise building at Ekamai Rd.</td>
<td>1.0m x 32m</td>
<td>390</td>
<td>975</td>
<td>23</td>
</tr>
</tbody>
</table>

Fig. 2 Progress of piling work for the Rama VIII main bridge

Fig. 3 Rock-socket bored pile construction for Mool River Project - Ubon Ratchathani Province

DEVELOPMENT IN DESIGN CONCEPT AND PARAMETERS

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In the initial stage of introducing bored piles in Thailand, the design concepts and parameters were mainly based on the available literatures from the research works carried out in other parts of the world such as Tomlinson (1957), Skempton (1959), Broms (1966), Bowles (1968), Meyerhof (1976) etc. The work of Chiruppapa (1968) was believed to be the first research data available for design parameters of bored pile in Bangkok soft clay. The author conducted the research based on 6 dry-processed small diameter bored piles with load cells installed at the pile toe in AIT campus in Phathumthani. Though the piles were simply constructed by dry-processed with casing method, this early-stage research study provided some important information such as load-settlement behavior, adhesion factor ($\alpha$) and bearing capacity factor (N) of bored pile in Bangkok soft clay.

With the passage of time, design method and parameters for local subsoil were improved as a result of research works carried out in 1980s. Differences between the behavior of driven piles and bored piles were well realized from these researches. (Ng, 1983) presented the load distribution characteristics of wet-processed bored piles founded in both first and second sand layers of Bangkok based on the instrumented (strain gauges) pile load test results. (Chiewcharnsilp, 1988) reported the shaft friction factor $\beta$ values of sand layers in Bangkok based on the instrumented load test results. In addition to the researches previously available (Chiruppapa, 1968; Suwanakul, 1969; Promboon, 1981; Ng, 1983; Chiewcharnsilp, 1988) Suchada (1989) determined the shaft friction factor (adhesion factor, $\alpha$) of Bangkok clay layers based on 11 bored piles of diameter ranging from 0.50 to 0.80m with the embedded length varying between 21.50m and 46m. It should be noted that the design parameters obtained from the research works of 1980s were mainly based on the estimation of shaft friction loads from static pile load test results with numbers of assumption since instrumented pile load test results were limited.

With the peak of construction-boom, large numbers of instrumented full-scale static load tests on bored piles were conducted throughout 1990’s which provided better understanding on behavior of these deep-seated foundations. Researches focused on the design parameters and methods were produced based on these test results. The design parameters were more accurate and well established that wet-process cast-in-place foundations became regarded as reliable foundations in construction industry of Thailand. With more confidence on soil parameters and behavior of these deep foundations, the designers designed to carry significantly higher loads for bored piles and barrettes constructed in late 1990 than those of 1980s. (Thasnanipan et. al., 1999) reported the failure mechanism of long bored piles in layered soil of Bangkok. The authors cited that for the bored piles embedded in the multi layered soils of Bangkok, estimation of ultimate shaft friction capacity needs to consider the brittle type of failure mechanism of stiff to hard clay layers and the $\alpha$ values selected need to be adjusted accordingly. Peak and residual $\alpha$ values mobilized in the stiff clay layers analyzed by the authors were plotted as shown in Figure 4 along with the suggested curves by different researchers. It can be seen from the figure that the residual $\alpha$ value of 2nd stiff clay layer (undrained shear strength values of 25 ton/m2) at the maximum test load drops below the curve suggested by (Suchada, 1989). So the $\alpha$ values proposed in Figure 4 for the stiff to hard clay layers gives overestimate of the ultimate shaft friction under these conditions.

Introduction of polymer-based slurry for wet-process bored piles marked a major breakthrough for both construction and design engineers. Thasnanipan et al.,(2002b), reported that bored piles constructed with polymer-based slurry have higher capacity than those constructed with bentonite slurry. Figure 5 shows the shaft friction factors $\beta$ of sand layers for polymer-based bored piles in comparison with the design line of bentonite bored piles.
Fig. 4 Comparison of adhesion factor $\alpha$, suggested by different researchers with the actual mobilized in the stiff clay layers (after Thansnanipan et. al 1999)

![Graph showing comparison of adhesion factor $\alpha$.]

Fig. 5 Back-calculated $\beta$ values of polymer bored piles at maximum test load plotted on design line of bentonite bored piles constructed in Bangkok subsoil (after Thansnanipan et. al 2002b)

![Graph showing back-calculated $\beta$ values.]

DEVELOPMENT IN QUALITY CONTROL TESTING

Improvement of testing equipment and powerful computer facilities are the key factors contributed to the development in quality control testing. Interpretation skills relevant to local soil condition and construction method of these tests were significantly improved in local industry. For instance, in early 1980, interpretation of the sonic integrity (seismic test) test results were needed to send to the specialists abroad which made the testing cost more expensive. Significant cost-saving were achieved in some major projects of late 1990, as more practical and precise interpretation were made to verify and establish the acceptance criteria in proving the quality of suspected piles with anomalies.
Koden drilling monitoring system

In Thailand, prior to the availability of Koden testing equipment, borehole verticality was checked by mechanical type equipment such as plumb float and adjustable globe. However, the reliability of these methods was doubtful. Koden drilling monitor system was believed to be first used in Thailand in 1980. The verticality of slurry-filled borehole can be monitored rapidly from the electronic plot of continuous profile by Koden equipment. This system is very useful for the verticality-control of long barrettes - unlike bored piles, guiding by temporary casing is not available for barrette drilling.

Sonic integrity test

Sonic integrity test, also known as seismic test is the most common method of integrity testing for both driven and bored piles in Thailand. Sonic integrity test is usually selected for both quality check (control test) and retrospective investigation. It is the cheapest in terms of cost and the simplest in terms of testing process. The main advantage of this test is that since no particular preparation is necessary during the pile construction phase it is more flexible to select which pile is to be tested. However, interpretation of sonic integrity testing needs considerable experience and knowledge in testing, subsoil condition and construction method. In many projects it is a part of the contractual requirement to conduct sonic integrity test. Minimum 10% to maximum 100% of production piles are commonly tested. It is also a reasonably acceptable method for applying as a retrospective investigation in determining integrity of the pile. The signal characteristics and their interpretations of sonic integrity test on piles founded in Bangkok subsoil were reported in details by Thasnasipan et al. (1998d).

Cross-hole sonic logging test

Sonic logging test is relatively expensive. It is mainly employed as a pre-planed site quality control testing. The major advantage of this method is that test can be carried out shortly after the pile construction. Hence, rectification measures can be implemented if the pile is detected with defect while the foundation contractor is on site. However, this method is generally not applicable if pile integrity is in question due to post-construction activities as access tubes are usually grouted after completion of the test. The first sonic logging test was believed to be conducted in 1982 for the wet-processed bored piles of Memorial Bridge Project where bad concrete zones were detected at depth about 20m and 1m (Ng, 1983). Use of Sonic logging test for checking pile integrity has increased in Thailand in recent years. The results from sonic logging test conducted on model piles in Bangkok helped to extend the knowledge of the signal characteristics and interpretation (Thasnanipan et. al., 2000b).

High strain dynamic load test

High strain dynamic integrity test has become a well-accepted method especially for evaluating the pile capacity in today’s foundation industry of Thailand and it is applied for both driven and bored piles. A large number of related technical papers and case histories of the test have been published and it is a part of standards and specifications such as ASTM D4945-89 (Standard Method for High-strain Dynamic Testing of Piles). Thasnanipan et. al. 2000b, reported the application of dynamic load testing in Thailand.

CONCLUDING REMARKS

Development of bored pile and barrettes in Thailand has been presented. According to the authors’ experience as a contractor, development in both construction and design aspects of these deep foundations in past decades are significant. In authors’ opinion, however, there are many works to be done to improve further with particular focus on constructibility issues, concrete technology for wet-processed bored piles and barrette, reliable but cost-effective
quality control testing and value-engineering. Starting from planning stage, site investigation, design, construction and inspection should be integrated such that designers, contractors and construction inspectors participating as a team with common goal. Appropriate and practical specification should be established jointly by these parties for local soil condition and construction method. Practical acceptance criteria should be developed to verify bored piles or barrettes with suspected anomalies. Continuing education should be promoted for both designers, inspection engineers and contractors. The Geotechnical Chapter of the Engineering Institute of Thailand under the royal patronage of his majesty the King Bhumibol Adulyadej, has started to establish the standard code of practice and guidelines for wet-processed bored piles which will serve as a yardstick for the deep foundation industry upon completion in near future.

REFERENCES


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Development and Achievements of Deep-seated Bored Piles and Barrettes Construction in Thailand for the Past Forty Years, A Country Report

Narong Thasnanipan, Zaw Zaw Aye, Chanchai Submaneewong and Thayan Boonyarak

SEAFCO Public Co., Ltd. Bangkok, Thailand
Weathered Crust
Medium to Stiff Clay
Medium Dense Sand (First Sand)
Bangkok Soft Clay
Hard Clay
Dense to Very Dense Sand (Second Sand)

Hydrostatic Line
Actual Piezometric Drawdown Line

Pore Pressure (kN/m²)

Su = 10-25 KPa
γt = 14-16kN/m³

Su = 50-140 KPa
γt = 17-21kN/m³

SPT-N = 20-40
γt = 20kN/m³

Su > 200 KPa
γt = 21kN/m³

SPT-N = 50-100
γt = 21kN/m³

Geotechnical Parameters

Development and Achievements of Deep-seated Bored Piles and Barrettes Construction in Thailand for the Past Forty Years, A Country Report
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Abstract: This paper presents the development and achievements of deep-seated bored piles and barrettes construction in Thailand based on the authors' experiences, in the past four decades. Literature related to bored piles and barrettes in Thailand published throughout the past forty years is summarized together with recent research works. Problems and difficulties in construction of wet-processed bored piles and barrettes in early days are reported. The areas of improvement, advancement and achievement in bored pile and barrette construction and main factors contributed to these developments are also presented. An attempt is also made to briefly discuss the future trend of bored pile and barrette foundations.

1   INTRODUCTION
During the past four decades, owing to the acceleration of development in mega infrastructure and numerous tall building projects, techniques and practice of cast-in-place deep foundations, particularly wet-process deep-seated large diameter bored piles and barrettes have experienced enormous development and achievements in Thailand. The versatility of the construction method and the high-load capacity which in turn offered the constructibility and cost-saving are the main factors contributed to the increasing use of those cast-in-place deep foundations. In the initial stage, there were a number of questions on design and construction aspects of these foundation systems particularly in Bangkok subsoil. With the passage of time, the construction equipment, installation techniques and testing of deep foundation elements have been developed. Large numbers of instrumented full-scale static pile load tests were conducted throughout the 1990s mainly in mega projects, which provided better understanding on behavior of these deep-seated foundations.

Research focused on the design parameters and methods were produced based on these test results. The design parameters became so well established that wet-process cast-in-place foundations became regarded as reliable foundations for practitioners involved in construction industry in Thailand. This paper presents the past and current practice of wet-processed bored piles and barrettes in Thailand together with a brief discussion on future trend of these cast-in-place deep foundations. The information contained in the paper is mainly from the mega projects constructed in Bangkok and adjacent areas.

2.   SUBSOIL AND EXISTING PIEZOMETRIC PROFILE
Subsoil profile and the present piezometric drawdown condition of Bangkok are presented in Fig. 1 below. A typical subsoil profile is relatively consistent in different localities in Bangkok. It is characterized by alternating layers of clay and sand deposits as shown in Fig. 1.

Fig. 1. Typical soil profile of Bangkok with piezometric drawdown condition (Thasnanipan, et al., 2002)

3.   CONSTRUCTION METHOD IN GENERAL
Due to the prevailing subsoil and groundwater conditions, deep-seated bored piles of toe depth over 24m are constructed by wet-process or slurry displacement method. Wet-process bored piles can be constructed by 2 methods, reverse-circulation and rotary drilling. As construction procedure of historic reverse-circulation method has been well documented in various studies and since it is less frequently utilized nowadays in Thailand, only a rotary-drilling method is briefly presented in this article as follows.

In the rotary drilling method, a temporary casing of appropriate length (12 to 18m in Bangkok depending on the thickness of soft clay) with required diameter (internal diameter not less than that of design bored pile diameter) is first installed to ensure the stability of the borehole in the top soft or loose soil layers. In some projects where the vibration is strictly restricted, a standard length of casing with oscillator or short temporary casing of 5 to 8 m is pushed down in combination with the pre-boring process. Drilling is commenced by auger to drill out the soil inside the temporary casing. Auger-drilling is commonly continued up to the top of...
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1 INTRODUCTION

During the past four decades, owing to the acceleration of development in mega infrastructure and numerous tall building projects, techniques and practice of cast-in-place deep foundations, particularly wet-process deep-seated large diameter bored piles and barrettes have experienced enormous development and achievements in Thailand. The versatility of the construction method and the high-load capacity which in turn offered the constructibility and cost-saving are the main factors contributed to the increasing use of those cast-in-place deep foundations. In the initial stage, there were a number of questions on design and construction aspects of these foundation systems particularly in Bangkok subsoil. With the passage of time, the construction equipment, installation techniques and testing of deep foundation elements have been developed. Large numbers of instrumented full-scale static pile load tests were conducted throughout the 1990s mainly in mega projects, which provided better understanding on behavior of these deep-seated foundations.

Research focused on the design parameters and methods were produced based on these test results. The design parameters became so well established that wet-process cast-in-place foundations became regarded as reliable foundations for practitioners involved in construction industry in Thailand. This paper presents the past and current practice of wet-processed bored piles and barrettes in Thailand together with a brief discussion on future trend of these cast-in-place deep foundations. The information contained in the paper is mainly from the mega projects constructed in Bangkok and adjacent areas.

2. SUBSOIL AND EXISTING PIEZOMETRIC PROFILE

Subsoil profile and the present piezometric drawdown condition of Bangkok are presented in Fig. 1 below. A typical subsoil profile is relatively consistent in different localities in Bangkok. It is characterized by alternating layers of clay and sand deposits as shown in Fig. 1.

3. CONSTRUCTION METHOD IN GENERAL

Due to the prevailing subsoil and groundwater conditions, deep-seated bored piles of toe depth over 24m are constructed by wet-process or slurry displacement method. Wet-process bored piles can be constructed by 2 methods, reverse-circulation and rotary drilling. As construction procedure of historic reverse-circulation method has been well documented in various studies and since it is less frequently utilized nowadays in Thailand, only a rotary-drilling method is briefly presented in this article as follows.

In the rotary drilling method, a temporary casing of appropriate length (12 to 18m in Bangkok depending on the thickness of soft clay) with required diameter (internal diameter not less than that of design bored pile diameter) is first installed to ensure the stability of the borehole in the top soft or loose soil layers. Drilling is commenced by auger to drill out the soil inside the temporary casing. Auger-drilling is commonly continued up to the top of...
the first water-bearing sand layer or bottom of the casing when using the short casing method. Drilling slurry or supporting fluid is then supplied to the borehole and drilling is proceeded with a bucket down to the design final depth of the pile. Before lowering the reinforcement cage, a special cleaning bucket is used to clean the base of borehole. If bentonite slurry is used, recycling method by air-lift or pump is applied as the base cleaning process. Reinforcement cages are then lowered into the borehole and concreting is carried out by tremie method.

A mechanical or hydraulic cable-suspended grab is commonly used for barrette construction. Excavation of the trench is carried out by the cyclic-process of lifting and lowering of the grab under gravity and tangential force of the clamshell operated by cables (mechanical grab) or hydraulic action (hydraulic grab). Different from bored pile construction, a guide wall of depth 1 to 1.5 m with inside clear dimensions slightly larger than the nominal size of the barrette is used to guide the grab during initial bites. Bentonite slurry is introduced into the trench as soon as the initial excavation commenced. The excavation is continued under the bentonite slurry to the final depth. After recycling of slurry and lowering the rebar cage, concreting is done by tremie method.

4. OVERVIEW OF APPLICATION

4.1 Bored piles

Using tripod rigs, small diameter dry-processed bored pile construction was started in early 1970. The first wet-process large diameter bored pile was constructed for Pinklao Bridge in Bangkok 36 years ago. Using reverse circulation method, a 1.50m diameter bored pile was installed up to 45m in the second sand layer. Three major bridges were constructed by bored pile using the reverse circulation method in Bangkok from early 1970 to 1980. The summarized information of these bridges is tabulated in Table 1 below.

Table 1. First 3 major bridges constructed by large diameter wet-processed bored piles in Bangkok.

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Year of Construction</th>
<th>Construction Method</th>
<th>Diameter (m)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pinklao Bridge</td>
<td>1971</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>45</td>
</tr>
<tr>
<td>Sathorn Bridge</td>
<td>1979</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>46</td>
</tr>
<tr>
<td>New Memorial Bridge</td>
<td>1982</td>
<td>Reverse circulation</td>
<td>1.50</td>
<td>49</td>
</tr>
</tbody>
</table>

The first wet-processed bored pile utilizing the rotary-drilling method down to first sand layer was constructed in Bangkok in the late 1970s for high-rise building project, the Royal Orchid Hotel located at the bank of Chao Phraya river. Since then bored piles constructed by rotary-drilling method have been extensively used for foundations of various heavy structures such as high-rise buildings, elevated expressways, overpass-bridges, underground car park buildings, waste-water treatment plants and most recently underground train stations of Bangkok’s first subway project. By mid 1980s, bored piles became the foundation of choice for heavy structures particularly in the urban area of Bangkok. The versatility of the construction method and the high-load capacity which in turn offered the constructability and cost-saving, are the main factors contributing to the increasing use of deep-seated large diameter bored piles and barrettes. Most of the early wet-process bored piles (1980s) in Bangkok were constructed up to 50 m depth. Sizes of bored piles constructed in early days ranged from diameter 0.6 to 1.5 m. Summarized information of early major high-rise building projects in Bangkok is presented in Table 2.

4.2 Barrette piles

In some projects where bored piles were not feasible due to the site constraints, applicable construction methods, and/or extensive bearing capacity requirements, the use of barrette foundations would make a suitable alternative. Barrettes with dimensions ranging from 0.80 m x 2.7 m to 1.5 m x 3.0 m for safe working load capacity from 11 000 to 25 000 kN have been used in some major projects.

The first barrette in Bangkok was believed to have been constructed in the late 1970s for the foundation of the Bangkok Bank Head Office Building at Silom road with the size of 0.6-0.8 x 2.5m and the toe depth of 33m. Barrettes can be constructed with flexible layout plans for both vertical and lateral loads. The layout pattern of barrettes can be arranged in a continuous row or column, radial, alternating long and short axis of barrette and a combination of two or more of such patterns. To minimize the need of additional equipment, a mechanical grab mounted on crawler crane can be used for constructing both barrettes (as piled-foundation) and diaphragm walls (as earth retaining structure) in some suitable projects without employing a bored piling rig, which could save the extra mobilization cost. In addition to the large bearing capacity requirements, on site difficulties such as limited head room where piling rigs cannot be utilized, under such situations like presence of overhead high voltage power cables, existing overpasses or structures for elevated expressways and planned subway stations, also demands the use of barrettes. Static load test up to 52 900 kN conducted on barrette set the record as the highest load ever tested for a single barrette foundation in Thailand (Thasnanipan et. al., 2002a). A summary of completed 35 projects constructed with barrettes with respect to selection criteria is presented in Table 3.

5. PROBLEMS AND DIFFICULTIES IN CONSTRUCTION OF WET-PROCESS BORED PILES AND BARRETTES IN EARLY DAYS

Basic but extensive problems were experienced in early stages of bored pile construction in Thailand as summarized below.

- Limited availability and capacity of equipment
- Lack of skills in operation of equipment (particularly the drilling rig)
- Adverse effect due to a slow rate of drilling (e.g. excessive formation of filter-cake by using bentonite slurry)
- Limited knowledge and less advanced techniques in the control of bentonite slurry
- Limited experience in construction method and related negative impact
- Quality of concrete used for tremie concreting
- Lack of experienced engineers and foremen
- Improper construction and quality control specifications and guidelines for deep-seated piles in local soil
- Improper design for constructability

Table 2. First high-rise building projects constructed with large diameter wet-processed bored piles in Thailand (from 1979 to 1983).

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Construction Method</th>
<th>Diameter (m)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Royal Orchid Hotel</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.00</td>
<td>33.0</td>
</tr>
<tr>
<td>(1979)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Taiping Tower</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>32.0</td>
</tr>
<tr>
<td>(1980)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>River City Hotel</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 – 1.00</td>
<td>27.5</td>
</tr>
<tr>
<td>Hotel (1982)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asoke Tower</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>50.0</td>
</tr>
<tr>
<td>(1983)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Time Square Building</td>
<td>Rotary-drilling with auger and bucket</td>
<td>0.80 - 1.50</td>
<td>50.0</td>
</tr>
<tr>
<td>(1983)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Summary of Barrette Selection in major projects.

<table>
<thead>
<tr>
<th>Selection Criterion</th>
<th>No. of Project</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Load Capacity</td>
<td>7</td>
<td>Incorporated with bored piles</td>
</tr>
<tr>
<td>Minimize Construction Equipment</td>
<td>7</td>
<td>Alternative for bored piles</td>
</tr>
<tr>
<td>Limited Head Room for Excavation</td>
<td>3</td>
<td>Under existing structures such as bridges, elevated expressway and power lines</td>
</tr>
<tr>
<td>Combination with Diaphragm Wall</td>
<td>16</td>
<td>As diaphragm wall legs</td>
</tr>
<tr>
<td>Foundation as well as portion of column</td>
<td>3</td>
<td>Provision for the futures requirement</td>
</tr>
</tbody>
</table>

6. IMPROVEMENT IN CONSTRUCTION

Over the past three decades, along with the development of wet-processed piling technology in other parts of the world, equipment, construction technique and design methods as well as better understanding of construction impact on the performance of this type of foundation have significantly improved in Thailand. Table 4 summarizes the areas of improvement in bored pile and barrette construction and main factors contributed to these developments. Some of these improvements were published both locally and internationally. The design, construction and behaviour of bored cast in-situ concrete piles in Bangkok Subsoil were presented by Thasnanipan et al., (1998a). The construction and performance of barrettes in Bangkok Subsoil was also reported by Thasnanipan et al., (1998b). Effect of construction time and bentonite viscosity on shaft friction of bored piles was also pointed out by Thasnanipan et al., (1998c). Thasnanipan et al., (2004a) also reported a comprehensive study on the effectiveness of two different toe-grouting methods, known as tube-à-manchette and drill-and-grout, applied in Bangkok.

Table 4. Summary of development in bored pile and barrette construction.

<table>
<thead>
<tr>
<th>Area of development / improvement</th>
<th>In the past (Early 1980’s)</th>
<th>At present</th>
<th>Main factor contributed to development</th>
</tr>
</thead>
<tbody>
<tr>
<td>Speed of construction</td>
<td>Minimum 3 days required to complete diameter 1.5m tip 50m bored pile</td>
<td>Less than 1 day to complete diameter 2.0 m tip 60m bored pile</td>
<td>Better equipment, operating skills as well as improved-knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Pile size and depth</td>
<td>Bored pile Maximum diameter 1.5m and common depth 25-50m for bored pile. <strong>Barrette</strong> Limited in size and depth</td>
<td>Bored Pile Maximum Diameter 2.0m and common depth 25-60m <strong>Barrette</strong> Various sizes &amp; depth over 60m</td>
<td>Better equipment, operating skills as well as improved-knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Base grouting</td>
<td>Not available</td>
<td>Available</td>
<td>Equipment availability and advance technology</td>
</tr>
<tr>
<td>Application of polymer-based slurry (for bored pile only)</td>
<td>Not available</td>
<td>Extensive use</td>
<td>Material availability and research</td>
</tr>
<tr>
<td>Construction impact on quality and performance</td>
<td>Not well understood</td>
<td>Improving</td>
<td>Experience from past projects and research</td>
</tr>
<tr>
<td>Quality control in construction process</td>
<td>Not well established and systematic</td>
<td>Well established and systematic</td>
<td>Experience from past projects and research</td>
</tr>
<tr>
<td>Quality control test method and interpretation</td>
<td>Fewer methods available and limited knowledge in interpretation</td>
<td>Better equipment available and better knowledge in interpretation</td>
<td>Advance equipment, experience from past projects and research</td>
</tr>
</tbody>
</table>
Experience from past projects and extensive research works provided better understanding of construction impact of quality and performance of these wet-processed deep foundations. Marked difference between outcome quality of bored piles constructed in 1980s and 1990s can be observed by significant fewer defects found in the latter. It can also be observed from load test results that bored piles constructed in late 1990 have higher capacities than those of 1980s as shown in Table 5.

Table 5. Load test results of bored piles constructed in 1970 to 1980 and 1990s.

<table>
<thead>
<tr>
<th>Year</th>
<th>Project Name</th>
<th>Pile size (Dia. &amp; Depth)</th>
<th>Design Load (kN)</th>
<th>Test Load (kN)</th>
<th>δ\text{max} (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1971</td>
<td>Pinklao bridge</td>
<td>1.5 m x 45 m</td>
<td>4100</td>
<td>8200</td>
<td>5</td>
</tr>
<tr>
<td>1995</td>
<td>Central Plaza Pinklao Tower</td>
<td>1.2 m x 45 m</td>
<td>7000</td>
<td>14000</td>
<td>27</td>
</tr>
<tr>
<td>1980</td>
<td>Taiping Tower</td>
<td>1.0 m x 32 m</td>
<td>3000</td>
<td>11250</td>
<td>118</td>
</tr>
<tr>
<td>1990</td>
<td>High-rise building at Ekamai Rd.</td>
<td>1.0 m x 32 m</td>
<td>3900</td>
<td>9750</td>
<td>23</td>
</tr>
</tbody>
</table>

Bored piles have been extensively used for foundations of the majority of the elevated expressways since 1991. Thousands of base-grouted bored piles were constructed for these infrastructure projects including Second Stage Expressway, constructed in 1991; Don Muang Tollway Extension, constructed in 1997; Bangna-Bangpakong Expressway constructed in 1998; and Wat Nakorn-In project, constructed in 2001. Over 700 bored piles of diameter ranging from 0.8m to 1.5m with depth from 35m to 54m were constructed between 1999 and 2000 using polymer-based slurry for the foundation of Rama VIII Bridge, one of the initiatives of his majesty the King Bhumibol Adulyadej. Progress of piling and superstructure works of the Rama VIII Bridge at the bank of Chao Phraya river is depicted in Fig. 2(a) and 2(b).

The availability of more reliable and powerful equipment and tools for drilling makes it possible to construct the rock-socket bored piles by rotary drilling method. Fig. 3(a) and 3(b) show the drilling rig equipped with a rock auger used for construction of highway bridge across Mool River in 2001, in the north-eastern part of Thailand. Drilling was carried out through weathered-sandstone by powerful rotary drilling rig with core barrel and rock auger.

![Fig. 2a. Rama VIII cable-stayed bridge construction – more than 700 deep-seated bored piles were constructed using polymer-based slurry.](image1)

![Fig. 2b. Rama VIII cable-stayed bridge and connecting elevated flyovers.](image2)

![Fig. 3a. Rock-socket bored pile construction for Mool River Project Ubon Ratchathani Province](image3)

![Fig. 3b. Rock-auger for Mool River bored pile construction](image4)
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<td>8200</td>
<td>5</td>
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<td>1980</td>
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Fig. 3a. Rock-socket bored pile construction for Mool River Project Ubon Ratchathani Province

Fig. 3b. Rock-auger for Mool River bored pile construction

Bored pile of diameter 2m founded at 62m depth for the foundation of Rama 8 bridge, a cable-stayed bridge across Chaophaya river constructed in early 2005 was the largest deep-seated bored pile in Bangkok installed by rotary drilling method at that time. In the same project, both conventional static pile load tests and bi-directional load tests were carried out on 2m diameter bored piles. Simplified version of load vs. settlement of these pile tests are presented in Fig.4. Apparently, from the test results, the test piles could carry a significantly higher load than the original pre-defined safe design load of 1600 ton. Thasnanipan et al. (2006) reported the experiences gained in practical construction of these largest diameters bored piles at the river bank. Pictures of some activities during construction of bored piles in the said project are presented in Figs. 5(a), 5(b), 6(a), 6(b), 7(a) and 7(b).

Fig. 4. Load-settlement relationship of diameter 2.0m bored piles (conventional static pile load test and bi-directional load test)

Fig. 5a View of progress in temporary casing installation for bored pile diameter 2m at the bank of Chaophaya River

Fig. 5b View of progress of bored pile diameter 2m drilling at the bank of Chaophaya River

Fig. 6a View of progress in rebar cage preparation for bi-directional load test

Fig. 6b View of progress in installation of rebar cage

Fig. 7a View of progress in rebar cage installation

Fig. 7b View of progress in installation of pile cap
Deep-seated T-shape and L-shape barrettes were constructed as leg-piles for diaphragm wall in recently completed projects in Bangkok. Construction of cruciform barrette (shown in Fig. 8(a) and 8(b)) for monopole type high-voltage power transmission line was described by Thasnanipan et al., (2000a).

The availability of more powerful and efficient hydraulic grabs offers faster construction of deep-seated barrettes in comparison with mechanical cable-hung grabs. Fig. 9 shows the construction of barrettes using hydraulic grab.

Due to the extensive bearing capacity requirements, T-shape barrettes (Thickness=1m, Width =3m) of 55m deep were used in one of the recent major projects. View of the guide wall and reinforcement installation of T-shape barrette are shown in Fig. 10(a) and 10(b). Fig. 11 shows the T-shape barrettes after exposing to the cut-off level.

The above-mentioned works reflect the development and achievement history of bored piles and barrettes in Thailand and offered useful information to the local construction industry as well as to that of the international deep foundation construction.
7. IMPROVEMENT IN DESIGN CONCEPT AND PARAMETER'S SELECTION

In the initial stage of introducing bored piles in Thailand, the design concepts and parameters were mainly based on the available literature from research carried out in other parts of the world such as Tomlinson (1957), Skempton (1959), Broms (1966), Bowles (1968), Meyerhof (1976) etc. The work of Chiruppapa (1968) was believed to be the first research data available for design parameters of bored piles in Bangkok soft clay. The author conducted the study based on 6 dry-processed small diameter bored piles with load cells installed at the pile toes on AIT campus in Phathumthani. Though the piles were simply constructed using the dry-process with casing method, this early-stage study provided some important information such as load-settlement behaviour, adhesion factor ($\alpha$) and bearing capacity factor ($N$) of bored piles in Bangkok soft clay.

With the passage of time, design method and selection of parameters for local subsoil were improved as a result of research works carried out in 1980s. Differences between the behaviour of driven piles and bored piles were well realized from these studies. Ng (1983) presented the load distribution characteristics of wet-processed bored piles founded in both the first and second sand layers of Bangkok based on instrumented (strain gauges) pile load test results. Chiewcharnsilp (1988) reported the shaft friction factor $\beta$ values of sand layers in Bangkok based on the instrumented load test results. In addition to the literature previously available (Chiruppapa, 1968; Suwanakul, 1969; Promboom, 1981; Ng, 1983; Chiewcharnsilp, 1988), Pimpassugdi (1989) determined the shaft friction factor (adhesion factor, $\alpha$) of Bangkok clay layers based on 11 bored piles of diameters ranging from 0.50 to 0.80m with the embedded length varying between 21.50m and 46m. It should be noted that the design parameters obtained from the research works of 1980s were mainly based on the estimation of shaft friction loads from plain static pile load test results with numbers of assumptions since instrumented pile load test results were limited.

With the peak of construction-boom in Thailand, particularly in Bangkok, large numbers of instrumented full-scale static load tests on bored piles were conducted throughout 1990’s which provided better understanding on behavior of these deep-seated foundations. Researches focused on the design parameters and methods were published based on these test data. With improved design methods the wet-process cast-in-place foundations came to be regarded as reliable foundations in the construction industry of Thailand. With more confidence on soil parameters selection and better understanding on behaviour of these deep foundations, the designer designed higher load capacity bored piles and barrettes in late 1990 than those in 1980s. Thasnanipan et al., (1999) reported the failure mechanism of long bored piles in layered soil of Bangkok. The authors cited that for the bored piles embedded in the multi layered soils of Bangkok, estimation of ultimate shaft friction capacity needs to consider the brittle type of failure mechanism of stiff to hard clay layers and the $\alpha, \beta$ values selected need to be adjusted accordingly. Peak and residual $\alpha, \beta$ values mobilized in the stiff clay layers analyzed by the authors were plotted as shown in Fig. 12 along with the suggested curves by different researchers. It can be seen from the figure that the residual $\alpha$ value of 2nd stiff clay layer (undrained shear strength values of 25 ton/m2), at the maximum test load, drops below the curve suggested by Pimpassugdi (1989). So the $\alpha$ values proposed by the author in Fig.12 for the stiff to hard clay layers overestimate the ultimate shaft friction under these conditions.

Introduction of polymer-based slurry for wet-process bored piles marked a major breakthrough for both construction and design engineers. Thasnanipan et al.,(2002b), reported that bored piles constructed with polymer-based slurry have higher capacity than those constructed with bentonite slurry. Fig. 13 shows the shaft friction factors $\beta$ of sand layers for polymer-based bored piles in comparison with the design line of bentonite bored piles. The higher load capacity of bored piles constructed with polymer-based slurry allows the use of a single, large diameter deep-seated bored pile in place of a group of smaller size shallow-seated bored piles or driven piles. Fig. 14 depicted the static pile load test result of 1.80m diameter bored pile of 60m depth constructed with polymer-based slurry in urban area of Bangkok. The test was conducted in late 2003. The maximum applied load 48 000 kN in this test is believed to be the highest static pile load test performed on a single bored pile in Thailand. Apparently, from the test result, the pile (diameter 1.80m x 60m deep) could carry a significantly higher load than the original pre-defined safe design load of 16 000 kN.
Improvement of testing equipment and powerful computer for bored piles in depth 60m in Bangkok (after Thansnanipan et. al 2002b)

Adhesion Factor, $\alpha$

<table>
<thead>
<tr>
<th>$\alpha$</th>
<th>$0.20$</th>
<th>$0.40$</th>
<th>$0.60$</th>
<th>$1.20$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_s \tan \delta$</td>
<td>$50$</td>
<td>$30$</td>
<td>$20$</td>
<td>$10$</td>
</tr>
</tbody>
</table>

Fig. 12 Comparison of adhesion factor $\alpha$, suggested by different researchers with the actual mobilized in the stiff clay layers (after Thansnanipan et. al 1999)

Fig. 13 Back-calculated $\beta$ values of polymer bored piles at maximum test load plotted on design line of bentonite bored piles constructed in Bangkok subsoil (after Thansnanipan et. al 2002b)

Fig. 14 Static pile load test result of 1.80m diameter bored pile of depth 60m in Bangkok

8. DEVELOPMENT IN PERFORMANCE MONITORING AND QUALITY CONTROL TESTING

Improvement of testing equipment and powerful computer facilities are key factors that contributed to the development in performance monitoring and quality control testing. Interpretation skills relevant to local soil conditions and construction methods of these tests were significantly improved in local industry. For instance, in early 1980, the sonic integrity (seismic test) test results were needed to send to the specialists abroad for interpretation which in turn made the testing cost more expensive. Significant cost-saving were achieved in some major projects of late 1990s, as more practical and precise interpretation were locally made to verify and establish acceptance criteria in proving the quality of suspected piles with anomalies.

9. KODEN DRILLING MONITORING SYSTEM

In Thailand, prior to the availability of Koden testing equipment, borehole verticality was checked by mechanical type equipment such as plumb float and adjustable globe. However, the reliability of these methods was doubtful. The Koden drilling monitor system was believed to be first used in Thailand in 1980. The verticality of slurry-filled borehole can be monitored rapidly from the electronic plot of the continuous profile by Koden equipment. This system is very useful for the verticality-control of long barrettes - unlike bored piles, guiding by temporary casing is not available for barrette drilling.

10. SONIC INTEGRITY TEST

Sonic integrity test, also known as seismic test is the most common method of integrity testing for both driven and bored piles in Thailand. Sonic integrity test is usually selected for both quality check (control test) and retrospective investigation. It is the cheapest in terms of cost and the simplest in terms of testing process. The main advantage of this test is that since no particular preparation is necessary during the pile construction phase, it is more flexible to select which pile is to be tested. However, interpretation of sonic integrity testing needs considerable experience and knowledge in testing, subsoil condition and construction method. In many projects it is a part of the contractual requirement to conduct sonic integrity test. Minimum 10% to maximum 100% of production piles are commonly tested. It is also a reasonably acceptable method as a retrospective investigation in determining integrity of the pile. The signal characteristics and their interpretations of sonic integrity test on piles founded in Bangkok subsoil were reported in details by Thanasipan et al. (1998d).

11. CROSS-HOLE SONIC LOGGING TEST

The first sonic logging test was believed to be conducted in 1982 for the wet-processed bored piles of the Memorial Bridge Project where had concrete zones were detected at depth about 20m and 1m Ng, 1983. Use of the Sonic logging test for checking pile integrity has increased in Thailand particularly in mega projects. Cross-hole sonic logging test is relatively expensive. It is mainly employed as a pre-planned site quality control testing. The major advantage of this method is that test can be carried out shortly after pile construction. Hence, rectification measures can be implemented if the pile is defective while the foundation contractor is on site. However, this method is generally not
applicable if pile integrity is in question due to post-construction activities, as access tubes are usually grouted after completion of the test. The results from sonic logging test conducted on model piles in Bangkok helped to extend the knowledge of the signal characteristics and interpretation (Thasnanipan et. al., 2000b). Thasnanipan et al., (2004b) also demonstrated the application of cross-hole sonic logging test in identifying over-cast length of bored piles prior to exposing the pile heads.

12. HIGH STRAIN DYNAMIC LOAD TEST

High strain dynamic integrity test has become a well-accepted method especially for evaluating the pile capacity in today’s foundation industry of Thailand and it is applied for both driven and bored piles. A large number of related technical papers and case histories of the test have been published and it is a part of standards and specifications such as ASTM D4945-89 (Standard Method for High-strain Dynamic Testing of Piles). Thasnanipan et al. (2000b), reported the application of dynamic load testing on piles in Thailand.

13. FUTURE TRENDS OF CAST-IN-PLACE DEEP FOUNDATIONS IN THAILAND

As the practitioners in construction industry have gained more experience and confidence in using higher capacity bored piles and barrettes, these types of deep foundations are expected to be of more in demand in the future. The higher load capacity achievable by deep-seated large bored piles and barrettes will allow the use of a single foundation element in place of a group of smaller size shallow-seated bored piles and driven piles. It is expected that improvement in installation techniques will be of key advance in the future of deep foundation industry in Thailand.

It is anticipated that barrettes will be more popular in the future for the following reasons.

− The availability of more powerful and efficient hydraulic grabs which will offer faster construction of deep-seated barrettes
− Less noise which will offer main advantages in congested and sensitive neighbourhood
− Less equipment requirement that will offer advantageous construction in limited space
− More versatility (e.g. able to construct in the area with limited headroom)
− Better in safety aspects as less equipment are required (especially for the construction along the public roads, subway stations, elevated expressway etc.)

14. CONCLUDING REMARKS

In Thailand, according to the authors’ experience as a deep-foundation contractor, development and achievement in both construction and design aspects of wet-process deep bored piles and barrette foundations in the past four decades were significant. With recognition of technical and economic advantages of using these high capacity cast-in-place foundations by local practitioners, it is expected that they will be more popular in the future construction industry of Thailand. However, in the authors’ opinion, there is much work to be done with particular focus on constructability issues, concrete technology for wet-processed bored piles and barrette, reliable but cost-effective quality control testing and value-engineering.

Starting from the planning stage, site investigation, design, construction and inspection should be integrated so that designers, contractors and construction inspectors can participate as a team with a common goal. Appropriate and practical specifications should be established jointly by these parties for local soil conditions and construction methods. Practical acceptance criteria should be developed to verify bored piles and barrettes with suspected anomalies. Continuing education should be promoted for designers, inspection engineers, and contractors. The Geotechnical Chapter of the Engineering Institute of Thailand under the royal patronage of his majesty King Bhumibol Adulyadej, has started to establish the standard code of practice and guidelines for wet-processed bored piles which will serve as a yardstick for the deep foundation industry upon its completion in the near future.

REFERENCES

Bowles, J. E. 1968. Foundation analysis and design, McGRav-Hill, New York
Ng, K. C. 1983. The construction problem and performance of large bored piles in second sand layer, M. Eng. Thesis No. GT 82-26, Asian Institute of Technology, Bangkok, Thailand


Geotechnical and Foundation Conference 2012

Barrette for Highrise Buildings and Heavy Structures in Bangkok Subsoil

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Based on the authors' experiences, this paper is intended to summarize the development of deep-seated barrettes construction in Thailand, in comparison with that of the past. Literature related to wet-processed bored piles and barrettes in Thailand published throughout the past 30 years is summarized together with recent research works.

Results of the static load test performed on barrettes in three projects are summarized. Increasingly use of barrettes for the highrise building projects is described. Recent application of heavy structures barrettes for elevated mass rail transport project, MRT Purple Line is also presented. An attempt is also made to briefly discuss the future trend of deep foundations in Thailand.

INTRODUCTION

During the past three decades, owing to the acceleration of development in mega infrastructure projects, techniques and practice of cast-in-place deep foundations, particularly wet-processed deep-seated large diameter bored piles and barrettes have experienced enormous progress in Thailand. The versatility of the construction method and the high-load capacity which in turn offered the constructability and cost-saving are the main factors contributed to the increasing use of those cast-in-place deep foundations. In the initial stage, there were a number of questions on design and construction aspects of these foundation systems particularly in Bangkok subsoil. With the passage of time, the construction equipment, installation techniques and testing of deep foundation elements have been developed. Large numbers of instrumented full-scale static pile load tests were conducted throughout the 1990s mainly in mega projects, which provided better understanding on behavior of these deep-seated foundations.

Research focused on the design parameters and methods were produced based on these test results. The design parameters became so well established that wet-process cast-in-place foundations became regarded as reliable foundations for practitioners involved in construction industry in Thailand. This paper focuses the past and current practice of barrettes in Thailand together with a brief discussion on future trend deep foundations in Bangkok subsoil. The information contained in the paper is mainly from the highrise buildings and infrastructure projects constructed in Bangkok and adjacent areas.
Barrette for Highrise Buildings and Heavy Structures in Bangkok Subsoil

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SUBSOIL AND EXISTING PIEZOMETRIC PROFILE

Subsoil profile and the present piezometric drawdown condition of Bangkok are presented in Figure 1 below. A typical subsoil profile is relatively consistent in different localities in Bangkok. It is characterized by alternating layers of clay and sand deposits as shown in Figure 1.

![Figure 1](image_url)

**Geotechnical Parameters**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Pore Pressure (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>Su= 10-25 KPa</td>
</tr>
<tr>
<td></td>
<td>γt = 14-16kN/m³</td>
</tr>
<tr>
<td>20</td>
<td>Su= 50-140 KPa</td>
</tr>
<tr>
<td></td>
<td>γt = 17-21kN/m³</td>
</tr>
<tr>
<td>30</td>
<td>Su&gt; 200 KPa</td>
</tr>
<tr>
<td></td>
<td>γt = 21kN/m³</td>
</tr>
<tr>
<td>40</td>
<td>SPT-N= 20-40</td>
</tr>
<tr>
<td></td>
<td>γt = 20kN/m³</td>
</tr>
<tr>
<td>50</td>
<td>SPT-N= 50-100</td>
</tr>
<tr>
<td></td>
<td>γt = 21kN/m³</td>
</tr>
</tbody>
</table>

**Figure 1.** Typical soil profile of Bangkok with piezometric drawdown condition (Thasnanipan, et. al., 2002)

BARRETTE CONSTRUCTION METHOD IN BANGKOK SUBSOIL

Due to the prevailing subsoil and groundwater conditions, barrettes are constructed by wet-process or slurry displacement method.

Both mechanical and hydraulic cable-suspended grab are used for barrette construction. In recent years, use hydraulic grabs are more common. Excavation of the trench is carried out by the cyclic-process of lifting and lowering of the grab under gravity and tangential force of the clamshell operated by cables (mechanical grab) or hydraulic action (hydraulic grab). Different from bored pile construction, a guide wall of depth 1 to 1.5 m with inside clear dimensions slightly larger than the nominal size of the barrette is used to guide the grab during initial bites. Bentonite slurry is introduced to the trench as soon as the initial excavation commenced. The excavation is continued under the bentonite slurry to the final depth. After recycling of slurry and lowering the rebar cage, concreting is done by tremie method.
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OVERVIEW OF BARRETTE APPLICATION IN CONSTRUCTION PROJECTS

In some projects where bored piles were not feasible due to the site constraints, applicable construction methods, and/or extensive bearing capacity requirements, the use of barrette foundations would make a suitable alternative. Barrettes with dimensions ranging from 0.80 m x 2.7 m to 1.5 m x 3.0 m for safe working load capacity from 11,000 to 23,000 kN have been used in some major projects.

The first barrette in Bangkok was believed to be constructed in the late 1970s for the foundation of the Bangkok Bank Head Office Building at Silom road with size of 0.6-0.8 x 2.5m and toe depth of 33m. Barrettes can be constructed with flexible layout plans for both vertical and lateral loads. The layout pattern of barrettes can be arranged in a continuous row or column, radial, alternating long and short axis of barrette and a combination of two or more of such patterns. To minimize the need of additional equipment, a mechanical or hydraulic grab mounted on crawler crane can be used for constructing both barrettes (as piled-foundation) and diaphragm walls (as earth retaining structure) in some suitable projects without employing a bored piling rig, which could save the extra mobilization cost. In addition to the large bearing capacity requirements, on site difficulties such as limited head room where piling rigs cannot be utilized, under such situations like presence of overhead high voltage power cables, existing overpasses or structures for elevated expressways and planned subway stations, also demands the barrettes. Static load test up to 52,900 KN conducted on barrette set the record as the highest load ever tested for a single barrette foundation in Thailand (Thasnanipan et. al. 2002a). A summary of completed projects constructed with barrettes with respect to selection criteria in 2003 in comparison with 2012 is presented in Table 1. As can be seen in the table, significant increase of barrette foundation can be realized particularly in the project with high load capacity requirement. It is also evident that barrette in combination with diaphragm wall is increasingly popular in the construction industry of Thailand.
Table 1 Summary of Barrette Selection in major projects

<table>
<thead>
<tr>
<th>Selection Criterion</th>
<th>Accumulated number of projects</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>as of 2003</td>
<td>as of 2012 August</td>
</tr>
<tr>
<td>High Load Capacity</td>
<td>4</td>
<td>21</td>
</tr>
<tr>
<td>Minimize Construction Equipment</td>
<td>6</td>
<td>14</td>
</tr>
<tr>
<td>Limited Head Room for Excavation</td>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>Combination with Diaphragm Wall</td>
<td>10</td>
<td>25</td>
</tr>
<tr>
<td>Foundation as well as portion of column</td>
<td>3</td>
<td>4</td>
</tr>
</tbody>
</table>

General configuration of barrettes as leg piles integrated with diaphragm wall and bored piles of building with deep basement is shown in Figure 3.

Figure 3 Schematic of barrettes as leg piles integrated with diaphragm walls and bored piles

PROBLEMS AND DIFFICULTIES IN CONSTRUCTION OF BARRETTES IN EARLY DAYS

Basic but extensive problems were experienced in early stages of barrette construction in Thailand as summarized below.

- Limited availability and capacity of equipment
- Lack of skills in operation of equipment (particularly the drilling rig)
- Adverse effect due to a slow rate of drilling (e.g. excessive formation of filter-cake by using bentonite slurry)
- Limited knowledge and less advanced techniques in control of bentonite slurry
- Limited experience in construction method and related negative impact
- Quality of concrete for tremie concreting method
- Lack of experienced engineers and foremen
- Improper construction and quality control specifications and guidelines for deep-seated piles in local soil
- Improper design for constructability

**IMPROVEMENT IN CONSTRUCTION OF BARRETTE**

Over the past three decades, along with the development of wet-processed piling technology in other parts of the world, equipment, construction technique and design methods as well as better understanding of construction impact on the performance of this type of foundation have significantly improved in Thailand. Table 2 summarizes the areas of improvement in bored pile and barrette construction and main factors contributed to these developments. Some of these improvements were published both locally and internationally. The design, construction and behavior of bored cast in-situ concrete piles in Bangkok Subsoil was presented by Thasnanipan et. al., (1998a). The construction and performance of barrettes in Bangkok Subsoil was also reported by Thasnanipan et. al., (1998b). Effect of construction time and bentonite viscosity on shaft friction of bored piles was also pointed out by Thasnanipan et al., (1998c). Thasnanipan et. al., (2004a) also reported a comprehensive study on the effectiveness of two different toe-grouting methods, known as tube-à-manchette and drill-and-grout, applied in Bangkok.

**Figure 4.** (a) Guide wall for cruciform barrette  (b) View of cruciform barrette after installation of reinforcement

Construction of cruciform barrette (shown in Figure 4) for monopole type high-voltage power transmission line was described in the work of Thasnanipan et. al., (2000a). Construction of deep-seated T-shape and L-shape barrettes as leg-piles for diaphragm wall are now more common in Bangkok.
Above-mentioned research works reflect the development history of bored pile and barrettes in Thailand and offered useful information to the local construction industry and perhaps to the international deep foundation engineering society.

**Table 2** Summary of development in barrette construction

<table>
<thead>
<tr>
<th>Area of development / improvement</th>
<th>In the past (Early 1980’s)</th>
<th>At present</th>
<th>Main factor contributed to development</th>
</tr>
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<tbody>
<tr>
<td>Speed of construction</td>
<td>Minimum 3 days required to complete barrette 1.5mx3m of toe depth 57m</td>
<td>Less than 2 day to complete barrette 1.2mx3m toe depth 60m</td>
<td>Better equipment, operating skills as well as improved-knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Pile size and depth</td>
<td>Barrette Limited in size and depth</td>
<td>Barrette Various sizes &amp; depth over 60m</td>
<td>Better equipment, operating skills as well as improved-knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Base grouting</td>
<td>Not available</td>
<td>Available</td>
<td>Equipment availability and advance technology</td>
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<tr>
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<td>Not well understood</td>
<td>Improving</td>
<td>Experience from past projects and research</td>
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<tr>
<td>Quality control in construction process</td>
<td>Not well established and systematic</td>
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<td>Quality control test method and interpretation</td>
<td>Fewer methods available and limited knowledge in interpretation</td>
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Experience from past projects and extensive research works provided better understanding of construction impact of quality and performance of these wet-processed deep foundations.

The availability of more powerful and efficient hydraulic grabs offers faster construction of deep-seated barrettes in comparison with mechanical cable-hung grabs. Figure 5 shows the construction of barrettes using hydraulic grab.
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**IMPROVEMENT IN DESIGN CONCEPT AND PARAMETERS SELECTION**

Usually the designers use the same design concept and parameters for barrettes and bored piles constructed under bentonite slurry. Following paragraphs describe the design practice of both elements commonly applied in the past and improvement with time.

In the initial stage of introducing bored piles in Thailand, the design concepts and parameters were mainly based on the available literature from research carried out in other parts of the world such as Tomlinson (1957), Skempton (1959), Broms (1966), Bowles (1968), Meyerhof (1976) etc. The work of Chiruppapa (1968) was believed to be the first research data available for design parameters of bored piles in Bangkok soft clay. The author conducted the study based on 6 dry-processed small diameter bored piles with load cells installed at the pile toes on AIT campus in Phathumthani. Though the piles were simply constructed using the dry-process with casing method, this early-stage study provided some important information such as load-settlement behavior, adhesion factor ($\alpha$) and bearing capacity factor (N) of bored piles in Bangkok soft clay.

With the passage of time, design method and selection of parameters for local subsoil were improved as a result of research works carried out in 1980s. Differences between the behavior of driven piles and bored piles were well realized from these studies. Ng (1983) presented the load distribution characteristics of wet-processed bored piles founded in both the first and second sand layers of Bangkok based on instrumented (strain gauges) pile load test results. Chiewcharnsilp (1988) reported the shaft friction factor $\beta$ values of sand layers in Bangkok based on the instrumented load test results. In addition to the literature previously available (Chiruppapa, 1968; Suwanakul, 1969; Promboon, 1981; Ng, 1983; Chiewcharnsilp, 1988), Pimpasugdi (1989) determined the shaft friction factor (adhesion factor, $\alpha$) of Bangkok clay layers based on 11 bored piles of diameters ranging from 0.50 to 0.80m with the embedded length varying between 21.50m and 46m. It should be noted that the design parameters obtained from the research works of 1980s were mainly based on the estimation of shaft friction loads from plain static pile load test results with numbers of assumptions since instrumented pile load test results were limited.
With the peak of construction-boom, large numbers of instrumented full-scale static load tests on bored piles and a few barrettes were conducted throughout 1990’s which provided better understanding on behavior of these deep-seated foundations. Researches focused on the design parameters and methods were published based on these test data. With improved design methods the wet-process cast-in-place foundations came to be regarded as reliable foundations in the construction industry of Thailand. With more confidence on soil parameters selection and better understanding on behavior of these deep foundations, the designer designed higher load capacity bored piles and barrettes in late 1990 than those in 1980s. Thansnanipan et. al., (1999) reported the failure mechanism of long bored piles in layered soil of Bangkok. The authors cited that for the bored piles embedded in the multi layered soils of Bangkok, estimation of ultimate shaft friction capacity needs to consider the brittle type of failure mechanism of stiff to hard clay layers and the $\alpha$ values selected need to be adjusted accordingly. Peak and residual $\alpha$ values mobilized in the stiff clay layers analyzed by the authors were plotted as shown in Figure 10 along with the suggested curves by different researchers. It can be seen from the figure that the residual $\alpha$ value of 2$^{nd}$ stiff clay layer (undrained shear strength values of 25 ton/m$^2$), at the maximum test load, drops below the curve suggested by Pimpasugdi (1989). So the $\alpha$ values proposed by the author in Figure 6 for the stiff to hard clay layers overestimate the ultimate shaft friction under these conditions.

**Figure 6** Comparison of adhesion factor $\alpha$, suggested by different researchers with the actual mobilized in the stiff clay layers (after Thansnanipan et. al 1999)
Adhesion Factor, $\alpha$

Tomlinson, 1957
Stas and Kulhawy, 1984
Suchada, 1989
for bored piles in Bangkok soils

Bored Cast Insitu Piles

Peak
Residual

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Figure 7 Back-calculated $\beta$ values of polymer bored piles at maximum test load plotted on design line ($\beta$ values in sand layer) of bentonite bored piles based on back-calculated (after Thansnanipan et. al 2002b)

DEVELOPMENT IN PERFORMANCE MONITORING AND QUALITY CONTROL TESTING OF BARRETTES

Improvement of testing equipment and powerful computer facilities are key factors that contributed to the development in performance monitoring and quality control testing. Interpretation skills relevant to local soil conditions and construction methods of these tests were significantly improved in local industry. For instance, in early 1980, the sonic integrity (seismic test) test results were needed to send to the specialists abroad for interpretation which in turn made the testing cost more expensive. Significant cost-saving were achieved in some major projects of late 1990, as more practical and precise interpretation were locally made to verify and establish acceptance criteria in proving the quality of suspected piles or barrettes with anomalies.

Koden drilling monitoring system

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Cross-hole sonic logging test

The first sonic logging test was believed to be conducted in 1982 for the wet-processed bored piles of the Memorial Bridge Project where bad concrete zones were detected at depth about 20m and 1m Ng, 1983. Use of the Sonic logging test for checking pile integrity has increased in Thailand particularly in mega projects. Cross-hole sonic logging test is relatively expensive. It is mainly employed as a pre-planned site quality control testing. The major advantage of this method is that test can be carried out shortly after pile construction. Hence, rectification measures can be implemented if the pile is defective while the foundation contractor is on site. However, this method is generally not applicable if pile integrity is in question due to post-construction activities, as access tubes are usually grouted after completion of the test. The results from sonic logging test conducted on model piles in Bangkok helped to extend the knowledge of the signal characteristics and interpretation (Thasnanipan et. al., 2000b). Thasnanipan et. al., (2004b) also demonstrated the application of cross-hole sonic logging test in identifying over-cast length of bored piles prior to exposing the pile top.

High strain dynamic load test

High strain dynamic integrity test has become a well-accepted method especially for evaluating the pile capacity in today’s foundation industry of Thailand and it is applied for both driven and bored piles. A large number of related technical papers and case histories of the test have been published and it is a part of standards and specifications such as ASTM D4945-89 (Standard Method for High-strain Dynamic Testing of Piles). Thasnanipan et. al. (2000b), reported the application of dynamic load testing on piles and barrettes in Thailand.

STATIC PILE LOAD TESTS ON BARRETTES

Figure 8, 9 and 10 show the load vs. pile head movement and load-transfer mechanism of two standard rectangular-shape barrettes and T-Shape barrette respectively.
Sonic integrity test

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Figure 8 (a) Load vs. pile head movement and (b) load-transfer mechanism of base-grouted barrette 1.50x3.00m founded at 57.5 m in Bangkok.
Table 3 Summary of load-settlement curve from three case studies

<table>
<thead>
<tr>
<th>Description</th>
<th>Test No. 1</th>
<th>Test No. 2</th>
<th>Test No. 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimension</td>
<td>1.50m x 3.00m</td>
<td>1.2m x 3.0m</td>
<td>1.0m x 2.0m x 3.0m (T)</td>
</tr>
<tr>
<td>Tip Level (m)</td>
<td>-57.5</td>
<td>-65.0</td>
<td>-55.0</td>
</tr>
<tr>
<td>Shaft friction area (m²)</td>
<td>518</td>
<td>546</td>
<td>770</td>
</tr>
<tr>
<td>End Bearing area (m²)</td>
<td>4.50</td>
<td>3.60</td>
<td>5.00</td>
</tr>
<tr>
<td>Slurry Used</td>
<td>Bentonite</td>
<td>Bentonite</td>
<td>Bentonite</td>
</tr>
<tr>
<td>Total sand layer thickness along the shaft (m)</td>
<td>18</td>
<td>33</td>
<td>6</td>
</tr>
<tr>
<td>Design load (ton)</td>
<td>2,300</td>
<td>2,500</td>
<td>2,600</td>
</tr>
<tr>
<td>Maximum pile head movement at maximum applied load (mm)</td>
<td>62</td>
<td>25</td>
<td>28</td>
</tr>
<tr>
<td>Load at 5mm pile head movement (ton)</td>
<td>2,515</td>
<td>2,470</td>
<td>2,600</td>
</tr>
<tr>
<td>Load at 20mm pile head movement (ton)</td>
<td>5,290</td>
<td>6,100</td>
<td>5,270</td>
</tr>
<tr>
<td>Load at fully mobilized shaft friction (ton)</td>
<td>5,060</td>
<td>6,470</td>
<td>4,600</td>
</tr>
<tr>
<td>Pile head movement at fully mobilized shaft friction (mm)</td>
<td>20</td>
<td>24</td>
<td>17</td>
</tr>
<tr>
<td>Unit end bearing at maximum test load (ton/m²)</td>
<td>70b</td>
<td>6b</td>
<td>130b</td>
</tr>
</tbody>
</table>

Note: a. Assumed shaft friction fully mobilized at plunging point of load-settlement curve
b. Unit end bearing not fully mobilized

Though it is not advisable to make major conclusions from 3 barrette tests, following summary can be drawn:

- Barrette head displacement (settlement) at design load is less or equal to 5mm
- Barrette head displacement (settlement) at assumed fully mobilized shaft friction is in the order of 1.3 to 2% of shorter width of barrette (thickness)
- End bearing does not contribute to the ultimate capacity of barrette
- Thickness of sand layer along the pile shaft may have influence on the barrette shaft friction capacity (as can be seen in Test No. 2 where sand layer thickness along the shaft is significantly thicker than other 2 test barrettes)

BARRETTES FOR HIGH-RISE BUILDING PROJECTS

More use of barrettes for highrise buildings can be observed in recent years. Points below are considered key factors contributed to increasing use of barrettes for highrise buildings particularly in congestive urban area.

- Buildings foot-prints are relatively small in comparison with the heights in majority of the highrise buildings. Hence higher load capacity foundation is required within available area
- Equipment utilized for barrette construction are relatively quiet and less vibration so that disturbance to the neighbouring area is less than that of piling equipment

Figure 9 Load vs. pile head movement of barrette 1.20x3.00m founded at 65 m in Bangkok

Figure 10 Load vs. pile head movement of T-shape barrette (1.00x2.00x3.00m) founded at 55m in Bangkok (constructed in tested in 2008)
Table 3 Summary of load-settlement curve from three case studies

<table>
<thead>
<tr>
<th>Description</th>
<th>Test No. 1 (1.5m x 3.00m) Base-grouted</th>
<th>Test No. 2 (1.2m x 3.00m) Barrette Test 2</th>
<th>Test No. 3 T-shape barrette (1.0m x 2.0m x 3.0m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimension</td>
<td>1.50m x 3.00m</td>
<td>1.2m x 3.00m</td>
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<td>Bentonite</td>
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<tr>
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<td>18</td>
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</tr>
<tr>
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<td>5,060</td>
<td>6,470</td>
<td>4,600</td>
</tr>
<tr>
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<td>24</td>
<td>17</td>
</tr>
<tr>
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- Equipment utilized for barrette construction are relatively quiet and less vibration so that disturbance to the neighbouring area is less than that of piling equipment
- Less equipment required for barrette construction – one set of barrette excavation equipment can be used for construction of barrette and diaphragm wall for the project with limited space
- Removal of excavated soil from the site can be easier as barrette grab can dispose directly into the dump trucks without needing to store on site provided that dumping trucks are able to move in and out from the site during excavation time (important issue in most of the congestive area where traffic restriction is stringent)

**Figure 11** Mat foundation of a high-rise building supported by barrette and bored pile

Figure 11 shows an example of mat foundation of a high-rise building supported by combination of barrette and bored pile. As explained earlier, one of major benefits of using barrette to support high-rise building is larger load bearing capacity of barrette compared with that of bored pile. Barrette can be designed for the high load concentration area while bored pile can be used in less concentrated load area as shown in the figure. In addition, barrette provides a versatile option to be constructed in various shapes such as T-shape (Fig. 10), X-shape or cruciform (Fig. 4) and L-shape to meet requirement of load bearing and bending moment capacity.

**BARRETTEs FOR HEAVY STRUCTURES - ELEVATED TRAIN PROJECT**

Bored piles have been extensively used for foundations of the majority of the elevated expressways since 1991. Thousands of base-grouted bored piles were constructed for these infrastructure projects including Second Stage Expressway, constructed in 1991; Don Muang Tollway Extension, constructed in 1997; Bangna-Bang Pli-Bangprakong Expressaway constructed in 1998; and Wat Nakorn-In project, constructed in 2001. Over 700 bored piles of diameter ranging from 0.8m to 1.5m with depth from 35m to 54m were constructed between 1999 and 2000 using polymer-based slurry for the foundation of Rama VIII Bridge, one of the initiatives of his majesty the King Bhumibol Adulyadej.

However, in 2010, large numbers of barrettes were constructed for new MRT project called Purple Line. Originally, the majority of foundations to support heavy elevated-
structures were designed for large bored piles of diameter 1m, 1.2m and 1.5m depending on
the loading requirement. After awarding the contracts to the main contractors, the value
engineering option using barrettes to replace bored piles proposed by the piling contractor
was accepted (as shown in Figure 12 and 13). Apart from the safety aspects, the key reasons
of changing from bored piles to barrettes are that (i) construction of barrettes requires less
equipment on the middle of the congestive road than bored piles (ii) pile cap (footing) for
barrette is much smaller than that of bored pile. Construction of over 1000 barrettes of
different sizes, 1x3m, 1.2x3m and 1.5x3m with toe depth ranging from 50m to 65m were
completed in the beginning of 2012.

Figure 12. Schematic of bored piles (original design) and proposed option of barrettes with
equivalent capacity– size of footing for barrette (20m2 in plan area) is significantly less than
that of bored pile (60m2 in plan area)

Figure 13. Construction of barrette for Elevated Train Project in Bangkok

Comparison of equipment and construction aspects bored pile and barrette is shown in Table
4 below.
Table 4. Comparison of equipment and construction aspects of barrette and bored pile for construction on the public roads

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Barrette</th>
<th>Bored Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Total high-tech equipment requirement</td>
<td>High</td>
<td>Medium</td>
</tr>
<tr>
<td>2</td>
<td>Experience and skill of operation crew</td>
<td>High</td>
<td>Medium</td>
</tr>
<tr>
<td>3</td>
<td>Risk involved (safety aspect)</td>
<td>Medium</td>
<td>High</td>
</tr>
<tr>
<td>4</td>
<td>Cost of equipment</td>
<td>High</td>
<td>Medium</td>
</tr>
<tr>
<td>5</td>
<td>Area requirement on road</td>
<td>Less</td>
<td>More</td>
</tr>
<tr>
<td>6</td>
<td>Site and road cleaness</td>
<td>Excellent</td>
<td>Average</td>
</tr>
<tr>
<td>7</td>
<td>Traffic problem due to method of soil disposal</td>
<td>Less</td>
<td>More</td>
</tr>
</tbody>
</table>

LATER LOAD TEST ON T-SHAPE BARRETTE AND DIAMETER 2M BORED PILE

The comparison of field test results and 3D finite element analysis on full-scale instrumented static lateral load tests on T-shape barrettes (Figure 14) and bored piles constructed in Bangkok were presented by Submaneeewong C (2009). Figure 15 presents the Load-movement graphs of barrettes and bored piles under lateral loading. The findings reported by the authors provide good reference for future design of similar elements subjected to lateral loading, particularly result of stiffness ratio vs shear strain level of barrette and bored pile as can be seen in Figure 16.

Figure 14. Lateral load test configuration for T-shape barrettes and instrumentation details
Figure 15. Movement at pile head under lateral load of barrettes and bored piles

Figure 16. Strain level of T-barrette and bored pile (reported by Submaneewong 2009) plotted on stiffness ratio vs shear strain for Bangkok Soft Clay reported by Prinzl and Davies (2006)
FUTURE TRENDS OF CAST-IN-PLACE DEEP FOUNDATIONS IN THAILAND

As the practitioners in construction industry have gained more experience and confidence in using higher capacity bored piles and barrettes, these types of deep foundations are expected to be of more demanding in the future. The higher load capacity achievable by deep-seated large bored piles and barrettes will allow the use of a single foundation element in place of a group of smaller size shallow-seated bored piles and driven piles. It is expected that improvement in installation techniques will be of key advance in the future of deep foundation industry in Thailand.

It is anticipated that barrettes will be more popular in the future for the following reasons.

- The availability of more powerful and efficient hydraulic grabs which will offer faster construction of deep-seated barrettes
- Less noise which will offer main advantages in congestive and sensitive neighbourhood
- Less equipment requirement which will offer advantages construction in limited space
- More versatility (e.g. able to construct in the area with limited headroom)
- Better in safety aspects as less equipment are required (especially for the construction along the public roads, subway stations, elevated expressway etc.)

CONCLUDING REMARKS

In Thailand, according to the authors’ experience as a deep-foundation contractor, development in both construction and design aspects of wet-process barrette foundations in past decades were significant. With recognition of technical and economic advantages of using these high capacity cast-in-place foundations by local practitioners, it is expected that they will be more popular in the future construction industry of Thailand. However, in the authors’ opinion, there is much work to be done with particular focus on constructability issues, concrete technology for wet-processed bored piles and barrette, reliable but cost-effective quality control testing and value-engineering.

Starting from the planning stage, site investigation, design, construction and inspection should be integrated so that designers, contractors and construction inspectors can participate as a team with a common goal. Appropriate and practical specifications should be established jointly by these parties for local soil conditions and construction methods. Practical acceptance criteria should be developed to verify bored piles and barrettes with suspected anomalies. Continuing education should be promoted for designers, inspection engineers, and contractors.

REFERENCES


REFERENCES


Submaneewong C. (2009) Behavior of T-shape barrette and bored pile subject to Vertical and Lateral Loading, Ph.D Dissertation, Chulalongkorn University, Bangkok, Thailand


Performance of a Braced Excavation in Bangkok Clay, Diaphragm Wall Subject to Unbalanced Loading Conditions

N. Thasnanipan, A. W. Maung, P. Tanseng and S.H. Wei
Seafco Co., Ltd., Bangkok, Thailand

Sponsored by;

Chinese Institute of Civil and Hydraulic Engineering
Southeast Asian Geotechnical Society
Performance of A Braced Excavation in Bangkok Clay,
Diaphragm Wall Subject to Unbalanced Loading Conditions

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S.H. WEI   SEAFCO Co., Ltd., Bangkok, Thailand

SYNOPSIS 0.8m thick cast-in-situ diaphragm wall having toe depth of 28m with two level temporary bracing was used to construct the basements of a structure which is located near the river and surrounded by buildings, including a historical one in Bangkok, Thailand. Due to the site condition, unbalanced lateral loading on the wall was expected and an excavation down to -12.7m below the existing ground level was carried out with instrumentation, consisting of (8) inclinometer tubes installed in the wall panels, settlement plates around excavation zone and tiltmeters and beam sensors on the existing structures. This paper presents computer model analysis and performance of the wall including results of instrumentation. Behaviour and performance of the wall is compared with those of other projects in Bangkok area.

INTRODUCTION

In Bangkok, the growing land price and need for space has necessitated deeper and larger basement excavations, even in some unfavorable subsoil and site conditions and in limited spaces. Subsoil conditions in Bangkok is generally a very soft clay of 12m to 18m thick layer underlain by stiff to hard clay and series of sand layers. Excavation in such soft soil requires efficient retaining structures and cast-in-situ diaphragm walls have therefore come in use frequently. This paper presents performance of a bracing excavation with diaphragm wall adjacent to the river and surrounding structures, including a historical building. This historical building, having archeological and cultural values, not only limited the height of the building but also influenced the construction time. Since the location of planned building is in a current limited height zone in which up to 4 storey height is permitted, 3 level basement was included to increase the usable floor area.

SITE CONDITION AND SUBSOIL

The building site is located nearby the Chao Phaya River and surrounded by a historical building and other existing structures supported by pile foundation (Fig. 1). Accurate information on foundation of these existing buildings was not available. The diaphragm wall along the river (D1) was constructed about only 4.0m away from the existing old river wall. The river bed near the river wall is about 2.2m-

Fig. 1  Layout of project site and instrumentation

3.0m in depth, sloping towards the mid-stream to a depth of about 10m to 12m. River water level is about 1.6m below the ground level during dry season and sometimes in the rainy season is above the ground level, causing inundation.

A primary site investigation was carried out by drilling two boreholes and two field vane shear tests. Prior to design for bracing and basement excavation, drilling of additional two boreholes and two field vane shear tests were performed to check variability of subsoil conditions. The subsoil
properties obtained from the boreholes and test data were summarized in Table 1.

Table 1. Summary of subsoil properties

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Layer Top in Depth m</th>
<th>W %</th>
<th>$\gamma_s$ kN/m$^3$</th>
<th>$c_v$ kPa</th>
<th>SPT N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft Clay</td>
<td>0.3</td>
<td>35-78</td>
<td>16-19</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>Medium Clay</td>
<td>7.2</td>
<td>30</td>
<td>19</td>
<td>71</td>
<td></td>
</tr>
<tr>
<td>Stiff Clay</td>
<td>14</td>
<td>22-34</td>
<td>19-21</td>
<td>43-300</td>
<td>14-52</td>
</tr>
<tr>
<td>Dense Sand</td>
<td>25</td>
<td>14-25</td>
<td>20-23</td>
<td>-</td>
<td>&gt;39</td>
</tr>
</tbody>
</table>

DIAPHRAGM WALL

A 800mm thick diaphragm wall was designed for excavation down to 12.7m below the ground with two levels of temporary bracing. The diaphragm wall toe was embedded down to 28m to achieve the overall stability of the excavation which is right on the river bank (Fig. 2).

Originally excavation stages were modeled by using a one-dimensional finite element computer program (Nonlinear Beam Column Analysis). Soil elements were modeled as spring and wall elements were modeled as beam. Four types of wall were designed to suit the temporary construction stages and permanent conditions. The walls were reinforced to withstand bending stresses up to 1000 kN.m/m in vertical direction. The maximum movement of the wall was expected to be 24.2mm.

For the wall sections adjacent to floor slab openings, especially in the water storage area, additional reinforcements were provided for bending stress in horizontal direction.

PILE FOUNDATIONS

A total of 165 bored piles consisting of 0.8m, 1.0m, 1.2m and 1.5m in diameter being founded in depth of 48m was constructed to support the structure. Out of 165 piles, 5 numbers of 0.8m diameter piles and 8 numbers of 1.0m diameter piles were incorporated with diaphragm wall panels as a leg to carry the load transferred through the wall.

INSTRUMENTATION

Due to the locality and particular conditions of the site, more instrumentation than those used by other projects in Bangkok were installed and monitored. Three types of instrumentation - Eight inclinometer tubes were installed in diaphragm wall panels to monitor displacement of wall, ten settlement plates of 1.0m and 5.0m in depth around excavation zone, and ten tilmeters and four vertical beam sensors on surrounding buildings were installed to monitor tilting. Layout of instrumentation is presented in Fig. 1.
BASEMENT EXCAVATION

The principal steps of basement excavation and construction sequence are described below:
1. Construct capping beam and excavation to 2.5m below ground level.
2. Install first level bracing at -1.5m and pre-load the struts
3. Excavate down to -7.0m
4. Install second level bracing at -6.5m and pre-load the struts
5. Continue excavation down to the final depth at -12.7m
6. Construct mat foundation (Basement 3), Basement 2 and remove second level bracing
7. Construct Basement 1 and remove the first level bracing

Since the excavation work is located adjacent to the river and surrounded by old buildings, the diaphragm walls are subject to three different lateral load conditions resulting from (1) full depth of the earth, (2) steep downward slope of riverbed, and (3) full depth of earth with possible surcharges from the adjacent buildings. In particular, the walls alongside the river (D1) and the opposite walls (D2), were expected to undergo an unbalanced loading condition. During temporary bracing design, two dimensional computer modeling were carried out to study the effects of unbalanced lateral loading on the wall and bracing. An additional two dimensional model analysis was carried out prior to designing a temporary bracing. The model analysis indicated relatively less movement of Wall D1 towards excavation (Fig. 3). However, the following measures were adopted in excavation work to prevent potentially adverse behavior of wall D1 and to keep the lateral wall movements within tolerable limits;

1. Using a simple, but efficient temporary bracing system
2. Pre-loading of the struts (200kN/m and 400kN/m for first and second levels respectively) on the one end of the strut on Wall D2 only (Fig. 1).
3. Excavating the soil in front of Wall D1 side first at any excavation stage
4. Frequent monitoring of wall movements
5. Minimizing construction time

During the initial excavation to 2.5m for installation of first level bracing, a historic foundation was unexpectively discovered. Excavation was suspended for about 3 months and resumed after further excavation was permitted by the archaeological department.

Excavation for final depth was made in the rainy season and a berm made of sand bags for flood protection was constructed around the perimeter of the wall. After lean mixed blinding concrete had been cast at the final excavation level, the river water level rose to a maximum level about 0.5m above the ground level.

TEMPORARY BRACING SYSTEM

A simple cross-lot bracing system with continuous wale beams was used to support the wall during basement construction stage (Fig. 1). 20m long steel king posts of H300x300 sections were driven into stiff clay to support the bracing system and temporary deck for excavation and construction equipment. A summary of steel sections used in bracing system is presented in Table 2.

<table>
<thead>
<tr>
<th>Bracing Level</th>
<th>Wale Beam</th>
<th>Strut</th>
<th>Expected Force (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>1xWF400x400</td>
<td>WF350x350</td>
<td>484.0</td>
</tr>
<tr>
<td>Second</td>
<td>2xWF400x400</td>
<td>WF400x400</td>
<td>789.0</td>
</tr>
</tbody>
</table>

Between wale beams and wall, lean mixed concrete were poured into the gaps between wale beams and wall to achieve a good load transfer. Pre-loading was carried out using hydraulic jacks. During pre-loading, the movements of the strut were measured. Horizontal movements of 0.82mm to 9.06mm and 0.15mm to 29.83mm in the direction of jacking for first level struts and second level struts respectively were recorded. For second level struts, the vertical movement of up to 1.68mm was measured at the jacking position.

INSTRUMENTATION RESULTS

Inclinometer Monitoring - Wall movements measured from inclinometers and predicted displacements by one-dimensional finite element analysis are shown in Fig. 4. Movements of Wall D1 and a section of Wall D3 close to the river, near the garden are found to be cantilever shape indicated by I-6, I-7 and I-8 respectively while movements
of Walls D2 and D3 have inward bulging shape indicated by I-1 to I-5.

Inclinometer No. I-7

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>PREDICTED</th>
<th>OBSERVATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.2</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>0.4</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>0.6</td>
<td>3.0</td>
<td>3.0</td>
</tr>
<tr>
<td>0.8</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>1.0</td>
<td>5.0</td>
<td>5.0</td>
</tr>
</tbody>
</table>

Fig. 4 Predicted and measured lateral wall movements

Generally the predicted and measured lateral wall movements are in a good agreement, except for the top portion of wall in which the measured wall movements exceed the predicted movements. The differences in wall movement were found to be caused by the following:

1. Advancement of excavation in front of Wall D1 further than other walls, allowing Wall D1 to stand longer prior to strut installation than the others.
2. Wall D1 was in cantilever condition for about 3 months after initial excavation for first level bracing.
3. No direct nor immediate pre-loading on Wall D1.
4. Slight over excavation to install first level strut for all walls.

The walls are in fix end conditions as fixity of walls indicated by all inclinometer reading are found to be at depth of about 20.0m which is 8m above the walls’ toe depth.

Maximum lateral wall movements after installation of struts were in the range (about 0.1% to 1.2% of excavation depth) of projects completed in Bangkok area.

Fig. 5 Wall movement and excavation depth ratio

Settlement Monitoring - Ground settlement of up to 16mm was observed after completion of excavation works. The maximum settlement was found to be near the location of the inclinometer I-6 which indicates the largest lateral movement at the top of the wall.

Tiltmeter and Beam Sensor Monitoring - Readings of tilmeter and beam sensors installed in the surrounding structures are shown with the progress of excavation in Fig. 6. A comparison between results from tilmeter and beam sensor and typical values for maximum building slope or settlement for damage risk assessment is presented in Table 3.

Table 3. Comparison between observed values and maximum damage risk assessment (Lake et. al.)

<table>
<thead>
<tr>
<th>Risk</th>
<th>Max. Slope of Building</th>
<th>Max. Settlement of building (mm)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category 1</td>
<td>&lt;1/500</td>
<td>&lt;10</td>
<td>Negligible : superficial damage unlikely</td>
</tr>
<tr>
<td>Vertical</td>
<td>1/2082*</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Beam Sensor</td>
<td>1/3165*</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Category 2</td>
<td>1/500</td>
<td>10 - 50</td>
<td>Slight possible superficial damage which is unlikely to have structural significant</td>
</tr>
</tbody>
</table>

* monitoring results for the project presented in this paper
No building settlements nor cracks on the buildings were observed till completion of the basement construction. The buildings were evaluated to be in risk category 1. The risk assessment and monitoring results confirm a good performance of the braced excavation.

DISCUSSION

Pre-loading is an effective way to reduce further wall movement after strut installation by providing a good intact between wall and supporting system. This is clearly seen in measured wall movements compared between I-7 and I-2, and I-3 and I-5 which are located in the walls generally facing each other (Fig. 4 and 1).

Delay in strut installation and construction time affect the wall movements, especially at initial excavation stage before installation of first level bracing.

Wall D1 which is alongside the river was found to be minimally affected by the unbalanced load conditions due to deep embedment (about 8m below the fixity) of the wall and presence of the old river wall. This was also shown by the two dimensional analysis (Fig. 3).

The predicted movements of the wall in cantilever condition at initial excavation stage in soft clay layer is considerably small due to the high modulus of soft clay adopted. In this case, construction time and sequence of excavation needed to be strictly controlled to keep the wall within the movement limit predicted by the one dimensional analysis.

Two dimensional analysis with facilities to model the geometry of site is required for the diaphragm wall subject to unbalanced lateral loading conditions.

CONCLUSION

Braced excavation using diaphragm wall subject to unbalanced loading due to adjacent river and surrounding buildings, was successfully achieved with proper instrumentation and monitoring.

Performance of the wall based on the instrumentation results are presented and discussed.

Back analysis was carried out with two dimensional modeling to determine the soil modulus. The soil modulus obtained from back analysis and the modulus adopted in wall designed were compared.

ACKNOWLEDGEMENT

The authors express their appreciation to the colleagues, especially to Mr. Ganeshan Baskaran and Mr. Muhammad Ashfaq Anwar for their invaluable suggestion and assistance in the preparation of this paper. Initial analysis works carried out by Dr. Vichai Vitayasupakorn and Mr.Young Zou are acknowledged.
REFERENCES


Prediction and performances of short embedded cast in-situ diaphragm wall for deep excavation in Bangkok subsoil

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Rolla, Missouri
PREDICTION AND PERFORMANCES OF SHORT EMBEDDED CAST IN-SITU DIAPHRAGM WALL FOR DEEP EXCAVATION IN BANGKOK SUBSOIL

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Bangkok Thailand  Thailand  Thailand  Thailand  

ABSTRACT

This paper presents a case history of performances and predictions of a diaphragm wall which is subject to serve differently from its initial purpose for which the wall was constructed. The updated requirements associated with the additional excavation created a situation whereby the wall would become short embedded and behave like free-end supported wall. As the wall had been already constructed during the time of modification it was only possible to modify the excavation method and bracing system. Accordingly, the wall stability and possible toe movement were analyzed, to accommodate the updated requirements, by using finite element methods. Continuous inclinometer monitoring has been carried out during the construction and results are being compared and analyzed for predicted values. Performance of the wall based on comparison between the inclinometer monitoring results during different excavation stages and predicted results, are discussed. It has been found that excavation depth for first bracing layer and construction period are very important for diaphragm wall performance. Large initial movements in the wall strongly influence the wall movements in the successive excavation works. Construction practice plays a major role in deep excavation work.

KEY WORDS

Diaphragm wall, Lateral movements, Inclinometer, Settlements, Free-end, Bracing, Excavation, Critical height, Cantilever
INTRODUCTION

Economic growth in Thailand during the past decade has been forcing the construction industry to look for all feasible methods to enhance the infrastructure of the country especially for the Bangkok city. Huge number of high-rise buildings with multilevel basements are being erected within a short time possible to accommodate the booming economy. Value of urban land is increasing in many folds beyond the limits. Unlike in developed countries, the building rules and regulations of this city are only in documented form and hardly established all in practice in a short time. As a result, developers and owners enjoy relatively more freedom to modify the structural layout, even during and after construction stages, to facilitate their growing demands which are being conceptualized within a short time duration in the fast developing city, aiming to maximize the area utilization. The project reported in this paper is an example for such case.

The project site is located in the vicinity of central business area of Bangkok. The structure is a multi-storied building for a business complex. The load of the structure would be carried by bored cast-in-situ bored piles and barrette piles with toe depth of 60m below the ground level, embedded into the second sand layer of the Bangkok subsoil.

The constructed cast in-situ diaphragm wall is 0.8m thick and the toe is embedded into stiff clay at a depth 18m below the ground level. Initially the wall was designed for construction of four basements including mat foundation of 3.5m thick. The maximum excavation depth was set to be 14m and a three level of temporary bracing system was considered to construct the basements with conventional bottom up construction method.

Embedded depth below the final excavation for the mat foundation was designed to be 4m. After construction of the wall it was decided to increase the number of basements to be five for part of the building area (Tower area) and excavation depth needed to be increased by 1.8m from the originally designed depth, leaving the wall embedment depth of 2.2m only.

SUBSOIL CONDITION

The sub soil conditions at the project site is not uncommon to usual conditions observed in Bangkok comprising a 15m thick soft to medium clay layer on the top underlain by stiff clay to a depth of about 22m. Below the stiff clay is a series of alternating layers of hard silty clay, sandy clay and dense sand.

SITE CONDITION

The project site is located in the corner of a main road and a street of the central business district in Bangkok. The planned building is facing both the main road and street. The building plan was set out in accordance with building code having a setback of 6m from an adjacent property boundaries. Behind it are two story buildings located more than 10m away. Deep excavation is ideal in such site condition. Figure 2 shows the layout plan of the building.

DIAPHRAGM WALL DESIGN

Originally diaphragm wall was designed with a 3 level temporary bracing to allow a maximum excavation depth of 14m below the existing ground for construction of mat foundation (see Fig. 3). Computer programs of finite element methods (WALLAP and CRISP) and finite difference method (FLAC) were used to simulate the staged excavation and predict the wall behaviour and performance. In the computer modeling, undrained strength of soil with total stress parameters derived from laboratory and field test results were
used. 0.8m thick Diaphragm wall with toe depth of 18m was found to be adequate enough for the planned basement construction in such soil condition.

Fig. 2  Layout Plan of Diaphragm Wall & Piles

Estimated maximum bending moment of 90 t-m/m on the excavation side and 56 t-m/m on soil side of the wall. The maximum deflection of the wall predicted was about 35-37mm with some degrees of toe movement up to 11mm.

Bored piles of 1.20m in diameter were in-cooperated within and below the toe of the wall to support the column load and dead load of the wall. Foundation piles (dia. 1.20m), including bored piles and barrette piles (2.70mx0.80m) were founded at the depth of 60m in dense sand layer. Four inclinometer tubes I-1 to I-4 (20m to 25m in length) were installed in the wall panels with bored pile legs to observe the behaviour and performance of the wall.

Diaphragm wall and foundation piles were constructed in parallel. After completion of diaphragm wall construction, but construction of bored piles were still in progress, overall design of the building was revised to suit the requirements of the property market. Number of basement floors was increased from 4 to 5 in the main tower area. As a result, the diaphragm wall constructed was deemed to support for the maximum excavation depth of 15.8m.

Firstly diaphragm wall design and construction stages were then reviewed. Since the wall has been constructed, excavation sequence and bracing system can only be modified so that the modified deeper excavation can be done without impairing the structure and stability of the wall.

Fig. 3  The Original Construction Sequence with three temporary bracing levels

MODIFIED CONSTRUCTION SEQUENCE

Finite element computer modeling was again carried out for the desired excavation depth. The model analysis showed that 4 level temporary bracing was necessary with strut pre-loading. It also indicated the diaphragm wall in free-end condition with some degrees of toe movement in a range of 23mm. The maximum predicted wall movement at the top was 77mm.

To prevent progressive toe movement with time, followings were recommended for excavation works.

(i) frequent monitoring of diaphragm wall movements when excavation close to the final excavation depth.

(ii) casting a reinforced lean concrete slab with a thickness of 20cm and 6m wide in front of the wall just after reaching the excavation depth of -13.8m.

(iii) excavation to -15.8m must be carried out leaving a 12m wide soil berm with lean mixed concrete slab on top of the berm all around the wall.

Apart from four inclinometer tubes installed in the wall, a total of 16 sets of settlement observation stations were installed around the wall perimeter. Each station comprised two to three settlement points located at 3m intervals and 3m away from the wall.
The risk of over excavation was carefully assessed not only for diaphragm wall but also for the existing buildings near by the site. Review of the site condition indicated that the existing buildings surrounding the project site were more than 10m away from the excavation zone and they were found subject to no risk.

BRACING SYSTEM

The bracing system included continuous waling beams along the diaphragm wall and longitudinal and transverse struts. The spacing of strut was generally 6.4m to 6.8m. Table no. 2 shows the type of wide flange structural steels used for the bracing.

Table 2 Structural steel sections used for temporary bracing

<table>
<thead>
<tr>
<th>Bracing Layer</th>
<th>Strut</th>
<th>Wale</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>2W300x300x94kg/m</td>
<td>2-W300x300x94kg/m</td>
</tr>
<tr>
<td>2nd</td>
<td>2W350x350x137kg/m</td>
<td>2W350x350x137kg/m</td>
</tr>
<tr>
<td>3rd</td>
<td>2W350x350x137kg/m</td>
<td>2W350x350x137kg/m</td>
</tr>
<tr>
<td>4th</td>
<td>W350x350x137kg/m</td>
<td>2W350x350x137kg/m</td>
</tr>
</tbody>
</table>

LATERAL MOVEMENTS OF THE WALL

Monitoring of wall movements was carried out by reading the inclinometer tubes profiles installed in the wall at every major construction stages. The wall constructed was designed for initial excavation of -2.5m to install the first bracing layer. However the excavation has been done to about -3.0m to -3.2m at once and a movement of 40mm was observed. Time delay during construction of capping beam and over excavation deeper than the critical height ($H_c = 2C/\gamma$) was unsupported around three months resulted an increase in wall movements to 55mm -70mm. Tension cracks were developed in the soil of active side about 2-3m away from the wall. After installation of struts and pre-loading them, the lateral movements of the wall ceased off. An increment of 2mm-4mm was recorded with further excavation stages (See Fig. 5). Struts from first to fourth levels were pre-loaded 40 t/m, 75 t/m, 75 t/m and 40 t/m respectively, 20%-25% of design load of bracing system, prior to further excavation. The pre-loading of the strut was very effective to reduce lateral movement of the wall and ensuring good intact between wall and bracing system.
were in a range of 1.2mm to 16mm while the predicted toe movement was about 23mm. Predicted and observed wall movements is tabulated as below:

Table 3  Predicted and observed wall movements

<table>
<thead>
<tr>
<th>Location on the Wall</th>
<th>Predicted for 14.0m Excavation (mm)</th>
<th>Predicted for 15.8m Excavation (mm)</th>
<th>Observed on Site (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>35-37</td>
<td>77</td>
<td>55-98</td>
</tr>
<tr>
<td>Toe</td>
<td>11</td>
<td>23</td>
<td>1.2-16</td>
</tr>
</tbody>
</table>

The maximum lateral movements were found in I-4. At the final excavation stage, movements of the wall increased considerably below the first bracing level down to toe of the wall. It was noted that in I-4 area the longitudinal and transverse struts of the bracing system become diagonal to the wall sections due to the geometry of wall layout. The stiffness of the bracing in this particular zone is considered to be much less than that of others.

The wall’s fixity was found to be at depth 20m-22.5m from inclinometer readings when the final excavation reached.

From the settlement observation data, it was found that the maximum ground settlements behind the wall were approximately 34% to 50% of the lateral wall movements, except for the area near the I-4 where the ground settlements equal the lateral wall movements.

Fig. 6B  Lateral movements of the wall shown by Inclinometer Tubes I-3 and I-4.

Fig. 7  Relationship between maximum ground settlements and maximum lateral wall movements

ANALYSIS ON PERFORMANCE OF THE WALL

Based on the deformation of the wall measured from the inclinometer readings, the bending moments were computed from the well known equation to check structural performance of the wall

\[ M = E_c I_{ef} \left( \frac{dS}{dy} \right) \]  

where  
- \( E_c \) = Modulus of Elasticity of concrete  
- \( I_{ef} \) = Effective moment of inertia of the wall section  
- \( S \) = slope given by inclinometer reading  
- \( y \) = elevation of the slope given

Generally the bending moments computed were within the design moments and the performance of the wall is found to be satisfactory. However a direct comparison between bending moment envelopes from computer simulation and bending moments derived from inclinometer readings shows some differences, especially at the strut levels and below the depth of 12m. These differences can be due to the following;

1. Stiffness of medium to stiff clay layer used based on the field and laboratory tests for simulation is much lower than the actual in-situ soil stiffness. In simulation for the wall design, influence on soil stiffness below the final excavation level by bored piles was not considered.

2. Efficiency and stiffness of bracing system, including pre-loading forces. In the original design calculation, much lower pre-loading forces were used.

3. Large initial movements during the wall in cantilever condition.

4. Inclinometer tubes (20-25m in length) were installed in diaphragm wall panels with bored pile legs. The bored pile...
legs provide the fixity of the wall and increase the bending moments at a location near the bottom of the wall.

5. Moment redistribution in the wall is considered not fully activated.

The relationship between wall movement and system stiffness (Fig. 9) shows that because of large movements during the wall is in cantilever condition, the excavation to 5.5m with one bracing deviates from the normal trend of braced excavation.

Fig. 10 Relationship between Normalized Maximum lateral movement versus factor of safety against basal heave.

The normalized lateral movements of the wall shown in Fig. 10 is within the range of 0.5-1.5% which is slightly higher than those of similar projects in Bangkok subsoil. The higher movement ratio occurred during excavation in the soft clay layer with slight over excavation and delay in first strut installation. It is observed that the first excavation depth and duration in bracing layer installation are very critical for the performance of the wall at further excavation stage.

CONCLUSION

1. Movements in first excavation stage is governing the performance of Diaphragm wall for further excavation.

2. First excavation deeper than the critical height of the soil needs to be supported immediately.

3. Apart from support system and soil properties, construction practice influenced the movement of the wall in this particular project.

4. A successful deep excavation with short embedded diaphragm wall has been achieved using a combination of prediction by computer simulations and instrumented field observations.
ACKNOWLEDGEMENT

The authors express their appreciation to the colleagues, especially to Mr. Ganeshan Baskaran, Mr. Young Zou, Mr. Muhammad Ashfaq Anwar, Mr. Pornpot Tanseng and Mr. Chetsadar Plongkrathok for their assistance in the preparation of this paper.

REFERENCES


Lessons from the collapse during construction of an inlet pumping station: Geotechnical instrumentation aspects

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ABSTRACT: In the wastewater treatment plant, an Inlet Pumping Station (IPS) is one of the most important structures to collect and pump all wastewater to the treatment plant. The IPS presented in this paper consists of 20m deep underground chambers. A reinforced concrete diaphragm wall system 1.0m thick and 25m deep was used in construction of the IPS. Four Inclinometer tubes were installed in the diaphragm wall to monitor the lateral wall movements during excavation work for basement construction. When excavation approached the final depth, excessive lateral wall movements were observed, but they were not taken seriously. Later, some cracks on the capping beam and noise from the bracing system were noticed. Then the IPS collapsed and was buried under sliding soil mass. At the time IPS collapsed five main bracing layers and only two of the five intermediate bracing layers had been installed. This paper emphasizes the importance of instrumentation. Additionally it presents the results of investigation into the causes of the collapse, as well as the reconstruction work.

1. INTRODUCTION

Bangkok metropolis is sectioned into four parts for the implementation of four mega projects, or "phases" that comprise a wastewater treatment system to serve the rapidly growing industrial and domestic needs. Three of four phases are under construction. All projects are a turnkey system where the main contractors are responsible for both design and construction. The engineers will be the consultants as representatives of the owner to review the conceptual and detailed design and to supervise the construction. The aim of this paper is to highlight the importance of instrumentation in deep underground construction, as proved by the collapse of the Inlet Pumping Station (IPS) at the second phase. This paper also briefly describes its reconstruction.

2. PROJECT DESCRIPTIONS

The second phase Bangkok wastewater project presented herein was started in 1995 to serve the central area of Bangkok City. The project was planned to be completed by 1999 (ie. within 4 years). In the sewage treatment plant (STP), the IPS is one of the most important structures to collect and pump all wastewater to the sewage treatment plant. Figure 1 presents the location of IPS on the sewage treatment plant (STP) site.

The IPS has two distinct chambers, a pumping chamber and an inlet/bypass chamber, all built underground. The pumping chamber is a circular shaft while the inlet/bypass chamber forms a box on the north side of the pumping chamber. The pumping chamber is 20.3m in diameter and 20.2m deep. It consists of a central shaft 8.75m in diameter and an annulus 4.875m wide. The inlet chamber is essentially a large box receiving the inlet sewers and also housing screens for removal of debris. The inlet chamber is 11.5m x 9.0m approximately in plan. The bypass chamber is adjacent to an inlet chamber and divided by a concrete wall from base level at...
19.5m below ground to roof level. The inlet/bypass chamber has three levels. The base level is at 19.50m below the ground. Levels 1 and 2 are at 10.5m and 5.5m depth respectively. The roof slab is at 1.0m depth.

The IPS was located at the northwest corner of the STP site, approximately four to ten meters from a canal. The canal is protected by concrete walls supported by battered concrete piles. Private residences consisting of wooden house were located close to the property line in the west.

Space limitation necessitated reinforced concrete diaphragm walls 1.0m thick and about 25.0m deep as permanent structures of the IPS. The diaphragm walls were also used as temporary structures during excavations up to 22.0m deep. The one-meter-thick wall was cast in panels with lengths of around 2.5 - 3.0m using the bentonite slurry trench construction method. The connection between adjoining panels was constructed by using U-shaped stop-ends with water bars.

For excavation work, five levels of temporary steel bracing were designed to support the straight wall portion and an additional five intermediate bracing layers was planned to frame the edges where the straight wall meets the circular enclosure (Fig. 2). Four inclinometer tubes were installed at critical points of the diaphragm wall to observe lateral movements during excavation for underground construction.

The bracing system was classified as temporary and thus no submission was made for review. The final temporary bracing arrangement (Fig. 3) was changed from the initial design in Figure 2. In the initial design, two lacing beams shown as a dash line in Figure 2 were planned to strengthen the strutting system. However these were cancelled in the final temporary bracing system (Fig. 3).

3. SOIL CONDITIONS

At the start of the project, the site consisted of very soft clay (mud) about 2 - 3m deep. The soft clay apparently extended westward, into the residential area of the wooden houses. When work on the project commenced, mud was removed from the site up to the property boundaries. Approximately a 3.0m thick of sand backfill was placed on the site. Then the soil conditions consisted of 3.0m thick sand backfill, followed by soft to medium dark grey clay approximately 16.0m deep. Then stiff to very stiff silty clay was encountered at about 30.0m depth, overlaying the first dense silty sand layer. The sensitivity of the soft to medium clay was in the order of 2.5 to 6.0. A piezometric level was found at about 21.0m below ground level, as a result of deep well pumping. A summary of general soil properties and soil stiffness is presented in Table 1.

4. COLLAPSE OF INLET PUMPING STATION

The reinforced concrete diaphragm wall and installation of four inclinometers were completed on 15th January 1997 by the diaphragm wall sub-contractor. After completion of the diaphragm wall, the same sub-contractor pointed out to the main contractor the importance of an adequate bracing system and staged construction sequences. The excavation and strutting was carried out by another subcontractor, while the other permanent structures such as capping ring beam were constructed by the main contractor. The capping ring beam had been casted to two thirds of its height by the main contractor on 8 June 1997. Two inclinometer tubes (I-1 and I-2) were found damaged and this was reported. But the
damaged tubes were left unrepaired and could not be used for monitoring.

The excavation began in mid-June, 1997. Backhoes were used for excavating inside the shaft and all materials were collected with a bucket. The bucket was hung from a service crane located behind the diaphragm wall perimeter near I-1 location (collapsed zone) on the ground. The excavation and installation of struts was carried out step by step. However, only 5 diagonal bracing layers, 5 main struts and two out of five intermediate struts were installed without instrumentation.

Lateral wall movements were recorded by I-3, located at one edge of circular enclosure and I-4, located in the intact zone (Fig.2). Wall movements in A-axis direction of the inclinometer access tube I-3 were indicative of the wall in cantilever/unsupported condition while those in B-axis direction were indicative of the wall being supported (Fig. 4a).

![Figure 4a. Inclinometer reading of I-3](image)

![Figure 4b. Inclinometer reading of I-4](image)

However, the wall movements in A-axis direction were considerably large and thus the main beams supporting the edges of the circular enclosure seemed not effective. The two differential movements indicated that the edges of the wall between the straight wall and the circular enclosure had been under torsional stress. It was reported that tension cracks on the capping beam were observed in late July 1997. I-4 recorded wall movements with a maximum of 13mm (Fig. 4b). Movements of the wall toe could not be measured due to lack of the inclinometer tube penetration into the soil below the wall toe level.

After observation of tension cracks on the capping beam, immediate installation of the remaining intermediate struts to support the edges of the circular enclosure and straight wall area were recommended. Occurrence of more cracks on the capping ring beam and noise from the bracing system was noticed. At this stage excavation depth inside the circular enclosure had been reached the final depth. However, only two of the five intermediate bracing layers had been installed at the time the IPS collapsed. Figure. 5 shows the photo of the bracing system taken one day before collapse. The IPS collapsed in the morning on 17 August 1997.

After the collapse, soil mass caved in. Several adjacent wooden houses were also damaged and slipped down (Fig. 6a). The soil mass buried the collapsed portions of the wall, the steel bracing, and the excavation equipment (Fig. 6b). Some sections of the wall on the western side collapsed while the remaining wall sections stood in place. The collapse also sheared off the capping beam at the collapse location, causing severe cracking in the capping beam section, and opening panel joints on remaining sections of the wall. No human casualties were reported in the incident as the loud noise prior to the

![Figure 5. Bracing system seen on 16, Aug. 1997 (after Thasnanipan, 1997)](image)
5. CAUSE OF COLLAPSE

The cause of collapse has been investigated by various parties involved. The preliminary causes of the collapse were considered to be:

- Improper shape of IPS permanent structure
- Failure of diaphragm wall
- Failure of internal temporary bracing during excavation
- Failure due to improper sequence of excavation
- Failure due to poor workmanship in temporary works

Three main causes of collapse are identified: the improper shape of the IPS, failure of the diaphragm wall, and improper temporary bracing.

In the case of improper shape of the IPS, similar underground structure had been completed in Frankfurt, Germany (Katzenbach et al. 1998) as shown in Figure 7. Pit excavation was about 20.0m deep and a similar temporary strutting system was also used.

Evidence also indicated that collapse of the IPS was not caused by the diaphragm wall. It was found that the results of analysis of the wall designed by both the designer and diaphragm wall sub-contractor were in conformity with completed projects in Bangkok. The completed projects have been reported by Teparaka et al. (1998) and Thasnanipan (1998). No defect or leakage was found on the exposed walls. Phien-Wej & Sriruanthong (1999) reported that at the final excavation depth in IPS reconstruction, the toe of the diaphragm wall in the collapsed zone was found in original position and there was no evidence of basal heave.

The importance of temporary bracing has been emphasized in various design guides, manuals and reports by Padfield & Mair (1984), William & Waite (1993) and Feld & Carper (1996). In the case of improper temporary bracing, this issue was identified as the main cause of collapse. Results of the analysis by means of FEM (Kanok-Nukulchai, et al. 1998) concluded that the main strut supporting the edges of the circular enclosure was grossly overstressed. The report also stated that these struts would have progressively reached their capacity at the initiation of the collapse and may have become totally ineffective. It was clearly indicated by the photographs taken just one day before the collapse of the IPS (Fig. 8).

Figure 8 shows separations of the built-up walling beams and struts, which would reduce the capacity of the monolithic section by 30%. The three lower intermediate struts required at the edges of the circular enclosure were not present even though the excavation works reached the final depth (Fig. 5). This improper bracing also contributed to the cracks observed on the capping beam. Additionally, lateral movements of the walls, shown by inclinometer...
monitoring from I-3 indicated that the walls were in cantilever mode. The wall behavior did not reflect predicted behavior indicating inefficiency of the bracing system (Fig. 9). The cracks occurred in the capping beams confirmed cantilever wall movements.

6. REMEDIAL ACTION / RECONSTRUCTION

Reconstruction of the IPS was started at the end of 1997. The process involved construction of an outer diaphragm wall perimeter to enclose the collapsed IPS location. The IPS could not be relocated due to presence of affiliated structures. Before starting the outer loop diaphragm wall, Jet Grouting techniques were used to stabilize the soil in the surrounding area for purposes of diaphragm wall construction as well as for excavation works (Fig. 10). Jet grouting and diaphragm wall construction was undertaken by another subcontractor. Jet grouting stabilized only soft clay up to about 16.0m deep. The outer loop diaphragm wall which was designed as a temporary structure was 1.5m thick and about 32.0 m deep.

During reconstruction, all parties realized the importance of instrumentation. Six inclinometers were installed, five numbers in the wall panels, and one in the jet grouted area in the collapsed zone (Fig. 10).

During excavation, two bracing layers with heavily built-up steel sections were used as diagonal struts at depths of 2.0m and 11.5m (Fig. 11). In both layers, strain gauges were also installed on two main struts and readings were recorded each time a depth of every 2.0m. had been excavated or every 2 days.

Installation and monitoring of (VWSG) on struts provided information on their performance during construction so that necessary action against instability could be taken.

Figure 9. Prediction vs. actual wall behavior (I-3)

Figure 10. Location of inclinometers and treated area by Jet grouting (after EIT 1999)

Figure 11. Arrangement of strutting system with stain gauges (after EIT 1999)

Figure 12. Excavation in progress for IPS reconstruction
Figure 12 shows the status of excavation and strutting before casting the new IPS structures inside the temporary diaphragm wall. The lateral wall movements were recorded by I-2 and I-6 with maximum wall deflections of about 50.0mm, 70.0mm respectively (Fig. 13). The reconstruction work was thus successfully completed with proper instrumentation.

7. CONCLUSIONS & RECOMMENDATIONS

It was learnt that ignorance of instrumentation for deep underground construction can cause a serious disaster.

Adequate instrumentation should be provided for deep excavation to avoid remedial work.

Monitoring and maintenance of instrumentation should be carried out throughout the work.

Parties involved in deep underground construction should have enough knowledge in geotechnical works and instrumentation.

Proper instrumentation enabled the successful reconstruction of IPS.

8. ACKNOWLEDGEMENT

The authors wish to express their appreciation to the colleagues, especially to Mr. Thiruchelvam Navaneethan for his invaluable advice and assistance in preparation of this paper.

9. REFERENCES

EIT (1999), Case study of problems related to settlement and failures Geotechnical Engineering in the Past 2 years, Subcommittee of Geotechnical Engineering, Engineering Institute of Thailand (EIT)
Behavior and performance of diaphragm walls under unbalanced lateral loading along the Chao Phraya River

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Behavior and Performance of Diaphragm Walls under Unbalanced Lateral Loading along the Chao Phraya River

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ABSTRACT: This paper presents the behavior and performance of diaphragm walls constructed for basements of two buildings along the bank of the Chao Phraya River bank, Bangkok, Thailand. These buildings were constructed at different periods under one contract. A case history on basement excavation of the first building has been reported previously by Thasnanipan et al. (1998). For the second building, more numbers of instrumentation including VWSGs in a diaphragm wall panel and pressure gauges in struts to monitor stress in the wall and the performance of bracing system were used respectively. The construction sequence adopted for excavation, performance of the walls and bracing based on instrumentation results are discussed.

1 INTRODUCTION

Retaining structures and supported systems for deep excavation in urban areas, particularly in the vicinity of a river, are often subjected to unbalanced lateral load due to asymmetric loading and surcharges. The various publications and design guides by GCO no. 1/90 (1996), Williams & Waite (1993) and Padfield & Mair (1991) give special guidance on overall stability of retaining structures and strutting systems in such conditions. Case histories on braced excavations under unbalanced/non-symmetrical lateral loading also have been reported by Thasnanipan et al. (1998), De Rezende Lopes (1985) and Kotoda et al. (1990).

This paper presents two basement excavation works up to 12.7m below the ground level for two separate buildings using braced diaphragm walls along the bank of the Chao Phraya River, Bangkok. The first one had previously been reported by Thasnanipan et al. (1998). Construction of the second building commenced about one year after completion of the first one.

2 SITE AND SUBSOIL CONDITIONS

The building sites are located 40.0m apart, and separated by an existing 5 storey building between them. They are located along the river and surrounded by existing structures including a historical building (Fig. 1). The existing buildings were supported by pile foundation. The distance between diaphragm walls along the river and the existing old river wall buildings respectively. They are also situated in the heart of an old established culturally-significant zone. Space and building height limitations necessitate multi-level basements to increase usable floor areas although the planned buildings are right on the river bank.

At the building sites, the river is about 205m wide and 10-12m deep mid-stream. The riverbed near the river wall is about 2.2-3.0m in depth with a gentle slope. The river water level in dry season is about 1.6m below ground level and in the rainy season sometimes rises above ground level, causing a flood. Primary site investigations included one borehole 60m deep and one field vane shear test for each site. For the first building, prior to designing temporary bracing and basement excavation work, drilling of additional three boreholes 20m deep and two field vane shear tests were carried out to check subsoil
variations. For the second building, two additional 60.0m deep boreholes were dug. The shear strength of soft clay in the planned second building site is higher than the soft clay in the first site. However, there is no significant variation in subsoil conditions within each site, particularly along the depth of planned diaphragm walls. Subsequently the profile of the sites and surcharges from surrounding buildings are of major concern for unbalanced lateral loading conditions in excavation works. Table 1 shows a summary of subsoil properties obtained from the boreholes and test data.

### Table 1. Summary of soil properties

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Layer</th>
<th>Top in Depth m</th>
<th>W</th>
<th>$C_u$ kPa</th>
<th>$C_{v}$ kN/m$^3$</th>
<th>$SPT$ N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft Clay</td>
<td>0-3.0</td>
<td>35-78</td>
<td>16-19</td>
<td>30</td>
<td>16-19</td>
<td>30</td>
</tr>
<tr>
<td>Med. Clay</td>
<td>12.7</td>
<td>30</td>
<td>19</td>
<td>71</td>
<td>19-21</td>
<td>71</td>
</tr>
<tr>
<td>Stiff Clay</td>
<td>14</td>
<td>22-34</td>
<td>19-21</td>
<td>43-300</td>
<td>14-52</td>
<td>43-300</td>
</tr>
<tr>
<td>Dense Sand</td>
<td>25</td>
<td>14-25</td>
<td>20-23</td>
<td>-</td>
<td>17S-60</td>
<td>14-52</td>
</tr>
<tr>
<td>Silty Clay</td>
<td>36.5</td>
<td>17-21</td>
<td>20-23</td>
<td>175-240</td>
<td>30-45</td>
<td>175-240</td>
</tr>
<tr>
<td>Dense Sand*</td>
<td>42-45</td>
<td>20-26</td>
<td>21</td>
<td>-</td>
<td>20-26</td>
<td>&gt;58</td>
</tr>
</tbody>
</table>

* Some 4-5m thick hard clay seams present at depth 48-52m

3 DIAPHRAGM WALLS

For both buildings, 800mm thick cast in-situ concrete diaphragm walls of 28m toe depth with two level temporary bracings were designed for base- ment excavation. The maximum excavation depths for the first and second buildings were 12.7m and 9.7m respectively. 28m deep walls were necessary for overall stability of excavation as they were located on the riverbank.

4 PILE FOUNDATIONS

Foundation piles having pile toe at 48m depth were constructed using bored piling under bentonite slurry. Quantities of piles are tabulated in Table 2.

### Table 2. Quantity of foundation piles

<table>
<thead>
<tr>
<th>Building</th>
<th>800mm</th>
<th>1000mm</th>
<th>1200mm</th>
<th>1500mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>32*</td>
<td>18*</td>
<td>31</td>
<td>6</td>
</tr>
<tr>
<td>Second</td>
<td>17*</td>
<td>33</td>
<td>5</td>
<td>23</td>
</tr>
</tbody>
</table>

* Some piles were incorporated in diaphragm wall panels.

5 INSTRUMENTATION

As construction sites were located in a very sensitive urban area and subjected to unbalanced lateral loading conditions, various types of instrumentation were installed and systematically monitored. Layouts of instrumentation for the first and second buildings are shown in Figures 3a and 3b respectively.

For the second building, 5 levels of vibrating wire strain gauges (VWSG) in pairs were installed in one diaphragm wall panel and 2 sets of earth pressure gauges were installed in struts to observe stress in the diaphragm wall and strut forces respectively. Types of instrumentation are presented in Table 3.

### Table 3. Quantity of Instrumentation

<table>
<thead>
<tr>
<th>Type of Instrumentation</th>
<th>First Building</th>
<th>Second Building</th>
<th>Measurements made</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inclinometer</td>
<td>8</td>
<td>6</td>
<td>Wall deflections</td>
</tr>
<tr>
<td>Settlement Plate</td>
<td>10</td>
<td>20</td>
<td>Ground settlements</td>
</tr>
<tr>
<td>Tiltmeter</td>
<td>10</td>
<td>10</td>
<td>Tilting of Buildings</td>
</tr>
<tr>
<td>Vertical Beam Sensor</td>
<td>5</td>
<td>5</td>
<td>Tilting of buildings</td>
</tr>
<tr>
<td>VWSG</td>
<td>-</td>
<td>1 location*</td>
<td>Strain in rebars</td>
</tr>
<tr>
<td>Earth Pressure Gauge</td>
<td>-</td>
<td>2 x 2</td>
<td>Sturt forces</td>
</tr>
</tbody>
</table>

* A pair of VWSG in 5 layers along the depth of wall panel
6 EXCAVATION WORK

Conventional bottom-up method with two levels of temporary bracing was adopted for excavation to construct foundation and basement floors of both buildings. Prior to excavation work for the first building, a two-dimensional computer analysis was carried out using PLAXIS finite element computer program to study possible behavior of the wall system under unbalanced lateral loading conditions. Such conditions were considered to be resulting from (1) full depth of the earth, (2) a step sloping river bed and (3) full depth of earth with possible surcharges from the adjacent buildings.

A major concern was that the diaphragm walls alongside the river would be thrust from the opposite walls. These bore a higher lateral load through axial force of struts as excavation progressed in stages. This wall behavior was indicated by computer modeling (Fig. 4), and previous reports by De Rezende Lopes (1985) and Kotoda et al. (1990). To prevent any adverse wall behavior alongside the river, the following measures were taken:

1. Using a simple and efficient temporary bracing system
2. Pre-loading on one end of struts on the opposite walls
3. Excavating first soil in front of the wall closest to the river at any excavation stage
4. Frequently monitoring of wall movements
5. Minimizing construction time

Moreover, an observational construction approach (Ikuta et al. 1994), using the "most probable" conditions and parameters in the design with a continuity plan for "most unfavorable" conditions was employed. Firstly a monitoring plan was established before excavation. Secondly the instrumentation monitoring data were used to trigger the contingency plan. Generally predicted wall movements were set as primary trigger values for the contingency plan. After successful completion of the first building, the same approach was also adopted in excavation work for the second building. For the first level bracing of the second building, single strut arrangement (less rigid but economical compared to that of the first building) was adopted with provision of strut-force monitoring.

A simple cross-lot bracing system with continuous wale beams was used for both buildings. 20m and 19m-long, steel king posts of H300 x 300 in section were used to support the working platform and bracing system for the first and second buildings respectively.

In both cases, the initial excavation to 2.5m for installing the first level bracing revealed historic foundations. Temporary bracing and excavation works

<table>
<thead>
<tr>
<th>Building</th>
<th>Bracing Level</th>
<th>Elevation</th>
<th>Strut Sections</th>
<th>Design Strut Force kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>I</td>
<td>-2.0</td>
<td>2 x WF350 x 350</td>
<td>484.0</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>-6.5</td>
<td>2 x WF400 x 400</td>
<td>789.0</td>
</tr>
<tr>
<td>Second</td>
<td>I</td>
<td>-2.0</td>
<td>1 x WF400 x 400</td>
<td>279.4</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>-7.0</td>
<td>2 x WF350 x 350</td>
<td>332.1</td>
</tr>
</tbody>
</table>
were delayed for about 3 and 2 months for the first and second buildings pending permission by the archaeological department for further excavation.

7 INSTRUMENTATION RESULTS

Construction activities and the corresponding results of instrumentation monitoring with construction time are presented in Figures 5a and 5b. These figures, particularly for the second building indicate a good relationship between construction activities on one hand and responses of the walls, existing buildings and ground on the other hand. It can be observed that significant changes in these responses generally occurred during the initial excavation stage in which walls were unsupported.

Figure 5a. Instrumentation results with construction time (First Building)

During the period of delay, monitoring of instrumentation was frequently carried out to compare the monitoring data and the trigger values for planning a contingency plan against possible risks to adjacent structures from unsupported excavations to 2.5m depth.

7.1 Inclinometer monitoring

For the first building, generally the predicted and measured lateral wall movements were in good agreement, except for those at the top portion of walls (Fig. 6a). The wall-top movements exceeded predicted movements in the case of both buildings due to delay in installing the first level bracing, leaving the walls in cantilever condition for about 2 months or more.

Figure 5b. Instrumentation results with construction time (Second Building)

For the second building, wall movements were monitored weekly, sometimes every 2-3 days when necessary. At I-6 location, wall movements were found to reach the trigger/predicted value, especially
at the top of wall when further excavation reached 7.5m depth after installation of the first level bracing.

Lateral wall movement reached a maximum rate of 6.5mm/day. Then a tension crack on the ground surface appeared about 8.0m away from the wall. A close examination of the bracing system indicated that one strut had swayed slightly about 130mm at a distance of 15.0m from the wall. Moreover, the sections of the strut had been connected at an angle. An additional strut was then immediately stacked on the defective one and then wall movements were found to cease. Installation of the second level bracing was immediately carried out for this area.

Regarding unbalanced lateral loading condition, the wall movements of the first building were found unaffected by this condition (Fig. 6a). However, for the second building, monitoring results from I-2 and I-4 suggested that the wall alongside the river had been pushed against the retaining soil by the opposite wall (Fig. 6b).

All inclinometer readings for both buildings indicated that the walls were in fixed-end condition with fixity at depths of about 20.0m. Maximum wall movements after installation of bracing ranged from 0.14% to 0.64% of the corresponding excavation depth for the first building, and 0.27% to 0.83% for the second building. These were also within the range (about 0.1% to 1.2% of excavation depth) of wall movements in other projects completed in Bangkok area.

7.2 **Pressure gauges**

Readings from the pressure gauges installed on struts of the second building were recorded daily. After removal of second level bracing upon completion of base slab and during construction of basement 2 floor, measured strut force of the first level bracing reached the trigger value. However, in general the readings indicated that the bracing system used was adequate.

<table>
<thead>
<tr>
<th>Item</th>
<th>First level kN/m</th>
<th>Second level kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Predicted by WALLAP</td>
<td>354.5</td>
<td>307.3</td>
</tr>
<tr>
<td>Allowed</td>
<td>279.4</td>
<td>332.1</td>
</tr>
<tr>
<td>Measured</td>
<td>331.3</td>
<td>206.2</td>
</tr>
</tbody>
</table>

7.3 **Vibrating Wire Strain Gauges**

Strains developed in the wall of the second building were measured by using VWSGs attached to main reinforcement bars of the wall panel where I-4 was also located. A comparison between the measured strains and the allowed strains of the wall was made as shown in Figure 5. The comparison indicated that strains induced in the wall were well below the allowed values.

7.4 **Settlement Plates**

For the first building, a maximum settlement of 16mm was recorded. For the second building, in most cases, ground settlements (surface and deep settlements at 1.0m and 5.0m depth) had been found to stop after installation of the first bracing. In some cases, the surface settlements stopped only after installation of the second level bracing, resulting in maximum settlements up to 30-50mm. The large settlements were found to be associated with surcharges from stockpiles of construction materials and in one case, from unsupported concrete steps, under demolition, which had become detached from the adjacent buildings.
7.5 Tiltmeter and Beam sensor

Readings are also shown in Figure 5. Results from monitoring and typical values for maximum building slope or settlement for damage risk assessment are compared in Table 6.

<table>
<thead>
<tr>
<th>Building</th>
<th>Max. Slope of</th>
<th>Max. Settlement of building</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tiltmeter</td>
<td>&lt;1/500</td>
<td>&lt;10</td>
<td>Negligible: superficial damage unlikely</td>
</tr>
<tr>
<td>Vertical</td>
<td>1/2082*</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Beam Sensor</td>
<td>1/3165*</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

| Risk Category 2 | 1/500 | 10 - 50 | Slight: possible superficial damage which is unlikely to have structural significance |

* monitoring results for the first building, ** for the second building presented in this paper

8 DISCUSSION

The planned excavation sequence and deep embedment (about 18m and 8m below the excavation and the fixture respectively) of the walls are considered to have contributed to minimizing the effect of unbalanced load conditions, particularly for the walls alongside the river.

Instrumentation results, particularly from the monitoring of wall movements have given effective warning against improper installation of bracing systems and excavation sequences. This allowed improvement or modification of the bracing system where necessary.

No damage was found in adjacent buildings, as confirmed by instrumentation results, particularly those from vertical beam sensors and tiltmeters. In comparing bracing systems for the first and second building works, stiffness of the first level bracing used for the second building is estimated to be only 50% of that for the first building while expected strut force is 25% less. Moreover single strut arrangement in the first level bracing for the second building is less rigid compared to dual strut arrangement which can act as a composite beam for the first building. Instrumentation on struts confirmed that bracing system used was adequate.

In both cases, early installation of bracing, particularly for the first level is very important for minimizing wall top movements, subsequently reducing accumulated wall movements for further excavation.

9 CONCLUSION

With proper instrumentation, two basement excavations using braced diaphragm walls subjected to unbalanced lateral loading from the river and adjacent buildings were completed.

Performance of the walls based on the instrumentation results are presented and discussed.

For underground construction work, monitoring with instrumentation plays a major role, especially when delay occurs during excavation. Moreover, overall construction cost, time and risk can be minimized with the use of instrumentation.

10 ACKNOWLEDGEMENT

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REFERENCES


Analysis of lateral wall movement for deep excavation in Bangkok subsoils

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ANALYSIS OF LATERAL WALL MOVEMENT FOR DEEP BRACED EXCAVATION IN BANGKOK SUBSOILS

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ABSTRACT

This paper presents the case histories and back analysis of diaphragm walls for deep-based excavations in Bangkok subsoils. Three case studies with different boundary conditions are detailed as under unbalanced loading, underpass project under very narrow clearance under the first stage expressway, and short embedded wall for modified deeper excavation. The back-figured values for the soil stiffness in terms of Young’s modulus are calculated as Eu/Su = 500 and 2000 for the soft Bangkok clay and stiff clay, respectively. The results have later on been confirmed with the first self-boring pressuremeter tests in Bangkok subsoils.

INTRODUCTION

Deep excavations in Bangkok City are still necessary even the country has faced severe economic crisis. There are still required the necessary civil works to serve the demands and also to solve the environmental problems such as the first underground subway project, and the wastewater treatment schemes. Subsoils condition in Bangkok is generally consists of a very soft dark gray of about 12-14 m thick layer underlain by stiff and very stiff clay to about 23-25 m depth then followed by the first dense silty sand layer. Excavation in such soft dark gray clay requires efficient retaining structures such as temporary sheet pile for shallow excavation, and cast in-situ reinforced concrete (RC) diaphragm wall as permanent structures for deep excavation. There are a lot of diaphragm wall activities under construction and under excavation stages for the first blue line subway project, Metropolitan Rapid Transit (MRT). This subway project consists of 18 underground stations, about 20 km long twin tunnels running from North of Bangkok at Bangsue station and turn around the business area, bus terminal and ended at the main state railway station namely Hua Lumpong station (Teparaksa, 1999). Excavation depth for the underground stations is between 22-33 m.

This paper aims to evaluate the soil stiffness parameters of the soft and stiff Bangkok clay for RC diaphragm wall based on back analysis of three case studies with different construction boundaries.

SOIL PARAMETERS FOR DIAPHRAGM WALL DESIGN

The RC diaphragm wall can be designed by various methods such as Finite Difference Method, Beam on Elastic Foundation Method, and also the Finite Element Method (FEM). The most well known and commonly used method is the two dimensional (2D) FEM method or plain strain method. The 3D FEM analysis is rarely used. The FEM analytical method can simulate the soil-structure interaction, and can predict the lateral wall movement as well as the
ground surface settlement. The appropriate soil modelings as well as the input soil parameters are the key issues to simulate the system. The most important soil parameter is the stiffness of each soil layer, which is normally specified in terms of Young’s modulus (E). The E value is very important especially for the soft clay layer that lead to induce the lateral wall movement and ground settlement during excavation works. Normally, E is assumed to be linear and determined from the conventional triaxial compression test and is very low as compared to the values obtained from the empirical correlation. This is because of the high strain levels at which the E is measured in the conventional triaxial test. The E value is recently known to be dependent on the order of shear strain level. Mair (1993) compared the soil stiffness at various shear strain levels for different structural systems as well as various kinds of laboratory tests as shown in Figure 1. In case of concrete diaphragm walls, the shear strain is of the order of 0.01% to 0.10% only.

The research works on the soil behaviors at the low strain level in soft Bangkok clay was presented by Shibuya et al (1997) based on laboratory tests as well the in-situ field vane shear tests. Authors presented the soil stiffness in terms of Maximum Shear Modulus (Gmax). Gmax is about 300*Su (300 times of Undrained Shear Strength) to 500*Su and if converted to the Undrained Young’s modulus (Eu) will be equal to 900*Su to 1500*Su for Poisson ratio (ν) of 0.5. Teramast (1998) also presented the soil stiffness at low strain levels based on the Bender Element Test as Gmax = 400*Su to 570*Su or Eu = 1200*Su to 1710*Su. As shown in Figure 1, Mair (1993) mentioned that the Bender Element Test is suitable for very low strain levels such as applied to the dynamics or earthquake problems. Most practical value of Eu for Bangkok subsoils is based on the empirical correlation proposed by Duncan & Buchigani (1976) as shown in Figure 2. The Eu values were presented relative to the undrained shear strength, which is dependent on the order of Over Consolidation Ratio (OCR). For Bangkok soft clay where OCR is about 1.6, Eu/Su = 300-600.

BACK ANALYSIS CASE STUDIES

As mentioned above that the soil stiffness varies and depends on the shear modulus. However, the values from laboratory tests give a high variation with the empirical correlation. Therefore, the back analysis from case histories of diaphragm wall construction is really
necessary to verify the appropriate soil stiffness. The back analysis was carried out for three case studies at the Thammasat University project, Dindang underpass project and Sathorn Complex project. Using the 2D-FEM program namely “PLAXIS” (Brinkgreve & Brand, 1996) back analysis for the said projects have been carried out. The analytical plain strain concept and the Mohr-Coulomb soil model or bi-linear modeling was used.

**Thamasart University Project**

The building site is located nearby the Chao Phaya River and surrounded by a historical building seated on shallow raft foundation as shown in Figure 3. The diaphragm wall along the river was constructed about 3-4 m away from the existing old and shallow river wall as shown in the building section in Figure 4. The river bed near the river wall is about 3 m in depth and sloping towards the mid-stream to a depth of about 10-12 m (Thasananipan et al, 1998). This riverbank lead to induce the unbalanced lateral earth pressure on the diaphragm wall system between land side and riverside. A 800 mm thick diaphragm wall was designed for excavation down to 12.7 m below the ground with two levels of temporary bracing at –1.5 m and –6.5 m depth. The diaphragm wall toe was embedded down to 28 m to achieve the overall stability of the excavation which is right on the river bank (Fig. 4). A site investigation was carried out by two boreholes and four field vane shear tests, the subsoil properties are summarized in Table 1.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Depth (m)</th>
<th>Wn (%)</th>
<th>Unit Weight, $\gamma_s$ (kN/m$^3$)</th>
<th>Cu (kPa)</th>
<th>SPT-N (Blows/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crust</td>
<td>0.0-2.5</td>
<td>50</td>
<td>19</td>
<td>30</td>
<td>-</td>
</tr>
<tr>
<td>Soft Clay</td>
<td>2.5-12.7</td>
<td>70-85</td>
<td>26</td>
<td>15-30</td>
<td>-</td>
</tr>
<tr>
<td>Medium Clay</td>
<td>12.7-14.0</td>
<td>50-55</td>
<td>17</td>
<td>30-45</td>
<td>-</td>
</tr>
<tr>
<td>Siff Clay</td>
<td>14.0-25.0</td>
<td>20-25</td>
<td>20</td>
<td>130-250</td>
<td>14-40</td>
</tr>
<tr>
<td>Dense Sand</td>
<td>&gt;25.0</td>
<td>-</td>
<td>20</td>
<td>-</td>
<td>&gt;39</td>
</tr>
</tbody>
</table>

**Instrumentation**

Due to the locality and particular conditions of the site, more instrumentation than those used by other projects in Bangkok were installed and monitored. Three types of instrumentation - Eight inclinometer tubes are installed in diaphragm wall panels to monitor displacement of wall, ten settlement plates of 1.0m and 5.0m in depth around excavation zone, and ten tiltmeters and four vertical beam sensors on surrounding buildings were installed to monitor tilting. Layout of instrumentation is presented in Fig. 3. Since the excavation work is located adjacent to the river and surrounded by old buildings, the diaphragm walls are subject to the unbalanced lateral load The model analysis indicated relatively less movement of Wall D1 towards excavation in the land side and move less in the river side as well as induce forward movement of river wall to the river (Fig. 5). This analysis led the engineer to solve the damages of the river wall due to unbalance loading by using the following construction methods.
1. Arrange the strut as a simple straight grid line
2. Pre-loading of the struts (200kN/m and 400kN/m for first and second levels respectively) on the one end of the strut on the land side wall.
3. Excavating the soil in front of Wall river side first at any excavation stage
Analytical Results

The analysis was trial by changing the soil stiffness and compares the results with the field performance. Figure 6 shows the results of FEM analysis compared with inclinometer measurements based on soil stiffness of Eu/Su = 500 and 2000 for soft Bangkok clay and stiff clay, respectively. Movements of Wall D1 close to the river are found to be cantilever shape indicated by I-7 while movements of Walls D2 and D3 have inward bulging shape indicated by I-1 to I-5. Generally the predicted and measured lateral wall movements are in a good agreement, except for the top portion of wall in which the measured wall movements exceed the predicted movements. The differences in wall movement were found to be caused by the following:

- Advancement of excavation in front of Wall D1 further than other walls, allowing Wall D1 to stand longer prior to strut installation than the others.
- Wall D1 was in cantilever condition for about 3 months after initial excavation for first level bracing.

Figure 6: Comparison between calculations and observation (Thammasat University Project)

Dindang Underpass Project

The Dindang underpass was the first Bangkok underpass project at Dindang intersection. Underpass the main road beneath the main foundations of the expressway. This project is also very interesting, specially designed equipment for D-wall construction was employed under very narrow clearance under the first stage expressway (Thasananipan et al. 1996). There was one temporary bracing layer at 2 m. depth and excavation to 4.9 m. depth. Figure 7 shows the results of FEM analysis compared with inclinometer measurements based on soil stiffness of Eu/Su = 500 and 2000 for soft Bangkok clay and stiff clay, respectively. The predicted and measured lateral wall movements are in a good agreement.
Figure 7: Comparison between prediction and observation (Dindang Underpass Project)

**Sathorn Complex project**

The project site is located in the vicinity of central business area of Bangkok. The structure is a multi-storied building for a business complex. The load of the structure would be carried by bored cast-in-situ bored piles and barrette piles with toe depth of 60m below the ground level, embedded into the second sand layer of the Bangkok subsoil. The constructed cast in-situ diaphragm wall is 0.8m thick and the toe is embedded into stiff clay at a depth 18m below the ground level. Initially the wall was designed for construction of four basements including mat foundation of 3.5m thick. The maximum excavation depth was set to be 14m and a three level of temporary bracing system was considered to construct the basements with conventional bottom up construction method. Embedded depth below the final excavation for the mat foundation was designed to be 4m. Figure 8 shows the layout plan of the building.

After construction of the wall it was decided to increase the number of basements to be five for part of the building area (tower area) and excavation depth needed to be increased by 1.8m from the originally designed depth, leaving the wall embedment depth of 2.2m only. The first author was invited to solve such a short embedded diaphragm wall (Teparaksa et al., 1998). The temporary bracing was changed from three layers to be four layers with using the soil berm of 12 m. wide to prevent the excessive wall movement as shown in Figure 9.
The sub soil conditions at the project site is not uncommon to usual conditions observed in Bangkok comprising a 15m thick soft to medium clay layer on the top underlain by stiff clay to a depth of about 22m. Below the stiff clay is a series of alternating layers of hard silty clay, sandy clay and dense sand. The soil properties used for Diaphragm wall design are summarized in the table 2 belows:

**Table 2  Soil Properties used for D-Wall Design**

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Unit Weight γs (kN/m³)</th>
<th>Undrained Shear Strength, Su (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft Clay, CH</td>
<td>16</td>
<td>10-29</td>
</tr>
<tr>
<td>Medium Clay, CH</td>
<td>18</td>
<td>35</td>
</tr>
<tr>
<td>Stiff Clay, CH</td>
<td>20</td>
<td>137-180</td>
</tr>
</tbody>
</table>
**Analytical Results**

The wall constructed was designed for initial excavation of -2.5m to install the first bracing layer. However the excavation has been done to about -3.0m to -3.2m at once and a movement of 40mm was observed. Time delay during construction of capping beam and over excavation deeper than the critical height \( H_{cr} = \frac{2C}{\gamma} \) was unsupported around three months resulted an increase in wall movements to 55mm -70mm. Tension cracks were developed in the soil of active side about 2-3m away from the wall. After installation of struts and pre-loading them, the lateral movements of the wall ceased off. Struts from first to fourth levels were pre-loaded 40 t/m, 75 t/m, 75 t/m and 40 t/m respectively, 20%-25% of design load of bracing system, prior to further excavation. The pre-loading of the strut was very effective to reduce lateral movement of the wall and ensuring good intact between wall and bracing system. Figure 10 shows the comparison of the results of FEM analysis compared with inclinometer measurements based on soil stiffness of \( \frac{E_u}{S_u} = 500 \) and 2000 for soft Bangkok clay and stiff clay, respectively. The measuring data was neglect the first movement due to the delay of the first cantilever diaphragm wall around three months. The predicted and measured lateral wall movements are in a good agreement.

![Figure 10: Comparison of wall movement from FEM analysis and inclinometer results](image)

**COMPARISONS OF ANALYSIS WITH PRESSUREMETER TEST**

The trial soil stiffness based on the back analysis shows that the Young modulus of soft Bangkok clay and stiff clay are \( \frac{E_u}{S_u} = 500 \) and 2000, respectively. Recently, six self-boring pressuremeter tests were performed along the route of Metropolitan Rapid Transit (MRT) northern contract by the Cambridge In-situ of Little Eversden (1997). The self-boring pressuremeter tests were carried out by reload/loading cycles in order to determine the soil stiffness at various shear strain levels. Figure 11 shows the results of pressuremeter test and
shows the variation of shear modulus with shear strain, Figure 11.1 is for soft clay while Figure 11.2 is for first stiff clay. For normal practice of diaphragm wall construction for deep basements in Bangkok soft clay, the shear strain is in the order of 0.1-0.2%, where the G/Su is about 160 or Eu/Su = 480. In case of the first stiff silty clay layer where shear strain is in the order of 0.05-0.1%, the G/Su is about 340 or Eu/Su = 1020. The soil stiffness of the soft clay from pressuremeter test results is very close to the results of back analysis, however, the values of stiff clay from pressuremeter test results is lower than the results of back analysis, this is might due to the effect of bored pile stiffness inside of the excavation area. The correlation from the self-boring pressuremeter was the non-linear soil model that will be used to carry out the future back analysis.

CONCLUSIONS

The analysis and the design of the diaphragm wall are normally based on the assumption of soil stiffness. Back analysis to verified the soil stiffness based on three different cases as under unbalanced loading, underpass project under very close to the foundations of the first stage expressway, and short embedded wall. The results shows that the soil stiffness in terms of Young modulus is Eu/Su = 500 and 2000 for soft Bangkok clay and stiff clay, respectively. These values were also corresponding to the results of self-boring pressuremeter tests of the first blue-line subway project in Bangkok.

REFERENCES


Mair, R. J. (1993): Development in geotechnical engineering research, application to tunnels and deep excavations, Proceedings of the Institution of Civil Engineers and Civil engineering.


Thasananipan N., Teparaksa W., Muang A. and Baskaran G., (1998): Design, construction and behavior of bored cast in-situ concrete pile in Bangkok subsoils, Proc. 4th Int. Conf. on Case Histories in Geotechnical Engineering, St. Louis, Missouri, USA.


Teparaksa W., Thasananipan N., Muang A., and Wei S. (1998): Prediction and performances of short embeded cast in-situ diaphragm wall for deep excavation in Bangkok subsoils, Proc. 4th Int. Conf. on Case Histories in Geotechnical Engineering, St. Louis, Missouri, USA.


Diaphragm wall and barrette construction for Thiam Ruam Mit Station Box, MRT Chaloem Ratchamongkhon Line, Bangkok

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DIAPHRAGM WALL AND BARRETTE CONSTRUCTION FOR THIAM RUAM MIT STATION BOX, MRT CHALOEM RATCHAMONGKHON LINE, BANGKOK

Narong Thasnanipan¹, Aung W. Maung² and Ganeshan Baskaran³

ABSTRACT

Construction experience of cast-in-situ diaphragm walls and foundation barrettes for the Thiam Ruam Mit Station, Chaloem Ratchamongkhon line of Metropolitan Rapid Transit Authority (MRTA) is briefly reported in this paper. The construction activities in the midst of heavily congested traffic and in the vicinity of underground utilities to meet the specified quality and safety controls are also presented.

INTRODUCTION

The Chaloem Ratchamongkhon line is the first underground mass rapid transit project in Bangkok, Thailand being constructed under the supervision of Metropolitan Rapid Transit Authority (MRTA). The project involves the construction of about 22km-long, twin bored, single track tunnels, 18 station boxes and cut-and-cover tunnels. The project was divided into two separate contracts, South and North. The south contract was awarded to BCKT Joint Venture consisting of Bilfinger + Berger Bauaktiengesellschaft, Ch. Karnchang Public Co., Ltd., Kumagi Gumi Co., Ltd. and Tokyu Construction Co., Ltd. The north contract was awarded to ION Joint Venture, comprising Italian Development Public Co., Ltd., Obayashi Corporation and Nishimatsu Construction Co., Ltd. This paper presents experience on construction of diaphragm walls and foundation barrettes for Thiam Ruam Mit Station (station no. 12) of the North contract where heavy traffic conditions prevail.

The station is one of the biggest stations and located on the Ratchadaphisek Road. The station box is enclosed by principally 1.0m thick side walls and 1.20m thick end walls 32.0-42.0m deep. A Tunnel Boring Machine (TBM) launching shaft was included in the north side of the station. To facilitate top-down construction, 63 Barrettes (1.2mx3.0m) embedded 44.5-55.0m deep in conjunction with pre-placed stanchions at the top were used.

Figure 1. Layout of MRT Chaloem Ratchamongkhon line

Three basement levels were planned for the station. The excavation depth inside the station box was up to 24.5m for tunneling and basement slab construction. As deep excavation required stringent tolerances for the diaphragm walls and barrettes, a high level of quality control was necessary.

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Construction of diaphragm walls and barrettes of the Thiam Ruam Mit station was carried out by an experienced local foundation contractor in which the authors worked while international contractors undertook similar work for the remaining 17 stations.

**SUBSOIL CONDITIONS**

The subsoil profile, as shown in Fig. 2 consists of made-ground up to 2.5m thick for pavement underlain by a series of clay and sand layers. Below the made-ground is 12.0m to 14.8m-thick Bangkok Soft Clay. The first stiff clay layer occurs below 14.0m depth and extends to 21.0m, overlying the first sand layer. The sand layer is about 20.0m thick at this location and has two thin (about 2-4m thick) very stiff clay layers at the top. Below the sand layer is stiff to hard clay layer which extends beyond 60.0m in depth. Within the clay layer, from 48.0m depth, a 14m thick sand layer occurs in the north and lenses out to the south. The properties of soil layers are summarized in Table 1.

![Profile of soil layers and station box](image)

**Figure 2. Profile of soil layers and station box.**

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Depth to Top of Layer (m)</th>
<th>w %</th>
<th>$\gamma_s$ kN/m$^3$</th>
<th>$C_u$ kPa</th>
<th>SPT- N Blow/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Made-Ground</td>
<td>0.0</td>
<td>40</td>
<td>17.0-19.0</td>
<td>30-45</td>
<td>5-9</td>
</tr>
<tr>
<td>Bangkok Soft Clay</td>
<td>0.2-2.6</td>
<td>64-100</td>
<td>14.6-16.8</td>
<td>15-31</td>
<td>-</td>
</tr>
<tr>
<td>Stiff Clay</td>
<td>14.2-17.3</td>
<td>16-51</td>
<td>18.3-21.8</td>
<td>107-250</td>
<td>13-17</td>
</tr>
<tr>
<td>Stiff to Hard Clay</td>
<td>40.8</td>
<td>16-20</td>
<td>21.0-21.1</td>
<td>61-410</td>
<td>11-61</td>
</tr>
</tbody>
</table>

The depth of ground water table at the site generally varies from 0.9m to 2.2m. Piezometers installed in the project area indicate the pore pressure is hydrostatic below 23m depth due to pumping of ground water.

**SITE CONDITION AND PREPARATION**

![Layout of station box](image)

**Figure 3. Layout of station box**

**Traffic Management**

The construction site was on a 35.0m-wide main road with 4 inbound and 4 outbound traffic lanes. During the construction period a maximum of only two traffic lanes could be occupied at a time, allowing 3 or 4 lanes for each traffic direction. The width of the working area available was about 11.0m. Sometimes, the
width of the work area needed to be widened locally to position the grab crane for diaphragm wall panel excavation.

**Utility Diversion**

The site is bounded by some underground utilities, including a 1.0m diameter water pipe and telecommunication cables in the close vicinity of the work area, particularly on the east side. Above ground, 69kV electricity lines, telephone cables, drainage and sewage pipes (1.5m dia.) lay on both sides. The water pipe ran along the planned diaphragm wall at about 3.0m depth and thus it was diverted by the other subcontractor to the west, outside the station box prior to construction. The face-to-face clearance distance between the diverted water pipe and the outer face of the station wall was only 35cm due to space constraints. A reinforced concrete wall to protect the pipe was constructed and it formed the bottom part of the guide wall for diaphragm wall panel excavation.

Two duct banks of the telecommunication cables were located 5-10cm away from the wall face at a depth of 3.04m. 15mm thick steel plates were used in these locations as both guide walls and protection systems for the duct banks.

**CONSTRUCTION SPECIFICATION**

Generally construction specifications were in line with the commonly practised specification for foundation works. Regarding construction tolerances, verticality of the walls and barrettes is specified within 1:200 while for pre-founded steel stanchions, the verticality allowed is within 1:400.

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Specification</th>
<th>Freshly Mixed</th>
<th>Prior to Placing Rebar &amp; Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>API RP13B-1 Section</td>
<td>&lt;1.05g/ml</td>
<td>&lt;1.10g/ml</td>
</tr>
<tr>
<td>Viscosity</td>
<td>3</td>
<td>29-50</td>
<td>29-50</td>
</tr>
<tr>
<td>Fluid loss</td>
<td>4</td>
<td>&lt;30ml</td>
<td>&lt;30ml</td>
</tr>
<tr>
<td>Sand content</td>
<td>7</td>
<td>&lt;1%</td>
<td>&lt;2%</td>
</tr>
<tr>
<td>pH</td>
<td>9</td>
<td>8-12</td>
<td>8-12</td>
</tr>
</tbody>
</table>

Properties of supporting fluid for wall and barrette excavation were maintained within the ranges as shown in Table 2. In addition, design assumptions required that total construction time below the soft clay layer was limited to minimize the effects of bentonite on shaft friction and in order to achieve optimum shaft friction capacity of foundation elements.

**CONSTRUCTION PHASES**

Throughout the project, the main construction schedule for the station comprised several interfacing and parallel work activities such as for utilities and traffic diversions, temporary decking, substructure work for station box and the TBM driving shaft which other subcontractors undertook. This meant that guide wall construction in particular needed to follow utility diversion as the same area was to be excavated for relocation of the utilities, removal of abandoned water pipes and construction of guide walls. The temporary decking and excavation for the TBM launching shaft followed the diaphragm wall construction.

**DIAPHRAGM WALL AND BARRETTE CONSTRUCTION**

**Equipment and Plant**

Mechanical rope-suspended grabs with crawler cranes (80tons) and service cranes (50-80tons) were used as main construction equipment. Grabs, 1.0mx2.0-3.0m and 1.2mx3.0m were used to cut the required dimensions of panel excavations. Bentonite slurry was used as supporting fluid for panel and barrette excavation. Silos having a total of 300 cu.m storage capacity and de-sanding and de-silting units (80cu.m/hr capacity) were used to supply the slurry required for up to two-panel excavation at a time.
Guide Walls

 Generally 1.5m deep guide walls were used for panel excavation. In the location of underground water pipes and duct banks, up to 3.0m-deep guide walls were used. Individual panels were sized with the following considerations: (1) type of reinforcement, (2) size of grabs used, (3) thickness and depth of panels, (4) location of slab and wall openings for entrances and TBM, (5) stability of trench in connection with location of underground utility and (6) location of public accesses and streets.

Panel Layout

 The walls were divided into 184 panels with horizontal lengths varying from 2.5m to 4.85m. L-shaped and T-shaped panels were used to form box structures of the station and TBM launching shaft (see Fig. 5).

Sequence of Construction

 Construction activities took place day and night. To maintain the traffic flow, comply with local authority regulations, construction sequence and activities were planned to allow concrete pouring and spoil removal to be done before and after rush hours, in most cases at night. Generally panel excavation was carried out at least 6.0m away from the recently cast panel or 16hrs after casting of the adjacent panels to avoid any damage induced by excavation.

Supporting Fluid

 Locally available bentonite powder was used for preparation of slurry. This product had been used for more than a dozen completed diaphragm wall projects. Approximately 496 tons of bentonite powder was consumed for excavation work. The last portion of slurry displaced by concrete or contaminated with cement was usually discarded. Soil volumes of about 26,274cu.m and 10,740cu.m were excavated for diaphragm walls and barrettes respectively.

Reinforcement

 Steel bars of SD50 were used for the main reinforcement bars. The reinforcement cages were 30.0m to 42.2m long and up to 4.5m wide, with 4 levels of box-out for slab connections for diaphragm wall panels. For barrettes, cages were 48.1 to 54.6m long. 4 to 5 cage sections with lengths of 6-12m were joined to form one continuous cage for individual panels and barrettes. The cage sections were fabricated off site and no more than two were joined for transporting to the site. Cage sections were connected prior to and during lowering into the trench. In order to place the box-out in position and to achieve an exact match of the cage sections, markings were made on the main bars of each cage. U-shaped bolts were used for cage connections. Couplers were generally used for slab connection and both couplers and bent-out bars were used for beam connections at the wall openings for entrances and slab openings. The large panel cages weighed up to 34.8 tons.

For panels with glass fibre reinforcement polymer (GFRP) for openings of TBM break-through, a temporary steel frame was necessary to hold the cage section due to different stiffness between GFRP and steel. The frame was then cut away section by section while lowering the cage. Diaphragm wall panels were rein-
forced with 136-218 kg/m² of steel bars while barrettes were reinforced with about 50 kg/m³ of steel bars. The wastage of steel was about 7% in which most quantity was consumed for cage hanging bars and frames for soft eye openings. A total of 4,756.15 tons of reinforcement bars was used for diaphragm wall and barrettes. Steel pipes 150mm in diameter were installed inside the diaphragm wall panels at 5 locations as a void former for inclinometer access tubes. 28 sets of vibrating wire strain gauges and total jack-out pressure cells were also installed in the wall panels of TBM launching shaft.

Concrete Casting

Ready mixed concrete grades 40 (cube strength 40 MPa at 28 days) and 35 (35 MPa at 28 days) to BS5328 were used for diaphragm walls and barrettes respectively. Two sets of tremie pipes were used in pouring concrete for both diaphragm walls and barrettes. The largest single concrete pour was about 232 cu.m for L-shaped panels. Total concrete volumes of 27,480 cu.m and 6,611 cu.m were poured for diaphragm walls and barrettes respectively. Concrete over-consumption in diaphragm wall caused by old, sand back-fill in the area of abandoned water pipes was estimated to be an average of 7.8% with a concrete wastage of up to 38.0% occurring in one panel. However, the average concrete wastage was 5.0%. For barrettes, the average concrete wastage was up to 14%. The high wastage compared to building and elevated highway projects (up to 7%) was caused by overcasting of concrete well above the cutoff level of 22m depth to ensure that sound concrete reached above the cutoff. Aggregates were used to backfill the open trench of barrettes above the concrete after casting.

Installation of Stanchion in Barrettes

Conventional plunging method was not adopted in view of the site conditions, low cutoff level, the stanchion design and horizontal bar arrangements in the reinforcement cage. Two specially designed installation frames with adjustable screws were used for installation of stanchions to ensure high accuracy for verticality and plan position. A removable 10m-long, steel column was fixed on top of the stanchion to check the verticality and alignment.

PRODUCTION RATES

The construction time required for a typical panel and barrette is detailed in Table 3.

Table 3. Average construction time for typical diaphragm wall panels (1.0x4.5x35.0m, cutoff 0.0m) and barrettes (1.2x3.0x44.5m, cutoff -22.5m).

<table>
<thead>
<tr>
<th>Activities</th>
<th>Diaphragm wall Hours</th>
<th>Barrette Hours</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavation</td>
<td>18</td>
<td>16.5</td>
</tr>
<tr>
<td>Checking verticality with Koden</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Desanding</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Cage installation</td>
<td>2.5</td>
<td>4.5</td>
</tr>
<tr>
<td>Stanchion installation</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>Tremie preparation</td>
<td>2.5</td>
<td>3.5</td>
</tr>
<tr>
<td>Concrete pouring</td>
<td>4</td>
<td>3.5</td>
</tr>
</tbody>
</table>

Figure 6. Guide frame for steel stanchion installation in barrettes

Throughout the construction period, a total of about 867 hrs, approximately 15% of construction time (excluding mobilization and site preparation period) was consumed by equipment maintenance and repair. Grab teeth were broken and changed often during chipping off the concrete overbreak behind the stop-end plate caused by loose material below the guide walls. Additionally, steel wire ropes for the grab also needed to be changed frequently as they were worn out by both the chiseling action and broken concrete pieces. 17% of construction period was rainy, but this usually had no significant effect on the production rate. A flash flood
occurred at one time by a heavy downpour. Standby time (14% of construction period) took place and was contributed by (1) obstructions by overbreak concrete during panel excavation and stop-end plate removal and (2) standing time after panel excavation reached below the soft clay to allow subsequent construction activities such as spoil removal and concrete delivery taking place outside rush hours. Traffic diversions were carried out in multi-phases and took at least 5% of construction period.

QUALITY, SAFETY AND ENVIRONMENTAL CONTROLS

As the alignment of diaphragm wall and barrettes is critical, every excavated panel and trench was checked with Koden monitoring equipment. Monitoring was carried out after excavation reached about 22.0m (ie. at the levels of base slab and barrette cutoff) and toe levels. Necessary corrections to trench verticality were made during excavation if required. Panel ends, especially for those panels with soft-eyes for TBM break-through were also checked to achieve high levels of accuracy for positioning. During excavation, reference posts were used for checking the grab position and observing rope position relative to the trench sides at all times. Prior to lowering the reinforcement cage into the trench, any mudcake built-up on the trench faces and panel joints was scraped off by a grab to which brushes were attached. Finally, any sediment or loose materials deposited at the bottom of the trench were also removed by using the grab.

Reference bars attached to the top of reinforcement cage were extended above ground level to check the position of the cage with survey equipment during installation of the cage into the trench.

Delivered concrete was checked for slump and cohesiveness prior to casting. As per specifications, adequate embedding length of tremie pipes in the concrete and slurry properties were maintained to achieve good-quality concrete casting. Plugging materials and shutters were introduced in the tremie pipes to separate first concrete pour and slurry in the trench. Samples of reinforcement bars and concrete were taken as specified for testing in the lab. 37 barrettes were provided with 6 steel tubes each for sonic logging to test the barrette concrete quality. No significant anomalies were detected by sonic logging test, and integrity and quality of barrette were found adequate.

As the site was in a busy public area, site and public accesses were kept clean all the time. Flagmen were also provided for traffic control at each construction zone. All workers were inducted in safety procedure prior to assuming duty. Construction activities were also supervised by a full-time safety officer. The safety officer ensured that all labor used mandatory personal protection equipment and observed the safety regulations. All heavy equipment and cranes were checked for safety and certified for operation prior to use and for regular maintenance. Additionally, regular safety patrols and inspection were also conducted by the main contractor and supervising engineers.

CONCLUSION

Construction work on the public road, involving various activities parallel with different teams/subcontractors demanded comprehensive construction sequences and planning to meet the targeted milestones. Proper coordination, planning, supervision and strict safety and environmental control by the parties involved thus brought about the successful completion of diaphragm wall and barrette construction on one of the busiest roads in Bangkok without any serious accident. When the paper was finalized, excavation inside the station box had been completed and it revealed good quality of workmanship without requiring any significant remedial work.

ACKNOWLEDGEMENT

The authors wish to express their appreciation to the MRTA and Nishimatsu Construction Co. Ltd. for their permission to publish this paper.

REFERENCES

Monitoring of diaphragm wall displacement and associated ground movement, brace excavation adjacent to historical building at the bank of Chao Phraya River

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ABSTRACT:
Modern buildings in Bangkok frequently require the excavation works for the basement facility adjacent to the existing structures. Diaphragm walls have been commonly used as permanent retaining walls for excavation works in various projects. Comprehensive monitoring system is of essential for any excavation works particularly for those adjacent to the sensitive buildings. This paper presents the monitoring of diaphragm wall displacement and associated movement of the ground and adjacent structures including historical building using a number of instruments for basement construction works located at the bank of Chao Phraya River. The bracing system and construction sequence adopted for excavation with the consideration of the presence of an unbalanced loading condition are also briefly discussed. Building damage risk assessment using a simple approach was carried out based on the instrumentation results and compared with the damage criteria established by published research work. A course of action undertook in response to the observed critical movement is also presented.

KEYWORDS:
Diaphragm wall, Instrumentation, Unbalanced Loading

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MONITORING OF DIAPHRAGM WALL DISPLACEMENT AND ASSOCIATED GROUND MOVEMENT, BRACED EXCAVATION ADJACENT TO HISTORICAL BUILDING AT THE BANK OF CHAO PHRAYA RIVER

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ABSTRACT: Modern buildings in Bangkok frequently require the excavation works for the basement facility adjacent to the existing structures. Diaphragm walls have been commonly used as permanent retaining walls for excavation works in various projects. Comprehensive monitoring system is of essential for any excavation works particularly for those adjacent to the sensitive buildings. This paper presents the monitoring of diaphragm wall displacement and associated movement of the ground and adjacent structures including historical building using a number of instruments for basement construction works located at the bank of Chao Phraya River. The bracing system and construction sequence adopted for excavation with the consideration of the presence of an unbalanced loading condition are also briefly discussed. Building damage risk assessment using a simple approach was carried out based on the instrumentation results and compared with the damage criteria established by published research work. A course of action undertook in response to the observed critical movement is also presented.

KEYWORDS: Diaphragm wall, Instrumentation, Unbalanced Loading

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1. **INTRODUCTION**
Excavation works in soft ground inevitably induced the ground movements which in turn may damage the adjacent existing structures. Diaphragm walls are commonly used as permanent retaining walls for basement excavation of many projects in Bangkok. It is necessary to use this type of rigid retaining wall in excavation work to minimize the ground movement particularly where the work is to be carried out in close proximity to the existing sensitive buildings. This paper presents the monitoring of diaphragm walls displacement and associated movement of the ground and the adjacent structures including historical building using a number of instruments for completed construction works located at the bank of Chao Phraya River, Bangkok. Design and construction aspects of the project have been reported by Thasnanipan et al. (1999).

2. **PROJECT LOCATION AND CONSTRAINTS IMPOSED ON SITE**
The project is situated in the heart of an old established culturally-significant area. The project consisted of basement excavation works for two separate buildings with the support of braced diaphragm walls.

![Figure 1 Layout of excavation sites and instrumentation](image)

Two basements are located 40m apart, separated by an existing 5 storey building. As shown in Figure 1, construction site is located along the bank of Chao Phraya River and surrounded by existing buildings including a historical building. Along the river, the distance between diaphragm walls and existing old river wall is about 4m and 8m for the first and second buildings respectively. Locations of the excavation sites themselves posed some constraints, which called for the need of careful consideration in establishing the design principles and sequence of construction. The major constraints imposed on sites are outlined below.

2.1 **Loading condition**
Thasnanipan et al. (1999) reported the presence of unbalanced loading condition. As the excavation areas are located immediately next to the river and surrounded by existing buildings, the walls alongside the river and the opposite walls were particularly expected to undergo an unbalanced lateral loading condition. No case history data of similar condition in the area were available at the time of design and construction of the basements.

2.2 **Risk of damage to the existing buildings**
The existing buildings located in the proximity of the excavation areas are of old structures. According to the available information, the buildings are supported by the piled foundation. However, no detailed information such as type, size and length of the piles was available. The most critical structure is the historical building which is a predominantly masonry and only 5m away from the outer face of the diaphragm wall of the first excavation site. As the building is important for its historical heritage, a damage category worse than negligible (as defined by Rankin 1988) would not be acceptable.
3. **SUBSOIL INVESTIGATION**
Two stages of site investigation were made. Preliminary stage of site investigation was performed during the tendering stage. Second stage of soil investigation was conducted prior to designing temporary bracing and basement excavation work. As subsoil properties obtained from the boreholes and test data were well reported by Thasnanipan et al. (1999), they are not presented again in this paper.

4. **DIAPHRAGM WALL AND BORED PILES**
For both buildings, 800mm thick cast in-situ concrete diaphragm walls of 28m toe depth with two level temporary bracings were designed for basement excavation. The maximum excavation depths for the first and second buildings were 12.7m and 9.7m respectively. Deep diaphragm walls of toe depth 28m were necessary to ensure the overall stability considering the location of the basement excavation at the bank of the river. Bored piles of diameter ranging from 800mm to 1500mm founded at 48m depth were constructed by wet process using bentonite slurry.

5. **INSTRUMENTATION AND MONITORING PROGRAM**
Instrumentation played major role in this project. The primary objective of the instrumentation program was to monitor the performance of the excavation to ensure that diaphragm wall as well as the entire excavation system were stable and that adjacent structures were not adversely affected. Furthermore, the instrumentation program was established to provide the main feedback in application of observational method. Types of instrumentation used in the project are presented in Table 1. Layouts of instrumentation for the first and the second buildings are shown in Figure 1.

<table>
<thead>
<tr>
<th>Type of Instrumentation</th>
<th>First Building</th>
<th>Second Building</th>
<th>Monitored Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inclinometer</td>
<td>8</td>
<td>6</td>
<td>Wall deflections</td>
</tr>
<tr>
<td>Settlement Plate</td>
<td>10</td>
<td>20</td>
<td>Ground settlements</td>
</tr>
<tr>
<td>Tiltmeter</td>
<td>10</td>
<td>10</td>
<td>Tilting of Buildings</td>
</tr>
<tr>
<td>Vertical Beam Sensor</td>
<td>5</td>
<td>5</td>
<td>Tilting of buildings</td>
</tr>
<tr>
<td>VWSG</td>
<td>-</td>
<td>1 location*</td>
<td>Strain in rebars</td>
</tr>
<tr>
<td>Earth Pressure Gauge</td>
<td>-</td>
<td>2 x 2</td>
<td>Strut forces</td>
</tr>
</tbody>
</table>

* A pair of VWSG in 5 layers along the depth of wall panel

6. **EXCAVATION METHOD AND BRACING SYSTEM**
Conventional bottom–up method with two levels of temporary bracing was applied for both buildings excavation. As reported by Thasnanipan et al. (1999), to prevent adverse wall behavior alongside the river due to the presence of unbalanced lateral loading condition, pre-loading was applied at one end of struts on the land side wall with simple and efficient temporary bracing system. As the excavation work of the first building was completed 1 year before the commencement of the second building, monitoring results of the first excavation work provided an ample opportunity to review the design assumption and fine tune the parameters used in the analysis of the diaphragm wall for the second building. The major modification was to use single strut for the first level of the second building (less rigid but economical compared to that of the first building) with the provision of strut-force monitoring, according to Thasnanipan et al. (1999). Summarized information of excavation and bracing systems of two buildings is tabulated below.

<table>
<thead>
<tr>
<th>Building</th>
<th>Excavation Depth</th>
<th>Bracing Level</th>
<th>Bracing Elevation</th>
<th>Strut Sections</th>
<th>Design Strut Force kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>-12.79m</td>
<td>I</td>
<td>-2.0</td>
<td>2 x WF350 x 350</td>
<td>484.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>II</td>
<td>-6.5</td>
<td>2 x WF400 x 400</td>
<td>789.0</td>
</tr>
<tr>
<td>Second</td>
<td>-9.70m</td>
<td>I</td>
<td>-2.0</td>
<td>1 x WF400 x 400</td>
<td>279.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>II</td>
<td>-7.0</td>
<td>2 x WF350 x 350</td>
<td>332.1</td>
</tr>
</tbody>
</table>
7. SETTLEMENT PREDICTION
Due to the presence of unbalanced loading condition, non-symmetric pattern of settlement distribution around the perimeter of excavation could be expected. However, considering the fact that pre-loading was to be used for the land side diaphragm wall but not for the riverside wall, settlement distribution was assumed to be symmetrical. Available methods were reviewed for predicting the ground movement with the consideration of two main factors such as, (1) simple and practical in application (2) enable to correlate with the predicted and measured diaphragm wall deflection.

7.1 Prediction of Surface Settlement
The method proposed by J. E. Bowels (1990) was selected as it meets the required criteria mentioned above. Bowels suggested the ground settlement induced by excavation as a function of ground loss due to the deflection of the retaining wall. Bowel demonstrated the calculation of settlements at specified distance by assuming parabolic variations of settlement within the influence distance. Using predicted diaphragm wall deflection, surface settlement behind the wall was computed by empirical formulas proposed by Bowels.

7.2 Prediction of Sub-surface Settlement
A simplified prediction of sub-surface settlement was made based on the calculated surface settlement described above. First, subsurface settlement influence line was constructed. As shown in Figure 2, settlement influence zone is assumed to decrease with depth from “D_0” at the surface and zero at the wall toe. With the assumption of linear relationship between the volume of deflected wall shape and the volume of settlement trough at any depth within settlement influence zone, subsurface settlement at different depths were calculated.

![Figure 2 Demonstration of settlement prediction from diaphragm wall deflection values](image)

8. INSTRUMENTATION RESULTS
8.1 Inclinometer Monitoring
The predicted and observed diaphragm wall deflections of the first building and second buildings are presented in Figure 3a and 3b respectively. As can be seen in the figures, observed wall movements exceeded the predicted values at the top portion of the walls for both buildings. The delay in installing the first level bracings, leaving the walls in cantilever conditions for the long period was the main reason of the large movement induced by the excavation at that stage.
At Inclinometer No. I-6 of the second building, wall movement was found to reach the trigger value at top portion when excavation reached 7.5m depth after the first level bracing installation. Lateral wall movement reached a maximum rate of 6.5mm/day and a tension crack on the ground was found about 8m away from the wall. A close examination of the bracing system indicated that one strut had swayed slightly about 130mm. Immediate actions were taken by stacking additional strut on the defective one and installing of the second level bracing carried out at that area. The wall movements were found to cease after these actions.

The wall deflections of the first building excavation found unaffected by the unbalanced lateral loading condition. However, for second building, monitoring results from inclinometer I-2 and I-4 suggested that the wall alongside the river was pushed against the retaining soil by the opposite wall.

8.2 Settlement and Building Movement

Figures 4a, 4b and 4c present the construction activities with time and corresponding monitoring data. As can be seen in the figures, monitoring results reflected the construction activities in general. It was also proved that performance and response of the diaphragm wall were largely influenced by the construction activities. It can be observed that significant changes in the responses were mainly occurred during the initial excavation stages in which walls were unsupported for long period. Monitoring data particularly that of settlement and building tilt measurement provided ample opportunity to assess the risk of the existing buildings damage. As the measured data were far less than the critical values, the excavation works were proceeded with confidence.

8.3 Strut Force Measurement

Daily monitoring was carried out for pressure gauges installed on struts. The comparison of predicted and measure strut forces are presented in Table 3. In general, the measured values indicated that the bracing system used was adequate.

<table>
<thead>
<tr>
<th>Bracing Level</th>
<th>Predicted Force (KN/m)</th>
<th>Measured Force (KN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Level</td>
<td>354.5</td>
<td>331.3</td>
</tr>
<tr>
<td>Second Level</td>
<td>307.3</td>
<td>206.3</td>
</tr>
</tbody>
</table>

Figure 3a. Lateral wall movements (1st Building) Figure 3b. Lateral wall movements (2nd Building)
9. BUILDING DAMAGE RISK ASSESSMENT

A simple approach was adopted for building damage assessment. Settlement and slope of the buildings due to the excavation works were predicted assuming the “green filed” condition. Neglecting restraints from foundation and structure, it was assumed that buildings follow the ground settlement trough at foundation level (estimated pile tip level). Using predicted and measured deflection values of diaphragm walls, surface and subsurface settlement profiles were prepared as shown in Figures 5 and 6. Building slopes likely to be induced by differential settlement were then predicted as presented in Table 4.

Figure 4 Monitoring results of instruments with construction time for First and Second Buildings
As can be seen in Table 4, the measured maximum tilt of the buildings reasonably agree with predicted building slopes derived from inclinometer measurement, particularly if the buildings were assumed to be settled by subsurface settlement at assumed pile tip levels. All buildings
were predicted to be in negligible risk category as classified by W.J. Rankin (1988). Field inspection confirmed that no damage was caused in all buildings from the minor movements induced by excavation works.

10. CONCLUSIONS

Instrumentation played a major role in execution and successful completion of excavation works adjacent to the existing sensitive buildings under unbalanced loading condition. Effective bracing system, excavation sequence and adequate embedment of the retaining walls were the main factors contributed in minimizing negative impact of the unbalanced loading condition.

A simple approach in prediction of ground settlement and assessment of the risk of building damage has been presented. The measured maximum tilt of the buildings reasonably agreed with the predicted values. It is necessary to use the rigid diaphragm wall in excavation work to minimize the ground movement particularly where the work is to be carried out in close proximity to the existing sensitive buildings.

REFERENCES

Performance of Buttress-Support Thin Diaphragm Wall for Underground Car Park in Bangkok

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Performance of Buttress-Support Thin Diaphragm Wall for Underground Car Park in Bangkok

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Abstract: This paper presents the geotechnical aspect of the construction of two-level underground car park building located in the culturally and historically significant area of Bangkok. Performance of buttressed-support diaphragm wall of 0.60m width is reported based on the inclinometer monitoring results. Intensive modification of construction sequence in actual work execution with "value engineering options" different from tender stage design is demonstrated along with application of observational method.

1 INTRODUCTION

Diaphragm walls have been utilized for underground permanent retaining structures in Bangkok for many years. The common width of the diaphragm wall in Bangkok ranges from 0.80m to 1.20m. Thin diaphragm wall of width less than 0.80m is rarely used in this growing-metropolis. Performance of buttress-support diaphragm wall of 0.60m width for two level underground car park building construction is discussed in this paper.

2 PROJECT OVERVIEW

The project is a two-level underground car park located in the center of Rattanakosin Island, the heart of an old established, historically and culturally significant area of Bangkok. As shown in Figure 1, construction site is surrounded by numbers of sensitive structures - in the south, Wat Suthat, one of Thailand’s most important temples and the Historical Giant-Swing, in the north, the City Hall and at South-east corner, the Historical Brahmin Temple. Rows of old shop-house buildings are closely located in the east and west boundaries of the project.

The project owner, Bangkok Metropolitan Authority (BMA) awarded the semi-turnkey basis construction contract to SEAFCO Co., Ltd. as a contractor. The contract consists of 3 major scope of works: (1) Construction of diaphragm wall, barrette and bored piles (2) Excavation works including temporary bracing design and installation (3) Construction of the entire basement structure having car park area of 18,552 m² and roof-level park of 10,936 m² plus cut-and-cover tunnel, underpass access to the City Hall. Since the project area was being used as a grade-level car parking space prior to the award of the contract, the key requirement was to construct the underground car park in two phases – to construct Phase 1 while leaving space for car parking in Phase 2, and to utilize semi-finished underground car park of Phase 1 during construction of Phase 2.

Diaphragm wall having 600mm width founded at 16m below ground level (B.G.L) was constructed simultaneously with dry-processed bored piles of diameter 600mm with toe depth 20m B.G.L. Barrettes having same toe depth as bored piles were installed at 8m spacing along with diaphragm wall panels. Sheet pile wall (14m deep) was used as a temporary retaining wall at the boundary of Phase 1 and 2 as shown in figure 2.

3 MAJOR CONSTRAINTS AND DESIGN PRINCIPLE

Location of the project site itself in the vicinity of sensitive buildings posed some constraints, which called for the need of careful consideration in establishing the design principles and sequence of construction. The architectural and utility aspects of the project called for the design of the basement with a number of openings from the ground surface to the final basement slab level to facilitate the ventilation system as shown in figure 1. Hence the roof slab can not be physically utilized as bracing in most of the area where the diaphragm wall is to be acting as a cantilever retaining wall in the permanent stage. It was analyzed in the preliminary analyses that the deflection of the diaphragm wall of 0.60 m width would be large if it was to be fully cantilevered. As the project is located in a sensitive area, ground movement induced by large deflection was unfavorable. It was therefore decided to use buttress to minimize the diaphragm wall deflection as shown in figure 2.

Fig. 1. Layout plan of the project showing adjacent buildings
SUBSOIL CONDITION

Similar to other localities in Bangkok a typical subsoil profile at the site is characterized by thick Bangkok soft clay layer at the top followed by thin layer of medium clay, and stiff clay layers as depicted in figure 2 (c). The first sand layer is found below 26m from the existing ground level.

TENDER STAGE DESIGN

The designers involved in the tender stage design made a fairly conservative design with two levels temporary bracing as shown in figure 3. Uncertainty of the performance of a thin diaphragm wall in soft clay layer was the likely reason to adopt the conservative design in the tender stage. It is not unreasonable to adopt the conservative design considering the rheological property of soft marine clay and likely long elapsed time of un-strutted diaphragm wall (relatively long un-strutted span of 5.6m between first strut and final excavation level for 600mm diaphragm wall) due to large volume of excavation work involved.

POST TENDER DESIGN REVIEW AND MODIFICATION FOR PHASE 1

Prior to the commencement of excavation works, value engineering review of the temporary works was undertaken by the contractor’s new team of design engineers together with the construction team. The main objectives were to minimize the material and construction sequence involved in temporary works so as to accelerate the excavation time thereby saving overall costs without compromising the safety aspect. After conducting a series of re-analyses with different conditions major modifications were made: (1) To change first strut level to –1.8m from the original tender stage design level –1.0m (2) To use only 1 temporary strut, omitting second level raking strut with the provision of sloping soil berm against diaphragm walls. Soil berm was to remove after completion of base slab construction in the majority of area – minimizing the elapsed time of partially un-strutted diaphragm wall between temporary strut and the final excavation level.

APPLICATION OF OBSERVATIONAL METHOD

In order to assess the most probable condition assumed in “value engineering design” and to take necessary actions if monitoring results reveal most unfavorable conditions (i.e. actual deflection of diaphragm wall reaches maximum acceptable limit), observational method was implemented on the followings basis.
- Reviewed the design parameters together with critical conditions posed on sites as well as most critical stage of excavation
- Designed and predicted the performance of diaphragm wall using the “most probable” conditions and parameters.
- Established the trigger criteria based on predicted diaphragm wall deflection
- Predefined the practical contingency plan for “most unfavorable” conditions where wall deflection reaches trigger levels
- Set out the instrumentation program with the consideration of above factors.
- Monitored the performance of diaphragm wall. Compared the monitoring results with the predicted and trigger values and re-assessed
- Implemented the contingency measures if monitoring result reaches action level of trigger values

Comprehensive and robust monitoring program was set up as a key element in application of the observational method and to ensure that modified construction sequence would not have adverse effect in temporary stage and on permanent design. A total of 6 inclinometers (3 in each phase) were installed in diaphragm wall together with some survey points.

FIG. 3 TENDER STAGE BRACING SYSTEM – DIAPHRAGM WALL WAS DESIGNED TO BE SUPPORTED BY 2 STRUTS (HORIZONTAL BRACING AND RAKER)
8 PREDICTION AND PERFORMANCE OF PHASE 1 DIAPHRAGM WALL

Figure 5 (a) shows the maximum predicted diaphragm wall deflection at different stages of excavation monitored by inclinometer No.1 (I-1 at East wall of Phase 1) together with predicted maximum deflection profile of 3 different conditions - tender stage design (2 temporary struts), modified design with buttress and modified design without buttress. It can be observed from figure that measured lateral movement pattern of diaphragm wall agreed well with that of prediction for modified design with buttress - meaning buttress-support has some influence on wall deflection. Deflection profile of South diaphragm wall which braced against temporary sheet pile wall at Phase 1 and 2 boundaries is presented in figure 5 (b). As can be seen in figure 5 (b), South diaphragm wall deflection is significantly higher than that of east wall, which is likely to be caused by the fact that South diaphragm wall is braced with more flexible sheet pile wall. Description of stages shown in the legend of figure 5 is summarized in table 1.

9 MODIFICATION OF PHASE 2 BRACING SYSTEM

Monitoring results of the Phase 1 excavation work provided an ample opportunity to review the design assumption, fine tune the parameters used in the analysis of the diaphragm wall for the Phase 2 and made modification of construction sequence. The major modifications are: (1) Removal of soil-berm at East and West diaphragm wall in shorter duration than that of Phase 1, and (2) Using raking struts instead of horizontal strut for North diaphragm wall a shown in figure 7.

10 PERFORMANCE OF PHASE 2 DIAPHRAGM WALL

Figure 8 (a) depicts the measured deflection of east diaphragm wall. As can be observed in figure 8 (a) in comparison with Figure 5 (a), the maximum deflection of east diaphragm wall in Phase 2 is larger than that of Phase 1. The likely reasons of this observation are;

- Un-strutted elapsed time for first temporary bracing in Phase 2 was longer than that of Phase 1.
- In Phase 1, horizontal struts were installed in north-south direction which temporary kingpost columns and strut were integrated in crisscross pattern with east-west struts – providing complete-support more rigid bracing system. Whereas in Phase 2, horizontal struts were installed only in east-west direction without having crisscross pattern with north wall – having less rigid bracing system than that of Phase 1.

With assurance of diaphragm wall performance from monitoring results of Phase 1, original plan of using horizontal struts for North diaphragm wall was modified by using raking struts instead of horizontal struts. As can be seen in figure 8 (b), deflection of North diaphragm wall (with raking strut support) is significantly higher than that of East diaphragm wall (with horizontal strut support). The main reason of larger movement of north diaphragm wall is due to the fact that it was supported only by the berm for the long period (about 52 days) before completion of raking struts so that soil-rheology effect took place during the long elapsed un-strutted period. Time-dependent deflection pattern due to soil-rheology effect can be observed in North diaphragm wall as illustrated in figure 9. North diaphragm wall moved progressively toward excavation before completion of raking struts (at 52 days) as can be seen in figure.
11 IMPLEMENTATION OF CONTINGENCY PLAN

Since deflection of North diaphragm wall (Phase 2) approached trigger levels, monitoring frequency was increased and the following contingency measures were implemented on site.

- Poured 15cm thick 1m wide lean concrete on the top of the berm along North diaphragm wall to provide bearing-effect
- Installed additional king-post and diagonal struts attached to the raking struts to provide more rigid support against diaphragm wall
- Soil-berm was removed locally in bays followed by construction of wale beam, tie beam and buttress as shown in figure 10

Movement of diaphragm wall movement was observed to be decreased by the above actions. No significant ground settlement was observed in the vicinity of the North diaphragm wall.

12 COST AND TIME SAVING FROM VALUE ENGINEERING OPTIONS

Significant cost and time saving were achieved from the value engineering options coupled with observational method implemented for both Phase 1 and 2. The major savings were achieved by less operation and material utilized in the following elements of temporary works.

- Cancellation of 2nd level raking struts against diaphragm wall for both phases
- Modification of bracing system – using raking struts instead of horizontal struts for North diaphragm wall in Phase 2

13 CONCLUSION

This research study reveals that a permanent diaphragm wall coupled with effective design and construction method supplemented by observational method could offer a logistically and financially attractive solution in construction of underground car park without disturbing the environment in the prominent historical area of Bangkok.
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Stability of a Trial Trench Excavated under Polymer Slurry in Bangkok Soft Clay

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Stability of a Trial Trench Excavated under Polymer Slurry in Bangkok Soft Clay

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Abstract: This paper presents findings on trench stability resulting from the use of polymer slurry. A trial trench 0.60x4.50x18.00m was excavated for diaphragm wall construction in Bangkok soft clay, using polymer as an alternative to bentonite slurry. Under consideration was its superior environmental advantages for soft clay soils typical of the region, notwithstanding a limited capacity to suspend soil particles. The trench was excavated with a cable suspended mechanical grab, filled with polymer slurry and left open for 4 days. Field measurement included recording of slurry properties, settlement surveys, inclinometer monitoring, periodical checking of the trench profile with Koden drilling monitoring equipment and trench depth sounding. Polymer slurry properties in the trench were found to change sharply within 12 hours, with density, pH and viscosity gradually decreasing after 12 hours. Sedimentation occurred within a few hours after completion of trench excavation and a 7.00m thick deposit was indicated at the bottom of trench after 10.5hrs. The deposit was cleared with the excavation grab but sedimentation recurred. The trench profiles plotted by Koden drilling monitoring equipment showed trench surface failure starting at depths between 7.00m and 7.50m on one of the long sides of the trench face 18 hours after trench excavation, with inclinometer readings indicating maximum soil yielding of about 6mm after 10 hours. A major collapse of the trench surface was observed approximately 36 hours after trench completion, with ground settlement of up to 9mm. Based on field measurement results and characteristics of the polymer used, the suitability of using polymer slurry is discussed.

1. INTRODUCTION

Polymeric fluids have been in use as support slurries to provide stability during excavation for bored piles and diaphragm walls. The merits of such polymeric fluids have been evaluated and reported by Beresford, J.J and et al in 1989. Polymeric slurries are environmentally compactable and acceptable. The used polymer fluid can be disposed directly into the municipal sewer without harmful consequences to water courses. Excavated materials can also be safely dumped in unregulated landfill sites. As the polymer slurry offers considerable advantages in terms of extending operation capabilities, logistics, environmental compatibility and overall cost-saving, as well as enhancement to frictional capacity of bored piles by providing intimate contact between pile shaft and surrounding soil, particularly in cohesionless soil without forming a filter cake, it has been used in bored piling work in Bangkok and elsewhere in Thailand (Ref). For diaphragm wall work, bentonite slurry is still used as the main supporting slurry in Thailand. Polymeric slurry usually purges itself of soil particles and particle settling takes place whilst in the trench. Thus a special process to remove the particles deposited at the bottom of trench is mandatory as a conventional mechanical grab will not effectively remove the sediments in such cases. For some basement excavation projects in Bangkok subsoils with a maximum excavation depth of up to 10.0m, diaphragm wall systems constructed predominantly in clay layers are required. Hence the use of polymer was considered a practical option for diaphragm wall excavation in clay. A trial trench (0.6x4.5x18m) was then excavated under polymer slurry in soft clay at Bangchan, north-eastern Bangkok, to evaluate the performance of the slurry.

Performance of trial trenches under bentonite slurry in soft clay have been reported previously by Dibiagio and Myrvoll (1972) and Thasnanipan et al (1998).

2. SITE CONDITIONS

A 25m deep borehole was drilled to investigate subsoil conditions at the trial trench location prior to trenching. The investigation indicated that a soft clay layer extended up to 13.5m below the existing ground level, overlaying a 3.0m thick stiff clay layer with SPT values ranging from 10 to 20. Sensitivity of soft clay ranged from 2.3 to 3.5. Below the stiff clay layer was very stiff clay 4.5m thick with SPT values ranging from 17 to 27, underlain by a fine to medium sand layer at a depth of 22.0m. SPT values obtained in the sand layer ranged from 27 to 54.

Fig. 1. Subsoil conditions at the trial trench site (Bangchan).

Natural water content in the soft clay generally ranges from 60-80%. However, at depths of 5m and 8m the natural water content is slightly above or about 100% respectively. Groundwater level observed in the borehole was at about 2.5m below the ground.
3. TRIAL TRENCH

The trial trench was located about 1.0m away from the borehole. A concrete guide wall of 1.5m deep was constructed with continuous reinforcement and cast against firm ground. The trial trench was 0.6m wide, 4.5m long and 18.0m deep. For field monitoring, all monitoring points, comprising 6 settlement survey points using one inclinometer tube were located on the opposite site of the excavation equipment to avoid disruption caused by the excavation activities during trenching operation. Locations of settlement survey points and inclinometer tube are shown in Figure 2.

Fig. 2. Layout of field monitoring locations.

Trenching was carried out on 16th June 1999 at about 14:00hrs under commercially available polymer slurry and completed at about 23:30hrs on the same day. A cable suspended mechanical grab (0.6mx2.0m) was used for trench excavation. 3 bites excavation process was used to achieve the required trench dimensions.

The polymer type used for trial trenching was a water-soluble synthetic polymer supplied as a concentrated liquid. As per by supplier recommendations, a mix ratio of 0.5kg of polymer per 1 cu.m. of slurry was used for slurry preparation. The properties of slurry were tested for compliance values specified for excavation prior to introducing it to the trenching process.

The slurry level in the trench was maintained up to ground level at all times. The rate of raising the grab during excavation was controlled to minimize the suction action which would impact on the trench stability. The trench profile was checked with Koden drilling monitoring equipment when excavation reached a depth of 9.0m. The polymer slurry was also tested for its quality. Trenching was then continued to the final depth of 18.0m. The verticality of the trench indicated by the drilling monitoring equipment was within 1:180.

4. FIELD TESTING PROGRAM

Field monitoring and slurry testing were generally carried out when trenching reached a depth of 9.0m and the final depth of 18.0m, and then every 6hrs after trenching was completed. The following field testing was performed.

- Checking trench profile with Koden drilling monitoring equipment – the equipment was stationed in a fixed position on the guide wall to monitor the changes in trench profiles. The recording accuracy of the equipment was in a range of +/- 0.2% (in this case the accuracy was within 1-2mm).
- Slurry testing – this was carried out on the viscosity, density and pH of the slurry sample taken from the trench at a depth of 9.0m.
- Ground settlement monitoring at 6 settlement points - Monitoring was carried out approximately every 2 hrs during excavation and every 6hrs after excavation.
- Checking trench depth – Depth sounding at 3 locations, the center and near both ends of the trench was carried out every 30minutes after excavation.
- Monitoring lateral soil movement with inclinometer – An inclinometer access tube was installed in the soil investigation borehole to a depth of 24.0m. Monitoring was carried out every about 2 hrs during excavation and at 2-6hrs of elapsed time after excavation.

5. TESTING AND MONITORING RESULTS

The trial trench excavation was carried out on 16th of June 1999 at 14:00hrs and completed at 23:30hrs. Sedimentation occurred within a few hours after completion of the trench and sediment built up to 11.0m in depth (i.e. deposit thickness was 7.0m) after 10.5hrs. The estimated rates of settlement of soil particles ranged from 1.0m to 1.4m per hour in thickness for the first 4hrs after trenching and then gradually decreased. Cleaning of the trench base was carried out 14hrs after trench completion. However, sedimentation recurred and built up to a depth of 13.0m after 12hrs. 24hrs after base cleaning (36hrs after excavation), sedimentation built up to a depth of 6.9m due to trench face failure.

The density of polymer slurry in the trench was found to change sharply within 12 hrs from 1.30g/ml to 1.08g/ml and then gradually decrease. 36hrs after trench completion, this reached 1.0g/ml. Similarly, viscosity of the slurry changed from 43sec to 32sec within 12 hrs and decreased to 29sec after 36hrs. A Marsh Funnel viscometer was used for indicative measurement of relative changes in viscosity only, rather than to evaluate the slurry performance. The pH value also changed from 8 to 7 after 12 hrs. After 36 hrs, the properties of the slurry remained constant till the trench was backfilled. It is considered that after 36hrs the properties of slurry indicated that the slurry in the
trench turned to water, purging all polymer constituents with soil particles.

Fig. 4. Plots of polymer slurry property changes with time.

Plots from the inclinometer readings indicated lateral movements of soil up to 5.8mm at a depth of 3.5m. However, actual soil movements at the face of the trench may have been greater than those measured by the inclinometer located 1.0m away. A maximum ground settlement of 9.0mm was observed 2.0m away from the trench after major failure of the trench face opposite site settlement points. No ground settlement was observed beyond the distance of 8.0m.

Fig. 5. Lateral soil displacements plotted from inclinometer readings.

The trench profile plotted by the monitoring equipment shows that on the opposite side of instrumented monitoring points (inclinometer and settlement survey points), trench surface failure started at depths of 7.0-7.5m about 18hrs after trench completion. The surface failure progressively increased and a major collapse was observed at depths between 2.0m and 8.0m after 38hrs. The trench surface failure was found to be a wedge failure, extending more than 1.0m into the soil face. Likewise, after 42hrs had elapsed, a localized collapse was also observed on the instrumented monitoring side of the trench at 6.0m below ground level, increasing progressively to a depth of about 5.0m below ground level till the monitoring was terminated at 96hrs after excavation.

Fig. 6. Plots of ground settlement readings.

Fig. 7. Profiles of trial trench plotted by Koden drilling monitoring equipment at different times.

6. DISCUSSION

Excavation of the trial trench was mainly in clay layer, which is non-permeable. Moreover, clay normally maintains a relatively stable ground condition apart from elastic deformation due to reduction of confining pressure. The movement can be controlled by hydrostatic pressure conceptually in diaphragm wall excavation. Compared to bentonite, polymer contributes relatively little additional density to the water in which it is prepared, and is just as effective in holding back the plastic deformation of the ground.

Empirical calculations were carried out for trench stability analysis based on the available soil test data with planned trench dimensions prior to excavation. The analysis indicated that polymer fluid pressure with a density of 1.0 t/m3 (reduced to density of water) was still adequate to maintain the stability of the trench. However, high natural water content approximating the liquid limit or exceeding moisture content of 100% in soft clay at depths between 3.5m and 9.0m was a major concern for trench stability. In the past, occurrence of localized failure or trench collapse in such soil conditions under bentonite slurry was experienced when pore pressure and/or density of slurry significantly changed after heavy rain.
Instability of trench was not encountered in the trial trenches which were excavated in soft clay and under-consolidated marine deposits under bentonite slurry as reported by Dibiagio & Myrvoll (1972) and Cowen et al (2001) respectively. The soil properties and movements of the trial trenches, including the trial trench reported herein are compared where possible and presented in Tables 1 and 2 respectively.

Table 1. Comparison of Soil Properties at Trial Trench Sites

<table>
<thead>
<tr>
<th>Depth m</th>
<th>Soil Properties</th>
<th>Studenterlunden Norway</th>
<th>Khlong Prapa, North Bangkok</th>
<th>Bangchan, North-East Bangkok</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-10</td>
<td>$\gamma$ t/m$^3$</td>
<td>1.60-1.86</td>
<td>1.60-1.80</td>
<td>1.37-1.80</td>
</tr>
<tr>
<td></td>
<td>$S_v$ t/m$^3$</td>
<td>4.0-8.0</td>
<td>1.3-2.5</td>
<td>1.6-2.0</td>
</tr>
<tr>
<td></td>
<td>WC %</td>
<td>40-45</td>
<td>40-70</td>
<td>40-105</td>
</tr>
<tr>
<td></td>
<td>LL %</td>
<td>45-50</td>
<td>60-70</td>
<td>50-85</td>
</tr>
<tr>
<td>10-20</td>
<td>$\gamma$ t/m$^3$</td>
<td>1.82-1.91</td>
<td>Below</td>
<td>1.6-2.0</td>
</tr>
<tr>
<td></td>
<td>$S_v$ t/m$^3$</td>
<td>2.0-5.0</td>
<td>14.0m is</td>
<td>2.0-18.0</td>
</tr>
<tr>
<td></td>
<td>WC %</td>
<td>40-42</td>
<td>medium</td>
<td>21-71</td>
</tr>
<tr>
<td></td>
<td>LL %</td>
<td>40-50</td>
<td>dense sand</td>
<td>43-84</td>
</tr>
<tr>
<td>20-30</td>
<td>$\gamma$ t/m$^3$</td>
<td>1.81-1.96</td>
<td>Below</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$S_v$ t/m$^3$</td>
<td>4.0-6.0</td>
<td>22.0m is</td>
<td></td>
</tr>
<tr>
<td></td>
<td>WC %</td>
<td>40-50</td>
<td>medium</td>
<td></td>
</tr>
<tr>
<td></td>
<td>LL %</td>
<td>40-55</td>
<td>dense sand</td>
<td></td>
</tr>
</tbody>
</table>

Table 2. Comparison of Ground movements

<table>
<thead>
<tr>
<th>Item</th>
<th>Studenterlunden Norway</th>
<th>Khlong Prapa, North Bangkok</th>
<th>Bangchan, North-East Bangkok</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trench Dimension in m</td>
<td>1.0x5.0x28.0</td>
<td>0.8x2.7x14.0</td>
<td>0.6x4.5x18.0</td>
</tr>
<tr>
<td>Max. Ground Settlement</td>
<td>8mm</td>
<td>-</td>
<td>9mm</td>
</tr>
<tr>
<td>Max. Soil yield in mm</td>
<td>(after 31 days)</td>
<td>(after 24 hrs)</td>
<td>(after 10hrs)</td>
</tr>
<tr>
<td>Supporting Fluid</td>
<td>Bentonite,</td>
<td>Bentonite</td>
<td>Polymer</td>
</tr>
<tr>
<td>Water after 32days</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Most of the monitoring points for Bangchan trial trench were located on one side of the long trial trench face. The trench profile as shown by drilling monitoring equipment indicated that the collapse of the trench face occurred initially at the side opposite the monitoring points. It is considered that the recorded maximum values of results may not have represented the actual maximum values of soil movements occurring around the trench, particularly on the side of the trench without monitoring instrumentation due to lack of monitoring data.

7. CONCLUSIONS

Based on the field monitoring results obtained from the trial trench excavated under polymer in Bangkok soft clay, the following conclusions were made:

- The trial trench was found stable up to 18hrs after completion of excavation under polymer fluid at this site and after that the stability of trench deteriorated with time.

It is considered that high natural water content of the soil, which approximated the liquid limit or exceeded 100% posed a risk to trench stability and contributed to the failure of the trial trench.

An efficient cleaning mechanism is considered necessary to remove the soil particles settling at the bottom of the trench under polymer slurry.

ACKNOWLEDGEMENT

The authors wish to acknowledge the contribution of EDE Company limited for field monitoring and assistance in trial trenching at Khlong Prapa and Bangchan sites.

REFERENCES


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Application of Top-down Construction Method for Deep Excavations in Bangkok

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SEAFCO Public Co., Ltd. Bangkok, Thailand
APPLICATION OF TOP-DOWN CONSTRUCTION METHOD FOR DEEP EXCAVATIONS IN BANGKOK

Narong Thasnanipan, Zaw Zaw Aye, Chanchai Submaneewong, Thayanan Boonyarak
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1. INTRODUCTION
Top-down construction method as the name implies, is a construction method, which builds the permanent structure members of the basement along with the excavation from the top to the bottom. Top-down method is mainly used for two types of urban structures, underground structures such as car parks, underpasses and subway stations and tall buildings with deep basements. Top-down method is the most appropriate solution in minimizing the duration and disruption to surface traffic and other urban activities, for the construction of the subway stations, underground car parks and underpasses where the structure is located directly underneath the existing roads. For tall buildings with deep basements, a full application of top-down method enables superstructure to build concurrently with excavation and construction of the basement giving a significant advantage in reducing the overall construction time.

2. TYPICAL SUBSOIL PROFILE OF BANGKOK
Bangkok subsoil is featured by alternating clay and sand layers of thick Quaternary deposits. As characteristics and properties of Bangkok subsoil layers have been well documented in many literatures, only a general description of Bangkok subsoil is presented in this article as follows.

Top layer consists predominantly of Fill-Materials, Clayey Sand or Silty Clay with some cement block rubble and rock fragments, is commonly found up to 2 to 4m depth. Soft to very soft, highly compressible dark gray marine clay lies beneath top layer and in some areas it lies under weathered crust layers of about 2m thick. Depending on the location, Soft Clay layer is extended up to 12-18m. About 2m thick Medium Clay layer can be observed between Soft Clay and underlying Stiff Clay. Generally Stiff Clay layer occurs directly underneath Medium Clay and its depth goes up to 22m. Below Stiff Clay layer, First Sand layer 5-8m in thickness can be found. This First Sand layer, however, is absent in some areas. Stiff to Hard Clay layer underlies First Sand and it is found to be about 5m thick. Second...
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Sand layer generally occurs at depths between 45 to 65m. Typical subsoil profile of Bangkok area is shown in Figure 1.

3. BORED PILE AND BARRETTES CONSTRUCTION METHOD

Wet process or slurry displacement method is always used for construction of deep-seated large bored piles and barrettes in Bangkok. For bored piles, temporary casing of 14-15m in length is used as a support in soft clay layer. Soil inside the casing is normally excavated by auger applying rotary drilling method and drilling is continued with the bucket under polymer-based slurry from the top of sand layer to the final depth. A mechanical cable-hung-grab or hydraulic grab mounted on crawler crane is used for excavation of trench with slurry support in barrette construction. Guide walls having a minimum depth of 1.2m are commonly used for initial guiding of grab excavation at the top of the trench. Different from bored pile, bentonite slurry is commonly used for barrette excavation in Bangkok. Tremie concreting is necessary for casting both bored piles and barrettes.

Stanchions are mainly installed in the large diameter deep-seated bored piles and barrettes founded in first or second sand layer. Bored piles of diameter 1.50m to 1.8m and rectangular-shape barrettes of 0.80x2.8m to 1.5x3.0m (thickness x width) founding at depth between 50m and 60m are commonly used to accommodate the stanchions.
4. ADVANTAGES OF TOP-DOWN CONSTRUCTION METHOD

Application of top-down method offers three main advantages as outlined below.
- It allows early commencement of super-structure construction without the need to wait for excavation reaching the bottom level. Overall construction duration can be significantly less than that of conventional method thereby saving the cost.
- It does not require temporary bracing system so that time requirement and cost for temporary works (both material and labor costs) are eliminated, in turn significantly saving cost.
- Minimize soil movement induced by excavation works which is an important factor for some sensitive locations.

5. STRUCTURAL MEMBERS REQUIRED FOR TOP-DOWN CONSTRUCTION

Design and construction principles for top-down method primarily call for two major structural elements.
- Columns with sufficient capacity must be pre-founded in bored piles or barrettes to sustain the construction load and to utilize as part of bracing system.
- Excavation for basement must be carried out with the support of permanent retaining wall so that basement floor slabs can be utilized as lateral bracing. Diaphragm wall of 0.8m to 1.2m in thickness with sufficient embedment in firm soil layers is commonly used as a retaining wall whereas prefabricated steel columns known as stanchions embedded in either large diameter deep-seated bored piles or barrettes are utilized as structural columns. Figure 2 illustrates the top-down construction method with utilization of stanchions and diaphragm wall for underground subway station.
6. TYPES OF STANCHION AND THEIR APPLICATION IN BANGKOK

Pre-fabricated “H” section steel columns and steel built-up-section columns are mainly used as pre-founded structural columns or stanchions. Pre-cast reinforce concrete columns are very seldom used (Manoharn S. & Aye Z. Z., 1994). Commonly used stanchion types and their application in Thailand can be viewed in published paper of Thasnanipan et. al (2000).

7. ALLOWABLE TOLERANCE FOR POSITION OF STANCHIONS

Allowable tolerance for stanchions is usually called by the designer and it is mainly governed by the structural tolerance of the steel as well as required position of the finished columns. In most projects, allowable vertical and horizontal deviation of stanchion are specified as 25 to 50mm whereas verticality is required between 1:200 and 1:400. There are a number of constraints to get highly accurate position of heavy and lengthy stanchions which have to be installed in deep-seated foundation piles constructed by wet process in Bangkok subsoil as described in the following sections.

8. STANCHION INSTALLATION METHODS

Stanchion installation method is usually selected by the piling contractor who take into consideration three main factors such as installation depth, size of stanchion and size of bored or barrette piles. Though installation details may be different from one contractor to another, stanchion installation can be categorized under two main methods, pre-concreting installation or placing stanchion prior to concreting or post-concreting or plunging installation.

8.1 Pre-concreting or pre-placing installation method

This method was applied in Thailand Cultural Center Station of the M.R.T Chaloem Ratchamongkhon Line. Stanchion was installed prior to tremie concreting, immediately after completion of barrette excavation and reinforcement lowering. The main reason for selection of this method in the mentioned project was due to the relatively large size of the barrettes. In some projects stanchion is attached to the last section of reinforcement and installed together with the reinforcement. General construction steps involved in this method are demonstrated in Figure 3.

Figure 3. General construction sequence of pre-concreting installation method
Specially designed guide frame used in installation of H-section steel column of 419x407x390kg/m, 24.7m long, inserted into barrettes applying pre-concreting method in Bangkok MRT project is shown in Figure 4. Effectiveness of guide frame plays one of the important roles in achieving positional accuracy of stanchion.

Figure 4 Guide frame used for pre-concreting installation in Thailand Cultural Center Station of the M.R.T Chaloem Ratchamongkhon Line

Figure 5 shows the installation of a 24m long built-up steel stanchion into the pile borehole prior to concreting.

Figure 5 Stanchion length 24m attached to the top portion of reinforcement cage installed into borehole prior to concreting
7. 2 Post-concreting installation or plunging method

In this method, stanchion is installed immediately after completion of bored pile concreting process. General construction sequence involved in this method is demonstrated in Figure 6. Guide frame is used to install the stanchion at the correct position.

![Figure 6. General construction sequence of pre-concreting installation method](image)

REVIEW OF THE STANCHION INSTALLATION METHODS USED IN OTHER PARTS OF THE WORLD

The stanchion installation methods used in other parts of the world have been presented in some published papers. Among these, Findlay (1989) reviewed a number of stanchion installation methods used in construction of large diameter bored piles for top-down construction particularly in UK. The author reported that steel columns can be placed with better accuracy by dry process bored piling method than wet process method (with support fluid).

Table 1. Summary of stanchion installation method from published literatures

<table>
<thead>
<tr>
<th>Reported by</th>
<th>Location</th>
<th>Stanchion installation method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ressi di Cerva and Tamaro (1991)</td>
<td>Boston, USA</td>
<td>Pre-concreting</td>
</tr>
<tr>
<td>Crawley and Stones (1996)</td>
<td>London, UK</td>
<td>Pre-concreting</td>
</tr>
<tr>
<td>Arz (1989)</td>
<td>Germany</td>
<td>Pre-concreting</td>
</tr>
</tbody>
</table>

From the available literatures it is noted that pre-concreting method is more common in use than post-concreting method.

PROBLEMS ENCOUNTERED IN POST-CONCRETING INSTALLATION METHOD

The major problem commonly encountered in this installation method is inability to install the stanchion at the design position due to one or combination of the following causes.

- Inclination of the borehole / trench
• Stanchion can not be inserted up to the required depth as concrete becomes hard due to the premature setting
• During lowering, stanchion is stuck by reinforcement cage and due to the hardening of the concrete, extraction of stanchion becomes impossible for reinstallation

As the installation of the stanchion is often associated with many unforeseen problems, it is likely in many cases that concrete becomes stiff or prematurely set during the installation. In Bangkok, premature setting of concrete is usually found to be attributed by;
• long delivery time due to traffic conjunction
• inappropriate mix
• severe weather
• disruption in concreting / equipment breakdown

PROBLEMS ENCOUNTERED IN PRE-CONCRETING INSTALLATION METHOD

Using this method can minimize some problems encountered during installation provided that adequate clearance is available between pile reinforcement and the stanchion for tremie pipes lowering. Inclination of borehole is also important for this installation method. Proper planning in concrete ordering is required to avoid unnecessary waiting of concrete trucks for stanchion placing which may affect the total batching time limitation. As stanchion has to be hung in the position until completion of concrete placement, appropriate and sufficient capacity of the hanging and positioning device should be provided to avoid falling and deviation of stanchion in the borehole. Temporary casing should not be used as a hanging system without additional support for the large stanchion.

FACTORS EFFECTING THE POSITIONAL ACCURACY OF THE STANCHION

It is hardly possible to install a stanchion so that its position and verticality is always as designed. As-built position of stanchion is found to be generally influenced by the installation method. Skill and experience of the contractor plays a major role in achieving positional accuracy of the stanchion provided that the installation is practical with the design elements such as size of foundation piles in relation to size of stanchion. The factors affecting the positional accuracy for two different methods were reported in details by Thasnanipan et. al (2000).
STANCHION POSITION EFFECTED BY EXCAVATION

In Bangkok, post-installation movement (horizontal movement) of the stanchion is frequently encountered and it is mainly caused by excavation induced soil displacement particularly at initial stage of excavation where diaphragm wall deflection is characterized by large cantilever rotation within Soft Clay layer. In some projects, deviated stanchions had to be pushed or jacked back to the original position and restrained by means of temporary support until permanent slab was cast.

APPLICATION OF TOP-DOWN METHOD IN CONGESTIVE URBAN AREA

Top-down method was successfully applied in Bangkok for a fifteen-storey commercial building with five-level underground car park located in an old established commercial area of Bangkok. Construction site is bounded by rows of old shop-house buildings estimated to be supported by short piles. Hence, ground movement induced by large deflection of retaining wall was unfavorable. Limited space availability on site and congestive neighborhood also posed another major constraint on construction logistic. By the local traffic regulation, concrete trucks and other heavy vehicles entering to the site were restricted during rush-hours of both morning and afternoon. These factors called for the need of careful consideration in establishing the design principles and sequence of construction. A numbers of construction sequences were considered. After having detailed review and conducting a series of analyses with different conditions, decision was made to apply the top-down construction method.

The main objectives were:
- to minimize the material and construction sequence involved in temporary works so as to accelerate the construction time
- to minimize the logistic problem imposed on site due to limited space in congestive neighborhood
- to minimize soil movement induced by excavation works which is important factor for the project adjacent to the old buildings of weak foundation

DIAPHRAGM WALL, BARRETTES AND BORED PILE CONSTRUCTION

Diaphragm wall having 800mm width founded at 24m below ground level (B.G.L) was constructed simultaneously with wet-processed bored piles of diameter ranges from 800mm to 1800mm with toe depth 60m B.G.L. Barrettes having same toe depth as bored piles were constructed along with diaphragm wall panels. Depending on the loading conditions, different sizes of permanent steel stanchions were installed by post-concreting method. Table 2 shows the detailed of foundation elements constructed in the project. Figure 8 shows layout of diaphragm wall and bored pile with stanchion. Figure 9 shows the aerial view of excavation works.
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<table>
<thead>
<tr>
<th>Type and Size of Foundation</th>
<th>Allowable Pile Capacity (ton)</th>
<th>Permanent Steel Stanchion Size</th>
<th>Total Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barrette 0.80 x 3.00m</td>
<td>1750</td>
<td>2 No. H-350x350mm</td>
<td>6</td>
</tr>
<tr>
<td>Bored pile dia. 1.80m</td>
<td>1600</td>
<td>H-400x400mm + 2 No. H-200x200mm + Steel Plate 80mm</td>
<td>3</td>
</tr>
<tr>
<td>Bored pile dia. 1.80m</td>
<td>1600</td>
<td>H-400x400mm + 2 No. H-200x200mm</td>
<td>15</td>
</tr>
<tr>
<td>Bored pile dia. 1.80m</td>
<td>1600</td>
<td>H-400x400mm</td>
<td>17</td>
</tr>
<tr>
<td>Bored pile dia. 1.65m</td>
<td>1450</td>
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<td>1250</td>
<td>H-400x400mm</td>
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<td>H-400x400mm</td>
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<td>No stanchion</td>
<td>9</td>
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<tr>
<td>Bored pile dia. 0.80m</td>
<td>500</td>
<td>No stanchion</td>
<td>16</td>
</tr>
</tbody>
</table>

Figure 8. Layout of bored pile with stanchion and diaphragm wall

Figure 9. Aerial view of the top-down construction in progress – concrete batching plant was set up on site to minimize the concrete supply interruption in congestive area with severe

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COMPARISON BETWEEN TOP-DOWN AND BOTTOM-UP METHOD

Master Construction Schedule of the major activities of top-down and bottom-up method is illustrated in a simple bar-chart form as shown in the figure below. Apparently, top-down construction method offered significant time saving as can be seen in the figure. This schedule comparison is based on the construction sequences by top-down and bottom-up method as demonstrated in Figure 10. Figure 11 shows construction progress after excavation reached to final level.

<table>
<thead>
<tr>
<th>Method</th>
<th>Major Activity</th>
<th>Duration (month)</th>
<th>Construction Period (Month)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>4</td>
</tr>
<tr>
<td><strong>Top-down</strong></td>
<td>D-wall and piling</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sub-structure</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Super-structure</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td><strong>Bottom-up</strong></td>
<td>D-wall and piling</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sub-structure</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Super-structure</td>
<td>13</td>
<td></td>
</tr>
</tbody>
</table>

Figure 10 Master construction schedule of major construction activities of top-down and bottom-up method

Apart from significant time saving, the cost associated with temporary works was significantly minimized by using top-down method. If bottom-up method was utilized, it would have required, 3 layers temporary bracing as shown in Figure 12. A total weight of steel H-beam type temporary bracings required for 3 layers was estimated to be 1,070 ton. Significant amount of scaffolding usage was also minimized by applying top-down method.

Figure 11 View of construction progress after excavation reached to final level
COMPARISON BETWEEN TOP-DOWN AND BOTTOM-UP METHOD

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### Method Major Activity Duration (month)

- **Top-down**
  - D-wall and piling: 4.5
  - Sub-structure: 10
  - Super-structure: 12

- **Bottom-up**
  - D-wall and piling: 4.5
  - Sub-structure: 16
  - Super-structure: 13

Apart from significant time saving, the cost associated with temporary works was significantly minimized by using top-down method. If bottom-up method was utilized, it would have required 3 layers temporary bracing as shown in Figure 12. A total weight of steel H-beam type temporary bracings required for 3 layers was estimated to be 1,070 ton. Significant amount of scaffolding usage was also minimized by applying top-down method.

---

**Figure 10** Master construction schedule of major construction activities of top-down and bottom-up method

**Figure 11** View of construction progress after excavation reached to final level

**Figure 12** Summarized construction sequence of top-down and bottom-up method

<table>
<thead>
<tr>
<th>Stage</th>
<th>Top-Down Method</th>
<th>Bottom-up Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Excavate to -1.7m and install temporary bracing at -1.5m</td>
<td>Excavate to -1.7m and install first level temporary bracing at -1.5m</td>
</tr>
<tr>
<td>2</td>
<td>Excavate to -4.5m and construct B1 Slab</td>
<td>Excavate to -7.2 m and install 2nd level temporary bracing at -7.0m</td>
</tr>
<tr>
<td>3</td>
<td>Remove 1st bracing at -1.5m</td>
<td>Excavate to -13.0m and install 3rd level temporary bracing at -12.8.0m</td>
</tr>
<tr>
<td>4</td>
<td>Excavate to -11.8m and construct B3 Slab</td>
<td>Excavate to -19.1 m and construct mat foundation</td>
</tr>
<tr>
<td>5</td>
<td>Excavate to -19.1m and construct Mat Foundation</td>
<td>Construct B4 Slab and columns</td>
</tr>
<tr>
<td>6</td>
<td>Construct B4, B2 and Ground Floor Slab</td>
<td>Remove 3rd level braking</td>
</tr>
<tr>
<td>7</td>
<td>Construct above floors</td>
<td>Construct B3 Slab and columns</td>
</tr>
</tbody>
</table>

---

Figure 12 Summarized construction sequence of top-down and bottom-up method
COMPARISON ON DIAPHRAGM WALL BEHAVIOR

Behavior of diaphragm wall with two different construction methods was studied. Figure 13 depicts the results of bending moment, shear force and displacement of diaphragm wall compared for top-down and bottom-up construction methods. Followings were concluded from the comparison.

- Reinforcement required for diaphragm wall with top-down method would be less than that of bottom-up method which in turn offered cost saving on material and labour
- Diaphragm wall deflection for top-down method was significantly less than that of bottom-up method which would minimize risk of damage to adjacent existing buildings

Predicted and measured diaphragm wall deflection is shown in Figure 14. As can be seen in the figure, actual measured deflection values are less than those of prediction. The shape of the deflection curves of prediction and actual measurement at each construction stage is fairly identical which suggests that design parameters and assumption used in analysis are of applicable.

![Figure 13 Bending moment, shear force and displacement of diaphragm wall comparison for top-down and bottom-up construction methods](image)

**Figure 13** Bending moment, shear force and displacement of diaphragm wall comparison for top-down and bottom-up construction methods
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CONCLUSION

Application of top-down construction method in Bangkok subsoil has been presented with particular emphasis on the two different stanchion installation methods. According to the author’s experience, pre-concreting installation method provided fewer problems in practical installation and achieved better positional accuracy. The designers such as structural engineers and architects should be aware of the accuracy achievable by practical installation method and take it into consideration at the design stage. Stanchion installation contractor should select the appropriate method, equipment as well as experienced personnel plus a well-formed plan with the consideration of all potential problems to achieve the successful construction of foundation structure, which is of primary importance for top-down technique.

Practical application of top-down method in recently completed project is presented highlighting the key advantages over bottom-up method. Significant cost saving from shorter construction period and minimum usage of material for temporary works offered by top-down construction method was remarkable achievement. In short, top-down method is the most appropriate solution in construction of the deep underground structures for protecting the modern urban environment.
REFERENCES


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Construction of Diaphragm Wall for Basement Excavation Adjacent to Tunnels in Bangkok Subsoil

Narong Thansnanipan¹, Aung Win Maung¹, Zaw Zaw Aye¹, Chanchai Submaneewong¹
Thayanan Boonyarak¹

¹ Seafco Public Company Limited, Thailand

ABSTRACT

In July 2005, an earth retaining structure with diaphragm wall was completed in the vicinity of Bangkok’s first subway system, the MRTA Chaloei Ratchamonkhon Line. Diaphragm wall design analysis and ground movements associated with diaphragm wall construction were reviewed. The review was carried out in accordance with technical requirements for engineering works within the specified protection zone of MRTA structures. Preliminary diaphragm wall analyses indicated that lateral movements of the wall should not exceed 26.54mm. A 6.0mm maximum movement allowance is specified for MRTA structures. The construction work was undertaken in compliance with technical requirements for engineering works within the specified protection zone of MRTA structures. Two inclinometer tubes with 20.0m in length were installed in the ground between the planned diaphragm wall and the south-bound tunnel. Surface settlements markers were also established in line with the inclinometer tube location on the pavement. Ground movements were monitored during diaphragm wall panel construction. Based on the monitoring results, the estimated maximum ground movement at the tunnel location due to wall panel construction was less than 0.5mm, indicating no impact on the tunnels by diaphragm wall construction.

1. INTRODUCTION

Deep excavation work adjacent to existing structures have become a common construction activity in most major cities as utilization of underground spaces, including underground car parks, basements, mass transit subway stations and tunnels increase. With mass transit systems in place, properties located along the commuter lines are considered the most favorable areas to be developed with maximum utilization of areas available. In turn, deep basement construction adjacent to the existing subway tunnels is also often necessary.

At present in Bangkok, such deep excavation work in the vicinity of the subway tunnels is considered relatively uncommon compared to major cities in developed countries. It is anticipated that this phenomenon will change in the near future. This paper presents an unprecedented case in Thailand of deep excavation adjacent to a subway tunnel. The proposed hotel tower reported herein was planned to be constructed with 2 basements using cast in-situ diaphragm wall located 7.8m to 9.6m away from the south-bound tunnel of the Mass Rapid Transit Authority, MRTA Chaloem Ratchamonkhon Line. The main foundation for the superstructure consists of 66 bored piles (1.5m diameter) and 7 barrettes (1.0x3.0m) embedded to depths of about 57.0m in the second sand layer of Bangkok Subsoil. For construction of basements and foundation mat of 3.5m in thickness, diaphragm walls (0.8mx20m deep) with 2 levels of temporary bracing are planned as a retaining system for a general excavation to -10.50m. The perimeter diaphragm wall is supported by 24 bored pile of 0.8m in diameter. Due to the presence of the MRTA tunnels in the close vicinity of the project, a review
was carried out on the expected performance of the retaining system, with regards to technical requirements for the engineering works within the protection zone of the MRTA.

In this paper, predicted diaphragm wall deflections and associated expected tunnel movements are presented. Pre-established geotechnical instrumentation program and recommended contingency measures are also described. Monitoring results of ground movement during different stages of diaphragm wall panel construction is also reported.

2. SITE CONDITIONS

The project site is located on the eastern side of Asoke Road, Sukhumvit, Bangkok, Thailand. On the eastern side are two one storey houses, 3 storey office building and open space car parking. The site is flanked by a 12 storey office building on the northern boundary and a 5 storey residential building on the southern boundary.

The MRTA twin tunnels with an outside diameter of 6.30m are located in front of the site about 15.0m below Asoke Road. Sound bound tunnel is adjacent to the site being 9.623m to 7.815m away from the planned diaphragm wall. The layout plan of the site is presented in Figure 1.

![Figure 1. Layout plan of the project](image)

3. SUBSOIL CONDITIONS

A site investigation was carried out with 3 boreholes drilled at the project site. The borehole information indicated that a soft clay layer about 10.0m in thickness occurred below 2.0m thick fill. A medium stiff clay layer with Su 2.9-3.9t/m² was encountered at a depth of 12.0m, overlaying a stiff clay layer at a depth of 15.0m. A 2.0-2.5m thick sand layer was intercepted at a depth of 19.5m overlying very stiff clay layers. Two dense sand layers with thickness of about 7.5m and 12.0m
occurred at depths of about 42.5m and 54.0m respectively in hard silty clay layers. A summary of subsoil conditions obtained from borehole data is presented in Table 1.1.

Table 1. Summary of Subsoil Conditions

<table>
<thead>
<tr>
<th>Depth below Ground (m)</th>
<th>Description of Material</th>
<th>Thickness (m)</th>
<th>Su (UC) (t/m²)</th>
<th>SPT N (blow/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2.0</td>
<td>Pavement/top soil</td>
<td>2.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2.0-3.0</td>
<td>Soft Clay 1</td>
<td>1.0</td>
<td>2.0</td>
<td>-</td>
</tr>
<tr>
<td>3.0-12.0</td>
<td>Soft Clay 2</td>
<td>9.0</td>
<td>1.2</td>
<td>-</td>
</tr>
<tr>
<td>12.0-15.0</td>
<td>Medium Stiff Clay</td>
<td>3.0</td>
<td>3.5</td>
<td>-</td>
</tr>
<tr>
<td>15.0-17.0</td>
<td>Stiff Clay</td>
<td>2.0</td>
<td>5.0</td>
<td>-</td>
</tr>
<tr>
<td>17.0-19.5</td>
<td>Stiff to Very Stiff Clay</td>
<td>5.0</td>
<td>12.0</td>
<td>18</td>
</tr>
<tr>
<td>19.5-22.0</td>
<td>Medium Dense Sand</td>
<td>2.5</td>
<td>-</td>
<td>20</td>
</tr>
<tr>
<td>22.0-35.0</td>
<td>Very Stiff Clay</td>
<td>13.0</td>
<td>20.0</td>
<td>30</td>
</tr>
<tr>
<td>35.0-42.5</td>
<td>Stiff Clay</td>
<td>7.5</td>
<td>12.0</td>
<td>18</td>
</tr>
<tr>
<td>42.5-50.0</td>
<td>Dense Sand</td>
<td>7.5</td>
<td>-</td>
<td>&gt;52</td>
</tr>
<tr>
<td>50.0-54.0</td>
<td>Hard Clay/ Silty Clay</td>
<td>4.0</td>
<td>29.9</td>
<td>45</td>
</tr>
<tr>
<td>54.0-66.0</td>
<td>Dense to Very Dense Sand</td>
<td>12.0</td>
<td>-</td>
<td>&gt;60</td>
</tr>
<tr>
<td>66.0-80.0*</td>
<td>Hard Silty Clay</td>
<td>-</td>
<td>-</td>
<td>&gt;45</td>
</tr>
</tbody>
</table>

Note:* at the end of borehole

4. TECHNICAL REQUIREMENTS FOR ENGINEERING WORKS WITHIN PROTECTION ZONE OF MRTA

Over 6 to 10 years, deep excavations with braced diaphragm walls have been carried out in Soft Bangkok Clay for multi-level basements, underground car parks, subway stations and deep shafts for ventilation and launching TBMs. After completion of the subway system, protection zones for the MRTA subway structures were established and technical requirements for future works planned within the protection zone were also issued. There are two protection zones, named A and B. Protection zone A is a zone located immediately above the MRTA structure, plus 3.0m clearance distance measured laterally from the outer faces of MRTA structure. Zone B is located outside the 3.0m clearance distance from the MRTA structure, but within the influence zone based on a line drawn from a point at the center level of MRTA tunnel on the 3m clearance boundary line at 1:1 slope to the ground for MRTA structures with base level no deeper than 20.0m depth. With the base level of MRTA tunnel located deeper than 21.0m below the ground, the influence zone below 21.0m is determined using a line with a slope 2:1 vertical to horizontal ratio from the clearance boundary line at the center level of the structure. Figure 2 illustrates the MRTA protection zones and Figure 3 shows a typical section of the project reported herein together with the protection zone.

The technical requirements generally cover construction method and technique allowed and approval, surcharge load limits, clearance distance to the structure and movement limits induced by the future works.

The most critical requirement for working adjacent to the MRTA structure is the differential movement resulting from the work shall not produce final distortion in the track or its plinth in excess of 3mm in 6.0m (1:2000), or a total movement in the MRTA structure or track not exceeding 6mm in any plan.

Due to the above mentioned requirements, the diaphragm wall design analysis was reviewed with finite element computer modeling to predict the movement of the tunnels caused by basement excavation work. Subsequently, impact assessment was carried out and necessary precautions and safety measures were outlined.

An assessment of works associated with diaphragm wall and bored piling for the project to meet the technical requirements for engineering works within the protection zone is summarized in Table 6.
5. DIAPHRAGM WALL ANALYSIS AND DESIGN

Diaphragm wall analysis and design was primarily carried out using the WALLAP computer program. Stability of wall, bending stresses and wall deflections were computed in relation to planned basement excavation sequence. Due to the presence of the MRTA subway tunnels about 7.8m away from the planned excavation, 2 dimensional analyses using the PLAXIS program was carried out to compute the response/displacement of the tunnels and ground settlements caused by the excavation works.
Based on the primary analysis with WALLAP, 0.8m thick diaphragm wall embedded to a depth of 20.0m with two levels of temporary bracing at -1.50m and 6.20m was required.

WALLAP has been used by the authors for analysis of braced excavations with diaphragm walls and sheet piled walls in Bangkok subsoils and elsewhere. More than 12 projects have been completed by the authors. Additionally 9 underground stations and a number of ventilation shafts of MRTA southern part has been completed with diaphragm walls designed by using the program (Schulz C. 2001).

Prior to determination of tunnel movements using PLAXIS, the same basic model (without foundation piles and tunnel) was used in analysis using both WALLAP and PLAXIS programs for a comparison between analysis results to establish a correlation for further analysis using PLAXIS program. The comparison of analyses indicated that the predicted maximum wall deflection computed by PLAXIS is about 39.0mm which is close to the 43.0mm computed by WALLAP. Figure 4 shows results of analysis from both WALLAP and PLAXIS programs.

![Comparison of Diaphragm Wall Analysis Results from WALLAP and PLAXIS](image)

**Figure 4.** Comparison of Diaphragm Wall Analysis Results from WALLAP and PLAXIS

### 5.1 Wall Deflection and Ground Movements

The characteristics of diaphragm wall movements in the completed projects in Bangkok have been reviewed by Phienwej et al (1998). It was reported that in terms of the ratio of $\delta_{\text{hmax}} / H$, the typical range of the maximum wall movement $\delta_{\text{hmax}}$ fell within 0.3-0.8% of excavation depth H.

For the planned project reported herein, primarily predicted maximum wall movement of 34.00mm, about 0.31% of the excavation depth (11.0m) by WALLAP was considered in the reasonable range and thus control of the wall deflection within the predicted range could be practically achievable, with adequate lateral supports and proper construction sequence and time controls if there is no existing subway tunnel nearby.

However, for calculation of lateral and vertical displacements of the ground, including those of the existing MRTA tunnels, a two dimensional simulation is necessary and thus PLAXIS program was selected. A number of computer simulations were carried out using the PLAXIS program to meet the technical requirements, particularly tunnel displacement criteria.
Initially computer modeling, using PLAXIS to estimate the displacement of the tunnels without soil improvement to the soft clay layer inside the excavation, indicated that tunnel displacement exceeded the limit of 6.0mm specified for the MRTA structure or tracks.

A series of computer models with different wall configurations such as increased wall thickness and embedment were carried out. It was found that the wall system did not significantly control the displacement of the tunnels within the specified limit, but the expected tunnel movement is generally governed by the wall deflection. Moreover, the computer modeling indicated that excavation for installation of the 2nd bracing layer and subsequent removal of the bracing after completion of base slab influenced the overall ground displacement and wall deflection, due to the presence of a thick soft clay layer between at depths of 3.0m to 12.0m. To effectively reduce wall deflection and ground displacement, improvement of the soft clay layer was considered necessary. We considered also that the site conditions allowed the soft clay to be improved to the strength of stiff clay with deep cement mixing inside the excavation zone close to the south-bound tunnel. The improve soil block could act as a counterfort to the wall during installation of the 2nd bracing layer. Additionally the improved soil block could also be used as a berm or contingency support for the wall during final excavation depth.

Predicted lateral and vertical displacements of the ground and tunnel movements are summarized and presented in Table 2.

<table>
<thead>
<tr>
<th>Model Type</th>
<th>Maximum Movement of ground (mm)</th>
<th>Maximum Movement of Tunnel (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple model without soil improvement</td>
<td>40.57</td>
<td>38.22</td>
</tr>
<tr>
<td>Improved Soft Clay layer @3.0-13.5m, 5.0m wide inside excavation.</td>
<td>26.54</td>
<td>25.79</td>
</tr>
</tbody>
</table>

5.2 Comparison with Similar Projects elsewhere

Publication of other cases resembling that of the planned project (Case A) in Bangkok has not been found and thus published papers on the similar/relevant projects elsewhere are reviewed and an assessment of movements in the MRTA structure or tracks induced by the planned excavation is carried out and summarized as below.
Although, general thickness of clay deposits vary with the projects, the nature of the work is similar, it is noteworthy that deep excavation work was carried out in soft soils adjacent to existing subway tunnels.

### Table 4. Summary of Excavation Configurations

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Project Name</th>
<th>Max. Excavation Depth (m)</th>
<th>Distance to Outer Face of Tunnel (m)</th>
<th>Depth to Tunnel Crown (m)</th>
<th>Wall Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Planned Project on Asoke Road</td>
<td>-11.0</td>
<td>7.8</td>
<td>15.0</td>
<td>0.8mx20m D-Wall</td>
</tr>
<tr>
<td>B</td>
<td>Inland Revenue Headquarters</td>
<td>-18.0</td>
<td>16.0</td>
<td>11.0</td>
<td>1.0mx24.5m D-Wall</td>
</tr>
<tr>
<td>C</td>
<td>New Dhoby Ghaut Station</td>
<td>-30.0</td>
<td>4.5</td>
<td>11.5</td>
<td>Φ0.8+1.0mx35.0m Secant pile wall</td>
</tr>
<tr>
<td>D</td>
<td>Cheung Sha Wan Gov. Office</td>
<td>-18.5</td>
<td>&lt;5.0</td>
<td>19.0-22.0</td>
<td>D-Wall</td>
</tr>
<tr>
<td>E</td>
<td>People Park Station</td>
<td>-23.067</td>
<td>12.9</td>
<td>6.959</td>
<td>D-Wall</td>
</tr>
<tr>
<td>F</td>
<td>New World Mansion</td>
<td>-13.0</td>
<td>3.0</td>
<td>8.0</td>
<td>26m D-Wall</td>
</tr>
<tr>
<td>G</td>
<td>Hong Kong Square</td>
<td>-14</td>
<td>7.0</td>
<td>-</td>
<td>0.8x24.0m D-Wall</td>
</tr>
<tr>
<td>H</td>
<td>The George Green Cut&amp;Cover</td>
<td>-9.0</td>
<td>1.0m to below excavation</td>
<td>13.7</td>
<td>Φ1.0mx19.0m Secant pile wall</td>
</tr>
<tr>
<td>Case No.</td>
<td>Project Name</td>
<td>Excavation Method</td>
<td>Max. Wall Deflection (mm)</td>
<td>Max. Tunnel Displacement (mm)</td>
<td></td>
</tr>
<tr>
<td>---------</td>
<td>------------------------------------</td>
<td>----------------------------</td>
<td>---------------------------</td>
<td>------------------------------</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Predicted</td>
<td>Actual</td>
<td>Predicted</td>
<td>Actual</td>
</tr>
<tr>
<td>A</td>
<td>Planned Project on Asoke Road</td>
<td>Bottom Up 2 level Strut + improved soil berm</td>
<td>26.54</td>
<td>-</td>
<td>5.97</td>
</tr>
<tr>
<td>B</td>
<td>Inland Revenue Headquarters</td>
<td>Top-down</td>
<td>30</td>
<td>25</td>
<td>11</td>
</tr>
<tr>
<td>C</td>
<td>New Dhoby Ghaut Station</td>
<td>Bottom Up 7 level Strut</td>
<td>24</td>
<td></td>
<td>15</td>
</tr>
<tr>
<td>D</td>
<td>Cheung Sha Wan Gov. Office</td>
<td>Top-down</td>
<td>&lt;25</td>
<td>&lt;25</td>
<td>17</td>
</tr>
<tr>
<td>E</td>
<td>People Park Station</td>
<td>Bottom Up 5 level Strut</td>
<td>35</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>New World Mansion</td>
<td>Bottom Up</td>
<td>-</td>
<td>&lt;12</td>
<td>-</td>
</tr>
<tr>
<td>G</td>
<td>Hong Kong Square</td>
<td>Bottom Up &amp; improved soil berm + 3 level strut</td>
<td>&lt;40</td>
<td>&lt;40</td>
<td>&lt;10</td>
</tr>
<tr>
<td>H</td>
<td>The George Green Cut&amp;Cover</td>
<td>Top-down</td>
<td>11</td>
<td>12</td>
<td>-</td>
</tr>
</tbody>
</table>

Review of the above completed projects, with the exception of the planned project (Case A), indicates that the specified limit of tunnel displacement ranges from 10mm to 20mm. Details of tunnel lining and specified limits of the railway track movement of these projects were not specifically reported. Only in Case No. C (Ref. Niu et al 2003), maximum movements of 7mm and 15mm were recorded for both the railway track and tunnel respectively.

Based on the projects mentioned above, controlling the maximum movements of the MRTA tunnel and railway track induced by the planned excavation for the project, within the specified limits is possible. However the excavation work must be carried out by competent contractors with contingency plans such as that outlined previously, with the geotechnical instrumentation and monitoring mentioned in the following section as a minimum requirement.

5.3 Geotechnical Instrumentation and Monitoring

The following instrumentation is considered essential and monitoring must be carried out throughout the basement construction period.

- **Inclinometer** - 2 inclinometers in the ground between the tunnel and planned diaphragm wall and (for diaphragm wall construction) 4 to 5 inclinometers in the diaphragm wall around the excavation.

- **Convergent Bolts & Tape Extensometer** – If necessary, adequate quantities of convergent bolts may be installed on the tunnel segments to monitor the convergent movement with tape extensometer at each construction stage throughout the basement excavation period.

- **Settlement Plates/markers** – A minimum of 4 settlement points on the ground across the tunnels needs to be installed in line with each inclinometer location and surveyed at each construction stage throughout the construction period.

- **Survey** – Movement of tracks must be surveyed at each construction stage throughout the basement excavation period.

- **Contingency Plan and Actions** - Since the predicted displacement of the tunnels is very close to the specified limits, monitoring of adjacent structures and ground movements during the staged
excavation is necessary, with checking against the predicted movements at every stage of construction. A contingency plan with trigger values of displacements being based on the predicted values must be prepared and necessary action must be taken immediately if the measured movements of diaphragm wall and tunnels exceed the trigger values set. It was suggested that a contingency plan including, but not limited to, the following be prepared and carried out accordingly when required.

- Preloading of struts as necessary
- Localized excavation and monitoring
- Provision of an additional bracing layer
- Speeding up the construction.

6. SUMMARY OF ASSESSMENT

An assessment of works associated with bored piling and diaphragm wall work to meet Technical Requirements for Engineering Works within the Protection Zone of the MRTA Chaloem Ratchamonnkhon Line is summarized in the table below.

Table 6. Summary of Assessment of Basement Excavation with Braced Diaphragm Wall

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Specified Requirement</th>
<th>Provision</th>
<th>Assessment/Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Diaphragm Wall Bored Piling Bracing and Excavation</td>
<td>Method Statements</td>
<td>Yes Yes outlined</td>
<td>Documents prepared &amp; submitted for approval</td>
</tr>
<tr>
<td>2</td>
<td>Calculations</td>
<td>Signed with Chartered Engineer</td>
<td>Yes</td>
<td>Conformed</td>
</tr>
<tr>
<td>2.1</td>
<td>Engineering Work in Zone B</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1.1</td>
<td>Surcharge at ground level</td>
<td>Not &gt; 50kPa (5t/m2)</td>
<td>15kPa (1.5t/m2)</td>
<td>Used in D-Wall analysis and Design</td>
</tr>
<tr>
<td>2.1.2</td>
<td>Surcharge on tunnel</td>
<td>Not &gt; 25KPa (2.5t/m2)</td>
<td>15kPa (1.5t/m2)</td>
<td>Used in D-Wall analysis and Design</td>
</tr>
<tr>
<td>2.1.3</td>
<td>Differential Movement</td>
<td>&lt; 1:2000 6mm in total</td>
<td>&lt; 6mm</td>
<td>Use observational approach and instrumentation program</td>
</tr>
<tr>
<td>2.1.4</td>
<td>Clear distance between pile/d-wall and MRTA structure</td>
<td>&gt; 3m or (5) pile diameter</td>
<td>&gt; 6m or (7) pile diameter</td>
<td>layout plan and drawings conform.</td>
</tr>
<tr>
<td>2.1.5</td>
<td>Additional surcharge load by piles</td>
<td>Not &gt; 50kPa @ designed GL</td>
<td>-</td>
<td>Pile cutoff is below 11.0m and generally outside Zone B. D-Wall is supported by pile legs to 57-60.0m.</td>
</tr>
<tr>
<td>2.1.6</td>
<td>Piling</td>
<td>Bored piling with auger under slurry</td>
<td>Yes</td>
<td>Method statement conforms.</td>
</tr>
<tr>
<td>2.1.7</td>
<td>Use of vibratory method</td>
<td>Not within 10m</td>
<td>&gt; 10m</td>
<td>Method statement conforms.</td>
</tr>
<tr>
<td>2.1.8</td>
<td>Future additional surcharge in Zone B</td>
<td>Use 1:1 slope of load distribution</td>
<td>Yes</td>
<td>Conformed</td>
</tr>
<tr>
<td>2.1.9</td>
<td>Approval for excavation adjacent to MRTA structures</td>
<td>&gt;3.5m near tunnels or station &gt;1.5m near others Suitable instrumentation</td>
<td>~ 11.0m with braced D-Wall Instrumentation program outlined</td>
<td>Approval is obtained.</td>
</tr>
<tr>
<td>2.1.10</td>
<td>Site investigation boreholes for location of MRTA structure</td>
<td>Fully grouted approved by the authority</td>
<td>-</td>
<td>The site is about 8m away from the MRTA structure.</td>
</tr>
<tr>
<td>2.1.11</td>
<td>Soil Improvement</td>
<td>Approval submission</td>
<td>Yes (by Main contractor)</td>
<td>It will be inside the perimeter D-wall as analysed.</td>
</tr>
</tbody>
</table>
Moreover we also concluded that the structural design of the diaphragm wall as analyzed and designed with the WALLAP program was adequate for the planned excavation work. However the maximum deflection of the wall adjacent to the existing tunnels had to be reduced to 20mm, to limit the displacement of the tunnel within the specified displacement as indicated by computer modeling with PLAXIS program. Additionally, soft clay adjacent to the south-bound tunnel inside the excavation at depths between 3.0m and 13.5m must be treated to improve the shear strength of the soil to that of stiff clay layer (about 10t/m²). We intended the soil improvement to minimize the wall movement and acts as a contingency measure in excavation. The observational approach is strongly recommended for a deep excavation in the proximity of exiting tunnels.

7. DIAPHRAGM WALL CONSTRUCTION

A review assessment of ground movements induced by diaphragm walling was also carried out during diaphragm wall design and analysis. Ground movements associated with diaphragm wall trenching in Bangkok clay has been studied and reported (Thasnanipan et al, 2004). It was reported that an inclinometer (24m long) was installed 1.0m away from the trial trench face for monitoring of soil movement during and after trenching. Additionally 6 surface settlement points with 2.0m spacing were also installed and monitored. The trial trench 0.6x4.5x18.0m was excavated under polymer slurry. The monitoring results indicated that vertical ground settlement did not extend beyond 8.0m from the trench face. The maximum vertical settlement of 9mm was found to occur at about 2.0m away from the trench face. The inclinometer readings which were recorded 1.0m away from the trench face indicated that the maximum lateral soil movement of 5.8mm occurred at the depth of about 3.5m below the ground. At 15.0m depth, the lateral movement of soil 1.0m away from the trench was about 1.0mm after 12 hours of trench excavation. Based on the above information, for the diaphragm wall construction adjacent to the existing MRTA tunnels located about 7.8m away from the outer face of planned diaphragm wall of the project site and at depths of about 15.0m below ground level, the following statements were made:

1. Vertical displacement of ground due to diaphragm wall trenching was expected to be within 8.0m from the trench face and thus not affect the tunnels which were located beyond 7.8m from the trenching.
2. Lateral soil displacement within 1.0m from the trench face at a depth of 15.0m depth was about 1.0mm. It was expected that the displacement would decreasing distance from the trench face. At the tunnel location, no lateral soil displacement was expected as the south-bound tunnel was about 8.0m from the trench face.
3. For trench stability, bentonite slurry was used instead of polymer slurry.

Diaphragm wall construction was carried out with a conventional rope-suspended mechanical grab under bentonite slurry in July 2005. Two inclinometer tubes were installed between the south-bound tunnel and the planned diaphragm wall, about 4.0m from the outer face. Since the subway tunnels were beneath Asoke Road, settlement marks were established on the pavement in line with the inclinometer location and crossing the tunnel alignment. Monitoring of ground movements was carried out during construction of diaphragm wall panels nearest to the inclinometer tubes. Figure 8 shows the layout of inclinometer and settlement marks. For construction of bored piles, 15m long temporary casing was used. The piles located within protection zone B were constructed after completion of adjacent diaphragm wall panels to protect from the ground vibration induced by temporary casing installation and extraction.

7.1 Soil Movement during Diaphragm Wall Construction

Soil movement during diaphragm wall panel construction were monitored (Table 7). Results of settlement survey at the settlement marks indicated no vertical movements of the ground.
Table 7. Summary of Monitoring Results (I-6 & I-7): Soil Movements at Inclinometer Locations

<table>
<thead>
<tr>
<th>Construction Activities</th>
<th>Movement @ 15.0m depth (mm)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>D-Wall panel excavation to -10.0m</td>
<td>-0.87</td>
<td>-0.40</td>
</tr>
<tr>
<td>D-Wall panel excavation to -20.0m</td>
<td>1.04</td>
<td>-0.81</td>
</tr>
<tr>
<td>After de-sanding</td>
<td>0.12</td>
<td>-0.38</td>
</tr>
<tr>
<td>After concrete pouring</td>
<td>-1.91</td>
<td>-2.18</td>
</tr>
<tr>
<td>24hrs after concrete pouring</td>
<td>-2.34</td>
<td>-1.62</td>
</tr>
<tr>
<td>36/48hrs after concrete pouring</td>
<td>-1.13</td>
<td>-2.55</td>
</tr>
</tbody>
</table>

Note: “-“ movement towards tunnel (opposite to direction of excavation)

Figure 7. Inclinometer Monitoring Results of I-6 and I-7

Figure 8. Layout of Instrumentation for Diaphragm Panel Construction

The following conclusions were made based on the above monitoring results.

1. No vertical displacement was observed at the surface settlement points; vertical displacement of ground due to diaphragm wall trenching seemed to be within 2.0m from the trench face and would not affect the tunnels which were located 7.8m beyond the trenching.

2. Maximum lateral soil displacement at 4.0m from the trench face at 15.0m depth was about 1.0mm towards excavation. It was expected that the displacement would decrease with distance increasing from the trench face. At the tunnel location, lateral soil displacement was estimated to be less than 0.5mm based on the extrapolation of displacement data.

3. After concrete casting of the panels, lateral soil movements were found to be towards the tunnels, possibly from pressure of wet concrete. Maximum movement at depth 15.0m at the inclinometer location was about 2.18mm after concrete pouring. After 36hrs of concrete casting, the maximum movement observed was about 2.55mm. At the tunnel location, lateral soil displacement was estimated to be less than 1.6mm based on the extrapolation of displacement data.

4. After the diaphragm wall panel was completed, residual movement of the soil towards the tunnels was observed. Due to the direction of this movement, it could be excluded from tunnel movement measurements induced by diaphragm wall deflection during basement excavation work.
8. CONCLUSION

At the time of drafting this paper, soil improvement with soil-cement columns was completed and excavation work has commenced with specified instrumentation and monitoring programs. An independent third party consultant has been appointed by the main contractor to review, inspect and supervise the excavation work.

Diaphragm wall and foundation bored pile construction was completed, in compliance with technical requirements for engineering work within the MRTA protection zone. Review and assessment of the deep excavation with diaphragm wall adjacent to the existing tunnels indicated that the project is feasible with adequate monitoring and contingency plans as outlined.

It is also concluded that diaphragm wall construction in close vicinity of existing subway tunnels was successfully completed with adequate instrumentation and monitoring.

Further research is recommended on the actual performance of the diaphragm wall when basement construction data and complete monitoring results are available.

REFERENCES


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Application of Observational Method in Diaphragm Wall Support Excavation

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Application of Observational Method in Diaphragm Wall Support Excavation

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1,2,3,4 Seafco Public Co., Ltd., Bangkok, Thailand

Abstract:
Application of observational method in managing geotechnical risk in diaphragm wall support excavation in Bangkok is presented in this paper. Intensive modification of construction sequence in actual work execution with “value engineering options” different from tender stage design along with application of observational method in Diaphragm-wall support two level underground car parking building located in the historically significant area of Bangkok is firstly demonstrated. Simple approach of observational method in risk assessment of buildings and structures backup by extensive instrumentation in tunneling and deep excavation works of Contract No. 1 of the M.R.T Chaloem Ratchamongkhon Line, the first underground mass transit system project of Bangkok (MRTA) is also reported. Key Factors contributed in successful application of observational method are discussed.

1. INTRODUCTION
Observational Method (OM) is not a very popular term among geotechnical engineers in Thailand, though the method has been applied in some major projects. The main reason of unpopularity of the method in Thailand may be due to the fact that traditionally there is less option opened for design modification as the project progresses. Geotechnical engineers may be more concerned on misused and limitation of the method and do not prefer to take risk. It is important to discuss among geotechnical engineers with regard to overall philosophy, key requirement, limitation and practical approach in application of observational method.

Tunneling works and deep excavation works have a high degree of geotechnical risk that needs to be identified, assessed, minimized and managed. Deep excavation and tunneling works generally interact with natural ground or geo-material and incorporating their characteristics as structural elements of their own stability. This interaction is more significant in underground excavation and tunneling projects than in other civil engineering projects. In this paper, application of observation method in managing geotechnical risk in two different projects is presented.

2. OBSERVATIONAL METHOD AND GEOTECHNICAL RISK MANAGEMENT
Historically, engineers used observational method to deal with uncertainties in the ground or geotechnical risks in early age of civil engineering. Peck (1967) pointed out the advantages of observational method by giving an example on a dam across the Svir River in Russia, from a documented record of the work of Graftio (1936).

Professor R. B. Peck set out procedures for the observational method (OM) as applied in soil mechanics in the Ninth Rankine Lecture (Peck, 1969). Peck described the limitation and drawbacks of observational method. Powderham (1996) reviewed the main features of the observational method of Peck and summarized the key requirements as follows;

(1) It must be possible to alter the design during construction
(2) The contractual condition must be compatible and allow design to be directly related to actual construction method
(3) An acceptable level of risk must be identified and controlled. In particular this requires a planned course of action for every foreseeable eventuality
Application of Observational Method in Diaphragm Wall Support Excavation

Zaw Zaw Aye¹, Chanchai Submaneewong², Thayanan Boonyarak³ and Chakkrit Chanchad⁴
¹,²,³,⁴ Seafo Public Co., Ltd., Bangkok, Thailand

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3. An acceptable level of risk must be identified and controlled. In particular this requires a planned course of action for every foreseeable eventuality
3. OBSERVATIONAL METHOD, RISK MANAGEMENT AND VALUE ENGINEERING

Figure 1 shows a generalized link between the elements of observational method and risk management with integration of value engineering.

4. VALUE ENGINEERING AND APPLICATION OF OM IN BASEMENT EXCAVATION WITH BUTTRESS-SUPPORT THIN DIAPHRAGM WALL

This project demonstrated the benefits of value engineering option powered by effective application of observational method in historically significant area of Bangkok. The project is a two-level underground car park located in the center of Rattanakosin Island, the heart of an old established, historically and culturally significant area of Bangkok. The project owner, Bangkok Metropolitan Authority (BMA) awarded the semi-turnkey basis construction contract “Lam Kon Muang Underground Car Park” to SEAFCO Co., Ltd. as a contractor. The contract consists of 3 major scope of works: (1) Construction of building foundation and retaining structure - diaphragm wall, barrette and bored piles (2) Excavation works including temporary bracing design and installation (3) Construction of the entire two-level underground car park building having car park area of 18,552 m² and roof-level park of 10,936m² plus cut-and-cover tunnel, underpass access to the City Hall. Geotechnical aspects highlighting the application of observational method and performance of buttress-support diaphragm wall of 0.60 m width for two-level underground car park building is discussed in this paper.

5. PROJECT REQUIREMENT AND MAJOR CONSTRAINTS

Since there is a limited availability of car parking space in the surrounding congested neighborhood and the project site was being used as a grade-level car parking space prior to the award of the contract, the key requirement was to construct the underground car park in two phases – to construct Phase 1 while leaving space for car parking in Phase 2, and to utilize semi-finished underground car park of Phase 1 during construction of Phase 2. This requirement posed the need of temporary retaining wall between Phase 1 and 2.
Construction site is surrounded by numbers of sensitive structures as shown in Figure 2, - in the south, Wat Suthat, one of Thailand’s most important temples and the Historical Giant-swing, in the north, the City Hall and at South-east corner, the Historical Brahmmin Temple. Rows of old shop-house buildings are closely located in the east and west boundaries of the project. Location of the project site itself in the vicinity of sensitive structures and buildings therefore posed some constraints, which called for the need of careful consideration in establishing the design principles and sequence of construction.

![Figure 2. Layout plan of the project showing adjacent buildings](image)

Location of the project site itself in the vicinity of sensitive structures and buildings therefore posed some constraints, which called for the need of careful consideration in establishing the design principles and sequence of construction. The architectural and utility aspects of the project called for the design of the basement with a number of openings from the ground surface to the final basement slab level to facilitate the ventilation system as shown in figure 3.

Hence the roof slab can not be physically utilized as bracing in most of the area where the diaphragm wall is to be acting as a cantilever retaining wall in the permanent stage. It was analyzed in the preliminary analyses that the deflection of the diaphragm wall of 0.60 m width would be large if it was to be fully cantilevered. As the project is located in a sensitive area, ground movement induced by large deflection was unfavorable. It was therefore decided to use buttress to minimize the diaphragm wall deflection as shown in figure 3.
6. DIAPHRAGM WALLS BARRETES AND BORED PILES CONSTRUCTION

Diaphragm wall having 600mm width founded at 16m below ground level (B.G.L) was constructed simultaneously with dry-processed bored piles of diameter 600 mm with toe depth 20 m below ground level. Barrettes having same toe depth as bored piles were installed at 8m spacing along with diaphragm wall panels. Sheet pile wall (14m deep) was used as a temporary retaining wall at the boundary of Phase 1 and 2 as shown in Figure 4.

7. SUBSOIL CONDITION AND DESIGN PARAMETERS

Typical subsoil profile at the site is characterized by thick Bangkok soft clay layer at the top followed by thin layer of medium clay, and stiff clay layers.
8. ORIGINAL TENDER STAGE DESIGN

The designers involved in the tender stage design made a fairly conservative design with two levels temporary bracing as shown in Figure 5. Uncertainty of the performance of a thin diaphragm wall in soft clay layer was the likely reason to adopt the conservative design in the tender stage. It is not unreasonable to adopt the conservative design considering time-dependent consolidation property of soft marine clay and likely long elapsed time of un-strutted diaphragm wall (relatively long un-strutted span of about 6.0 m between first strut and final excavation level for 600 mm diaphragm wall) due to large volume of excavation work involved.

Figure 5. Tender stage bracing system – diaphragm wall was designed with soil-berm and 2 struts support (horizontal bracing and raker)
9. VALUE ENGINEERING OPTION FOR PHASE 1

Value engineering review of the temporary works was undertaken by the contractor’s new in-house design engineering team prior to the commencement of Phase 1 excavation works. Rigorous attention to detail of the design concept and constructability was made in the pre-construction discussions between design engineers and construction team.

The main objectives for value engineering options were to minimize the material and construction sequence involved in temporary works so as to accelerate the excavation time thereby saving overall costs without compromising the safety aspect. After conducting a series of re-analyses with different conditions major modifications were made: (1) To lower the first strut level to –1.8 m from the original tender stage design level –1.0 m (2) To use only 1 temporary strut, omitting second level raking strut with the provision of sloping soil berm against diaphragm walls. Soil berm was to remove after completion of base slab construction in the majority of area – minimizing the elapsed time of partially un-strutted diaphragm wall between temporary strut and the final excavation level. Modified bracing system of Phase 1 as an outcome of value engineering review is illustrated in Figure 6.

![Figure 6. Value engineering option - Modified Bracing system for Phase 1](image)

10. IMPLEMENTATION OF THE OBSERVATIONAL METHOD

During the review of tender stage design for Phase 1, it was recognized that two-phase excavation works in this project was ideally suitable for application of the observation method. The flexibility of the contractual requirement in temporary design which allowed the contractor to modify the design and construction method also provided the favor for the observational method.

In order to assess the most probable condition assumed in “value engineering design” and to take necessary actions if monitoring results reveal most unfavorable conditions (i.e. actual deflection of diaphragm wall reaches maximum acceptable limit), the observational method was implemented on the followings basis.

- Reviewed the design parameters together with critical conditions posed on sites as well as most critical stage in excavation and basement construction work
- Predicted the performance of diaphragm wall with “most probable” as well as “most unfavorable” conditions and parameters.
- Established the trigger criteria based on predicted diaphragm wall deflection
- Predefined the practical contingency plan for “most unfavorable” conditions where wall deflection reaches trigger levels
- Set out the instrumentation program with the consideration of above factors
• Monitored the performance of diaphragm wall. Compared the monitoring results with the predicted and trigger values and reassessed
• Implemented the contingency measures if monitoring result reaches action level of trigger values

Process of OM in Phase 1 is illustrated in a flowchart in figure 7.

Figure 7. Flowchart showing the OM process involved in Phase 1

Figure 8 shows the predicted diaphragm wall lateral displacement or deflection of Phase 1 (east, west and south diaphragm wall) at two conditions together with trigger levels and tender stage prediction. It should be noted that the diaphragm wall deflection was predicted to be maximum or most critical after removing the horizontal temporary strut.

Most probable condition was established for the predicted deflection of diaphragm wall with full influence of buttress support – assuming buttress effectively supports as permanent strut in diaphragm wall analysis model. Most unfavorable condition was set out for the predicted deflection of diaphragm wall without considering influence of buttress – buttress was excluded in the model. Diaphragm wall reinforcement was designed based on the most unfavorable condition.

Figure 8. Prediction of diaphragm wall performance in pre-construction stage with trigger criteria in comparison with tender stage prediction
In establishing the criteria for trigger values, it was necessary to consider the broad context in which diaphragm wall exists, design assumption and concept, likely behaviour of the wall itself or its predicted performance and effectiveness of selected temporary bracing system. Trigger levels were established to provide the design team and construction team, an opportunity for early review and resetting of the monitoring frequency as well as for implementing the contingency measure as necessary.

In general, exceeding alert trigger levels must initiate a review of design data, construction progress and monitoring frequencies with the consideration of possible measures to limit further deflection. Exceeding action trigger levels must initiate further review of above mentioned points and if necessary to initiate a planned course of action or contingency measures.

Effective and good communications between the design team and construction crew were made all along the excavation stages with clear responsibilities in construction control. The contingency measures included immediate backing filling of excavated soil and installing of temporary struts in the critical area.

11. MONITORING PROGRAM

In planning the monitoring system and program, it is important to consider the parameters to be measured which reflect the actual performance of the diaphragm wall support excavation. It is also necessary to take account the practical measurement applicable for established trigger criteria, number and frequency of measurement required to carry out a meaningful interpretation of wall behaviour which would be integrated in the implementation of observational method.

Comprehensive and robust monitoring program was set up as a key element in application of the observational method and to ensure that modified construction sequence would not have adverse effect in temporary stage and on permanent design. A total of 6 inclinometers (3 in each phase) were installed in diaphragm wall together with some survey points. In order to make effective use of the established trigger levels, an adequate number of measurement were carried out at appropriate frequency. Typical monitoring frequencies for inclinometer set up as guideline for the project is outlined below.

- Measured immediately before commencing excavation in the vicinity of instrument
- Minimum readings of 2 times a week while excavation in progress
- Minimum readings of 1 time a week when no excavation the vicinity of instrument
- Minimum readings of 3 times a week when measured deflection values exceeded alert trigger levels
- Minimum reading 1 time a day when measured deflection values reached action trigger levels

As the most critical stage was predicted at the time horizontal temporary bracings were removed, a full attention was paid to the inclinometer monitoring with the following special criteria and frequencies.

- First temporary strut removal was to carry out at the diaphragm wall panel where inclinometer was located
- Measured immediately before removal of temporary strut at the closest distance to the instrument.
- Measured every 6-8 hours immediately after removing the first strut
- Second strut to be removed must be the one located immediately adjacent to first strut which had removed
- Not to remove the second strut until inclinometer measured deflection values had stabilized
- Not to remove more struts unless measured deflection values were stabilized and within alarm trigger levels

In addition to inclinometer measurement, diaphragm wall movement was also monitored by the survey points strategically marked on the wall panels. Ground settlement and surface cracks behind the diaphragm wall were also visually checked by the construction team as daily basic.
12. PERFORMANCE OF PHASE 1 DIAPHRAGM WALL

After carrying out the comprehensive desk studies and establishment of systematic monitoring program presented above, Phase 1 excavation work was carefully commenced. Figure 9 shows the maximum accumulated diaphragm wall deflection at different stages of excavation monitored by inclinometer No.1 (I-1 at East wall of Phase 1) together with trigger levels and predicted maximum deflection profile of 3 different conditions - tender stage design (2 temporary struts), modified design with buttress and modified design without buttress. It can be observed from figure that measured lateral movement pattern of diaphragm wall agreed well with that of prediction for modified design with buttress - meaning buttress-support has significant influence on wall deflection.

Figure 9. Phase 1 East Wall-Monitored diaphragm wall deflection at different stages with trigger levels

Figure 10. Phase 1 South Wall - Monitored diaphragm wall deflection at different stages with trigger levels
Deflection profile of South diaphragm wall which braced against temporary sheet pile wall at Phase 1 and 2 boundaries is presented in Figure 10. As can be seen in Figure 10, South diaphragm wall deflection is significantly higher than that of east wall, which is likely to be caused by the fact that South diaphragm wall is braced with more flexible sheet pile wall. Description of stages shown in the legend of Figure 9 and 10 is summarized in Table 3.

Table 3. Description of stages shown in Figure 9 & 10

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Excavate to –2.2m and installed temporary strut</td>
</tr>
<tr>
<td>2</td>
<td>Excavate to –4m</td>
</tr>
<tr>
<td>3</td>
<td>Excavate to –6.6m with berm</td>
</tr>
<tr>
<td>4</td>
<td>Removal of berm</td>
</tr>
<tr>
<td>5</td>
<td>Removal of temporary strut after completion of buttress</td>
</tr>
</tbody>
</table>

Figure 11. Phase 1 excavation work in progress with historical Buddhist temple Wat Suthat in background (Phase 2 area was being used as car parking space)

13. OM IN PHASE 2 - MODIFICATION OF BRACING SYSTEM

Monitoring results of the Phase 1 excavation work provided an ample opportunity to review the design assumption, fine tune the parameters used in the analysis of the diaphragm wall for the Phase 2 and made modification of construction sequence. The major modifications are: (1) Removal of soil-berm at East and West diaphragm wall in shorter duration than that of Phase 1, and (2) Using raking struts instead of horizontal strut for North diaphragm wall as shown in Figure 12.
Deflection profile of South diaphragm wall which braced against temporary sheet pile wall at Phase 1 and 2 boundaries is presented in Figure 10. As can be seen in Figure 10, South diaphragm wall deflection is significantly higher than that of east wall, which is likely to be caused by the fact that South diaphragm wall is braced with more flexible sheet pile wall. Description of stages shown in the legend of Figure 9 and 10 is summarized in Table 3.

Table 3. Description of stages shown in Figure 9 & 10

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Excavate to –2.2m and installed temporary strut</td>
</tr>
<tr>
<td>2</td>
<td>Excavate to –4m</td>
</tr>
<tr>
<td>3</td>
<td>Excavate to –6.6m with berm</td>
</tr>
<tr>
<td>4</td>
<td>Removal of berm</td>
</tr>
<tr>
<td>5</td>
<td>Removal of temporary strut after completion of buttress</td>
</tr>
</tbody>
</table>

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Monitoring results of the Phase 1 excavation work provided an ample opportunity to review the design assumption, fine tune the parameters used in the analysis of the diaphragm wall for the Phase 2 and made modification of construction sequence. The major modifications are: (1) Removal of soil-berm at East and West diaphragm wall in shorter duration than that of Phase 1, and (2) Using raking struts instead of horizontal strut for North diaphragm wall as shown in Figure 12.

Process involved in application of OM in Phase 2 is shown in figure 13.

Figure 12. Bracing system of Phase 2 diaphragm walls

Figure 13. Flowchart showing the OM process involved in Phase 2

14. PERFORMANCE OF PHASE 2 DIAPHRAGM WALL

Figure 14 depicts the measured deflection of east diaphragm wall of Phase 2. As can be observed in Figure 14 in comparison with Figure 9, the maximum deflection of east diaphragm wall in Phase 2 is larger than that of Phase 1.
The likely reasons of this observation are;
• Un-strutted elapsed time for first temporary bracing in Phase 2 was longer than that of Phase 1.
• In Phase 1, horizontal struts were installed in north-south direction which temporary kingpost columns and strut were integrated in crisscross pattern with east-west struts - providing complete-support more rigid bracing system. Whereas in Phase 2, horizontal struts were installed only in east-west direction without having crisscross pattern with north wall – having less rigid bracing system than that of Phase 1.

![Figure 14. Phase 2 East Wall - Monitored diaphragm wall deflection at different stages with trigger levels](image)

Figure 14. Phase 2 East Wall - Monitored diaphragm wall deflection at different stages with trigger levels

With assurance of diaphragm wall performance from monitoring results of Phase 1, original plan of using horizontal struts for North diaphragm wall was modified by using raking struts instead. As can be seen in Figure 15, deflection of North diaphragm wall (with raking strut support) is significantly higher than that of East diaphragm wall (with horizontal strut support). The main reason of larger movement of north diaphragm wall is due to the fact that it was supported only by the berm for the long period (about 52 days) before completion of raking struts so that soil-berm became soften during the long elapsed un-strutted period.

![Figure 15. Phase 2 North Wall - Monitored diaphragm wall deflection at different stages with trigger levels](image)

Figure 15. Phase 2 North Wall - Monitored diaphragm wall deflection at different stages with trigger levels

Table 4. Description of stages shown in Figure 14 & 15

<table>
<thead>
<tr>
<th>Stage</th>
<th>East Diaphragm wall</th>
<th>North Diaphragm wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Excavate to −2.2m and installed temporary strut</td>
<td>Excavate to −2.2m at d-wall, and to −6.6m with sloping berm</td>
</tr>
<tr>
<td>2</td>
<td>Excavate to −4m</td>
<td>Installed raking strut</td>
</tr>
<tr>
<td>3</td>
<td>Excavate to −6.6m with berm</td>
<td>Removal of berm</td>
</tr>
<tr>
<td>4</td>
<td>Removal of berm</td>
<td>Removal of raker after completion of buttress</td>
</tr>
<tr>
<td>5</td>
<td>Removal of temporary strut after completion of buttress</td>
<td></td>
</tr>
</tbody>
</table>

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Figure 15. Phase 2 North Wall - Monitored diaphragm wall deflection at different stages with trigger levels

Time-dependent deflection pattern due to softening and deformation of soft clay can be observed in North diaphragm wall as illustrated in Figure 17. North diaphragm wall moved progressively toward excavation before completion of raking struts (at 52 days) as can be seen in Figure 17.

Figure 16. View of buttress-support diaphragm wall prior to temporary bracing removal
15. IMPLEMENTATION OF CONTINGENCY PLAN

Since deflection of North diaphragm wall (Phase 2) approached action trigger levels, monitoring frequency was increased and the following contingency measures were implemented on site.

- Poured 15cm thick 1m wide lean concrete on the top of the berm along North diaphragm wall to provide bearing-effect
- Installed additional king-post and diagonal struts attached to the raking struts to provide more rigid support against diaphragm wall
- Soil-berm was removed locally in bays followed by construction of wale beam, tie beam and buttress as shown in Figure 18.

Figure 17. Phase 2 – Time dependent wall deflection of North diaphragm wall – diaphragm wall was supported only by soil-berm for 52 days before completion of raker installation.

Figure 18. Perspective of North diaphragm wall constructed in bay
Movement of diaphragm wall was observed to be decreased and eventually stabilized by the above actions. No significant ground settlement was observed in the vicinity of the North diaphragm wall.

Figure 17. Phase 2 – Time dependent wall deflection of North diaphragm wall – diaphragm wall was supported only by soil-berm for 52 days before completion of raker installation.

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Figure 18. Perspective of North diaphragm wall constructed in bay

Figure 19. View of raker and soil-berm support – soil-berm was removed locally bay by bay

Figure 20. View of buttress-support diaphragm wall after removal of temporary bracing
16. TIME AND COST SAVING FROM VALUE ENGINEERING OPTIONS AND THE APPLICATION OF OBSERVATIONAL METHOD

Significant cost and time saving were achieved from the value engineering option coupled with observational method implemented for both Phase 1 and 2. The major savings were achieved by less operation and material utilized in the following elements of temporary works.

- Cancellation of 2nd level raking struts against diaphragm wall for both phases
- Modification of bracing system – using raking struts with soil-berm support instead of horizontal struts for North diaphragm wall in Phase 2

17. KEY FACTORS CONTRIBUTED IN SUCCESSFUL APPLICATION OF OBSERVATIONAL METHOD

Outcome of a through desk study at post-tender stage provided an effective value engineering option which offered significant cost and time saving for overall construction program. Effective and good communications between the design team and construction crew played a key role in successful application of observational method to completion of the project. Systematic monitoring program with clear defined trigger criteria was also the important element in implementing the observational method. This case study reveals that a thin permanent diaphragm wall coupled with effective design and construction method supplemented by the observational method and robust monitoring program could offer a logistically and financially attractive solution in construction of underground car park without disturbing the environment in the prominent historical area of Bangkok.

18. REFERENCES

Clough C.W. and T.D. O'Rourke, 1990. Deep Excavation and Tunneling. 7th ICSME, Mexico City
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ABSTRACT

Diaphragm wall support deep excavation works of some major projects including Bangkok first subway project are presented in this paper. Available literatures related to diaphragm wall and deep excavation works in Bangkok are summarized together with recent research works. Problems and difficulties in construction of diaphragm walls and deep excavation works in early days are reported. The areas of development in diaphragm wall construction and main factors contributed to the area are also presented. An attempt is also made to briefly discuss the future trend of diaphragm wall construction and deep excavation works in Bangkok. This paper is intended to serve as a source of reference for the practitioners in the construction industry with regard to the application of diaphragm walls in deep excavation works in urban area of Bangkok.

INTRODUCTION

As in other major cities, growing land price and need of underground space for commercial and infrastructure developments necessitated deeper underground excavation works in Bangkok. Diaphragm wall as an embedded earth-retaining structure is one of the most suitable solutions to facilitate the deep excavation works in urban area of the growing metropolis. The common thickness of diaphragm wall in Bangkok ranges from 0.80m to 1.20m. Though not very common, in some cases, thick diaphragm wall of 1.50m and thin diaphragm wall of 0.50 to 0.60m were also constructed. Toe depths of diaphragm walls are in the range of 16m to over 30m depths depending on the final elevation of the excavation. This paper aims to provide the summaries of the technical data on diaphragm wall construction and deep excavations in Bangkok with particular focus on the progressive development in past decades. Literatures published throughout past 20 years are summarized along with the recent research works. The information and data presented in this paper are based on the author's personal experience, observation from other completed projects and from published literatures.

SUBSOIL AND PIEZOMETRIC PROFILES IN BANGKOK

Subsoil profile and the present piezometric drawdown condition of Bangkok are presented in Fig. 1 below. A typical subsoil profile is relatively consistent in different localities in Bangkok. It is characterized by alternating layers of clay and sand deposits as shown in the Fig. 1.
Diaphragm Wall Support Deep-Excavations
In Bangkok Subsoil

Narong Thasnanipan¹, Zaw Zaw Aye¹, Aung Win Maung¹, Thayanan Boonyarak²
Phongpat Kitpayuck¹, Nutthapon Thasnanipan¹
¹Seafco Public Company Limited: zaw@seafco.co.th
²Hong Kong University of Science and Technology: thayanan@hotmail.com

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Diaphragm wall support deep excavation works of some major projects including Bangkok first subway project are presented in this paper. Available literatures related to diaphragm wall and deep excavation works in Bangkok are summarized together with recent research works. Problems and difficulties in construction of diaphragm walls and deep excavation works in early days are reported. The areas of development in diaphragm wall construction and main factors contributed to these are also presented. An attempt is also made to briefly discuss the future trend of diaphragm wall construction and deep excavation works in Bangkok. This paper is intended to serve as a source of reference for the practitioners in the construction industry with regard to the application of diaphragm walls in deep excavation works in urban area of Bangkok.

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Subsoil profile and the present piezometric drawdown condition of Bangkok are presented in Fig. 1 below. A typical subsoil profile is relatively consistent in different localities in Bangkok. It is characterized by alternating layers of clay and sand deposits as shown in the Fig. 1.
Fig. 1, Subsoil and piezometric profile of Bangkok

THE NEEDS OF DIAPHRAGM WALL FOR DEEP EXCAVATIONS IN BANGKOK

Generally, requirement and selection of basement retaining walls are based on the following factors.

- Depth of final excavation of basement – shallow or deep excavation
- Subsoil condition
- Construction sequence and permanent condition of basement structure
- Environmental conditions and constraints – effect of retaining wall installation and basement excavation to neighbouring properties and structures
- Constructability – availability of space and physical condition of existing site

(Peck et. al. 1977) defined that deep excavations are those whose depths are deeper than 6m. Sheet piles, contiguous-pile walls, secant pile walls and sinking caisson are alternative retaining structures available in Bangkok for deep excavations. Prevailing subsoil (existence of very soft to soft clay) and ground water condition are the main factors for using diaphragm walls for deep basement excavation in Bangkok. For most of the deep excavation works in urban environment, advantages offered by diaphragm walls weigh more favourably for both technical and economical reasons while other methods have distinct limitations. Major advantages of diaphragm wall are listed below.

- Can be used as permanent structural wall
- Water retainable
- Can be installed to deeper depths and for load bearing element
- Less temporary propping needed
- Can be applied for top-down construction method
- Rigid structure so that ground movement induced by basement excavation is less than other flexible retaining wall
- Vibration and noise generated from installation of diaphragm wall is less than other methods

Key limitations of diaphragm wall can be summarized as:

- Diaphragm wall itself is water retainable structure but not 100% water-proof. Additional measures are required if basement required high-grade water-proof wall
- Availability of limited numbers of specialist diaphragm wall contractors
- Finished wall surface is influenced by the subsoil
DIAPHRAGM WALL CONSTRUCTION METHOD

The diaphragm wall construction method allows for the formation of a reinforced concrete wall beneath the ground surface. The construction of the diaphragm wall is carried out from ground level, panel by panel forming series of cast-in-place underground RC walls. RC Guide walls of 1m to 1.5m deep are mainly used to assist the trenching operation. Each panel is excavated by using cable operated mechanical clamshells suspended from crawler cranes or by more modernized hydraulic grabs. Throughout the excavation Bentonite slurry is maintained at the top of the excavated trench for wall stability. On completion of excavation, the Bentonite slurry which may have become contaminated with soil, is cleaned by recycling through de-sanding equipment. The reinforcement cages are then lowered into the slurry filled trench, with each unit spliced to the other, to form a continuous cage to the required depth. Tremie pipes are then installed to the base of the panel and concrete is cast from the panel toe up to the required cut-off. During casting the displaced Bentonite slurry is drawn off and stored for reuse. Adjacent panels are then excavated using the same procedure. Stop-ends are used to provide the formation shear-key and continuity between adjoining panels. Fig. 2 is a summarized demonstration of process involved in diaphragm wall construction. The selection of panel width is mainly depending upon the available grab size, ground condition and geometric constraints of the basement footprint. Panel width of 3m to 6m are commonly used in Bangkok. Where practical, it is advisable to use same panel width for the ease of preparation and handling of the reinforcement cage.

![Diaphragm wall construction process](image)

Fig. 2 – Diaphragm wall construction process

OVERVIEW OF DIAPHRAGM WALL PROJECTS IN BANGKOK

The first diaphragm wall in Bangkok was believed to have been constructed in the late 1970s for the basement retaining walls of the Bangkok Bank Head Office Tower on Silom Road. Table 1 summarized the diaphragm wall projects constructed in the early days.
Table 1. List of the early diaphragm wall projects in Bangkok

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Diaphragm Wall Dimension (m)</th>
<th>Excavation Depth (m)</th>
<th>Year of construction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Thickness</td>
<td>Depth</td>
<td></td>
</tr>
<tr>
<td>Bangkok Bank HQ</td>
<td>0.50</td>
<td>14.00</td>
<td>7.00</td>
</tr>
<tr>
<td>International Trade Center</td>
<td>0.82</td>
<td>26.00</td>
<td>18.50</td>
</tr>
<tr>
<td>Srivara Hightech Tower</td>
<td>1.02</td>
<td>26.00</td>
<td>17.20</td>
</tr>
<tr>
<td>Silom Precious Tower</td>
<td>1.02</td>
<td>28.00</td>
<td>20.00</td>
</tr>
<tr>
<td>Jewelry Trade Center</td>
<td>0.82</td>
<td>28.00</td>
<td>15.60</td>
</tr>
<tr>
<td>Oriental Tower</td>
<td>0.82</td>
<td>28.00</td>
<td>15.50</td>
</tr>
<tr>
<td>Sukhumvit 33 Tower</td>
<td>0.82</td>
<td>26.00</td>
<td>14.00</td>
</tr>
<tr>
<td>Thammasat University Library Building</td>
<td>0.80</td>
<td>28.00</td>
<td>12.70</td>
</tr>
</tbody>
</table>

Fig. 3 illustrates the embedded length of diaphragm wall in relation to the excavation depth of the projects listed in Table 1. It is to be noted that except shallow excavation required for Bangkok Bank Head Office, diaphragm wall embedded deeper into hard clay layer in all projects constructed in the early days. Fig. 4 shows the embedded length of diaphragm wall in relation to the excavation depth of the projects listed in Table 2.

From 1991, with the booming construction industry, usage of diaphragm walls for deep basements of the high-rise buildings in Bangkok has been significantly increased. In year 1997, due to the economic crisis, with significant decline of property sector, diaphragm wall construction was concentrated only for infrastructure projects.

Fig. 3, Diagram showing the excavation depths in relation to embedded length of diaphragm walls in early days (1977 to 1994)
Table 1, List of the early diaphragm wall projects in Bangkok

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Diaphragm Wall Dimension (m)</th>
<th>Excavation Depth (m)</th>
<th>Year of Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bangkok Bank HQ</td>
<td>0.50</td>
<td>14.00</td>
<td>1977</td>
</tr>
<tr>
<td>International Trade Center</td>
<td>0.82</td>
<td>26.00</td>
<td>1985</td>
</tr>
<tr>
<td>Srivara Hightech Tower</td>
<td>1.02</td>
<td>26.00</td>
<td>1991</td>
</tr>
<tr>
<td>Silom Precious Tower</td>
<td>1.02</td>
<td>28.00</td>
<td>1992</td>
</tr>
<tr>
<td>Jewelry Trade Center</td>
<td>0.82</td>
<td>28.00</td>
<td>1992</td>
</tr>
<tr>
<td>Oriental Tower</td>
<td>0.82</td>
<td>28.00</td>
<td>1992</td>
</tr>
<tr>
<td>Sukhumvit 33 Tower</td>
<td>0.82</td>
<td>26.00</td>
<td>1992</td>
</tr>
</tbody>
</table>

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Table 2, List of some diaphragm wall projects constructed from 1997 to 2008 in Bangkok (Basement Excavations Deeper than 10m)

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Diaphragm Wall Dimension (m)</th>
<th>Excavation Depth (m)</th>
<th>Year of Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Charoen Pump Station</td>
<td>0.80</td>
<td>20.00</td>
<td>1997</td>
</tr>
<tr>
<td>Sathorn Pump Station</td>
<td>0.80</td>
<td>20.00</td>
<td>1997</td>
</tr>
<tr>
<td>MWA GMC-7A</td>
<td>0.80</td>
<td>21.00</td>
<td>1997</td>
</tr>
<tr>
<td>Bank of Thailand</td>
<td>0.80</td>
<td>20.00</td>
<td>2002</td>
</tr>
<tr>
<td>Hampton Condominium</td>
<td>0.80</td>
<td>20.00</td>
<td>2003</td>
</tr>
<tr>
<td>Central World Plaza</td>
<td>1.00</td>
<td>18.00</td>
<td>2004</td>
</tr>
<tr>
<td>Boe Bae Bumrung Meung Plaza</td>
<td>0.80</td>
<td>24.00</td>
<td>2004</td>
</tr>
<tr>
<td>Esplanade</td>
<td>0.80</td>
<td>18.00</td>
<td>2005</td>
</tr>
<tr>
<td>Por Teck Tung Office</td>
<td>0.80</td>
<td>16.00</td>
<td>2006</td>
</tr>
<tr>
<td>Noble Remix</td>
<td>0.80</td>
<td>20.00</td>
<td>2007</td>
</tr>
<tr>
<td>Life@Sathorn</td>
<td>0.80</td>
<td>18.50</td>
<td>2008</td>
</tr>
<tr>
<td>Royal Maneeya</td>
<td>1.00</td>
<td>30.00</td>
<td>2005</td>
</tr>
<tr>
<td>Sathorn Square</td>
<td>0.80</td>
<td>21.00</td>
<td>2007</td>
</tr>
<tr>
<td>Renaissance</td>
<td></td>
<td></td>
<td>2008</td>
</tr>
<tr>
<td>The Sukhothai Residences</td>
<td>1.00-1.20</td>
<td>20.00-22.0</td>
<td>2008</td>
</tr>
<tr>
<td>Bangsue Wastewater Treatment</td>
<td>1.20-1.50</td>
<td>24.70</td>
<td>2009</td>
</tr>
<tr>
<td>Grand Rama 9 Square</td>
<td>1.00-1.50</td>
<td>22-25.00</td>
<td>2010</td>
</tr>
<tr>
<td>MahaNakhon Hill &amp; Tower</td>
<td>0.6-0.80</td>
<td>16.00-18.0</td>
<td>2011</td>
</tr>
<tr>
<td>MahaNakhon Retail Cube</td>
<td>0.80</td>
<td>22.0</td>
<td>2011</td>
</tr>
<tr>
<td>UBC III &amp; EM2</td>
<td>0.80-1.00</td>
<td>18.0-22.0</td>
<td>2011</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Extensive application of diaphragm wall for deep excavations can be observed between 1997 and 2001 for the first Bangkok subway project where 18 deep stations and associated structure plus some sections of cut-and-cover tunnels were constructed by diaphragm wall. (Phienwej et. al., 2006) discussed the difference between the length of diaphragm wall to depth of excavation between two contracts of the first Bangkok subway, Chaloemratchamongkhon Line. The authors cited that it is partly due to the difference in design criteria adopted. Fig. 5 illustrates the embedded length of diaphragm wall in relation to the excavation depth of MRT Stations. Excavation depth over 30m carried out in Silom Station set the deepest excavation ever done in Bangkok subsoil.

DIAPHRAGM WALL CONSTRUCTION PROBLEMS IN THE EARLY DAYS

Basic but extensive problems were experienced in early stages of diaphragm wall construction as summarized below.

- Lack of experienced engineers and foremen
- Limited availability and capacity of equipment

Table 3, Summary of development in diaphragm wall construction in Bangkok

<table>
<thead>
<tr>
<th>Area of development/improvement</th>
<th>Main factor contributed to development</th>
</tr>
</thead>
<tbody>
<tr>
<td>Speed of construction</td>
<td>Better equipment, operating skills as well as improved-knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Size and depth of diaphragm wall</td>
<td>Better equipment, operating skills as well as improved-knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Slurry management</td>
<td>Experience from past projects and research studies</td>
</tr>
<tr>
<td>Construction impact on quality and performance</td>
<td>Experience from past projects and research studies</td>
</tr>
<tr>
<td>Quality control in construction process</td>
<td>Experience from past projects and research studies</td>
</tr>
<tr>
<td>Quality control test method and interpretation</td>
<td>Advance equipment, experience from past projects and research studies</td>
</tr>
</tbody>
</table>

In some projects where headroom is limited, special diaphragm wall grabs and equipment were employed. Fig. 6 shows the specially-designed diaphragm wall equipment used in the project with limited low headroom.

Fig. 6, Specially designed baby-grab with short-boom for limited low-headroom area

Fig. 5- Diagram showing the excavation depths in relation to embedded length of diaphragm walls of 18 subway stations of the first Bangkok Mass Rapid Transit Project
• Lack of skills in operation of equipment
• Limited knowledge and less advanced techniques in the control of bentonite slurry
• Limited knowledge and experience in construction method and related negative impact
• Quality of concrete used for tremie concreting
• Improper construction and quality control specifications and guidelines for local soil
• Improper design for constructability
• Limited availability of performance monitoring equipment (e.g. efficient and reliable inclinometer and other instruments)

DEVELOPMENT IN DIAPHRAGM WALL CONSTRUCTION

Over the past three decades, along with the development of diaphragm wall construction technology in other parts of the world, equipment, construction technique and design methods as well as better understanding of construction impact on the performance of embedded retaining wall have significantly improved in Thailand. Table 3 summarizes the areas of improvement in diaphragm wall construction and main factors contributed to these developments.

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<tr>
<td>Size and depth of diaphragm wall</td>
<td>Better equipment, operating skills as well as improved-knowledge in construction method, management and local soil condition</td>
</tr>
<tr>
<td>Slurry management</td>
<td>Experience from past projects and research studies</td>
</tr>
<tr>
<td>Construction impact on quality and performance</td>
<td>Experience from past projects and research studies</td>
</tr>
<tr>
<td>Quality control in construction process</td>
<td>Experience from past projects and research studies</td>
</tr>
<tr>
<td>Quality control test method and interpretation</td>
<td>Advance equipment, experience from past projects and research studies</td>
</tr>
</tbody>
</table>

In some projects where headroom is limited, special diaphragm wall grabs and equipment were employed. Fig. 6 shows the specially-designed diaphragm wall equipment used in the project with limited low headroom.
Diaphragm wall is an ideal solution for deep circular shafts. Though alternative method such as sinking caisson may offer cheaper direct cost, significant time saving and maximum safety can be achieved by using diaphragm wall in construction of deep shafts. View of diaphragm wall support deep shaft is presented in Fig. 7.

Fig.7. Deep circular shaft constructed by diaphragm wall

Diaphragm walls were only solution for deep excavations required for subway stations of the first Bangkok MRT. (Thasnanipan et. al., 2000) presented the construction of diaphragm wall for one of the largest subways stations of the MRT. Construction parameters and technical details of the diaphragm wall were reported by the authors.

DEVELOPMENT IN DEEP EXCAVATION SUPPORTED BY DIAPHRAGM WALLS

In Bangkok, most of the diaphragm wall support deep excavations are carried out by multi-propping or of braced-excavation. Depending on the excavation depths and sequence, bracing span-lengths commonly ranges from 4m to 8m. Improvements in diaphragm wall support deep excavation works are obvious and significant in past decade. Developments in the following areas are notable.

• Application of top-down construction method in diaphragm wall support deep excavation works
• More complex excavation works to deeper depths
• More understanding on excavation induced ground movement and risk of damages to adjacent structures
• Application of ground improvement method to minimize the ground movement
• Application of observational method integrated with value engineering option and comprehensive instrumentation program
• More understanding on water-retainable capacity of diaphragm wall

Application of top-down construction method in urban area of Bangkok was presented by (Thasnanipan et. al., 2006). Along with technical aspects of the top-down application for 19.10m deep excavation works, the authors highlighted the cost saving gain from shorter construction period and minimum usage of material for temporary bracing works. View of excavation process with permanent slabs having braced the excavation can be seen in Fig. 8.

Fig.8. Application of top-down method for deep excavation work in Bangkok (Thasnanipan et. al., 2006)

All 18 deep underground stations of the first Bangkok MRT were constructed between 1997 and 2000 by top-down method using diaphragm walls as permanent structural support. Fig. 9 shows the view of subway station construction with application of top-down method. Skeleton slabs provided large openings for which offered logistically significant advantage in deep excavation works.

Fig.9. View of subway station construction with application of top-down method

Complex and challenging deep excavation works at Silom MRT Station was reported by (Hall et. al. 2001). The authors demonstrated the complicated construction sequence involved in the deepest excavation works in Bangkok subsoil supported by NS-Box Diaphragm Wall. The process involved in underpinning of existing flyover was also described by the authors.

Ground movement prediction and building risk damage assessment for the deep excavation works can be found in the works of (Aye et. al., 2006). With demonstration of the procedure used in staged assessment on risk of damage by excavation induced ground movement, the authors proposed a simple method
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to predict both vertical and horizontal surface and subsurface ground movement based on the deflection profile of diaphragm wall. Fig. 10 shows the proposed method for prediction of subsurface ground movement.

\[
D_0 = 2.5 H_e
\]

\[
S_{WO} = \frac{4V_O}{D_0}
\]

\[
S_{io} = S_{WO} \left( \frac{x}{D_0} \right)^2
\]

\[
D_y = y \frac{D_0}{H_w}
\]

\[
V_{Ty} = V_y \frac{V_{Io}}{V_O}
\]

Fig. 10. Demonstration of subsurface settlement prediction from diaphragm wall deflection values proposed by (Zaw Zaw Aye et. al., 2006)

(Thasnanipan et. al. 2006) reported an unprecedented case on use of soil-cement columns as ground improvement to minimize ground movement induced by diaphragm wall support deep excavation adjacent to existing Bangkok MRT tunnels. The authors presented the technical requirement of the deep excavation work in the protection zone of MRT tunnels along with actual monitoring results of similar projects in other parts of the world. This integration of ground improvement application to diaphragm wall deep excavation works is expected to be more popular in the future as need of basement facilities are increasing along the existing MRT tunnels. The performance of diaphragm wall and impact on tunnels meeting all technical criteria required imposed by the Metropolitan Rapid Transit Authority of Thailand (MRTA) was subsequently reported by (Teparaksa et. al., 2006)

Fig. 11. Typical section of diaphragm wall support deep excavation adjacent to MRT tunnel showing protection zone, (Thasnanipan et. al., 2006)

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Application of observational method integrated with value engineering and risk management for deep excavation works was demonstrated by (Aye et al., 2006). The authors concluded that well organized plan with systematic approach of observational method backup by extensive instrumentation and close cooperation among the involved parties were the key factors contributed to successful completion of the reported projects.

(Thasnanipan et al., 2008) presented the comprehensive study of water-resisting deep basement in Bangkok soil. The authors highlighted the preventive measures for water leakage commonly occurred in basement with reference to BS8102. The authors also recommended the designers to consider additional measures to prevent water leakage and dampness for diaphragm wall support excavation as diaphragm wall itself is not fully water-proof.

(Pongrujikorn, 2006) reported the development of underpass construction in urban area of Thailand with emphasis on the projects in Bangkok. The author stated that diaphragm wall has been proved to be a reliable option for most cases in underpass construction.

IMPROVEMENT IN DESIGN PARAMETERS

In the initial stage of introducing diaphragm wall in Thailand, the design concepts and parameters were mainly based on the available literature from research carried out in other parts of the world. With the passage of time, design method and selection of parameters for local subsoil were improved as a result of research works carried out in 1990s. Differences between the behaviour of flexible pile walls and diaphragm walls were well realized from these studies. Development in powerful computer facilities and commercially available FEM programs are key factors contributed to the advancement in analysis of diaphragm wall support deep excavations.

Detailed advance FEM analyses of diaphragm wall support deep excavations in Bangkok subsoil can be found in the work of (Balasubramaniam et al., 1992). The authors demonstrated the FEM simulation for both top-down and conventional braced excavation with temporary support. The authors concluded that behavior of supported excavation in soft clay is mainly controlled by the cantilever mode of deformations and it is 80% of the maximum deflection of diaphragm wall.
With the peak of construction-boom in Thailand, particularly in Bangkok, large numbers of diaphragm wall support deep excavations were systematically monitored by geotechnical instrumentation throughout 1990’s which provided better understanding on behavior of this type of embedded retaining wall. Researches focused on the design parameters and methods were published based on these data. With improved construction and design methods, diaphragm wall came to be regarded as reliable excavation support retaining structure in the construction industry of Thailand. With more confidence on soil parameters selection and better understanding on behaviour of diaphragm wall, significant cost savings were achieved by more efficient design. This can be seen by shorter embedded depths of diaphragm walls in recent projects than those in early days as shown in Fig. 4 in comparison with Fig. 3.

(Teparaksa et. al., 1999), back-analysed the soil stiffness parameters of the soft and stiff Bangkok clay for diaphragm wall support excavation based on three cases studies. The authors proposed the soil stiffness values for soft and stiff clay in terms of Young modulus in relation to undrained shear strength which provided useful reference for the design engineers involved in diaphragm wall and deep excavation works.

IMPROVEMENT IN APPLICATION OF INSTRUMENTATION

In early days, use of instrumentation in diaphragm wall support excavations was limited. Increasing needs of deep excavation works adjacent to sensitive buildings and structures called for more comprehensive instrumentation and monitoring program.

(Thasnanipan et. al., 1998) reported the use of inclinometer, tiltmeter, vertical beam sensor, vibrating wire strain gauge, settlement plate and earth pressure gauge in monitoring of diaphragm wall support deep excavations and adjacent sensitive buildings at the bank of Chao Phraya River. The important role of instrumentation for deep excavation in presence of unbalance loading condition was emphasized by the authors.

Ignorance of instrumentation data resulted in catastrophe collapse of deep excavation for Inlet Pumping Station was discussed by (Teparaksa et. al., 1999). The authors presented the findings of investigation into the causes of failure showing excessive movement recorded by inclinometer prior to the collapse.

Extensive application of geotechnical instrumentation was more evident in construction of 18 deep subway stations for the first Bangkok MRT between 1997 and 2001. (Aye et. al., 2006) summarized the instrumentation program used in monitoring of ground movement induced by deep station excavations. The authors discussed the systematic approach of damage risk assessment of adjacent existing buildings and structures with application of extensive instrumentation.

(Thasnanipan et. al., 2004) reported the effective application of inclinometer monitoring data in construction of underground car park in the historical area of Bangkok. The authors described that thin diaphragm wall with buttress support (Fig. 13) applied in this project was the first of its kind in Bangkok as described by the authors.

![Fig.13. View of thin diaphragm wall supported by buttress (Thasnanipan et. al., 2004)](image-url)
The authors presented a good data set of diaphragm wall movement profile showing time-dependent progressive deflection pattern due to softening and deformation of soft clay caused by delay in installation of raker support as shown in Fig. 14. The authors demonstrated by comparing actual monitoring results with preset trigger levels contributed to the understanding of the condition of the excavation as work progressed. This approach served as the basis to judge whether contingency actions are to be implemented. To ensure the safety of the excavation, the contingency measures exercised upon diaphragm wall deflection reaching preset trigger levels were reported by the authors.

The actual monitoring data of different type of instrumentation presented in above literatures served as valuable information and technical references for all parties involved in diaphragm wall and deep excavations industry.

![Cumulative Displacement vs Depth](image)

Fig.14. Time dependent wall deflection of diaphragm wall – diaphragm wall was supported only by soil-berm for 52 days before completion of raker installation (Thasnanipan et. al., 2004)

**OBSERVATION OF GROUNDWATER CONDITION IN RECENT PROJECTS**

Recently, deep excavations deeper than 18.0m in depth at two projects for basement constructions being 5.25km apart in Bangkok subsoil encountered presence of groundwater after reaching about 17.5m. Subsoil information obtained by soil boring in these projects indicated a series of stiff silty and sandy clay between 15m and 24m, overlying a dense sand layer. Construction progress was significantly affected by presence of groundwater in silty clay. Fully saturated silty clay turned liquid when disturbed by excavation activities, causing difficulties in casting a concrete blinding layer for foundations construction. Dewatering and ground improvement to the subgrade layer below foundation mat and footings needed to be carried out. Bearing capacity of disturbed silty clay with high water content is very low so that cobble size concrete blocks and crushed rocks to be compacted in to the soil for improving the subgrade. Then a lean concrete blinding layer was cast on the subgrade part by part while dewatering.

A general piezometric profile in Bangkok shown in Figure 1 indicates a drawdown at -20m. Additional peizometric readings from the piezometers installed at depths in the range of 28.0m to 33.0m and at 40m from other sites in Bangkok indicate that piezometric levels are around -15.0m and -23.0m respectively. Piezometers installed at depths 28.0m to 33.0m shows piezometric levels increase to -12.0m within six months. However, the groundwater encountered in the above mentioned sites was considered to be perched water in semi permeable soils layer being vertically cut off by diaphragm walls, with an assumption that the groundwater level was reduced gradually by pumping out.
Fig.15. Groundwater encountered in basement excavations deeper than -17.50m.

CONCLUSIONS

According to the authors’ experience as deep-foundation contractors and observation as researchers, development in both construction and design aspects of diaphragm wall and deep excavations in Bangkok in past decades were significant. With recognition of technical and economical advantages of using this type of embedded retaining walls by local practitioners, it is expected that they will be more popular in the future construction industry of Thailand. However, in the authors’ opinion, there is much work to be done with particular focus on constructability issues, concrete technology for diaphragm walls, reliable but cost-effective quality control testing and value-engineering. Starting from the planning stage, site investigation, design, construction and inspection should be integrated so that designers, contractors and construction inspectors can participate as a team with a common goal. Appropriate and practical specifications should be established jointly by these parties for local soil conditions and construction methods. As a commercial and political center of the country with a rapidly growing population, Bangkok will continue to witness more underground construction for its infrastructure development in which diaphragm wall support excavation will clearly play a great role.

REFERENCES


Underpass Construction in Congestive Area of Bangkok

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Diaphragm wall has been used as permanent retaining wall for a vehicle underpass at major intersection in urban area of Bangkok besides deep basement structures of high-rise buildings, subway stations, deep shafts and other forms of underground structures. This paper presents the key technical features of recently completed vehicle underpass along one of the busiest roads in Bangkok. The design aspects of earth-retaining structure integrated with top-down construction method are reported. Predicted and measured deflections of diaphragm wall at different stages of construction are presented. Practical construction method adopted under the existing flyover is highlighted. Difficulties encountered during the construction are briefly described. This project marked the first application of post-tension roof slabs along with top-down construction technique for vehicle underpass construction by cut-and-cover method in Thailand.

1. Introduction

To combat appalling traffic congestion, Bangkok Metropolitan Authority has been implementing various measures in both urban and sub-urban areas. Flyovers are frequently used to improve the traffic flow along the road at many major intersections. Though underpass is not commonly used, it is the only solution in some critical traffic jam areas. The first underpass in the form of depressed-road for rail-road junction was believed to be constructed about 35 years ago at Bangsue, northern outskirt of Bangkok. Several years later, vehicle underpass was constructed using cast-in-place diaphragm wall by Bangkok Metropolitan Authority (BMA) at Din Daeng intersection in 1994. Since then more than 20 underpasses have been constructed by BMA and other agencies in Bangkok and other major cities in the country.

This paper presents the design and construction of recently completed vehicle underpass along one of the busiest roads in Bangkok. The presenting project in this paper demonstrated the effective use of underground space in congestive urban areas of Bangkok and exemplified good solutions for environmental preservation.

2. Project Requirement and Major Constraints

The project works involved construction of a vehicle-underpass and associated utility diversion and road expansion at one of the major intersections in the eastern part of Bangkok. Figure 1 shows the alignment and location of the underpass overlaid on aerial photo of the site (modified from GoogleEarth). Four-lane underpass was to construct in the form of 758 m long cut-and-cover tunnel along six-lanes existing Srinakarin Road. As can be seen in the figure, at the intersection, tunnel section of underpass was to construct under existing flyover of Udomsuk Road.
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As the main six-lanes Srinakarin Road is one of the major roads linking eastern parts and inner Bangkok, traffic density along this road is high. Presence of two major shopping malls in the close proximity to the intersection is also another factor contributed to congestive traffic condition in the area.

Location of the project in the heavy traffic area and need of construction under the existing flyover with limited headroom therefore posed major constraints, which called for the need of careful consideration in establishing the design principles and sequence of construction. As the construction is to be carried out along the major public road, ground movement induced by cut-and-cover tunnel excavation must be minimized.

![Figure 1](alignment-and-location-of-vehicle-underpass.jpg)

**Figure 1** Alignment and location of vehicle underpass (aerial photograph modified from GoogleEarth)

3. **Sub-soil Condition**

Similar to other localities in Bangkok a typical subsoil profile at the site is characterized by the alternating layers of clay and sand deposits as shown in Figure 2. Undrained shear strength \( S_u \) and SPT N-value obtained from 5 SI boreholes were plotted and design line was derived as depicted in Figure 2. Typical subsoil profile at the site is characterized by thick Bangkok soft clay layer at the top followed by thin layer of medium clay underlying by alternative of stiff clay and dense sand layers. Undrained shear strength of soft clay is obtained by using unconfined compressive strength test from Shelby tube sample. In stiff to hard clay layer, correlation of undrained shear strength with SPT N-value is adopted where undrained shear strength is equals to 6.85N (kPa) for high plasticity clay (Sambhandaraksa & Pitupakorn, 1985). Angle of internal friction \( \phi' \) of dense sand is estimated to be 32˚ by using SPT correlation (Peck et al., 1974). The design soil parameters are tabulated in Table 1.
Figure 2 Soil profile, undrained shear strength and SPT N-value

Table 1 Design soil parameters

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Depth (m)</th>
<th>$\gamma_t$ (kN/m$^3$)</th>
<th>$S_u$ (kPa)</th>
<th>$\phi'$ (degree)</th>
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<tbody>
<tr>
<td>Fill</td>
<td>0-2</td>
<td>18.5</td>
<td>25</td>
<td>0</td>
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<td>Soft clay 1</td>
<td>2-10</td>
<td>16.5</td>
<td>17</td>
<td>0</td>
</tr>
<tr>
<td>Soft clay 2</td>
<td>10-14</td>
<td>17.5</td>
<td>22</td>
<td>0</td>
</tr>
<tr>
<td>Medium stiff clay</td>
<td>14-18</td>
<td>19.0</td>
<td>40</td>
<td>0</td>
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<tr>
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<td>18-20</td>
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<td>0</td>
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<tr>
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<td>20.0</td>
<td>180</td>
<td>0</td>
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<td>27-32</td>
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<tr>
<td>Hard clay</td>
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<td>180</td>
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<td>Very dense sand</td>
<td>40-45</td>
<td>20.0</td>
<td>0</td>
<td>32</td>
</tr>
</tbody>
</table>
4. Review of Tender Stage Design and Modification

Review of the practical construction method was undertaken by the contractor’s in-house design team. Rigorous attention to detail of the design concept and constructability was made in the pre-construction discussions among the client, consultants, design engineers and construction team. Original tender called for the conventional construction method – to complete the excavation to final excavation depth and construct the roof-slab in the tunnel section of the underpass by using precast pre-stressed concrete panels. Installation of 2.5m x 18.8m heavy precast panels underneath the existing flyover was considered difficult and high risk involved. After reviewing different options with consideration of the constraints imposed on site and constructability, top-down construction with post-tension slabs as roof-slab of the tunnel section was selected.

4.1 Numerical analysis for diaphragm wall design

Figure 3 illustrates an example of finite element mesh of deep approach with sump pit. In other zone, basic geometry of the mesh follows that of excavation depth, wall depth and construction sequence of each zone. Numerical analyses are carried out by using PLAXIS V7.2 (Brinkgreve and Vermeer, 1998) finite element program. Plane strain total stress analyses using elasto-plastic with Mohr-Coulomb failure criterion constitutive model are adopted. Undrained shear strength of clay (Su) and internal friction angle of sand (φ’) are selected according to subsoil properties as previously discussed. Young’s modulus ratio (E_u/S_u) of each clay ranges from 250 to 750 which is estimated based on expected shear strain of diaphragm wall. Boundary condition consists of roller support applied at both vertical sides and pin support applied at the bottom of the mesh. Numerical procedure follows the construction sequence in the field conditions. The primary objective of numerical analysis is to verify structural capacity and overall stability during construction. It should be noted that time dependent behavior of clay is not simulated.

Figure 3 Finite element mesh of deep approach with sump pit
5. Construction of Diaphragm Wall, Barrettes and Bored Piles

Figure 4 shows the diaphragm walls, bored piles and barrettes layout plan. Diaphragm wall was designed as earth-retaining wall for the entire section of the underpass. Diaphragm wall having 800mm thick founded at 25m below ground level (B.G.L) was constructed in three phases. Both Phase 1 (north section) and Phase 2 (south section) were constructed in two stages (different working stages for east and west diaphragm walls) to meet the traffic diversion requirement. North section of the underpass diaphragm wall was constructed in Phase 1 in the initial stage of the project. In Phase 2, south section of diaphragm wall was constructed prior to construction of cross-wall for a deep sump-pit - diaphragm wall with 6m deep cut-off-level perpendicular to longitudinal alignment of the main underpass diaphragm wall.

As the tunnel section of the underpass at the intersection was to be constructed under limited headroom due to the presence of existing flyover, careful planning was needed for Phase 3 diaphragm wall construction. Main factors were the limited time available for traffic diversion at the intersection and the need of special equipment to practically construct the diaphragm wall under the flyover where headroom was only 6m. Specially-designed mechanical grab was used to construct the diaphragm wall in this low-headroom section as shown in Figure 5. Another major challenge involved in construction of diaphragm wall under the existing flyover was the obstruction of existing water supply pipe at 8m below ground level along the diaphragm wall alignment. Removing the pipe buried 8m below ground level by open-cut or sheet-pile supported excavation option was considered time consuming, unpractical as well as risky, taking the consequences involved into account (unpractical and risky to have 8m open-cut excavation in soft clay, unpractical to install 16m deep sheet pile low headroom. After reviewing different options to remove the obstructed water supply pipe, decision was made to remove the pipe by sending a man inside the pipe - to cut the obstructed part in smaller pieces and close the pipe to stop the flow of bentonite into the pipe during diaphragm wall construction. Risk assessment and safety measures with contingency plan were made before actual pipe removal was done. The obstructed section of the buried pipe was successfully removed.

Figure 4 Diaphragm wall, bored pile and barrette layout plan of vehicle underpass
Figure 5 Special-design diaphragm wall grab working under existing flyover

Figure 6 demonstrates the 3D perspective of stanchions embedded in bored pile and barrettes. Total 43 bored piles of 1000mm diameter founded at depth 29m below ground level were designed to support roof-slab of tunnel section. Bored piles were constructed by wet-processes method using polymer-based slurry. Steel stanchions were installed in bored piles to facilitate top-down construction. As it was not practical to utilize bored pile drilling machine underneath the flyover, within the low-headroom section of the underpass, three barrettes were installed to replace bored piles.

Figure 7 shows the plan of cut-and-cover tunnel constructed by using top-down method integrated with post-tension slab. Total length of tunnel zone is approximately 180 m along the longitudinal profile of underpass or about 25% of overall underpass length.

As discussed in the previous section, top-down construction requires modifications from the original design. Instead of using temporary prop to support the diaphragm wall during excavation of tunnel section, roof-slab was used as temporary support. Four openings were provided at an evenly spacing of about 40 m for access of soil excavation underneath the roof-slab.

Construction sequences of tunnel zone are shown in Figure 8. Major construction activities in each stage are given as follows.

Stage 1:
- Construct diaphragm wall and bored pile with embedded temporary stanchion

Stage 2:  
- Construct upper part of median wall (spine-beam) on the top of stanchion
- Construct roof slab and apply post-tension force
- Excavate to formation level

Stage 3:  
- Construct base slab
- Construct median wall between temporary stanchions
- Remove temporary stanchion and construct the remaining median wall

Figure 6 Three-Dimensional perspective of stanchions embedded in bored pile and barrettes
6. **Application of top-down method integrated with post-tension slab**

Figure 7 shows the plan of cut-and-cover tunnel constructed by using top-down method integrated with post-tension slab. Total length of tunnel zone is approximately 180 m along the longitudinal profile of underpass or about 25% of overall underpass length.

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![Plan view of tunnel zone showing post-tension roof slab constructed prior to excavation and openings for excavation works](image)

**Figure 7** Plan view of tunnel zone showing post-tension roof slab constructed prior to excavation and openings for excavation works

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- **Stage 1**: Construct diaphragm wall and bored pile with embedded temporary stanchion
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  - Construct roof slab and apply post-tension force
  - Excavate to formation level
- **Stage 3**: Construct base slab
  - Construct median wall between temporary stanchions
  - Remove temporary stanchion and construct the remaining median wall
Figure 8 Construction sequences in tunnel zone

6.1 Comparison of roof slab between original tender design and alternative design
Comparison between tender design and alternative design of roof slab are shown in Figure 9. Figure 9a shows precast prestress slab in the tender design. The width of each precast slab is 2.5 m. A gap of 0.2 m between each precast slab is filled with concrete after placing of precast slab. Figure 9b shows post-tension slab used as alternative in this project. Each construction joint is located at a distance of about 12.5 m. By using alternative design, numbers of joints were significantly reduced. A major benefit of reduction of number of joint is possibility of water leakage from roof slab through joint is dramatically reduced. In addition, using the roof slab as permanent prop provide significant larger axial stiffness compared with when temporary steel prop is used. The evidence of increasing axial stiffness on performance of diaphragm wall is discussed later in the following section.

Figure 9 (a) Precast prestress slab; original design (b) post-tension slab; actual application (c) typical tendon profile (d) tendon and grouting hose

Figure 10a and 10b shows a typical construction of post-tension roof slab.

6.2 Excavation underneath the roof slab
Figure 11a and 11b show slab opening and temporary prop in tunnel zone. Spacing between each opening was determined by optimizing efficiency of excavation and providing space to facilitate the construction. Excavation was carried by using a small excavator inside the tunnel (Fig. 11a) or using a long reach boom excavator on the roof slab (Fig. 11b). Presence of roof slab provides additional space and increase efficiency of excavation works in comparison with the original design where temporary working platform is to be used.
Before starting excavation underneath the roof slab, tension is applied to tendon embedded inside the roof slab (Fig. 9c). Subsequently, grout is injected via grouting hose. Figure 9d shows a typical tendon profile which is similar in both original and alternative design. A profile of tendon was designed to resist loading condition of a beam with end and middle supports. In order to apply roof slab as a permanent prop for excavation, rigorous structural capacity checking for combined bending moment and axial force in the slab is required. Figure 10a and 10b shows a typical construction of post-tension roof slab.

![Tendon installation for roof-slab](image1)
![Grouting after post-tensioning the slab](image2)

**Figure 10** (a) Tendon installation for roof-slab (b) grouting after post-tensioning the slab

### 6.2 Excavation underneath the roof slab

Figure 11a and 11b show slab opening and temporary prop in tunnel zone. Spacing between each opening was determined by optimizing efficiency of excavation and providing space to facilitate the construction. Excavation was carried by using a small excavator inside the tunnel (Fig. 11a) or using a long reach boom excavator on the roof slab (Fig. 11b). Presence of roof slab provides additional space and increase efficiency of excavation works in comparison with the original design where temporary working platform is to be used.

![Slab opening with temporary support](image3)

**Figure 11** Slab opening with temporary support (a) using a small excavator inside the tunnel; (b) using a long reach boom excavator on the roof slab
6.3 Temporary support of the roof slab

Figure 12a shows temporary support of roof slab. Load from the roof slab is transferred to the upper part of the median wall which works as a spine-beam. Additional reinforcement in the upper part of median wall was required during construction stage. The upper part of the wall was supported by stanchions made of steel H 350x350-137 kg/m embedded in φ1.0 m diameter bored pile. The remaining part of median wall was constructed after completion of base slab (Fig. 12b). After completion of median wall, temporary stanchions were removed since vertical load from roof slab was designed to be transferred through the median wall.

![Stanchion](Image)

**Figure 12** (a) Temporary stanchions supporting post-tension roof slab (b) median wall and stanchion

7. Performance of diaphragm wall

Figure 13 shows location of instrumentation in this project. Instrumentation consists of inclinometer, surface settlement point, strain gauge and piezometer. Instrumentation in diaphragm is installed in three zones which are tunnel, deep approach and deep approach with sump pit. Each zone of monitoring is selected based on its construction sequences. Table 2 summarizes construction sequence in each zone according to the location of instrumentation which is T1, T2, D1, D2 and DS1. In this paper, only some of measured results are reported for conciseness of paper.

![Diagram](Image)

**Figure 13** location of instrumentation
Table 2 Summary of construction sequence in each zone

<table>
<thead>
<tr>
<th>Tunnel (T1 &amp; T2)</th>
<th>Deep approach (D1 &amp; D2)</th>
<th>Deep approach with sump pit (DS1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construct roof slab</td>
<td>Excavate to -1.5 m</td>
<td>Excavate to -1.5 m</td>
</tr>
<tr>
<td>Excavate to final depth (-6 m)</td>
<td>Install temporary bracing at -1.0 m</td>
<td>Install temporary bracing at -1.0 m</td>
</tr>
<tr>
<td>Construct base slab</td>
<td>Excavate to final depth (-6 m)</td>
<td>Excavate to -5.5 m</td>
</tr>
<tr>
<td></td>
<td>Construct base slab</td>
<td>Install temporary bracing at -5 m</td>
</tr>
<tr>
<td></td>
<td>Remove temporary bracing</td>
<td>Excavate to final depth (-9.5 m)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Construct sump pit slab</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Construct base slab</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Remove temporary bracing</td>
</tr>
</tbody>
</table>

7.2 Lateral movement of diaphragm wall

Lateral movement of diaphragm wall in three zones are shown in Figure 14. Measured results from three major construction stages which are excavation to final depth, base slab construction and four months after completion of base slab are shown. Computed results at the final stage of construction in each zone are compared with the measured results. Maximum lateral displacement of diaphragm wall is 24, 41 and 30 mm in tunnel, deep approach and deep approach with sump pit zone, respectively.

By comparing diaphragm wall performance in two different construction sequences with the same diaphragm wall geometry and excavation depth, it is apparent that lateral movement in bottom-up construction (Fig. 14b) is about 2 times larger than that in top-down construction (Fig. 14a). Not only magnitude of wall movement is larger in bottom-up excavation compared with that of top-down, but also the rate of movement. Rate of wall movement after final excavation depth reached in deep approach is about one order of magnitude larger than that in tunnel. The major reason of this observation is considered to be contributed by higher stiffness of prop at the top of diaphragm wall. Prop stiffness in tunnel section which is the post-tension roof slab is about 16 times higher than that of temporary steel prop used in deep approach section. A major difference in wall deflection was observed at the top of the wall. In tunnel section, only a small deflection was observed at the top of the diaphragm wall and maximum wall deflection was observed at 9 m below ground surface. In deep approach section, movement at the top of the wall was about 24 mm due to shortening of the bracing. Subsequently, earth pressure is transferred to the formation level (6 m below ground surface) causing maximum movement at depth 7 m.

In deep approach with sump pit (Fig. 14c), although it was constructed by using bottom-up method and excavation depth (9.5 m) is deeper than that of deep approach - maximum wall movement is about 73% of that in deep approach. Two major reasons are identified to explain this observation. First, there are two levels of temporary steel prop in deep approach with sump pit compared resulting in 25% smaller average prop spacing (H/number of prop) compared with that in deep approach. Second, excavation length of sump pit is only 18 m in longitudinal direction of the wall which is almost the same as excavation width (17 m). Thus geometry
effects may contribute to reducing of lateral wall movement compared with plane strain condition in deep approach.

Computed result overestimates wall movement in tunnel zone while it underestimates that in deep approach zone. The differences are expected to be due to two major causes. First, constant soil stiffness is adopted. The reduction of soil stiffness with increasing shear strain is not simulated. Second, time dependent soil properties are not modeled. It is apparent that time dependent behaviour in deep approach is the major component of wall movement. Nevertheless, computed results reasonably agree with measured result in terms of overall trend.

*Figure 14* Lateral wall movement in (a) tunnel zone; top-down method (T1); (b) deep approach bottom-up method with 1-layer bracing (D2); (c) deep approach with sump pit bottom-up method with 2-layers bracing (DS1)

### 7.3 Comparison of measured wall movement with other case histories

Figure 15 shows maximum measured wall movement by using two different construction methods and sequences reported previously in other case histories (Phienwej et al., 1995). Previous study conducted by Phienwej et al., 1995 suggests that maximum lateral wall movement using bottom-up construction can be up to 0.4% of excavation depth (H). On the other hand, wall movement in top-down construction is estimated to be only 0.2%H.

In this study, movement in bottom-up construction in deep approach and top-down construction in tunnel exceeds 0.4%H and 0.2%H line, respectively. Wall movement using top-down construction is closer to 0.27%H line proposed by Wang et al., 2010 based on measured field results in Shanghai soft clay. Formation level of excavation in this study is in soft clay and only one prop is provided. Thus, passive resistance is mainly from soft clay causing larger lateral movement compared with other case histories. Another possible reason of large lateral wall
movement is mainly due to the construction time or time-dependent movement as construction of underpass naturally takes more time due to traffic diversion and other factors.

Figure 15 Measured lateral wall movement compared with other case studies in Bangkok [Modified from Phienwej et al., 1995]

7.4 Bending moment of diaphragm wall

Figure 16 shows comparison of measured and computed bending moment of diaphragm wall. Measured bending moment is from vibrating wire strain gauges (VWSG) installed at reinforcement of diaphragm wall at depth of 6 and 17.5 m where bending moment is expected to be large. Measured strain is larger than 150 με thus wall flexural stiffness reduction factor of 80% is adopted. Deduced bending moment is from two times differentiation of measured lateral wall movement (Fig. 16) using the same stiffness reduction factor as measured from strain gauge.

In tunnel zone (Fig. 16a), measured bending moment at depth 6 m is slightly smaller than deduced bending moment suggesting that deduced bending moment is reasonable. It should be noticed that deduced bending moment exceeds computed bending moment from 8 to 13 m below ground surface due to smaller stiffness in soft to medium clay in numerical analysis.

In deep approach zone (Fig. 16b), measured bending moment at 6 m from strain gauge is about 30% larger than that in tunnel zone. Deduced bending moment in deep approach also confirm the measured result. Bending moment from depth 3 to 7 m significantly exceeds the computed bending moment due to lacking of time dependent properties of clay in numerical analysis.
At depth 17.5 m, measured, deduced and computed bending appears to be in a good agreement. In addition, deduced bending moments in both tunnel and deep approach zones are still within allowable bending moment capacity of diaphragm wall.

![Bending moment plots](image)

**Figure 16** Bending moment of diaphragm wall in (a) tunnel zone (T1); (b) deep approach zone (D2)

### 7.5 Excess pore water pressure

Measured excess pore water pressure normalized with initial pore water pressure is shown in Figure 17. Pore water pressure is measured from standpipe piezometer installed at a distance of 1 m away from diaphragm wall at sump pit with tip of piezometer at 10 m. Initial pore water pressure is about 70 kPa which is considered to be reasonable if hydrostatic pore water distribution is assumed given ground water table is at 3 m below ground surface (Teparaksa, 1999). Wall displacement at depth of 10 m is shown to compare with response of piezometer.

During the first 120 day after initial reading, although there is constant increasing of wall movement, there is no significant change of pore water pressure. This observation may be due to unloading of horizontal stress causes negative excess pore water pressure and shearing in soft clay causes positive excess pore water pressure. Consequently, two components counteract each other causing only a slight change of excess pore water pressure. From day 120 to 150, lateral
wall displacement increase significantly when excavation reaches to final depth of 10 m. Subsequently, there is a sharp increment of excess pore water pressure from day 160 to 200. This is likely to be caused by shearing in compression mode dominates effect of horizontal stress unloading. After base slab is constructed (at day 210), wall movement and excess pore water pressure appear to be stable.

![Graph](image)

**Figure 17** Normalized excess pore water pressure at 10 m deep outside of diaphragm wall at sump pit (DS1)

### 8. Conclusion

Design and construction of vehicle underpass in the congestive area of Bangkok is presented. Inclinometer measurement confirmed that top-down construction with permanent slab (stiffer bracing) induced less deflection than that temporary steel bracing with lower stiffness. This project reveals that a permanent diaphragm wall coupled with effective design modification of associated structure members and construction method could offer a logistically and technically attractive solution in construction of vehicle underpass in the congestive traffic area of Bangkok. Careful consideration in post-tender review of technical options and selecting the most practical construction method backup by sound design input and good performance of construction team are important factors contributed to successful completion of the project.
References
Sonic integrity test on piles founded in Bangkok subsoil – Signal characteristics and their interpretations

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SONIC INTEGRITY TEST ON PILES FOUND IN BANGKOK SUBSOIL

SIGNAL CHARACTERISTICS AND THEIR INTERPRETATIONS

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ABSTRACT

Sonic integrity testing is a common method used to test the integrity of piles in Bangkok. An acoustic technique equipped with computer software, developed by TNO, PDI, IFCO, etc., is commonly used to test the piles. This paper presents the signal characteristics and their interpretations from sonic integrity test performed on mainly bored piles varying in length, size, construction method and founded soil strata. Toe reflection of small bored piles (diameter of 0.35m to 0.60m) constructed in Bangkok subsoil can be observed for pile length of up to 20m to 24m. Beyond 35m below the ground, visibility of toe reflection is uncommon for large bored piles (diameter of 0.80m and larger). Commonly predicted defects and frequency of occurrences are tabulated. Data sets have been selected from the Data Bank of a foundations specialists’ organization to which the authors are associated with. Recently completed and current projects in Bangkok have been selected so that the data set could be well represented by over 8000 piles. 285 piles (3.3%) out of 8,689 piles have been interpreted for poor concrete or inclusions, cracks and size reduction. Among these, cracks contribute the major portion (2.2% or 194 piles). Cracks are mainly induced by either improper excavation adjacent to the piles or trimming the pile head by ill fated ways. Small diameter piles are the most suffered by cracks. Prominent sectional variations have been indicated by the signals at depth where temporary casing ends. Intermittent variations also are common at the interface of soft clay and medium to stiff clay layers.

KEYWORDS

Integrity, Sonic signal, Toe reflection, Pile impedance, Discontinuity
INTRODUCTION

Recent economic growth is being embraced by high-rise buildings in Bangkok, the city of Thailand, supplemented by other fast developing structures such as elevated highways and railways, bridges etc. As the Bangkok subsoil suffers a lot of constraints especially by the presence of thick soft clay on the top, almost every structure relies on pile foundation. Tens of thousands of piles are being constructed and tested annually in this city. For old structures, short friction piles (6-8m long) floating in the soft clay layer were commonly used. In modern practice, driven concrete piles with tips extending to the first sand layer or to stiff clay layer at a depth about 20-30m are normally employed for light to medium sized structures. For heavy structures or high-rise buildings deep large diameter (0.80m-1.5m) bored piles with tip extending to more than 45m are commonly used. Safety and overall stability of any finished structure highly depend on the foundation and assessing the quality of foundation is an inevitable aspect at the early stages of the structure. An acoustic technique equipped with computer softwares, developed by TNO, PDI, IFCO, etc., is commonly used to test the pile integrity.

TYPICAL SUBSOIL CONDITION IN BANGKOK

The Bangkok subsoil generally consists of 12-18m soft marine clay deposit underlain by medium to stiff clay. The dense second sand layer at depths 20m to 35m is commonly used. Between these sand layers, stiff to hard clay layers are present.

CONSTRUCTION METHOD

Two types of construction method, dry and wet process are commonly practiced in bored piling in Bangkok Soil.

Dry process uses steel temporary casing (about 15m in length) to protect the soft clay from caving in. The bore is drilled either auger or using bucket. For small piles, shells with drop weight is used to make the bore. Concrete is poured into borehole which is in dry condition after placing the reinforcement cage.

Wet process too normally uses steel temporary casing of 14-15m in length. A borehole is made by rotary method using auger or auger bucket under bentonite slurry. Concrete is poured with tremie method.

Dry process is employed mainly for small piles of 0.35m to 0.60m in diameter. Length of the piles are up to 25m and extends further if necessary, depending on subsoil condition, absence of sand layer with groundwater, etc.

For large bored piles of 0.80m to 1.50m in diameter, piles are constructed to depths of 30m to over 60m. Wet process is commonly used and piles are founded in sand layers.

Compressive cylindrical strength of concrete used in bored piles in Bangkok soil is in a range of 240-280 ksc (24-28 Mpa)

PRINCIPLES OF SONIC INTEGRITY TESTING

The test method is based on the time-domain, one dimensional wave theory. The compressive wave is propagated axially by the blow of hand-held impact device on to the pile head. The wave travels down along the pile shaft and reflects upward at the pile toe. In case where pile impedance variations occur due to changes in pile cross sectional area or properties of pile materials, or presence of discontinuities, part or whole of down-ward traveling wave reflects at the impedance variations and returns to the pile top before the first reflection from the pile toe. The reflected wave is detected by an accelerometer mounted on the pile top. Signal traces of time-velocity are usually recorded. As piles are installed in the ground the reflected signal is usually influenced by shaft friction contributed by the surrounding soil layers.

The formulae commonly used in analysis of integrity are;

\[ L - \frac{c}{2} \]

Where \( L \) is pile length, \( c \) is velocity of stress wave and \( T \) is travel time of the wave from pile top to toe and reflect back to the top.

Impedance of pile \( z \) (pile resistance to change in stress wave velocity) is given by

\[ z = E \frac{A}{c} \]

where \( A \) is cross-sectional area of piles and \( E \) is Young’s Modulus of the pile material

\[ E = \rho c^2 \]

from Equation (2) and (3)

\[ z = \rho c A \]

where \( \rho \) is density of pile.

The length of pile is determined from the travel time of the reflected wave from the pile toe and the wave velocity in concrete. The velocity wave can be estimated from the concrete strength of piles or from the reference pile with known length. In the same way the location of discontinuities and irregularities in piles can be also determined. Equation (4) shows that the change in pile impedance could be due to variation in pile cross-sectional area or in concrete quality.
CHARACTERISTICS OF SONIC TEST SIGNAL IN BANGKOK SUBSOIL

Soil influence

Influence by different soil strata on signals have been observed in many cases. Stiff clay layer at depth 15-25m (undrained shear strength of 10-22 t/m²) influences prominently and higher degree of signal reflections have been observed in small bored piles than that of large bored piles. Figures 1 and 2 explain these conclusions.

Fig. 1. Influence of soil friction on sonic signals (a pre-cast driven pile 0.40x0.40x25.0m in clay layers)

Toe reflections

Toe reflections are clear when the toe depth is 20-22m for piles of 0.35m-0.50m in diameter and up to 24m for 0.60m piles. In general, when pile length over diameter ratio (L/D) is within 40, toe reflections are clear and hence the pile length is predicted. However, for piles founded in depth deeper than 30-35m the toe reflection is uncommon as damping of signals is caused by deeper soil layers. Further, magnitude of toe reflection is prominent for larger piles compared to small piles with same length and constructed at same site. Figure 2 shows the signal characteristics of different size piles constructed just a few meters apart with same length.

Fig. 2. Toe reflections from different pile sizes, pile no. 6 (φ 0.50m x 22.0m) and pile no. 24 (φ 0.35m x 22.0m)

Size variations

Size variations interpreted by signals are often consistent with casting records and drilling monitoring results available.

Figure 4A and 4B show the comparison of borehole profiles and sonic test records.

In some cases, even though pile toe is extended beyond 35m in depth, the toe reflection can be observed in the test signals of toe grouted piles having L/D within 40.

Figure 1 is an example for such case where the bored piles were base grouted and of 40m in length (toe at 55m, trim level at -15m). However, it should be noted that these reflections are prominent only for few piles. Influence on toe reflection by grouted toe also depends on stiffness of the soil, underneath the toe, which has been grouted.

Fig. 3. Toe reflection observed from a toe grouted pile (pile φ 1.00m x 40.0m, cutoff –15.0m, toe depth 55.0m)

Fig. 4A. Size increase in pile shaft at 15m is shown by both sonic integrity test and drilling monitoring records (Pile φ 1.0m, cutoff –3.0m, toe depth –45m.)
Fig. 4B Uniform pile size along the pile shaft is shown by both drilling monitoring record and sonic integrity test record (Pile φ0.80m, cutoff 3.0m, toe depth -45m).

Common causes for size variations in bored piles are:

1. Effects from temporary casing
   - slight dimension changes of casing and bucket or auger
   - inadequate length to protect the soft clay
   - surface intact change at transition at the casing end
2. Delayed feeding of slurry and improper slurry level maintenance for wet process
3. Long time duration of maintaining open bore
4. Non continuous concrete pouring
5. Soil conditions such as inclusion of sand lenses in clay layers.

Figure 5 shows reduction of pile size at about 14m. A completed bored pile would have the size of outer casing at the top portion while the lower portion would be of bucket/auger size which is slightly less than the casing size. This size variation has been indicated by the signals at the transition zone. However dimensional variations by improper concrete slumping upon extracting the temporary casing and gradual dimensional changes at the transition zone are hardly indicated by the signal.

Figure 6 shows bored pile sections formed at the lower end of temporary casing. These bored piles were cast together with a steel column for construction of deep basements with diaphragm walls using top-down method. These piles were exposed when excavation reached to 15m below the ground level.

Figure 6 The photograph illustrates the pile size changes caused by difference in diameter of temporary casing and auger used in pile construction.

The test signal in Fig. 7 shows size decrease caused by inadequate casing length used in soft clay layer. Such defect is found often in piles constructed by dry process using casings of inadequate length to protect the entire soft clay layer of varying thickness.

Figure 7 The signal shows decrease in pile size at 10.6m (pile φ0.35m x 26.0m).

Figure 8 Pile with localize size increase at 13.5m indicated by the sonic signal. (pile φ0.80m, cutoff 1.4m, toe depth 35.85m).
In contrast to the size reduction, caused by smaller auger/bucket size than the casing size, relative size increase also have been observed. Casting records for piles indicated in Fig. 8 noted about 30% over break of concrete volume.

Discontinuities-cracks

Common causes attributed for these are;
1. disruption in concrete pouring
2. contamination of concrete with bentonite slurry or soil
3. lifting up of insufficiently workable concrete or hardened concrete upon extracting the temporary casing

Figure 9A shows a discontinuity in pile shaft caused by disruption during concrete pouring. Concrete supply was interrupted by poor coordination between the piling contractor and concrete suppliers. By the time to resume the concreting the lower section has been hardened. The discontinuity found was later closed by coring and cement grouting. Figures 9A and 9B show the signal interpretation before and after closing the discontinuity.

Figure 10A Sonic test signal showing size decrease and or a discontinuity at 7.5-9.0m and poor quality pile section near the to (Pile 0.35m x 20.0m)

Figure 10B Sonic signal on the failed test pile showing a construction joint between pile cap and pile top at 1.40m. Size decrease or a discontinuity is also indicated.

Figure 11A shows presence of poor quality concrete at the pile top due to contamination of concrete with bentonite slurry. Such defect is often found in piles with shallow cutoff level and hence adequate over cast length to have a sound concrete at cut off level, sometimes, cannot be made. Normally 1.0m to 2.0m over-casting of concrete is practiced.

After trimming back the pile top, sonic test was performed and the test signal indicates the sound integrity of the same (see Fig. 11B)

Fig 9A Sonic signal showing a discontinuity at 11.0m caused by interruption in concrete pouring(pile φ 0.80m x38.8m) and multiple reflections from the discontinuity

Fig 9B Sonic integrity test shows the discontinuity at 11.0m has been closed by grouting.

Figures 10A and 10B show the characteristics of signals observed at a site where dry process was adopted. Many piles at this particular site were interpreted for poor integrity especially at the bottom portion. From the construction records it was noted that boring was done in water bearing sand layer without casing or bentonite slurry causing concrete contamination with water and/or segregation.

Further, static pile load test results indicate the test pile (Fig 10B) failed well before designed maximum test load. Figure 10B also shows the joint between pile cap and pile.

Fig 11A The signal showing poor quality concrete at pile top section (Pile φ0.60m x18.0m)

Fig 11B The signal acquired after trimming top portion. The reflections from the lower part of the pile and toe become visible.
Physical pile damage due to excavation or improper chipping to trim level

Most discontinuities or cracks in bored piles after installation of piles can be caused by construction activities associated with basement excavation adjacent to them and improper trimming to design cut-off level.

The test signal in Fig. 12 shows a crack or discontinuity in the pile caused by soil movements occurred by adjacent excavation work.

Fig. 12 Sonic signal showing a discontinuity crack at about 7.8m, caused by lateral soil movements.

Fig. 13 High strain dynamic load test signal showing a crack in the pile at about 8.0m.

An inadequate retaining system for excavation work has caused the lateral movement to the pile top about 80cm and a crack has developed in this pile. The crack was also detected by the high strain dynamic load test performed to determine the pile capacity of the cracked pile (see Fig. 13).

From Fig. 12 and Fig. 13 the duration of pulse in the sonic integrity test signal is about half of that in dynamic test signal. The short duration (high frequency) pulses can resolve reflections from narrow impedance changes. In such case presence of a crack in the pile is more clearly indicated by sonic integrity test signal (low strain) than the dynamic test signal (high strain).

An assessment of pile defects based on the sonic integrity test records on 8,689 bored piles in Bangkok subsoil and their construction records including soil conditions was carried out. The assessment indicates that integrity of 285 piles (3.3%) out of 8689 piles has been suspected. They are summarized in the table below;

<table>
<thead>
<tr>
<th>Type of Defect</th>
<th>Causes</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poor concrete at pile top (0-3m)</td>
<td>cutoff level near the ground or not enough overcast</td>
<td>0.1</td>
</tr>
<tr>
<td>Cracks/discontinuities</td>
<td>Excavation works for pile trimming and Construction activities</td>
<td>2.2</td>
</tr>
<tr>
<td>Size reduction</td>
<td>Inadequate casing length or soft clay layer variation in thickness</td>
<td>1.0</td>
</tr>
</tbody>
</table>

From the above assessment, most of the defects are found to be in small piles with a diameter of 0.60m or smaller. Study of piling and excavation plans show that piles with a crack indicated by the sonic integrity were located in the vicinity of excavation boundaries.

A total of 1491 piles were found with localize increments in pile size. All of size variations including size reductions are found to be at the transition interface of temporary casing and soil layer, where soil strata changes also occur.

CONCLUSION

Sonic integrity test is an effective method to assess the pile integrity. Computerized acoustic technique developed by TNO, PDI and IFCO is commonly used in Bangkok for both driven piles and bored piles. About 3.3% of 8689 piles have been interpreted for either size reduction or poor quality of concreting or cracks/discontinuities.

Toe reflections are generally clear for small piles with slenderness ratio of 40. But only in few cases, especially toe grouted, toe reflection has been observed for longer piles with toe embedded below 30-35m.

Signal interpretations have been compared with soil investigation results and construction records. Causes for defects could be identified and conformed to casting records.

Minor defects, such as small cracks can be predicted in many instances and nature of them need to be verified by further visual inspection.

The predicted defects and possible causes verified from the construction records would help the piling and excavation contractors to be aware of the causes of defects associated with construction practice.

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REFERENCES


Application of Stress-Wave Theory to Piles

Quality Assurance on Land and Offshore Piling

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Sussumu Niyama
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Sonic Integrity test of piles-integrity effected by basement excavation in Bangkok soft clay

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Sonic integrity test of piles-integrity effected by basement excavation in Bangkok Soft Clay

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ABSTRACT: Modern buildings in Bangkok nowadays are designed with basement facility and different elevation of pile foundation which require excavation works in the immediate vicinity of the constructed bored piles. Adverse effects on piles from adjacent excavation works in terms of excessive movement of piles and pile damages have been observed in some projects. Assessment of bored pile damages using sonic integrity test is focused in this paper with demonstration of test results from three case histories. Method of remedial works for damaged piles and additional testing for pile capacity justification where available are also briefly discussed. Stress induced in piles due to the soil movement caused by excavation works is also examined with the aid of computerized finite element program. Location of cracks detected by sonic integrity tests is correlated with the analyzed bending moment in pile, which exceeds the cracking moment capacity of pile.

1 INTRODUCTION

Due to the prevailing condition of subsoil (existence of considerably weak stratum at top part) piles are commonly used for foundation of buildings and structures in Bangkok. Deep-seated large diameter bored piles of over 35 m depth with 0.8 – 1.8m in diameter have been mainly used for heavy structures such as high rise buildings, elevated expressways, flyovers or overpass bridges and more recently for underground subway stations of the first Bangkok MRTA system. As a part of pre-planned quality assurance regime and retrospective investigation, integrity of the piles are tested to obtain the information with regard to the potential deficiencies of the constructed piles which may have formed during actual pile construction process or may have been attributed by other activities after construction of the piles. Designs of the modern buildings in Bangkok frequently call for the basement facility and for the different elevation of pile foundation, which require excavation work mainly in top soft clay layer. In some projects, adverse effect from excavation works adjacent to constructed piles has been observed. This paper presents the assessment of bored pile damages which were induced by adjacent excavation using sonic integrity testing method.

2 GENERAL DESCRIPTION OF BANGKOK SUBSOIL

Bangkok subsoil consisted of alternating clay and sand layers of Quaternary deposits extending down to about 550m depth where bedrock generally exists (Balasubramanium, 1991). Made-Ground consists predominantly of Fill-Materials, Clayey Sand or Silty Clay with some cement block rubble and rock fragments, is commonly found up to 4m depth. Soft to very soft, highly compressible dark gray marine clay lies beneath Made-Ground and in some areas it lies under weathered crust layers of 2m thick. Depending on the location, this layer is extended up to 12-18m. About 2m thick Medium Clay layer can be observed between Soft Clay and underlying Stiff Clay. Generally Stiff Clay layer occurs directly underneath Medium Clay and its depth goes up to 22m. Below Stiff Clay layer, First Sand layer 5-8m in thickness can be found. This First Sand layer, however, is absent in some areas. Stiff to Hard Clay layer underlies First Sand and it is found to be about 5m thick. Second Sand layer generally occurs at depths between 45 to 65m. The Generalized Fence Diagram of Subsoil covering the central area of Bangkok is illustrated in Figure 1.
3 BORED PILE CONSTRUCTION IN BANGKOK

In Bangkok, bored piles are constructed mainly by two methods, dry and wet process. Dry process is applied commonly for small piles of diameter 0.35 to 0.60m with relatively shorter lengths. In dry process, a pile is bored to the required depth within impermeable layers (usually up to 25m) by either rotary or percussion method whilst temporary casing (about 15m in length) is installed in soft clay layer for protection of soil caving. After installing reinforcement cage, concrete is poured into the borehole under dry condition. Tripod method, shell with drop weight is also commonly used in dry process.

Wet process method as its name implies makes the pile under wet condition by using drilling slurry (bentonite or polymer). Temporary casing of 14-15m in length is also used as a support in soft clay layer and soil inside the casing is normally excavated by auger applying rotary drilling method. Drilling is continued with the bucket under the slurry from the top of sand layer to the final depth. Tremie concreting is necessary for pile installed under wet process. Bored piles constructed by wet process are generally of large diameter (0.8m to 1.8m diameter) deep-seated (30m to over 60m) and are normally founded in either first or second sand layer. Bored piles having compressive cylindrical strength of concrete in a range of 240-280 ksc (24-28 MPa) with reinforcement ranging 0.5% to 1.2% of pile sectional area, are commonly used in Bangkok.

4 SONIC INTEGRITY TESTING IN BANGKOK

Sonic integrity test is widely used for integrity testing of both driven and bored piles in Bangkok. It is employed as a part of quality control and or as a retrospective investigation when some problem becomes apparent. For a quality control regime, the number required for integrity testing depends upon the technical requirement of the particular projects ranging from minimum 10% to maximum 100% of total constructed bored piles. A sonic integrity tester with built-in computer having high quality signal acquisition system and computer programs developed by TNO, PDI, IFCO etc. are commonly used for pile integrity testing.

OVERVIEW OF EXCAVATION INDUCED PILE DAMAGE IN BANGKOK

In general, pile defects are caused at two stages, during the pile construction process and after construction of the piles (post-pile-construction).

Thasnanipan et al. (1998b) reported that integrity of 285 bored piles (3.3% of 8,689) were found to be of doubtful quality according to an assessment on the results of sonic integrity test with the additional information obtained from the pile construction records of bored piles in Bangkok subsoil. The results of the findings are summarized below.

Table 1. Summary of pile assessment on 8,689 bored piles (Thasnanipan et al. 1998b)

<table>
<thead>
<tr>
<th>Type of Defect</th>
<th>Causes</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poor concrete at pile top (0-3m)</td>
<td>Cutoff level near the ground or inadequate overcast</td>
<td>0.1</td>
</tr>
<tr>
<td>Size reduction</td>
<td>Insufficient casing length or soft clay layer variation in thickness</td>
<td>1.0</td>
</tr>
<tr>
<td>Cracks/discontinuities</td>
<td>Excavation works for pile trimming and construction activities</td>
<td>2.2</td>
</tr>
</tbody>
</table>

As can be seen in the above table, higher percentage of defects is caused by post-pile-construction activities and more commonly by movement of surrounding soils induced by excavation works in the vicinity of the piles.

Modern buildings in Bangkok frequently require the excavation works for basement facility and for the use of different elevations of pile foundation. Excavation works are mainly carried out by using cut slope, sheet pile wall, diaphragm wall and secant pile wall. Since most of the bored piles are designed mainly to carry an axial load, lateral and tensile force which may impose on pile during basement excavation are sometimes not considered or overlooked. The lateral displacement of ground caused by excavation naturally induced an additional lateral force and bending moment in the pile. Once the induced bending moment exceeds the cracking moment capacity, piles are subject to be cracked. Unexpected tensile force acting upon the pile due to an excessive heave or uplift force induced by excavation can also cause cracking of pile. In Bangkok, pile cracks caused by external forces are normally found to be at the level where Soft and Medium to...
Stiff Clay boundary is present. Figure 2 illustrates the typical features of external forces, which cause a crack in the piles. Pile damage associated with sheet pile wall excavation is presented in this paper.

Figure 2. Typical features of external forces causing pile cracks in Bangkok subsoil

6 APPLICATION OF SONIC INTEGRITY TEST IN PILE-INTEGRITY CHECKING

Sonic integrity test is simple, rapid, efficient and cost effective in examination of the pile integrity. It is reasonably reliable for the integrity testing of the pile especially for the upper part where potential of damage is higher as far as defect caused by the external effects (lateral displacement of soft clay due to adjacent excavation works, unexpected force from improper pile head trimming) is concerned.

6.1 Basic principles of sonic integrity test

Sonic integrity testing also called as low-strain integrity testing examines the response of a pile to a light external impulse force. Either time-domain or frequency-domain can be used in analysis of measured data by sonic integrity test. In Bangkok, time-domain method is mainly applied in interpretation of test data.

Sonic integrity test is undertaken by striking the head of the pile with a light hand-held hammer and recording the response of the pile to this impulse blow by means of a sensor or accelerometer placed in good contact with the pile head. Stress wave theory is a basic principle of this low-strain integrity testing. The hammer blow induces a compressive stress-wave into the pile, which propagates axially along the pile shaft and reflects back toward the pile head at a change of impedance within the pile. In the case where pile impedance variation occurs due to changes in pile cross sectional area or properties of pile materials or presence of discontinuities, part or whole of down ward wave reflects at the impedance variations and returns to the pile head before the first reflection from the pile toe.

Basically, following formulae are used in analysis of sonic integrity test.

\[ L = c \frac{T}{2} \]  

where, \( L \) is a length from pile head to reflected surface, \( c \) is a velocity of stress wave and \( T \) is a travel time of the wave for length \( L \).

From this formula, length \( L \), can be computed from the travel time of reflected wave and the wave velocity of concrete which may be estimated from concrete strength of piles or from the reference pile of known length.

\[ z = \frac{E A}{c} \]  

\[ E = \frac{\rho c^2}{\rho} \]  

where, \( z \) is an impedance of pile, \( E \) is Young Modulus of pile material, \( A \) is cross-sectional area of pile, \( c \) is the propagation velocity of the stress wave and \( \rho \) is density of pile.

From Equations (2) and (3)

\[ z = \rho c A \]  

As can be seen from Equation (4), the change in pile impedance could be due to variation in pile cross-sectional area or in concrete quality.

6.2 Interpretation of sonic integrity test

In general the typical pile features detectable by sonic integrity test includes;

- Pile size variation
- Pile toe reflection (for pile length verification)
- Pile material variation
- Soil influence
- Discontinuity and/or cracks

The typical sonic integrity signals of above features have been reviewed and presented by Thasnanipan et al. (1998a). As some features have a similar pattern to each other, it is always important to check and compare the test data of pile in question with those of other piles within the same job site to establish “site signature”. In some cases correlations should be made with similar projects where damaged piles were investigated with firmed evidence (coring or excavation to detect damage level). Interpretation should also be made with sound knowledge of pile construction technique, subsoil condition and other factors (problem encountered during and post pile construction) which may influence the sonic test signals. Pile construction records are also useful in interpretation of piles with detected anomaly. Conclusion should be made after careful and thorough review of test results in conjunction with other available information. Further
investigation (if applicable excavate down to level of detected crack for visual inspection) may require before final conclusion is made.

As a main focus of this paper, the typical features associated with the pile damage (discontinuity/cracks) detected by sonic integrity test is discussed in detail with the data obtained from three cases as presented in the following sections.

6.3 Pile damage assessment
From the results of sonic integrity test, degree of damage or level of crack may be assessed. The assessment of crack generally involved – setting of amplification in the same range for all piles and comparison of input impulse, amplitude of anomaly and intensity of reflection from the anomaly (i.e. multiple reflection). Additional information obtained from further investigation as described in above section is also used in pile damage assessment.

7 CASE STUDIES
7.1 Case I – Sheet pile excavation supported by soil berm

7.1.1 Subsoil condition
A 12m thick soft clay layer occurs beneath fill material and weathered crust of 1.0 to 2.5m thick. Unconfined shear strength (\(S_u\)) of soft clay layer ranges 1t/m\(^2\) at top and 2.5t/m\(^2\) at bottom whereas the natural water content is about 60-80%. Medium clay layer of 1 to 2m in thickness, having shear strength of 2.5t/m\(^2\) to 3.5t/m\(^2\) underlies soft clay layer. Below medium clay, at the depth between 18 to 27m, stiff clay with traces of fine sand having SPT N value of 15 to 37 is observed. Thick layer of fine sand interbedded with clayey sand occurs underneath stiff clay layer.

7.1.2 Pile foundation
Pile foundation of eight high rise buildings in this project comprises 904 cast-in-place bored piles of diameter 1.0m and 1.5m, founded at 59m below ground level. Pile tips are embedded in dense sand layer and the entire length of piles are reinforced with steel reinforcement of 0.72% of pile sectional area for the top part and reduced gradually to 0.23% for the bottom section. To support the underground water tanks, 88 pre-cast concrete piles (I-30cm) having 21m length were also driven down to stiff clay layer.

7.1.3 Excavation method and observed failure
Excavation was initially carried out by using sheet pile (Type FSP III) with soil berm support or slope. Before reaching to –8.1m maximum depth of excavation, sheet pile failure and associate severe lateral movement as well as settlement of ground including tension cracks of more than 300mm in width were observed at some locations. Minor damages were also investigated at the adjacent properties. Temporary raking struts were immediately installed in response to these excessive ground movements caused by sheet pile failure. A typical scheme showing excavation works of Case I is presented in Figure 3.

Figure 3. A picture showing slope excavation with sheet pile support – Case I

7.1.4 Sonic integrity test and pile damage assessment
More than 50% of piles were tested with sonic integrity test. Test results were thoroughly reviewed and “site signature”, most common features of the majority of piles, is established. It is to be noted that test signals from piles of less potential for damage (least effect from excavation) are selected in establishment of “site signature” or reference signal of good piles. Based on the “site signature” of good piles, defect piles were identified and assessed. Table 2 shows the summary of pile damage assessment by sonic integrity test.

<table>
<thead>
<tr>
<th>Pile Dia. (m)</th>
<th>No. of piles with defect</th>
<th>Defect depth below ground level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Least prominent crack</td>
<td>Less prominent crack</td>
</tr>
<tr>
<td>1.0</td>
<td>4</td>
<td>8</td>
</tr>
<tr>
<td>1.2</td>
<td>-</td>
<td>10</td>
</tr>
<tr>
<td>1.5</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The typical sonic integrity test results of undamaged pile and damaged pile classified as least prominent crack, less prominent crack and prominent crack are shown in Figure 4. Majority of piles with crack detected were located in the periphery of the excavation zone. Piles, particularly with prominent cracks, were observed to be deviated from their original location up to 600mm towards excavation.
7.1.5 Remedial measures and pile capacity justification

Bored piles with cracks were cored to the depth below the crack and grouted with non-shrink cement. Sonic integrity test was conducted few days after rectifying the pile. Sonic integrity signals of the defected pile tested before and after the remedial work is presented in Figure 5. After the rectification works, high strain dynamic load test was performed on two piles with large horizontal deviation and suspected prominent cracks. The dynamic test results indicate that tested piles could be capable of carrying the design load with factor of safety higher than 1.5.

7.2 Case II – Pile damage caused by sheet pile excavation with one level temporary support

7.2.1 Subsoil condition

Soft Clay layer extends to 15m below ground surface with undrained shear strength increases from 1t/m² at the top and 2t/m² at the bottom. Medium Clay layer of 2 to 3m in thickness is found between Soft Clay and the underlying Stiff Clay. Stiff Clay layer occurs up to depth 42m with the SPT ‘N’ values ranges between 14 and 40.

7.2.2 Pile foundation

402 bored piles of diameter 1m and 1.5m, founded at 55m below ground, being embedded in dense sand layer were used as foundation in this project. Reinforcement steel bars of 0.5% pile cross sectional area were provided for the entire length of bored piles.

7.2.3 Excavation method and observed failure

Sheet pile wall of 14m in length with 1 level bracing was utilized for maximum 8m excavation. Though the design required for installation of struts at 1m below ground level, the actual excavation was carried out up to 3m depth without installing the temporary struts causing sheet piles deflected in large magnitude.

7.2.4 Sonic integrity tests and pile damage assessment

Sonic integrity test was performed for all piles after trimming to the designed cutoff level. It is evident that piles with crack detected by sonic integrity test were mostly located in the vicinity of the excavation boundary.

A summary of the number of damaged piles and their degree of damage assessed are presented in Table 3.
Table 3. Summary of pile damage assessment by sonic integrity test – Case II

<table>
<thead>
<tr>
<th>Pile Dia. (m)</th>
<th>No. of piles with defect</th>
<th>Defect depth below ground level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Least prominent crack</td>
<td>Less prominent crack</td>
</tr>
<tr>
<td>1.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.5</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

7.2.5 Remedial measures
Damaged piles were cored to the level below the defected crack and grouting was performed.

7.3 Case III – Pile damage caused by sheet pile excavation with two level temporary support

7.3.1 Subsoil condition
Underneath the top 2m of weathered crust, Soft Clay layer can be observed up to 15m below the ground surface. Medium Clay layer underlies Soft Clay with the thickness of 2m. Undrained shear strength of Soft Clay ranges between 1t/m² and 3t/m² whereas it is about 50t/m² in Medium Clay. Beneath Medium Clay, Stiff Clay layer is found up to 25m where the boundary of Hard Clay is investigated. Within Stiff Clay stratum softer zone of Medium Clay is identified at the depth between 19 and 21m characterized by reduction in SPT values which appear consistent with increases in moisture content.

7.3.2 Pile foundation
A total of 143 bored piles of 0.6m diameter were installed with pile tip at about 26m below the ground to support a residential apartment. Dry process was employed to construct the piles. Cut-off level of bored piles ranges from 0.85m to 5.95m depth. Entire lengths of all piles are fully reinforced with steel bars grade SD40 of 0.35% of pile sectional area.

7.3.3 Excavation method and observed failure
Temporary retaining wall using FSP III sheet pile type of 14m in length with two level bracing placed at 1m and 3.5m below ground level was employed for 6m depth excavation. A part of pile layout within excavation area showing sheet pile wall and bracing system is illustrated in Figure 7. No major failure in terms of large tension cracks and ground settlement was observed at the surface adjacent to the excavation though some excessive sheet pile deflection was visually inspected at few locations. Hence, it was not expected by the visual inspection at site that excavation work has effected the integrity of piles.

7.3.4 Sonic integrity tests and pile damage assessment
Though sonic integrity test was employed as a part of quality control regime in the original plan, it was eventually applied for a retrospective investigation after encountering many piles with defect. Hence all piles were undergone sonic integrity test. According to the test results, 84% of piles located within the excavation zone were identified to be with crack. Sonic integrity test results of good pile, pile with less prominent crack, and pile with prominent crack are illustrated in Figure 8. As can be seen in Figure 8, for pile with prominent crack, a distinct and sharp reflection is evident at depth about 5.5m below pile top. Repetition of reflection at multiples of this depth can also be observed for this pile. These features are comparatively not distinct for the pile with less prominent crack. An excessive basal heave, insufficient reinforcement to resist the tensile stress and relatively small pile size are considered to be the main reasons causing the cracks in this case. Table 4 shows a summary of pile damage assessment by sonic integrity test.
Table 4. Summary of pile damage assessment by sonic integrity test – Case III

<table>
<thead>
<tr>
<th>Pile Dia. (m)</th>
<th>No. of piles with defect</th>
<th>Defect depth below ground level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Least prominent crack</td>
<td>Less prominent crack</td>
</tr>
<tr>
<td>0.60</td>
<td>-</td>
<td>38</td>
</tr>
</tbody>
</table>

7.3.5 Remedial measures and pile capacity justification

Coring was carried out up to the depth below the detected crack to verify the pile damage and then it was grouted with non-shrink cement. To justify the capacity of damage piles, high strain dynamic load test was carried out on 25 piles in which 24 piles with detected cracks and 1 pile without crack detection. Summary of dynamic load test is presented in Table 5.

Table 5. Summary of dynamic load test results of damaged piles – Case III

<table>
<thead>
<tr>
<th>No. of pile</th>
<th>Mobilized capacity / Design ultimate capacity of 240 ton (t.m)</th>
<th>Cracking moment (t.m)</th>
<th>Ultimate moment (Whitney) (t.m)</th>
<th>Moment by FEM model (t.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Less than 50%</td>
<td>1</td>
<td>79</td>
<td>149.7</td>
</tr>
<tr>
<td>II</td>
<td>Less than 10%</td>
<td>2</td>
<td>188</td>
<td>117.2</td>
</tr>
<tr>
<td>III</td>
<td>More than 100%</td>
<td>22</td>
<td>13.3</td>
<td>255.7*</td>
</tr>
</tbody>
</table>

Note: (*) moment in pile located outside of sheet pile wall within the active side of soil mass.

8 FINITE ELEMENT ANALYSIS WITH COMPUTER MODEL SIMULATION

Finite element analysis was carried out using two-dimensional modeling. With the aid of PLAXIS computer program staged excavation was simulated and pile/soil movements as well as bending stress induced in the piles were examined for above presented projects. Model profile of Case III used in FEM analyses is presented in Figure 9.

Figure 9. Finite element model profile of Case III

Stiffness of foundation piles per linear meter (E*S/\per cent spacing in row) was used as a parameter for plate elements, where E and I is Young Modulus and moment of inertia of pile respectively. The soil between piles in a row is ignored. Mohr-Coulomb constitutive model is adopted using undrained soil parameters derived from soil investigation at site and associated lab test results.

9 CORRELATION BETWEEN RESULTS FROM FINITE ELEMENT AND SONIC INTEGRITY TEST

The analysis results indicate that bending moment in piles induced by excessive ground movement due to excavation works are higher than that of cracking capacity of pile in all cases. Table 6 presents the summary of the analyses.

Table 6. Moment capacity of piles and induced moment in piles due to excavation simulated by finite element method

<table>
<thead>
<tr>
<th>Case</th>
<th>Pile size (m)</th>
<th>Rebar (%)</th>
<th>Cracking moment (t.m)</th>
<th>Ultimate moment (Whitney) (t.m)</th>
<th>Moment by FEM model (t.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1.0</td>
<td>0.75</td>
<td>30</td>
<td>79</td>
<td>149.7</td>
</tr>
<tr>
<td>II</td>
<td>1.5</td>
<td>0.50</td>
<td>102</td>
<td>188</td>
<td>117.2</td>
</tr>
<tr>
<td>III</td>
<td>0.60</td>
<td>0.35</td>
<td>6.5</td>
<td>9.2</td>
<td>255.7*</td>
</tr>
</tbody>
</table>

Note: (*) moment in pile located outside of sheet pile wall within the active side of soil mass.

As can be seen in Table 7, the crack location in pile indicated by sonic integrity test is generally consistent with the location of computed bending moment, which exceeds the cracking moment capacity of piles for all three cases.

Table 7. Locations of crack and computed bending moments in piles in the vicinity of excavation boundary

<table>
<thead>
<tr>
<th>Case</th>
<th>Integrity test depth of crack in piles near excavation boundary (m)</th>
<th>Depth of maximum bending moment by FEM (m)</th>
<th>Depth of bending moment exceeding pile’s cracking moment from FEM (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>12.7 &amp; 13.7</td>
<td>18.0</td>
<td>11.0 &amp; 13.0</td>
</tr>
<tr>
<td>II</td>
<td>12.7 &amp; 13.7</td>
<td>15.0</td>
<td>12.8 &amp; 14.5</td>
</tr>
<tr>
<td>III</td>
<td>11.5 &amp; 12.0</td>
<td>15.5</td>
<td>13.0 &amp; 14.0</td>
</tr>
</tbody>
</table>

It is evident that detected cracks are located near or at the depths where boundary of Soft and Medium or Stiff Clay is present.

In case III, FEM analysis results suggest that piles are subject to some degrees of basal heave as illustrated in Figure 10. This is found to be caused by inadequate embedded length of sheet pile. As can be seen in Figure 11, the results also show that...
bending moment induced by excavation exceeds the ultimate moment capacity of pile. Sonic integrity test results of some piles indicate the presence of more than one crack at some depths, suggesting piles would have experienced both bending and tensile stress due to the excavation.

Observation of actual construction works on sites suggests that over excavation prior to installation of support and inadequate retaining system are the main causes leading to excessive ground movement, a primary reason of pile damage. Excavation contractors should be well aware of the consequences of excessive soil movement induced by improper construction practice.

The designer of the foundation should also be aware of the potential problem on site and should take the actual construction practice into consideration in the design. For instance, if piles are expected to experience the excessive tensile or bending stress from the most practical excavation method, sufficient reinforcement should be provided in the design at the first place. The foundation designer should closely be involved in the design of the retaining system and construction control on site during the excavation period so that any possible negative effect can be minimized if not entirely avoided.

REFERENCES


Application of Stress-Wave Theory to Piles

Quality Assurance on Land and Offshore Piling

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Non-destructive integrity testing on piles founded in Bangkok subsoil

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Non-Destructive integrity testing on piles founded in Bangkok subsoil

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ABSTRACT: Pile foundation is compulsory in Bangkok, Thailand, to support almost all types of structure due to prevailing subsoil condition. Large bored piles of 600mm to 1800mm in diameter and barrettes (0.8mx2.7m-1.5mx3.0m) with toe level up to 60m depth are commonly used to support heavy structures covering high-rise buildings, subway stations, elevated highways, etc. In order to test or check the quality of such high load carrying foundation piles constructed in multi-layered soil conditions, three main types of non-destructive integrity tests such as sonic integrity, sonic logging and high strain dynamic load tests are usually employed. Sonic integrity test is the most common method in integrity testing on both driven and bored piles. Sonic logging test is occasionally employed as a part of pre-planned site quality control system. High strain dynamic load test is commonly used for pile capacity evaluation. This paper presents an overview of these three commonly used non-destructive tests on piles in Bangkok subsoil. Some findings from the author’s involvement in research works are also presented.

1 INTRODUCTION

Three techniques namely sonic integrity, sonic logging and high strain dynamic load tests are the commonly used non-destructive integrity tests applied in Bangkok, Thailand. The effectiveness and suitability in terms of both cost and method of application itself are the primary reasons of the greatest growth in use of these tests. An overview of sonic integrity, sonic logging and high strain dynamic load tests conducted on piles particularly bored piles founded in Bangkok subsoil is presented in this paper.

2 TYPICAL SUBSOIL CONDITION IN BANGKOK

Subsoil profile is relatively consistent at different localities in Bangkok. A typical subsoil profile is characterized by the alternating layers of clay and sand deposits as soil succession shown in Figure 1. Weathered Crust of 1m to 3m thick occurs as uppermost layer underlain by Very Soft to Soft Clay commonly known as Bangkok Soft Clay (BSC). Thickness of Soft Clay layer varies from 12m to 18m depending on the location. Underneath Soft Clay, lies Medium to Stiff Clay. Dense to Very Dense Sand layer also known as First Sand layer is found below Stiff Clay layer at depths about 25m to 38m. Second Dense to Very Dense Sand layer occurs at about 46m depth. Between these sand layers, Hard Clay layer is present.

Figure 1. Typical soil profile of Bangkok plain with piezometric draw down conditions (after Thasnanipan et al., 1998a).

The depth of bedrock is not well determined in the Bangkok area but has been reported to be at least
550m deep (Balasubramaniam, 1991). Existing pore water pressure conditions in upper part of BSC are hydrostatic from nearly 1m below the ground level. Then the hydrostatic conditions change to piezometric draw down near bottom level of BSC.

3 COMMON CONSTRUCTION METHOD

The common practice of bored pile construction method in Bangkok can be divided into wet and dry processes.

For the wet process, bored piles are constructed using bentonite or polymer slurry as drilling fluid over the full length of the piles, to ensure the stability of the excavation at all stages. Top 15m-thick soft clay was temporarily cased. Drilling for the piles was done with a conventional drilling auger or auger bucket. To clean the sediments at the borehole base the airlift technique or cleaning bucket is applied. After lowering the rebar cage, concrete is poured with tremie method.

Generally the dry process is employed for small piles (0.35m to 0.6m in diameter and 25m in length). This method uses a steel temporary casing of about 15m in length to protect the soft clay from caving in. Boring is carried out by rotary drilling or percussion tripod rig.

Bored piles are commonly constructed with reinforcement of 0.5-1.2% of sectional area of pile.

4 COMMON DEFECTS CAUSED IN BORED PILES

Common pile defects such as size reduction/necking, discontinuity, soil/slurry inclusions, etc. may be caused during pile construction. Most of the cracks in bored piles caused at post construction stage are usually induced by construction activities associated with basement excavation and improper trimming of pile head to design cut-off level. Causes of defects in bored piles during construction are;

- Inadequate length to protect the soft clay layer
- Delayed feeding of slurry and improper slurry level maintenance for wet process
- Long time duration of maintaining open borehole
- Non continuous concrete pouring or disruption in concreting
- Lifting up of insufficient workable concrete or hardened concrete upon extracting the temporary casing
- Contamination of concrete with drilling slurry or soil
- Soft pile toe due to improper base cleaning

5 THE SELECTION OF SUITABLE METHOD

In general, non-destructive tests are conducted to verify the quality of the pile. Turner (1997) highlighted the factors in selection of pile testing in CIRIA Report 144 as follows;

- The perceived nature of possible features or defects within the pile
- The ability of the test method to detect the feature or defect under investigation
- The cost of testing and examination
- The ease of use and interpretation

Like other indirect measurement of pile integrity, sonic integrity and sonic logging tests are only capable of identifying structurally significant features. Rejection or acceptance of individual piles should not rely only on the results of these tests. Further investigation, engineering evaluation and judgement are highly recommended to confirm the defect detected by these indirect techniques. Both methods however are very useful for cost-effective screening test to identify piles with potential defect.

Sonic integrity test is usually selected for both quality check (control test) and retrospective investigation. It is the cheapest in terms of cost and the simplest in terms of testing process. The main advantage of this test is that since no particular measures/preparation is necessary during the pile construction phase it is more flexible to select which pile is to be tested. However, interpretation of sonic integrity testing needs considerable experience and knowledge in testing, subsoil condition and construction method.

Sonic logging test is relatively expensive. It is mainly employed as a pre-planned site quality control testing. The major advantage of this method is that test can be carried out shortly after the pile construction. Hence, rectification measures can be implemented while the foundation contractor is on site. However, this method is generally not applicable if pile integrity is in question due to post construction activities as access tubes are usually grouted after completion of the test.

High strain dynamic load test is usually selected to verify the load carrying capacity of piles. Pile integrity can also be determined by high strain dynamic load test. The major advantages of this test in comparison with static pile load test are those of cost, time and space requirement. In some projects, dynamic load test was applied to justify the capacity of pile after rectifying its defect detected by other integrity tests. Table 1 shows the suitability of test method for different type of common defects.
Table 1. The suitability of test method for different type of defects.

<table>
<thead>
<tr>
<th>Type of defects</th>
<th>Test method and suitability for defect detection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sonic integrity</td>
</tr>
<tr>
<td>Crack</td>
<td>Suitable</td>
</tr>
<tr>
<td>Pile size change</td>
<td>Suitable</td>
</tr>
<tr>
<td>Poor quality concrete or Contaminated concrete</td>
<td>Possible</td>
</tr>
<tr>
<td>Soft pile toe</td>
<td>-</td>
</tr>
</tbody>
</table>

6 SONIC INTEGRITY TEST

6.1 Overview of the test in Bangkok

Sonic integrity test, also known seismic test is the most common method of integrity testing for both driven and bored piles in Bangkok. In many projects it is a part of the contractual requirement to conduct sonic integrity test. Minimum 10% to maximum 100% of production piles are commonly tested. It is also a reasonably acceptable method for applying as a retrospective investigation in determining integrity of the pile.

6.2 Interpretation of the test results

As has been reported in various literatures, it is to be emphasized that sonic integrity test is highly dependent on the experience in both field testing and interpretation. The signal characteristics and their interpretations of sonic integrity test on piles founded in Bangkok subsoil were reported by Thasnasipan et al. (1998b). The authors presented the interpretation of various features of sonic integrity test signals in comparison with soil investigation data and construction records.

6.3 Some findings from sonic integrity test in Bangkok

Toe reflections are generally clear for the piles with length to diameter ratio (L/D) within 40, but toe reflection was hardly detected for piles founded at depth deeper than 30-35m as reported by Thasnasipan (1998b). However, it was found that toe reflection is detectable for some deep-seated bored piles of about 50m in length if pile was tested at early age. Figure 2 shows the significant toe reflection of 52.0m-long pile tested few days after concreting. This is considered mainly due to the fact that at early stage the bonding between soil and pile has not yet fully developed so that damping effect or attenuation of the sonic signal by shaft friction is relatively low.

Use of temporary casing usually causes larger size at the top portion (from pile top to casing bottom) for completed piles in Bangkok subsoil. This is caused by the size difference between casing and the drilling tool (bucket/auger) as the top section of pile formed by the outside diameter of the casing is usually 5-10cm larger than the lower section of pile with nominal diameter formed in the borehole made by drilling tool (Figure 3). Test signal shown in Figure 3 indicates a clear reflection at approximate 11m from pile top (about 15m below ground level) with a repeat or multiple reflection at about 2 times of that distance (approximate 22m from pile top). As the polarity of the reflection is negative, the first interpretation is that this pile has decrease in cross section or crack at 15m below ground level. However, with the consideration of pile construction method, it is concluded that negative reflection is caused by variation in pile cross section at the bottom end of 15m long temporary casing. The sonic signals with anomaly acquired on those piles are often misinterpreted as indicative of defect piles, leading to an argument.

For the analysis of sonic integrity test, velocity of sonic wave in pile material (concrete) is typically assumed 4000m/s in most cases. Basically, lower strength of concrete gives lower wave propagation velocity. The trend of sonic wave velocity increment with pile age derived from a number of 600mm diameter bored piles with known lengths is presented in Figure 4.
7 SONIC LOGGING TEST

7.1 Overview of the test in Bangkok

Use of Sonic logging test for checking pile integrity has increased in Thailand in recent years. It is used for cast-in-place foundation elements such as bored piles, barrettes, diaphragm walls and caissons. Both PVC and steel tubes can be used as access tubes. Though cost of steel tube is higher than that of PVC, steel tubes are considered more suitable than PVC tubes for two reasons; (1) less potential for bending and damage especially for deep-seated bored piles and barrettes with heavy steel reinforcement and (2) better bonding with concrete. These access tubes are sometimes also used for pile base grouting. The access tubes are attached to the reinforcement cage, which is installed prior to concreting as a normal practice for cast-in-place piling.

Two access tubes are required as a minimum for sonic logging test. For a good coverage of the test pile, recommended number of tubes for different sizes of bored piles is shown in Table 2.

<table>
<thead>
<tr>
<th>Pile Diameter (mm)</th>
<th>Minimum No. of tubes</th>
<th>Tube spacing (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D ≤ 750</td>
<td>2</td>
<td>180</td>
</tr>
<tr>
<td>750 &lt; D ≤ 1000</td>
<td>2</td>
<td>120</td>
</tr>
<tr>
<td>1000 &lt; D ≤ 1500</td>
<td>4</td>
<td>90</td>
</tr>
<tr>
<td>1500 &lt; D ≤ 2500</td>
<td>6</td>
<td>60</td>
</tr>
<tr>
<td>2500 &lt; D</td>
<td>8</td>
<td>45</td>
</tr>
</tbody>
</table>

A typical problem encountered in sonic logging test is the access tube blockage which often comes to know only at the time sonic testing is conducted one week or more after pile construction. The blockage of access tube is mainly caused by intrusion of concrete from leakage at either bottom cap or tube connection and intrusion of soil or other material from the top cap of the tube. Sonic test instruments (transducer and receiver) sometimes may not be able to insert down to the bottom of the access tubes due to the bend of the tube itself. Though the problem associated with the access tube blockage can be overcome by drilling through the obstruction in the worst case (but not possible if tube is bent), it should be eliminated in the first place by careful fabrication, installation and protection of the tubes.

7.2 Interpretation of test results

The results of a series of laboratory tests to determine the effects of various defects or inhomogeneities within a concrete section were reported by Stain and Williams (1991). The test results were carried out on small panels constructed to model various pile construction defects and anomalies. The results from these panels were related to tests on control panels formed from homogeneous concrete.

Faiella and Superbo (1998) reported the analysis of the sonic logging test results of over 6800 piles collected from 37 sites in Italy. Based on the results of analyses, the authors presented the defect classification criteria for piles monitored by different number of tubes as illustrated in Tables 3 (a) and 3 (b).

Table 3 (a). Defect classification criteria monitored by means of 2 tubes (Faiella and Superbo, 1998)

<table>
<thead>
<tr>
<th>T_a/T_s</th>
<th>T_s/D</th>
<th>Type of defect</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;1.15</td>
<td>-</td>
<td>Non-homogeneous concrete</td>
</tr>
<tr>
<td>1.15-1.45</td>
<td>&gt;3</td>
<td>Light</td>
</tr>
<tr>
<td>1.45-2.0</td>
<td>&lt;1</td>
<td>Light</td>
</tr>
<tr>
<td>&gt;2.0</td>
<td>&lt;0.5</td>
<td>Probable serious</td>
</tr>
</tbody>
</table>

where:

- \( T_a \) = Travel time of sonic waves in the anomalous zone
- \( T_s \) = Travel time of sonic waves in the sound concrete
- \( T_a/D \) = Thickness of the anomalous zone
- \( P_a/P_t \) = Number of measurement paths affected by sonic anomalies
- \( P_t \) = Total number of travel paths for each pile
- \( D \) = Pile diameter

Table 3 (b). Defect classification criteria monitored by means of 3 and 4 tubes (Faiella and Superbo, 1998).

<table>
<thead>
<tr>
<th>T_a/T_s</th>
<th>P_a/P_t</th>
<th>T_s/D</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;1.15</td>
<td>-</td>
<td>Non-homogeneous concrete</td>
</tr>
<tr>
<td>1.15-1.45</td>
<td>&gt;3</td>
<td>Light</td>
</tr>
<tr>
<td>1.45-2.0</td>
<td>&lt;1</td>
<td>Non-homogeneous concrete</td>
</tr>
<tr>
<td>&gt;2.0</td>
<td>&lt;0.5</td>
<td>Probable serious</td>
</tr>
</tbody>
</table>

where:

- \( T_a \) = Travel time of sonic waves in the anomalous zone
- \( T_s \) = Travel time of sonic waves in the sound concrete
- \( T_a/D \) = Thickness of the anomalous zone
- \( P_a/P_t \) = Number of measurement paths affected by sonic anomalies
- \( P_t \) = Total number of travel paths for each pile
- \( D \) = Pile diameter
Srivanavit et al. (1999) reported the interpretation of test signals in comparison with the actual integrity of 9 model tests conducted in Bangkok. Model piles are of 800mm diameter with 1m length. The summary of the model test results including the description of model piles as well as discussions by authors are illustrated in Table 4.

Table 4. The summary of sonic logging model test results

<table>
<thead>
<tr>
<th>Case</th>
<th>Concrete &amp; material properties</th>
<th>Profile</th>
<th>Discussion of signal</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Standard good concrete pile with clean tubes and clean water.</td>
<td>![Image]</td>
<td>Sonic logging profile shows continuous signal, no anomaly.</td>
</tr>
<tr>
<td>2</td>
<td>Good concrete pile with dirty tubes and clean water.</td>
<td>![Image]</td>
<td>Sonic logging profile shows defect at pile head due to bleeding of concrete.</td>
</tr>
<tr>
<td>3</td>
<td>Good concrete pile with dirty tubes and dirty water (contaminated with bentonite slurry) filled.</td>
<td>![Image]</td>
<td>Sonic logging profile shows defect at pile head due to bleeding of concrete, similar to case 2.</td>
</tr>
<tr>
<td>4</td>
<td>Good concrete with Layer of bentonite slurry, sand, gravel fill (30cm. Thickness whole Section) at middle of the pile.</td>
<td>![Image]</td>
<td>Sonic logging profile shows signal loss from 0.4m to 0.7m due to delay of sonic wave.</td>
</tr>
<tr>
<td>5</td>
<td>Good concrete with Clay on surface of one tube (50 cm. from top of concrete).</td>
<td>![Image]</td>
<td>Sonic logging profile shows signal delay at 0.5m depth.</td>
</tr>
<tr>
<td>6</td>
<td>Concrete mixed with bentonite slurry.</td>
<td>![Image]</td>
<td>Sonic logging profile shows non-uniform signal, not clear signal due to poor concrete and inhomogeneous concrete.</td>
</tr>
<tr>
<td>7</td>
<td>Thin smear of 2mm. bentonite slurry on tubes (50 cm. from top of concrete).</td>
<td>![Image]</td>
<td>Sonic logging profile shows signal loss from depth 0-0.5m (more travel time).</td>
</tr>
<tr>
<td>8</td>
<td>Thick smear of 5mm. bentonite slurry on tubes (50 cm. from top of concrete).</td>
<td>![Image]</td>
<td>Sonic logging profile shows continuous signal loss and signal delay at depth 0-0.5m, similar to case 7.</td>
</tr>
<tr>
<td>9</td>
<td>Weak concrete (high w/c ratio).</td>
<td>![Image]</td>
<td>Though weak (low strength) concrete is used, sonic logging profile shows continuous signal, because of homogeneity of the tested media.</td>
</tr>
</tbody>
</table>

7.3 Factors to be considered in interpretation of sonic logging test

Turner (1997) pointed out the need of both theoretical knowledge and practical experience in interpretation of the test results in CIRIA Report 144. Suggestion is also made in this report that the anomaly shown in sonic test signals can be caused not only by changing in physical properties of the materials, but also by factors within the measuring system itself. These factors are:

- Free movement of the probes within access tubes
- Mismatched probe positions especially at pile toe
- Measurement resolution
- Incorrect position of access tube
- Air gaps or different material around access tubes
- Aggregate variation (in the case of base grouting)
It is to be noted from the author’s suggestion based on the above factors that only variations of transmit time more than 15 to 20% of the norm should be regarded as warranting further investigation.

8 HIGH STRAIN DYNAMIC LOAD TEST

8.1 Overview of the test in Bangkok

High strain dynamic integrity test has become a well-accepted method especially for evaluating the pile capacity in the foundation industry today. A large number of related technical papers and case histories of the test have been published and it is a part of standards and specifications such as ASTM D4945-89 (Standard Method for High-strain Dynamic Testing of Piles). In Bangkok, dynamic load test is applied for both driven and bored piles. A summary of the number of dynamic load test carried out during 1991 to 1997 is shown in Table 5. It is to be noted that quantities shown are collected from the available sources and the actual tested numbers are likely to be slightly more than those indicated in the table. As can be seen in the table, application of dynamic load test increases year by year in Thailand.

The pile driving analyzer (PDA) with computer software by Pile Dynamic INC, USA is mainly used in Bangkok.

Table 5. Yearly minimum quantity of dynamic load test conducted in Thailand during 1991 to 1997

<table>
<thead>
<tr>
<th>Year</th>
<th>No. of dynamic load test done</th>
</tr>
</thead>
<tbody>
<tr>
<td>1991</td>
<td>84</td>
</tr>
<tr>
<td>1992</td>
<td>134</td>
</tr>
<tr>
<td>1993</td>
<td>242</td>
</tr>
<tr>
<td>1994</td>
<td>369</td>
</tr>
<tr>
<td>1995</td>
<td>464</td>
</tr>
<tr>
<td>1996</td>
<td>473</td>
</tr>
<tr>
<td>1997</td>
<td>695</td>
</tr>
</tbody>
</table>

For driven pile it is usually performed at two stages, during driving (initial driving monitoring) and some period after installation of pile (restrike test). For bored piles, a specially designed pile cap is normally required as an integral part of the pile head to avoid pile head damage. Ram weights of 4 ton to 20 ton are commonly used in Bangkok. The common ram weights used for different pile sizes are shown in Table 6.

Table 6. Common ram weights used for different size of piles

<table>
<thead>
<tr>
<th>Pile size (m)</th>
<th>Ram weight (ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 0.50</td>
<td>4.00</td>
</tr>
<tr>
<td>0.50 – 0.80</td>
<td>8.00</td>
</tr>
<tr>
<td>1.00 – 1.50</td>
<td>20.00</td>
</tr>
</tbody>
</table>

The dynamic load test is also occasionally used as an additional investigation when pile is found with defect by other integrity tests. Figure 5 shows the photo of dynamic load test carried out on barrette of size (1.0x2.7m) with toe depth 48.94m located at the bank of water supply canal. A 20 ton hammer was used to activate the 2050 ton test load in this project.

Figure 5. Dynamic load test carried out on a barrette (1.0m x 2.7m x 48.94m).

8.2 Comparison and correlation of dynamic and static pile load test

A comparison between dynamic and static load test results has been reported by various researchers. Seidel and Rausche (1984) reported the results of dynamic and static load tests performed on drilled shafts of the West Gate Freeway in Melbourne, Australia. A 20 ton hammer with drop heights between 1.6 and 2.5m was used for the dynamic tests of 12 shafts ranging from 1100mm and 1500mm in diameter and 35m to 64m in length. The authors reported that dynamic activation of static pile resistance forces exceeded 3000 ton for some 1500mm diameter shafts. Skin friction predictions from dynamic load tests and values obtained from instrumented shafts under static load tests were remarkably similar and pile head load-movement relationships obtained from both test methods were comparable as reported by the authors.

Prebaharan et al. (1990) reported the results of dynamic and static tests conducted on bored piles at the Marina Bay Station of the Singapore Rapid Transit System. According to the report, dynamic load tests were carried out for bored piles of 1000mm diameter and 25m to 50m length founded in Old Alluvium of Singapore Island. Agreement between dynamic and static load tests results permitted replacement of 44 out of 51 tests with dynamic load test from originally planned static load tests as stated by the authors.

Vasivarthana and Kampananon (1997) presented the efficiency and reliability of dynamic load test carried out in Bangkok. Figures 6 (a) and 6 (b) show the load settlement curves of dynamic and static load tests conducted in Bangkok on large and small di-
ameter piles respectively. As can be seen in the figures, load-settlement characteristics obtained from dynamic load test agree well with those of static load test for both large and small diameter bored piles.

![Graph](image)

Figure 6 (a). Load-settlement curves of dynamic and static load tests on large diameter piles (φ 0.80m x 50.0m).

![Graph](image)

Figure 6 (b). Load-settlement curves of dynamic and static load tests on small diameter piles (φ 0.35m x 18.91m).

9 CONCLUSIONS

Use of sonic integrity, sonic logging and high strain dynamic tests in Bangkok has been presented.

Though sonic integrity test is a simple and cost effective method, the reliability of the test results is highly dependent on the experience of the person in both field-testing and interpretation.

The results from sonic logging test conducted on model piles in Bangkok helps to extend the knowledge of the signal characteristics and interpretation.

High strain dynamic test has become a well-accepted method especially for load testing of piles. Agreement between static and dynamic load test results enhances the confidence in using the dynamic load test as shown by rapid increment in number of tests conducted in seven years in Thailand, mainly in Bangkok.

ACKNOWLEDGEMENT

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REFERENCES


Problems of Pile Foundations Building

Damages to piles associated with excavation works in Bangkok soft clay

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Seaeco Company Limited Bangkok, Thailand

ISBN 5-88151-161-1
ABSTRACT

In Bangkok, Thailand, pile foundations are commonly used to support all type of structures. The excavation for construction of pile caps and mat foundations are carried out mostly in the soft clay layer. Sheet pile walls with temporary bracing systems are commonly used for excavation works (depth ranging form 4.0-11.3m) in the built-up areas of central and outskirts of Bangkok metropolis. Sometimes open cut excavation is adopted for shallow excavation (2.0-6.0m) in the areas where no buildings exist in the close vicinity of the site. Lateral movements of soils take place in such excavations causing pile deviation and cracks to the piles, especially located in the vicinity of boundary of excavation. Such defects have been found more frequently in smaller diameter piles than the larger diameter piles. These tension cracks in piles are commonly observed above the level of interface of soft and stiff clay layers where excavation induced bending moment in pile exceeds the pile’s cracking moment capacity. Sonic integrity testing is usually employed to test the piles’ integrity. High strain dynamic load test is sometimes employed to check the performance of piles with such defects. This paper presents four cases in which damages caused to piles by soil movement associated with excavation works. Pile damages, especially tension cracks indicated by integrity testing, analysis on stresses in pile due to the soil movements and behavior of retaining systems are also examined. Correlation between location of crack and bending moment in pile which exceeds the pile cracking moment capacity also made.

KEYWORDS

Pile deviation, Lateral movement, Sonic integrity test, Dynamic load test, Pile damage, Bending moment, Cracking moment
INTRODUCTION

Almost every structure in Bangkok sits on pile foundations. Excavation works for preparing the pile cap, footing and/or mat are usually within the soft clay commonly found in Bangkok. However for structures requiring many basement levels and underground facilities, the excavation works are undertaken to below the soft clay more than 20m from the ground. Various soil retaining systems are adopted in such excavation works to suit the site and soil conditions and depth of excavation. Analysis and design for the retaining systems sometimes are overlooked by the practicing engineers for the possible movements, both vertical and lateral, of soil adjacent to the excavation boundaries. Moreover monitoring of soil movements, with exception of existing structure near by, is often ignored. Such ignorance leads to damage the piles already constructed or installed in soft soil stratum. As a result secondary defects found in the piles after completion of piling work, including pile top deviations far from allowable tolerance become an argument between the piling contractor, excavation subcontractor and the employer or supervising engineer.

PILE FOUNDATIONS

Both bored and driven piles are commonly used for foundations of structures in Bangkok subsoil. The subsoil generally consist of a 12-18m thick very soft clay layer overlying medium to stiff clay layer. A sand layer occurs at depth of 20m to 35m below the stiff clay layer. A second layer of dense sand is generally found below 50m where the toe of deep bored piles are founded for high rise or heavy structures. Stiff to hard clay layers occurs intermittently between the sand layers. Large diameter bored piles having diameter of 800mm to 1500mm are extended down to 35m to 60m respectively being embedded in the sand layers. For low rise buildings driven or small diameter bored pile of 35mm to 600mm are commonly used with footing for pile groups or for individual piles. The length of pile is generally to 20 to 28m embedded in stiff clay or first sand layer.

The working load capacity of bored piles is usually limited by the allowable stress of the concrete. Cutoff level of such piles generally varies from about 2.0m to 6.0m below ground surface. For the structures with deep basements, bored piles are trimmed down to 20.0m.

Generally bored piles are constructed with reinforcement of 0.5-1.2% of sectional area of pile and concrete having cube strength of 30-32 MPa. Wet process, using bentonite slurry is normally employed for drilling in water-bearing sand layers and concrete pouring is done under the slurry. Dry process normally used for construction of small diameter bored piles with depth not greater than 26m.

TYPES OF EXCAVATIONS

Common type of excavation found in Bangkok subsoil are tabulated as below,

<table>
<thead>
<tr>
<th>Type of Excavation</th>
<th>Excavation below G.L. (m)</th>
<th>Site Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cut Slope</td>
<td>3.0-6.0</td>
<td>open area, no adjacent buildings</td>
</tr>
<tr>
<td>Sheet Piles Wall</td>
<td>4.0 to 11.3</td>
<td>adequate site clearance</td>
</tr>
<tr>
<td>Jet Grouted Wall</td>
<td>5.0 to 10.5</td>
<td>adequate site clearance</td>
</tr>
<tr>
<td>Secant or Contiguous Pile Wall</td>
<td>4.0 to 20.5m</td>
<td>in built-up area</td>
</tr>
<tr>
<td>Diaphragm Wall</td>
<td>7.0 to 23.0m</td>
<td>in built-up area</td>
</tr>
</tbody>
</table>

First two types of excavations are found to induce higher lateral and vertical soil movements compared to the remaining types (Ref. 2) and sometimes cause a localize failure of the soil mass retained due to improper construction practice. In this paper pile damage associated with the first two types in four cases are studied and presented.

INTEGRITY OF PILES

Sonic integrity test is usually employed to test the integrity of piles after piles have been trimmed to the design cutoff level. Generally a minimum of 10% to a maximum of 100% of piles are tested. A sonic integrity tester with built-in computer having high quality signal acquisition system has been commonly used for pile integrity testing. However, severity of the defect/crack or any irregularity in piles is usually determined comparatively based on magnitude of reflection amplitude of individual piles from such a defect/crack or irregularity, with reference to the input pulse. Figure shows the sonic test records of a good pile and piles with crack in different severity from Cases IV.

CASE I - CUT SLOPE (Bangna-Trad Highway KM. 26)

Subsoil Condition - A 17m thick soft clay layer occurs with unconfined average shear strength (Su), ranging from 0.69 t/m² at the top to 2.20 t/m² at the bottom of the layer. The field vane shear test indicates in-situ shear strength of that layer is 1.40t/m² to 3.80t/m² with sensitivity of 2.7 to 3.7. The natural water content of soft clay layer exceeds 100%. Below the soft clay layer is medium clay layer with Su value of 4.0 t/m² to 6.0t/m². Then a stiff clay layer having Su value of 13.0t/m² is found at the depth of 21m-22m overlying the first sand layer which extends to the depth of about 34m to 37m. The second sand layer occurs at the
depths of about 52.0m - 61.0m. Between sand layers a thick hard clay layer is present, but sporadically.

Pile foundation - A total of 428 concrete bored piles of diameters 1.0m, 1.2m and 1.5m were installed with pile tip at 53m to 65m below the ground, being embedded in second dense sand layer to support a high rise structure. Cutoff level of piles ranges from 4.6m to 5.9m in depth. Entire length of piles were fully reinforced with steel reinforcement of 1% and 0.5% of pile sectional area in top 33m and lower section of pile shaft respectively.

Soil Retaining System - Cut slope excavation with two levels of 1:3 slope and a berm was adopted to achieve the maximum excavation to -5.9m below the existing ground level as the site is a large open space being located outside built-up area. Slope stability analysis using effective stress soil parameters indicated that a safety factor of 1.2 for temporary condition can be achieved by driving 6m long I-12 RC soldier piles at a spacing of 1.5m at the toe of the bottom slope. Moreover, the ground level around the perimeter of excavation was reduced down to 0.5m. Excavation was carried out on temporary working platforms supported by 24m long steel king posts (H300x300) driven into the ground.

Pile Damages - All piles were tested with sonic integrity test. Test results are summarized in the Table below;

<table>
<thead>
<tr>
<th>Pile Dia. (m)</th>
<th>Least Prominent Crack</th>
<th>Less Prominent Crack</th>
<th>Prominent Crack</th>
<th>Depth below GL(m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>4</td>
<td>8</td>
<td>7</td>
<td>6.0-13.8</td>
</tr>
<tr>
<td>1.2</td>
<td>10</td>
<td>2</td>
<td></td>
<td>6.1-7.6</td>
</tr>
<tr>
<td>1.5</td>
<td>1</td>
<td></td>
<td></td>
<td>9.9</td>
</tr>
</tbody>
</table>

Most piles with large crack detected were located in the outer row of foundation piles (Fig. 1) and located under the traffic ramps of temporary working platform. It was found that the crack in these piles were caused by localized soil sliding which was occurred by the heavy traffic load. Pile deviations in order of 50cm to 100cm towards excavation were observed.

An inspection after the soil sliding revealed that king posts were not installed to support parts of the ramps behind the slope on the ground. Apart from the traffic ramp area, a number of localized soil slidings up to 3m to 4m occurred in some areas. They are found to be caused by loose materials filled in bored hole above pile casting level during casting. Less Prominent crack detected in some piles, particularly within excavation zones are considered to be caused by improper pile trimming method. A backhoe was used to break off the over cast section by hitting the pile head after chipping of the concrete cover at the cutoff level around the pile shaft and separating the dowel bars from the overcast section.

Soil around the suspected pile with prominent crack were excavated down to detected crack level. These piles were trimmed down to below the crack and re-constructed to the cut off level. Tie beams were provided for the piles with crack and horizontal deviation more than allowable tolerance.

CASE II - SHEETPILE EXCAVATION SUPPORTED BY SOIL BERM (Huay Kwang Area)

Subsoil Condition - A 12m thick soft clay layer occurs with unconfined average shear strength (Su), ranging from 1.0 t/m² at the top to 2.50 t/m² at the bottom of the layer. The natural water content of soft clay layer is about 60-80%. Below the soft clay layer is medium clay layer with Su value of 2.5 t/m² to 3.5t/m². Then a stiff clay layer with traces of fine sand having SPT N value of 15 to 37 is found at the depth of about 18m-overlying sand layers which occur below 27m in depth.

Pile foundation - A total of 904 concrete bored piles of diameters 1.0m and 1.5m were installed with pile tip at about 59m below the ground, being embedded in dense sand layer to support a total of eight high rise buildings located at the particular size. Cutoff level of bored piles ranges from 5.1m to 8.1m in depth. Entire length of piles are fully reinforced with steel reinforcement of 0.72% of pile sectional area for the top and reduced gradually to 0.23% for bottom section of pile shaft. A total of 88 pre-cast concrete (I-30) piles with 21m in length was also driven to support the underground water tanks. The cutoff level of the pre-cast piles was 3.1m below the ground level.

Soil Retaining System - The foundation structure is about 10m to 20m away from the adjacent property boundaries. Initially excavation with cantilever sheet pile wall (FSP III
with 14m in depth) supported by a soil berm or slope was adopted to achieve the maximum excavation to -8.10m below the existing ground level. However after excavation to some depths, due to sheet pile failure at some locations and excessive vertical and lateral soil movements, temporary raking struts were then immediately installed. Tension cracks wider than 300mm were observed on the ground about 4.0m away from the excavation zone. Minor damages also occurred to the adjacent properties due to soil movement.

**Pile Damages** - More than 50 % of piles were tested with sonic integrity test.

**Table 3** Summary of Pile damage Detected by Sonic Integrity Test - Case II.

<table>
<thead>
<tr>
<th>Pile Dia. (m)</th>
<th>No. of Piles with Defect</th>
<th>Least Prominent Crack</th>
<th>Less Prominent Crack</th>
<th>Prominent Crack</th>
<th>Depth below GL (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>1</td>
<td>23</td>
<td>4</td>
<td>13-16.6</td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>-</td>
<td>1</td>
<td>1</td>
<td>15.6</td>
<td></td>
</tr>
</tbody>
</table>

Large numbers of piles with crack detected were located in the periphery of the excavation zone (within the soil berm).

**Figure 2**. Location of Damaged Piles (only a quarter of Project Site shown here) Case II

Figure 2 shows a portion of the piling plan affected by soil movements. Pile deviations of up to 600mm, particularly in pile with prominent crack towards excavation were observed.

Bored piles with crack were cored to the depth below the crack and grouted. High strain dynamic load test was carried out on two piles suspected with crack and large horizontal deviation after remedial work. The dynamic test results indicate that these piles could be capable of carrying the design load with factor of safety higher than 1.5.

**Figure 2.1**. A Typical Scene showing excavation works Case II

**CASE III SHEETPILE EXCAVATION WITH ONE LEVEL TEMPORARY SUPPORT (Yannawa)**

**Subsoil Condition** - In this project site, a 14-15m thick soft clay layer occurs with unconfined shear strength (Su) ranging from 1.0 t/m² to 2.00 t/m² at the top and bottom of the layer respectively. Below is stiff to very stiff clay layers extend down to about 42m in depth. The subsoil condition below 36m from the ground level is variable, having a 13m thick sand layer sporadically. A stiff to hard dark grey clay is found at the depth between 40m and 49m. Generally a thick dense sand layer occurs below 50m in depth.

**Pile foundation** - A total of 402 concrete bored piles of diameters 1.0m and 1.5m were installed with pile tip at about 55m below the ground, being embedded in dense sand layer to support a building structure. Cutoff level of bored piles ranges from 0.75m to 7.80m in depth. Entire length of all piles are fully reinforced with steel reinforcement of 0.5% of pile sectional area.

**Soil Retaining System** - Due to different cutoff levels of piles, sheet pile (14m) with one level of temporary bracing at about 1m below the ground level was used for each excavation zone. Figure 3 shows a portion of pile layout which includes an excavation zone with braced sheet pile wall. Prior to installation of temporary struts, excavation was carried out to about 2.5 to 3m in depth. The over excavation without support caused excessive movements of soil and sheet pile wall along grid line C10 (Fig.3). Deflected sheet piles were re-aligned and struts were installed immediately with pre-loading.

**Pile Damages** - All piles were tested with sonic integrity test after pile were trimmed to the designed cutoff level. Piles with crack detected were mostly located in the vicinity of the excavation boundary (Fig. 3 and Ref. 6). Moreover, lateral pile deviation in a range of 100-840mm towards excavation were also observed in those piles located in the vicinity of the sheet pile wall (Fig. 3).
Table 4. Summary of Pile damage Detected by Sonic Integrity Test - Case III.

<table>
<thead>
<tr>
<th>Pile Dia. (m)</th>
<th>Least Prominent</th>
<th>Less Prominent</th>
<th>Prominent</th>
<th>Defect Depth below GL (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>-</td>
<td>-</td>
<td>5</td>
<td>7.5-9.4</td>
</tr>
<tr>
<td>1.5</td>
<td>-</td>
<td>-</td>
<td>14</td>
<td>8.4-21.0</td>
</tr>
</tbody>
</table>

Damaged piles were cored to the depth below the crack and grouted. Modification of reinforcement for the mat was done for defect piles with large horizontal deviation.

CASE IV - SHEETPILE EXCAVATION WITH TWO LEVEL TEMPORARY SUPPORT (Sukhumvit Area)

Subsoil Condition - A 14-15m thick soft clay layer occurs with unconfined shear strength ($S_u$), ranging from 1.0 t/m² at the top to 3.00 t/m² at the bottom of the layer. The natural water content of soft clay layer is generally about 37 - 87%. Below is very thick stiff to very hard clay layers interbedding with a 3m thick medium clay layer at the depth of 19m.

Pile foundation - A total of 143 concrete bored piles of 0.6m diameter were installed with pile tip at about 26m below the ground to support a residential apartment. Dry process was employed to construct the piles. Cutoff level of bored piles ranges from 0.85m to 5.95m in depth. Entire length of all piles are fully reinforced with steel reinforcement of 0.35% of pile sectional area.

Soil Retaining System - Type FSP III sheet pile (14m) with two level of temporary bracing at about 1.0m and 3.5m below the ground level was used. Figure 4 shows a portion of pile layout which includes an excavation zone with braced sheet pile wall.

Five bored piles with crack suspected were cored to verify the crack and grouted later. High strain dynamic load test was also carried on a total of 25 piles in which 24 piles with crack and 1 pile without crack. Summary of dynamic load test results is presented in the Table 5.

Table 5. Summary of Pile damage Detected by Sonic Integrity Test - Case IV.

<table>
<thead>
<tr>
<th>Pile Dia. (m)</th>
<th>Least Prominent Crack</th>
<th>Less Prominent Crack</th>
<th>Prominent Crack</th>
<th>Defect Depth below GL (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>-</td>
<td>38</td>
<td>33</td>
<td>6.5-13.8</td>
</tr>
</tbody>
</table>

Figure 5. Sonic Integrity Test Records showing Good pile, Pile with less Prominent Crack and Prominent Crack respectively - Case IV.

Table 6. Dynamic Load Test Results of Damaged Piles - Case IV

<table>
<thead>
<tr>
<th>Mobilized Capacity/Design Ultimate Capacity of 240 ton</th>
<th>No. of Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 50%</td>
<td>1</td>
</tr>
<tr>
<td>less than 10%</td>
<td>2</td>
</tr>
<tr>
<td>More than 100%</td>
<td>22</td>
</tr>
</tbody>
</table>
The results indicate that bending moments in piles induced by excavation are higher than the cracking moment capacity of the piles in all cases. Summary of analysis is presented in the Table below;

<table>
<thead>
<tr>
<th>Case</th>
<th>Pile Size (m)</th>
<th>Rebar (%)</th>
<th>Cracking Moment (t.m)</th>
<th>Ultimate Moment (Whitney) (t.m)</th>
<th>Moment by FEM Model (t.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1.0</td>
<td>1.0</td>
<td>30</td>
<td>100</td>
<td>86.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>109.5*</td>
</tr>
<tr>
<td>II</td>
<td>1.0</td>
<td>0.75</td>
<td>30</td>
<td>79</td>
<td>149.7</td>
</tr>
<tr>
<td>III</td>
<td>1.5</td>
<td>0.5</td>
<td>102</td>
<td>188</td>
<td>117.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>255.7*</td>
</tr>
<tr>
<td>IV</td>
<td>0.6</td>
<td>0.35</td>
<td>6.5</td>
<td>9.2</td>
<td>13.3</td>
</tr>
</tbody>
</table>

Note: (*) moment in pile located in active side of soil mass either outside of sheet pile wall or cut slope.

In Cases I and III, piles located in the active side of soil mass are found to be under the bending stress about 10% and 36% higher than the ultimate bending capacity of piles respectively.

In Case II, the computed bending moments in the pile closest to the sheet pile wall and piles in second row is 89% and 27% respectively higher than the ultimate bending
capacity of piles. The sonic integrity test also detected presence of crack in 35% of the piles which are located in first two outer rows (see Fig. 2).

In Case IV, the modeling results show that excavation induced bending moments in pile exceed the ultimate moment capacity of piles. This is confirmed by the observed pile damages (see Fig. 5). Moreover the model results suggest that these piles are subject to some degrees of basal heave due to flexibility of and soil condition at embedment of sheet pile wall. Sonic integrity test results shows presence of more than one crack in some piles at depths, suggesting piles would have experienced combined stresses. Comparison between Case III and Case IV illustrates that cracks are found more frequently in small piles than large piles in a case of soil movements due to excavation.

The crack location in piles indicated by sonic integrity test generally is found to be consistent with the location of computed bending moment which exceeds the cracking moment capacity of piles for all cases.

**Table 7. Locations of Crack and Computed Bending Moments in Piles in the Vicinity of Excavation Boundary**

<table>
<thead>
<tr>
<th>Case</th>
<th>Integrity Test Depth of Crack in Piles near Excavation Boundary (m)</th>
<th>Depth of Max. B.M by FEM (m)</th>
<th>Depth of B.M exceeding Pile’s Cracking Moment (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>6.0 &amp; 13.8</td>
<td>19.5</td>
<td>13.5</td>
</tr>
<tr>
<td>II</td>
<td>12.7 &amp; 13.7</td>
<td>18.0</td>
<td>11.0 &amp; 13.0</td>
</tr>
<tr>
<td>III</td>
<td>12.4 &amp; 19.0</td>
<td>15.0</td>
<td>12.8 &amp; 14.5</td>
</tr>
<tr>
<td>IV</td>
<td>11.5 &amp; 12.0</td>
<td>15.5</td>
<td>13.0 &amp; 14.0</td>
</tr>
</tbody>
</table>

FEM analysis indicates the maximum bending moment in piles occurs at the level close to the interface between soft clay and stiff clay layers. For lateral movements of piles and soil, a direct comparison can not be made between model and observed data as the accurate field monitoring data are not available. Performance of soil retaining systems by model analysis are summarized in the Table below;

**Table 8. Summary of Lateral Movements of Retaining System**

<table>
<thead>
<tr>
<th>Case</th>
<th>Type of Retaining System</th>
<th>Level of Strut</th>
<th>Excav. Depth (m)</th>
<th>Max. Lateral Movement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Cut Slope</td>
<td>-</td>
<td>5.4</td>
<td>117</td>
</tr>
<tr>
<td>II</td>
<td>Sheet Pile</td>
<td>1</td>
<td>5.7</td>
<td>175</td>
</tr>
<tr>
<td>III</td>
<td>Sheet Pile</td>
<td>1</td>
<td>8.0</td>
<td>56</td>
</tr>
<tr>
<td>IV</td>
<td>Sheet Pile</td>
<td>2</td>
<td>6.0</td>
<td>52</td>
</tr>
</tbody>
</table>

For Case III, model analysis also confirmed that without temporary bracing, sheet pile would fail when soil berm is removed for trimming pile adjacent to the wall.

Instrumentation such as installation of inclinometer and strain gauges in piles and active side of soil mass is suggested for the future projects to analyze behaviour and performance of piles subject to lateral loads induced by excavation.

**CONCLUSION**

This paper presents four case studies on bored piles damaged by excavation induced lateral soil movements in Bangkok soft clay layer. Performance and response of pile are studied by FEM modeling and confirmed with pile integrity test results. Causes of pile damages found are also discussed.

In addition to FEM analysis, the followings were found from the actual site construction activities,

1. Over excavation prior to installation of support
2. Inadequate retaining system which can effectively control soil movement.
3. Inadequate support for construction traffic load (Ref. 4)
4. Improper pile trimming method (Ref. 4)
5. Inadequate pile design for lateral load during construction
6. Inadequate reinforcement in piles to resist soil heave
In designing of bored piles, the above mentioned causes, including deviation of piles due to excavation induced soil movement are to be taken into account.

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REFERENCES


Quantifying Pile Head Condition before Basement Excavation by Cross-Hole Sonic Logging Tests

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Natamon Kampanananon

STS Instruments Co., Ltd.
QUANTIFYING PILE HEAD CONDITION BEFORE BASEMENT EXCAVATION BY CROSS-HOLE SONIC LOGGING TESTS

Narong Thasnanipan¹, Zaw Zaw Aye², Thayanan Boonyarak³ and Natamon Kampananon⁴

ABSTRACT

In wet process bored piles construction, it is necessary that concrete is continued to be placed until good quality of concrete is formed above the design pile cut-off level to avoid contamination of slurry within the design pile length. Controlling the excessive overcast length to maintain the appropriate pile top level is one of the major problems particularly for the piles with low cut-off level. It is also impossible to check the quality of concrete at the top part of the piles prior to exposing the pile head for pile cap construction or basement excavation. If the pile head condition of recently cast bored piles could be justified in the initial stage of the project, it would be better and easier for the piling engineers to efficiently decide the optimum overcast length of the pile which in turn would provide effective saving of unnecessary concrete consumption. Application of sonic logging tests in justifying overcast pile length is demonstrated in this paper.

Keywords: cross-hole sonic logging test, wet-processed bored piles, cut-off level, contaminated concrete, pile integrity

1. INTRODUCTION

The first sonic logging test in Thailand was believed to be conducted in 1982 for the wet-processed bored piles of the Memorial Bridge Project where bad concrete zones were detected at two levels, depth about 20m and 1m from the pile top as reported by Ng (1983). In the past, sonic logging test was only employed as a pre-planed site quality control testing for large infrastructure projects in Bangkok. The primary reason of using sonic logging test in only large scale projects was due to its relatively expensive cost since cost of testing the entire length of deep-seated bored piles are high – higher costs involved in installation of lengthy access tubes and the test itself. With the passage of time, the application of this quality control test is being extended. As interpretation skills relevant to local soil condition and construction method of this test have been significantly improved in local industry, the piling engineers begins to use sonic logging tests for further assessment of constructed bored piles.

2. WET-PROCESSED BORED PILE CONSTRUCTION METHOD

Due to the prevailing subsoil and groundwater conditions, deep-seated bored piles of toe depth over 24m are constructed by wet-process or slurry displacement method. In the rotary drilling method, a temporary casing of appropriate length (12 to 18m is commonly used in Bangkok depending on the thickness of soft clay) with required diameter (internal diameter not less than that of design bored pile diameter) is first installed to ensure the stability of the borehole in the top soft or loose soil layers.

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Drilling is commenced by auger to drill out the soil inside the temporary casing. Auger-drilling is commonly continued up to the top of the first water-bearing sand layer or bottom of the casing. Drilling slurry or supporting fluid is then supplied to the borehole and drilling is proceeded with a bucket down to the design final depth of the pile. Before lowering the reinforcement cage, a special cleaning bucket is used to clean the base of borehole. If bentonite slurry is used, recycling method by air-lift or pump is applied as the base cleaning process. Reinforcement cages are then lowered into the borehole and concreting is carried out by tremie method.

3. CLASSIFICATION OF THE SONIC LOGGING TEST ANOMALY

Based on the experience of testing specialist on over 3000 deep-seated bored piles in past 15 years in Thailand coupled with the comparison between sonic logging test results and direct examination from cored samples of zones detected with anomaly, the category and criteria for interpretation of cross-hole sonic logging test results have been established as shown in Table 1. Figure 1 depicted the sonic signal profile of 4 categories classified in Table 1.

Table 1. Category of sonic logging signals and criteria

<table>
<thead>
<tr>
<th>Category</th>
<th>Criteria</th>
<th>Integrity</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>No or slight increase in transmission time not more than 10%</td>
<td>Homogeneous concrete</td>
</tr>
<tr>
<td>B</td>
<td>Minor increase in transmission time &gt;10% but &lt;25%</td>
<td>Minor anomaly</td>
</tr>
<tr>
<td>C</td>
<td>Transmission time more than Category B but transmission time can still be defined</td>
<td>Major anomaly</td>
</tr>
<tr>
<td>D</td>
<td>Transmission time can not be defined</td>
<td>Defect</td>
</tr>
</tbody>
</table>

Figure 1. Sonic profile showing different category (A, B, C and D) as classified in Table 1

4. PILE HEAD CASTING LEVEL OF WET-PROCESSED BORED PILES IN BANGKOK

In order to ensure the sound concrete at the specified pile cut-off level, wet-process bored piles are commonly cast well above the cut-off level. Though most of the piling engineers wish to minimize the overcast length, their primary concern is a formation of contaminated concrete at the design pile cut-off level which would need the piling contractor to come back to the site to carry out the remedial work while the superstructure contractor has commenced his work. There could be a claim from the superstructure
contractor for having delay to rectify the imperfect pile heads. Hence, most of the bored piling engineers prefer to cast the pile significantly higher than specified pile top level as shown in Figure 2.

Figure 2. View of exposed diameter 1.20m bored piles for mat foundation - Most of the piles were constructed with excessive overcast length (over 4m above design pile cut-off level)

The need of using the long temporary casing in Bangkok subsoil also makes it difficult to maintain the top of overcast at the desired level. Table 2 shows the casting tolerance above cut-off levels according to The Institution of Civil Engineers (1996) United Kingdom.

Table 2. Casting tolerance above cut-off levels for wet-processed bored piles (ICE, UK, 1996)

<table>
<thead>
<tr>
<th>Cut-off level below commencing surface, H, m</th>
<th>Casting tolerance above cut-off level, m</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.15 – 10.00</td>
<td>1.0 + H/12 + C/8 where C = length of temporary casing below the commencing surface</td>
<td>Piles cast under water or support fluid</td>
</tr>
</tbody>
</table>

5. COMMON DEPTH OF ANOMALY FORMED BY CONTAMINATED CONCRETE IN BORED PILES

As construction method and quality control process for wet process bored piles are being well established in local construction industry of Thailand, defect (discontinuity or contaminated concrete) in lower part of pile shaft rarely occurs in these days. Hence investigating the top part of pile is more important than testing the entire length of the deep-seated piles in Bangkok subsoil. Moreover, if sonic access tubes are installed only in the top part of bored piles, the costs associated with both the material and test process itself would be significantly reduced and integrity testing by sonic logging method would be more feasible. According to the authors’ experience from over 5,000 exposed bored piles in Bangkok subsoil, to achieve good pile head condition within design cut-off level, bored piles constructed under polymer-based slurry require shorter overcast lengths than those constructed under pure bentonite slurry. In authors’ opinion, the tolerable overcast lengths of bored piles constructed under polymer-based slurry should be less than those specified in Table 2. Two cases were studied to determine the common depth of significant anomaly (Category C or D) of bored piles in Bangkok subsoil.

5.1 Case 1

According to the sonic logging test results obtained from 400 piles of diameter 1.50m with depth over 50m constructed under polymer-based slurry in Bangkok, only 2 piles (0.5% of total contract piles) were detected with C Category in the design pile length. The level of the detected C category for these 2 piles
was found to be within 2m below cut-off level of the pile. The overcast length above cut-off level of 95% of piles were within 2.5m.

5.2 Case 2

The sonic logging test conducted on 174 bored piles of diameter 1.0m with depth over 50m constructed under polymer-based slurry reveals that no pile was detected with C category below the cut-off level. The overcast length of 76% of piles were within 2.2m above design cut-off level.

Table 3. Summary of sonic test results from 2 cases – bored piles constructed under polymer-based slurry in Bangkok

<table>
<thead>
<tr>
<th>Case</th>
<th>Total no. of piles</th>
<th>Pile diameter / depth</th>
<th>Number of piles detected with C category below design cut-off level by sonic logging test</th>
<th>Maximum overcast length of majority of pile from sonic logging test result</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>(pile)</td>
<td>(%)</td>
</tr>
<tr>
<td>Case 1</td>
<td>400</td>
<td>1.50m / 52m</td>
<td>2</td>
<td>0.5</td>
</tr>
<tr>
<td>Case 2</td>
<td>174</td>
<td>1.0 m / 50m</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

This finding suggests that contaminated concrete rarely occur below the cut-off level if overcast length of the pile is about 2.5m for polymer-based bored piles. Quality control of the construction process such as slurry mixing process and composition, properties of slurry, base cleaning, maintaining adequate tremie length, concrete quality and concreting procedure however must be according to the standard practice to achieve this result. Practical application of polymer-based slurry for deep-seated wet-processed bored piles in multi-layered soil of Bangkok has been reported by Thasanipan et. al. (2003).

Figure 3. View of polymer-based bored pile diameter 1.8m with overcast length about 2m – contaminated concrete was found only at the pile top

6. QUANTIFYING THE PILE HEAD CONDITION BY SONIC LOGGING TEST PRIOR TO BASEMENT EXCAVATION

Designs of the modern buildings in Bangkok frequently call for the basement facility which require design cut-off level of the piles well below the existing ground level. In wet process bored piles construction, it is necessary that concrete is continued to be placed until good quality of concrete is formed above the design pile cut-off level to avoid contamination of slurry within the design pile length. Controlling the excessive overcast length to maintain the appropriate pile top level is one of the most difficult task particularly for the piles with low cut-off level. Checking the condition of concrete at the top part of the piles prior to exposing the pile head for pile cap construction or basement excavation by direct inspection is impossible.
If the pile head condition of recently cast bored piles could be justified in the initial stage of the project, it would be better and easier for the piling engineers to efficiently decide the optimum overcast length of the pile which in turn would provide effective saving of unnecessary concrete consumption. Cross-hole sonic logging test as an early indicator may be the ideal method to check the optimum overcast length of recently constructed bored piles in controlling the excessive overcast length.

Sonic logging test could also be used as a performance-indicator for the piling contractor. In one major project, during the excavation works for 8 to 10m deep basement construction, the main contractor claimed that over 50% of exposed bored piles from total 174 contract piles were found to be with excessive overcast length (over 4 to 6m from design pile cut-off level). All bored piles in this project were constructed under polymer-based slurry and sonic logging test was conducted on every pile. The piling contractor carefully reviewed all the sonic logging test results and found the followings;

- 76% of piles were within 2.2m overcast length
- 18% of pile were with overcast length between 2.2m and 3m
- Only 10 piles (5%) exceeded overcast length of 3m.

Over 4m to 6m overcast length claimed by the main contractor was caused by the solid-like mixture of backfill material mixed with highly contaminated concrete at the top of the piles as boreholes were backfilled with crushed-rocks. Figure 4 shows the typical sonic logging test results of one of the piles claimed by the main contractor as excessively overcast piles. As can be seen in the figure, homogenous concrete was detected only up to 2m above design cut-off level whereas non-homogeneous (Category C or D) was observed above 2m. Field observation also confirmed that majority of piles were with highly contaminated at top 2m above design cut-off level. The main contractor finally accepted the proof of sonic logging test result to justify the overcast length.

Figure 4. Typical sonic logging test profile of bored pile claimed as excessively overcast (overcast length 4.4m above cut-off level) by the main contractor

7. RECOMMENDED PROCEDURE OF QUANTIFYING THE PILE HEAD CONDITION BY SONIC LOGGING TEST

As presented in earlier section, if the construction method strictly follows the quality assurance and control procedure (QA & QC), contaminated concrete is rarely formed within the design bored pile length. It should be sufficient to test the pile integrity of the top 10 to 15m by sonic logging test so that cost associated with both access tubes and the test itself can be significantly reduced. It is also not necessary to cast the concrete with excessive overcast length as most of the modern bored piles are constructed under polymer-based slurry. Followings are the recommended process in minimizing the formation of contaminated concrete within the design pile length for wet-processed bored piles in Bangkok subsoil.
• Carefully check and use the required temporary casing length considering the thickness of soft clay layer (common thickness of Bangkok soft clay layer is 15 to 18m)
• Pre-install the sonic logging access tubes covering only the top 15m for majority of the piles. Small percentage of piles may have full length of sonic access tubes to the pile tip to randomly check the integrity of the entire pile shaft
• Check the borehole profile of each pile by drilling monitor (koden equipment) to roughly assess the required concrete volume (to check the size and shape of borehole by drilling monitor to estimate required concrete volume)
• Strictly follow and control the slurry mixing process, slurry properties and quality check test
• Use appropriate concrete mix with good quality and strictly follow and control the tremie concreting process
• Carefully withdraw the temporary casing
• Construct the pile of higher design cut-off level first (if possible) and perform the sonic logging test 10 days after concreting
• If possible (for high cut-off level piles or cut-off level within 3m below existing ground) excavate with sufficient support to visually inspect the pile head and compare with sonic logging test. This process may not be necessary though it will be very useful to assess the sonic logging test result to correlate with actual condition of pile top at early stage of pile construction
• The piling engineer must carefully review the borehole profile, actually poured concrete volume and sonic logging test of each completed pile to assess and justify the construction method, site characteristic and quality assurance procedure for further improvement

8. CONCLUSION

Cross-hole sonic logging test is a useful quality control test to examine the integrity of the wet-process bored piles in early stage of the project. Discontinuity in the form of contaminated concrete in the lower part of pile shaft rarely occurs in these days as construction method and quality control process for wet process bored piles have been well established in local construction industry in Thailand,. Therefore, it is recommended that sonic logging test should be mainly used to investigate the integrity of the pile at the top portion and a random check is performed only for few piles to examine the integrity of entire pile shaft. An extended application of cross-hole sonic logging test in quantifying pile head condition prior to exposing the pile has been presented. The piling engineers should pay more attention to control the excessive overcast length to minimize the cost associated with cutting extra length of pile head and disposal of the waste. The piling engineers should be trained to effectively apply the sonic logging method not only as integrity-proof test but also as a tool to quantify the pile head condition at the early stage of the piling work.

REFERENCES

Institution of Civil Engineers, 1996. Specifications for piling and embedded retaining walls, The Institution of Civil Engineers and Crown, Thomas Telford Services Ltd. London, United Kingdom

Ng, K. C. 1983. The construction problem and performance of large bored piles in second sand layer, M. Eng. Thesis No. GT 82-26, Asian Institute of Technology, Bangkok, Thailand


Emerging Technologies

Advantages and Limitation of Integrity Tests for Large Diameter Bored Piles in Bangkok Subsoil

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STS Instrument Co., Ltd.
ADVANTAGES AND LIMITATION OF INTEGRITY TESTS FOR LARGE DIAMETER BORED PILES IN BANGKOK SUBSOIL

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Natamon Kampananon, STS Instruments Co., Ltd., Bangkok, Thailand

INTRODUCTION

Due to the prevailing condition of subsoil deep-seated large diameter bored piles of over 50 m depth with 0.8 - 1.8m in diameter are mainly used for heavy structures such as high rise buildings, elevated expressways, flyovers or overpass bridges and more recently for underground subway stations of the first Bangkok MRTA system. As a part of pre-planned quality assurance regime and retrospective investigation, integrity of the piles are tested to obtain the information with regard to the potential deficiencies of the constructed piles which may have formed during actual pile construction process or may have been attributed by other activities after construction of the piles. Three techniques namely sonic integrity, sonic logging and high strain dynamic load tests are the commonly used non-destructive integrity tests applied in Bangkok, Thailand. The effectiveness and suitability in terms of both cost and method of application itself are the primary reasons of the greatest growth in use of these tests.

SUBSOIL AND EXISTING PIEZOMETRIC PROFILE

A typical subsoil profile is relatively consistent in different localities in Bangkok. It is characterized by alternating layers of clay and sand deposits of Quaternary age extending down to about 550m depth where bedrock generally exists.

Existence of soft to very soft highly compressible dark gray marine clay makes difficulties in foundation construction works. This notorious soil layer lies beneath made-ground in most urban area and in some areas it lies under weathered crust layers of 2m thick. Depending on the location, this layer is extended up to 12-18m. About 2m thick Medium Clay layer can be observed.
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In Bangkok due to the prevailing condition of subsoil (existence of considerably weak stratum at top part) deep-seated large diameter bored piles of over 50 m depth with 0.8 - 1.8m in diameter have been mainly used for heavy structures such as high rise buildings, elevated expressways, flyovers or overpass bridges and more recently for underground subway stations of the first Bangkok MRTA system. As a part of pre-planned quality assurance regime and retrospective investigation, integrity of the piles are tested to obtain the information with regard to the potential deficiencies of the constructed piles which may have formed during pile construction process or may have been attributed by other activities after construction of the piles. An overview of three commonly used non-destructive tests such as sonic integrity, sonic logging and high strain dynamic load tests on deep-seated piles in Bangkok subsoil is presented in this paper. Advantages and limitations of each integrity test are discussed.

INTRODUCTION

Due to the prevailing condition of subsoil deep-seated large diameter bored piles of over 50 m depth with 0.8 - 1.8m in diameter are mainly used for heavy structures such as high rise buildings, elevated expressways, flyovers or overpass bridges and more recently for underground subway stations of the first Bangkok MRTA system. As a part of pre-planned quality assurance regime and retrospective investigation, integrity of the piles are tested to obtain the information with regard to the potential deficiencies of the constructed piles which may have formed during actual pile construction process or may have been attributed by other activities after construction of the piles. Three techniques namely sonic integrity, sonic logging and high strain dynamic load tests are the commonly used non-destructive integrity tests applied in Bangkok, Thailand. The effectiveness and suitability in terms of both cost and method of application itself are the primary reasons of the greatest growth in use of these tests.

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between Soft Clay and underlying Stiff Clay. Generally Stiff Clay layer occurs directly underneath Medium Clay and its depth goes up to 22m. Below Stiff Clay layer, First Sand layer 5-8m in thickness can be found. This First Sand layer, however, is absent in some areas. Stiff to Hard Clay layer underlies First Sand and it is found to be about 5m thick. Second Sand layer generally occurs at depths between 45 to 65m. Typical subsoil profile and the present piezometric drawdown condition of Bangkok up to 60m depth are presented in Figure 1.

CONSTRUCTION METHOD

Due to the prevailing subsoil and groundwater conditions, deep-seated bored piles of toe depth over 24m are constructed by wet-process or slurry displacement method using rotary drilling machine. A temporary casing of appropriate length (12 to 18m in Bangkok depending on the thickness of soft clay) with required diameter (internal diameter not less than that of design bored pile diameter) is first installed to ensure the stability of the borehole in the top soft or loose soil layers. In some projects where the vibration is strictly limited, a standard length of casing with oscillator or short temporary casing of 5 to 8 m is pushed down in combination with the pre-boring process. Drilling is commenced by auger to drill out the soil inside the temporary casing. Auger-drilling is commonly continued up to the top of the first water-bearing sand layer or bottom of the casing when using the short casing method. Drilling slurry or supporting fluid is then supplied to the borehole and drilling is proceeded with a bucket down to the design final depth of the pile. Before lowering the reinforcement cage, a special cleaning bucket is used to clean the base of borehole. If bentonite slurry is used, recycling method by air-lift or pump is applied as the base cleaning process. Reinforcement cages are then lowered into the borehole and concreting is carried out by tremie method.

OCCURRENCE OF POTENTIAL DEFECTS IN DEEP-SEATED WET-PROCESSED BORED PILE

In general, pile defects can be caused at two stages, during the pile construction process and after construction of the piles (post-pile-construction). Pile defects that may arise during the construction of wet-processed bored pile are size reduction/necking, discontinuity, and soil/slurry inclusions. The major causes of these defects are;

- Inadequate length to protect the soft clay layer (length of temporary casing)
- Delayed feeding of slurry and improper slurry level maintenance for wet process
- Long time duration of maintaining open borehole
- Non continuous concrete pouring or disruption in concreting
- Lifting up of insufficient workable concrete or hardened concrete upon extracting the temporary casing
- Contamination of concrete with drilling slurry or soil
- Soft pile toe due to improper base cleaning

Most of the defects or cracks in bored piles caused at post construction stage are usually induced by construction activities associated with adjacent excavation work with inadequate earth retaining system and improper trimming of pile head to design cut-off level.

Thasnanipan et al. (1998b) reported that integrity of 285 bored piles (3.3 % of 8,689) were found to be of doubtful quality according to an assessment on the results of sonic integrity test with the additional information obtained from the construction records of bored piles in Bangkok subsoil. The results of the findings are summarized in Table 1. As can be seen in the table, higher percentage of defects is caused by post-pile-construction activities and more commonly by movement of surrounding soils induced by excavation works in the vicinity of the piles.

Table 1. Summary of pile assessment on 8,689 bored piles (Thasnanipan et al. 1998b)

<table>
<thead>
<tr>
<th>Type of Defect</th>
<th>Causes</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poor concrete at pile top (0-3m)</td>
<td>Cutoff level near the ground or inadequate overcast length</td>
<td>0.1</td>
</tr>
<tr>
<td>Size reduction</td>
<td>Insufficient casing length or soft clay layer variation in thickness</td>
<td>1.0</td>
</tr>
<tr>
<td>Cracks/discontinuities</td>
<td>Excavation works for pile trimming and construction activities</td>
<td>2.2</td>
</tr>
</tbody>
</table>
SELECTION OF SUITABLE TEST METHOD

Turner (1997) highlighted the factors in selection of pile testing in CIRIA Report 144 as follows:
• The perceived nature of possible features or defects within the pile
• The ability of the test method to detect the feature or defect under investigation
• The cost of testing and examination
• The ease of use and interpretation

Like other indirect measurement of pile integrity testing methods, sonic integrity and sonic logging tests are only capable of identifying structurally significant features. Rejection or acceptance of individual piles should not rely only on the results of these tests. Further investigation, engineering evaluation and judgment are highly recommended to confirm the defect detected by these indirect techniques. Both methods however are very useful for cost-effective screening test to identify piles with potential defect.

High strain dynamic load test is usually selected to verify the load carrying capacity of piles. Pile integrity can also be determined by high strain dynamic load test. The major advantages of this test in comparison with static pile load test are those of cost, time and space requirement.

Table 2. The suitability of test method for different type of defects.

<table>
<thead>
<tr>
<th>Type of defects</th>
<th>Sonic Integrity</th>
<th>Sonic logging</th>
<th>High strain dynamic load test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crack</td>
<td>Suitable</td>
<td>Possible</td>
<td>Suitable</td>
</tr>
<tr>
<td>Pile size change</td>
<td>Suitable</td>
<td>Possible</td>
<td>Suitable (if severe defect)</td>
</tr>
<tr>
<td>Poor quality concrete or Contaminated concrete</td>
<td>Possible</td>
<td>Suitable</td>
<td>Suitable (if severe defect)</td>
</tr>
<tr>
<td>Soft pile toe</td>
<td>-</td>
<td>Not usually</td>
<td>Suitable</td>
</tr>
</tbody>
</table>

In some projects, dynamic load test was applied to justify the capacity of pile after rectifying its defect detected by other integrity tests. Table 2 shows the suitability of test method for different type of common defects.

SONIC INTEGRITY TEST

Sonic integrity test, also known seismic test is the most common method of integrity testing for both driven and bored piles in Bangkok. In many projects it is a part of the contractual requirement to conduct sonic integrity test. Minimum 10 % to maximum 100 % of production piles are commonly tested. It is also a reasonably acceptable method for applying as a retrospective investigation in determining integrity of the pile.

Features Detectable by Sonic Integrity Test for Deep-seated Bored Piles

Following negative features of deep-seated bored piles can be commonly detected by sonic integrity tests.
• Discontinuity / crack
• Size reduction or necking
• Discontinuity / Contaminated concrete

The characteristic of sonic integrity signals of above features have been thoroughly reviewed and presented by Thasnanipan et al. (1998a). As some features have a similar pattern to each other, it is always important to check and compare the test data of pile in question with those of other piles within the same job site to establish “site signature”. In some cases correlations should be made with similar projects where damaged piles were investigated with firm evidence (coring or excavation to detect damage level). Interpretation should also be made with sound knowledge of pile construction technique, subsoil condition and other factors (problem encountered during and post pile construction) which may influence the sonic test signals. Pile construction records are also useful in interpretation of piles with detected anomaly. Conclusion should be made after careful and thorough review of test results in conjunction with other available information. Further investigation (if applicable excavate down to level of detected anomaly for visual inspection) may require before final conclusion is made.

Discontinuity / Crack Detectable by Sonic Integrity Test

Modern buildings in Bangkok commonly require the excavation works for basement facility and
for the use of different elevations of pile foundation. Excavation works are mainly carried out by using cut slope, sheet pile wall, diaphragm wall and secant pile wall. Since most of the bored piles are designed mainly to carry an axial load, lateral and tensile force which may impose on pile during basement excavation are sometimes not considered or overlooked.

![Figure 2](image)

Figure 2. Typical features of external forces causing pile cracks in Bangkok subsoil

Excessive lateral displacement of ground caused by improper excavation naturally induced an additional lateral force and bending moment in the pile. Once the induced bending moment exceeds the cracking moment capacity, piles are subject to be cracked. Unexpected tensile force acting upon the pile due to an excessive heave or uplift force induced by excavation can also cause cracking of pile. In Bangkok, pile cracks caused by external forces are normally found at the level where Soft and Medium to Stiff Clay boundary is present (typically at 12 to 16m below the existing ground level) as illustrated in Figure 2.

**Size Reduction or Necking**

Common causes for size reduction in bored piles are:

- Insufficient length of temporary casing to protect the soft clay
- Delayed feeding of slurry and improper slurry level maintenance for wet process
- Long time duration of maintaining open bore
- Non continuous concrete pouring
- Soil conditions such as inclusion of sand lenses in clay

**Contamination of concrete or Discontinuity of Pile Material**

A discontinuity in pile shaft caused by disruption during concrete pouring can be seen in Figure 4. Concrete supply was interrupted by poor coordination between the piling contractor and concrete suppliers. As previously poured concrete in the lower pile section had been hardened when concreting resumed, the tremie pipe was also blocked. Blockage was cleared after completely pulling out the entire tremie from borehole. Reinstallation of tremie pipe in the borehole filled with bentonite slurry eventually caused the formation of contaminated layer between old and new concrete.

![Figure 4](image)

Figure 4. Sonic signal showing a discontinuity at 11.0m caused by interruption in concrete pouring - multiple reflections of signal can be observed from the discontinuity

The detected discontinuity was later rectified by coring and cement grouting. Figures 5 shows the sonic test signal of the same pile after closing the discontinuity.
Based on the experience of testing specialist on over 3000 deep-seated bored piles in past 15 years in Thailand along with the comparison between sonic logging test results and direct examination from cored samples of zones detected with anomaly, the category and criteria for interpretation of cross-hole sonic logging test results have been established as shown in the table below.

Table 5  Category of sonic logging results and criteria

<table>
<thead>
<tr>
<th>Category</th>
<th>Criteria</th>
<th>Integrity</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>No or slight increase in transmission time not more than 10%</td>
<td>Homogeneous concrete</td>
</tr>
<tr>
<td>B</td>
<td>Minor increase in transmission time &gt;10% but &lt;25%</td>
<td>Minor anomaly</td>
</tr>
<tr>
<td>C</td>
<td>Transmission time more than Category B but transmission time can still be defined</td>
<td>Major anomaly</td>
</tr>
<tr>
<td>D</td>
<td>Transmission time can not be defined</td>
<td>Defect</td>
</tr>
</tbody>
</table>

Table 4. Recommended number of tubes for different pile size

<table>
<thead>
<tr>
<th>Pile Diameter (mm)</th>
<th>Minimum No. of tubes</th>
<th>Tube spacing (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D ≤ 750</td>
<td>2</td>
<td>180</td>
</tr>
<tr>
<td>750&lt;D ≤ 1000</td>
<td>3</td>
<td>120</td>
</tr>
<tr>
<td>1000&lt;D ≤ 1500</td>
<td>4</td>
<td>90</td>
</tr>
<tr>
<td>1500&lt;D ≤ 2500</td>
<td>6</td>
<td>60</td>
</tr>
<tr>
<td>2500&lt;D</td>
<td>8</td>
<td>45</td>
</tr>
</tbody>
</table>

Figure 6. Sonic profile showing A, B, C and D categories
Turner (1997) pointed out the need of both theoretical knowledge and practical experience in interpretation of the test results in CIRIA Report 144. Suggestion is also made in this report that the anomaly shown in sonic test signals can be caused not only by changing in physical properties of the materials, but also by factors within the measuring system itself. These factors are:

- Free movement of the probes within access tubes
- Mismatched probe positions especially at pile toe
- Measurement resolution
- Incorrect position of access tube
- Air gaps or different material around access tubes
- Aggregate variation (in the case of base grouting)

It is to be noted from the author’s suggestion based on the above factors that only variations of transmit time more than 15 to 20% of the norm should be regarded as warranting further investigation.

HIGH STRAIN DYNAMIC LOAD TEST

High strain dynamic load test can be employed as a part of pre-planned quality assurance regime and retrospective investigation. High strain dynamic integrity test has become a well-accepted method especially for evaluating the pile capacity in the foundation industry today. A large number of related technical papers and case histories of the test have been published and it is a part of standards and specifications such as ASTM D4945-89 (Standard Method for High-strain Dynamic Testing of Piles).

Vasinvarthana and Kampananon (1997) presented the efficiency and reliability of dynamic load test carried out in Bangkok. Figures 7 and 8 show the load settlement curves of dynamic and static load tests conducted in Bangkok on large and small diameter piles respectively. As can be seen in the figures, load-settlement characteristics obtained from dynamic load test agree well with those of static load test for both large and small diameter bored piles.

![Figure 7. Load-settlement curves of dynamic and static load tests on large diameter piles (φ 0.80m x 50.0m).](image)

![Figure 8. Load-settlement curves of dynamic and static load tests on small diameter bored piles (φ 0.35m x 18.9m).](image)

Pile integrity can also be determined by high strain dynamic load test. Figure 9 and 10 illustrate the discontinuity (crack) detected at the same pile by sonic integrity and high strain dynamic load tests respectively.

It is to be noted from Figure 9 and 10 that the duration of pulse in the sonic integrity test signal is about half of that in dynamic test signal. The short duration (high frequency) pulses can resolve reflections from narrow impedance changes. In such case presence of a crack in the pile is more clearly indicated by low strain with sonic integrity test signal than the high strain by dynamic test signal.
load test for both large and small diameter bored piles. Load-settlement characteristics obtained from dynamic and static load tests conducted in a dynamic load test carried out in Bangkok. Vasinvarthana and Kampananon (1997) High-strain Dynamic Testing of Piles). A number of case histories of the test have been published showing a crack in the pile at about 8.0m, caused by excessive lateral soil movement induced by adjacent excavation work.

Figure 10. High strain dynamic load test signal showing a crack in the pile at about 8.0m

DISCUSSION

Sonic integrity test can be applied for both quality check (control test) and retrospective investigation. It is the cheapest in terms of cost and the simplest in terms of testing process. In Bangkok a single testing crew can carry out the test over 70 piles within a day at one site if all piles tested are easily accessible and readily prepared. The main advantage of this test is that since no particular measures or preparation is necessary during the pile construction phase it is more flexible to select which pile is to be tested. However, interpretation of sonic integrity testing needs considerable experience and knowledge in testing, subsoil condition and construction method. The major limitations of sonic integrity test are;

- Signal characteristics is influenced by damping effect due to the stiffer soil layers at deeper depth. Hence toe reflection is hardly detectable for piles founded at depth deeper than 35m. Similarly signal reflections from anomalies are generally clear for the piles with length to diameter ratio (L/D) within 40.

- Some defect features can be of similar signal pattern and it can not detect the axial extent of anomaly. Therefore crack may be misinterpreted as contaminated concrete or void which can lead to an argument between piling contractor and excavation contractor.

Sonic logging tests are much more expensive than sonic integrity tests as the cost of access tubes and test itself are comparatively high for deep-seated bored pile. It is mainly applicable as a pre-planed site quality control testing. The major advantage of this method is that test can be carried out shortly after the pile construction. Hence, rectification measures can be implemented while the foundation contractor is on site. However, this method is not applicable for retrospective investigation. It is less useful if pile integrity is in question due to post construction activities as small horizontal cracks are hardly detectable by this test method.

The major advantage of high strain dynamic load test is its capability to verify both the load carrying capacity and integrity of the pile. The main advantages of this test in comparison with static pile load test are effectiveness in terms of cost, time and space requirement. It can be also applied to justify the capacity of pile after rectifying its defect detected by other integrity tests. However, testing and interpretation of the results must be made by qualified testing specialists who have adequate knowledge in local soil conditions and construction process of wet-processed bored piles.

CONCLUSIONS

Integrity tests commonly applied for large diameter bored piles in Bangkok are presented highlighting the advantages and limitations of each method. It is to be emphasized that integrity test are highly dependent on the experience of the testing firm in both field testing and interpretation. Interpretation should be made with sound knowledge of pile construction technique, subsoil condition and other factors such as problem encountered during and post pile construction which may influence the test results and interpretation. Pile construction records are also useful in interpretation of the piles with detected anomaly. Conclusion should be made after careful and thorough review of test results in conjunction with other available
information. Further investigation may require before final conclusion is made.

REFERENCES


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Application of Low-strain Integrity Tests for Quality Assurance of Deep Foundations in Thailand

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Application of Low-strain Integrity Tests for Quality Assurance of Deep Foundations in Thailand

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Keywords: Sonic integrity test, Sonic logging, Parallel seismic test

ABSTRACT: Low-strain integrity tests have been widely used in Thailand since the 1980s, when large-diameter bored piles were introduced for the earliest high rise buildings in Bangkok. The most commonly used integrity test methods on deep foundations in Thailand are sonic integrity tests (locally known as seismic tests) and sonic logging tests. The sonic integrity test is specified as a compulsory quality assurance test for bored piles in most projects. Additionally it is also carried out on some reference piles soon after piling work is completed, particularly in projects which require deep excavation for further construction stages by other than the piling contractor. This practice is adopted to prevent contractual dispute between the piling contractor and the other construction contractor regarding pile damage contributed by soil movement associated with deep excavation work. Sonic logging tests are commonly specified for all large bored pile with diameters up to 2.00m and barrette piles for bridges and elevated highways and viaduct foundations. In addition parallel seismic tests are now often utilized to determine pile length of old bridges and flyover foundations to check alignment clearance for tunnels designed along the existing roads and canals. This paper presents a brief history of low-strain integrity tests used in Thailand and discusses interpretation of test signals in conjunction with pile construction method and subsoil conditions.

1 INTRODUCTION

Thailand has been one of the economically fast growing nations in South East Asia since the 1980s, with the economic growth fuelling a boom in civil construction. However, the majority of large scale construction work has been mainly in Bangkok, the capital of Thailand, where over 10% of the total population resides. Bangkok is located in a sediment basin comprising thick layers of soft clays and a series of inter-beding sand and clay layers in hundreds of meters of deep. The depth of bedrock is not well determined in the Bangkok area but has been reported to be about 550m deep (Balasubramaniam, 1991). Deep foundations consisting of large diameter bored piles and barrettes reaching a depth of 70m in such subsoil support heavy structures are therefore concentrated in Bangkok.

With improvements in the network of highways connecting all provinces over the last 20 years, civil construction is now active in all major provincial cities and tourist destinations. In these areas, foundation constructions are generally much shallower than those used in Bangkok as better soil conditions with relatively shallow bedrock is present. In order to assess the quality of foundation piles, a number of low-strain integrity test methods have been applied according to pile type. A monograph of the low-strain integrity testing described in this paper has been published by Turner M. J. (1997).

2 FOUNDATION PILES

In Thailand, two major types of deep foundation piling are pre-cast driven piles and bored piles. Pre-cast piles are generally used outside urban or less built-up areas where there are no noise or vibration constraints for pile driving. In Bangkok, bored piles with tips extending to the first sand layer or to stiff clay layer at a depth of about 20 -30 m are normally employed for light to medium-sized structures located in vibration-restricted areas. For heavy structures or high-rise buildings deep large diameter (0.80m-2.00m) bored piles with tips extending to more than 45m are commonly used. Two types of construction method, dry and wet process are typically applied in bored piling.

The dry process uses steel temporary casing (about 15m in length, especially in Bangkok) to
protect the soft soil from caving in. The bore is drilled either using an auger or bucket. For small piles, shells with drop-weight are used to make the bore. Concrete is poured into a borehole which is in dry condition after placing the reinforcement cage.

The wet process also uses steel temporary casing. A borehole is made with rotary drilling using auger or an auger bucket under bentonite or polymer slurry. Concrete is poured with tremie method under drilling slurry.

The dry process is employed mainly for small piles of 0.35m to 0.60m in diameter. The length of the piles are up to 25m and extends further if necessary, depending on subsoil conditions, absence of sand layer with groundwater, etc.

For large bored piles of 0.80m to 2.00m in diameter and barrettes (up to cross sections of 1.5x3.0m), piles are constructed to depths of 30m to over 60m. The wet process is commonly used and piles are founded in sand layers. Compressive cylindrical strengths of concrete used in bored piles is in a range of 240-280 ksc (24-28 Mpa.).

3 SONIC INTEGRITY TESTS

3.1 Overview of sonic integrity testing in Thailand

The sonic integrity/echo test (locally known as seismic test), performed with an accelerometer and hammer, has been used in Thailand since the 1980s when large-diameter bored piles were introduced for the earliest high-rise buildings in Bangkok. In addition to construction records and specification for the bored pile construction for quality assurance, a sonic integrity test was also employed. In the early days of high-rise construction, the sonic integrity test equipment showed a test record on the display unit only. The record was photographed by a Polaroid camera for later interpretation. The quality of test records was visually poor and sometimes comprehensive conclusions on test results were difficult to make. A few years later, advancements in both computer hardware and software enabled the user to obtain high resolution data with signal enhancement and digital processing for interpretation of test data. Test results can thus be accurately interpreted, particularly for types of defect or anomalies and their position, and pile length.

Other derivatives of low-strain integrity tests, such as frequency response tests were introduced by a few testing companies. However, the frequency-based testing was not as popular as the time-based testing due to unfamiliarity with both analysis and interpretation of the test results.

In the period 1992-1997, with economic growth in Asia, massive development in construction of multi-storey buildings, elevated highways, bridges, etc. took place in Bangkok. Large quantities of bored piles and driven piles were installed annually. The sonic integrity test was then widely used for quality assurance of these piles.

After 1997, the construction industry in the private sector was significantly affected by an economic crisis. However infrastructure work such as mass transit system and airport construction projects was still active.

The sonic integrity test is used mainly to detect defects, changes in the physical properties and length of the bored piles. For driven piles, it is used for detecting pile defects such as cracks and joint splitting induced by over driving and/or pile inclination.

3.2 Signal characteristics and interpretation

The sonic integrity test indicates toe reflections of pile clearly for small piles with slenderness ratio of 40. In few cases, especially toe grouted piles toe reflection has been observed for longer piles with toe embedded below 30-35m.

In some cases, even though pile toe is extended beyond 35m in depth, the toe reflection can be observed in the test signals of toe grouted piles having L/D within 40.

Figure 2 is an example of such a case where the bored piles were base grouted and of 40m in length (toe at 55m, trim level at -15m). However, it should be noted that these reflections are prominent only for a few piles. Influence on toe reflection by grouted toe also depends on stiffness of the soil underneath the toe, which has been grouted.
In a few cases, it was found that toe reflection is detectable for some deep-seated bored piles of about 50m in length if the pile was tested at an early age. Figure 3 shows the significant toe reflection of 52.0m-long pile tested a few days after concreting. Note young pile age, the bonding between soil and pile has not yet fully developed so that damping effect or attenuation of the sonic signal by shaft friction is relatively low.

Influence by different soil strata on signals has been observed in many cases (Thasnanipan et al 1998). Stiff clay layer at depths of 15m-25m (undrained shear strength of 10-22 t/m$^2$) significantly influences signal reflections. This was in small rather than large bored piles. Figures 4 and 5 explain these conclusions.

Test signals shown in Figure 7 indicate a clear reflection at approximately 11m from pile top (about 15m below ground level) with a repeat or multiple reflections at about 2 times that distance (approximately 22m from pile top). As the polarity of the reflection is negative, the first interpretation is that this pile has decreased in cross section or crack at 15m below ground level. However, based on the pile construction method, it is concluded that negative reflection is caused by variation in pile cross section at the bottom end of the 15m-long temporary casing. The sonic signals with anomaly acquired on those piles are often misinterpreted as indicative of defective piles, leading to dispute.
Figure 7. A typical pile shape constructed in Bangkok soil with resulting sonic integrity test signal profile.

Figure 8 shows bored pile sections formed at the lower end of temporary casing, confirming size changes occur below the casing. These bored piles were cast together with a steel column for construction of deep basements with diaphragm walls using top-down method. These piles were exposed when excavation reached to 15m below ground level.

Figure 8. The photograph illustrates pile size changes caused by difference in diameter of temporary casing and auger used in pile construction.

Size variations shown by signals are often consistent with casting records and available drilling monitoring results. Figures 9A and 9B show the comparison of borehole profiles and sonic test records.

Figure 9A. Size increase in pile shaft at 13m is shown by both sonic integrity test and drilling monitoring records. (Pile \( \phi 1.0\text{mx}40\text{m}, \text{cutoff} \ -5.0\text{m}, \text{toe depth} \ -45.0\text{m.} \))

Figure 9B. Uniform pile size along the pile shaft is shown by both drilling monitoring record and sonic integrity test record. (Pile \( \phi 0.80\text{mx}42.0\text{m}, \text{cutoff} \ -3.0\text{m}, \text{toe depth} \ -45.0\text{m.} \))

Sonic integrity test can clearly reveal a crack or discontinuity in piles. Most cracks or discontinuities in bored piles after installation can be caused by construction activities associated with basement excavation adjacent to them and improper trimming to design cut-off level. In the past, from the assessment of pile defects, most of the defects are found in small piles with a diameter of 0.60m or less. Study of piling and excavation plans show that piles with a crack indicated by the sonic integrity were located in the vicinity of excavation boundaries (Thasananipan et al, 2000a). These piles were cracked by soil movement due to both reinforcement inadequate for resisting bending stresses and improper soil excavation practice. Since geotechnical experience has been gained from several construction projects with deep basements, pile design has been improved over the years and pile defects are significantly reduced.

The test signal in Figure 10 shows a crack or discontinuity in the pile caused by soil movements due to adjacent excavation work.

Figure 10. Sonic signal showing a discontinuity (crack) at about 7.8m, caused by lateral soil movements.

Figure 11A shows a discontinuity in the pile shaft caused by disruption during concrete pouring. Concrete supply was interrupted by poor coordination between the piling contractor and
concrete suppliers. By the time the concreting was resumed the lower section had hardened. The discontinuity was later closed by core-drilling and cement grouting. Figures 11A and 11B show the signal interpretation before and after closing the discontinuity.

![Figure 11A](image)

Figure 11A. Sonic signal showing a discontinuity at 11.0m caused by interruption in concrete pouring (pile \( \phi 0.80 \times 38.80 \)) and multiple reflections from the discontinuity.

![Figure 11B](image)

Figure 11B. Sonic integrity test shows the discontinuity at 11.0m has been closed by grouting.

Figure 12 shows the characteristics of signals observed at a site where dry process was adopted. Sonic integrity test signals were considered indicative of poor pile integrity at the site. Construction records revealed that boring was done in water bearing sand layer without casing or bentonite slurry causing concrete contamination with water and/or segregation.

![Figure 12](image)

Figure 12. Sonic test signal showing size decrease and or a discontinuity at 7.5-9.0m and poor quality pile section near the toe (Pile \( \phi 0.35 \times 26.0m \)).

3.3 Current applications

The sonic integrity test has been accepted by engineers and has been widely used in the deep foundation works in Thailand for over 25 years. The test is also applied to testing of barrette piles in conjunction with sonic logging. For barrettes, multiple testing points with signal stacking are used for obtaining the test data.

Due to cost effectiveness, simplicity of the method and ease of performing the test, the sonic integrity test is now specified for testing 100% of piles in most of the projects for quality assurance.

Additionally, a sonic integrity testing is also carried out on some piles located in the boundary of deep excavation zones soon after piling work is completed. The test records use a reference pile to document pile integrity prior to deep excavation work to prevent dispute between piling contractor and excavation contractor for any pile damage caused by improper construction practice.

4 CROSSHOLE SONIC LOGGING

4.1 Overview of Crosshole sonic logging test in Thailand.

The first crosshole sonic logging test is believed to have been conducted in 1982 for quality insurance of bored piles based on the test results for the Memorial Bridge Project in Bangkok as reported by Ng (1983). Usually the test is employed as quality control testing for foundation piles and diaphragm walls of infrastructure projects such as subways, elevated highways and bridges. Due to the relatively high costs involved in installation of a number of steel pipes along the entire length of piles as sonic emitter and receiver access tubes, it is employed for large-scale projects. To be cost effective, use of the access tubes for sonic logging tests is sometimes extended for pile base grouting and for checking soil conditions below the pile base.

During the 1990s, sonic logging tests were extensively used for foundation piles of elevated highways and bridges in Thailand. After 2000, large bored piles up to 2.0m in diameter and barrettes are now used for foundations of buildings, viaducts and cable-stayed bridges. Whereas previously, bored piles of 1.5m in diameter were routinely used for construction of large buildings. Crosshole sonic logging tests have become compulsory for quality insurance for these large piles.

Two access tubes are required as a minimum for the sonic logging test. A recommended number of tubes for good coverage of different pile sizes is shown in Table 1.

<table>
<thead>
<tr>
<th>Pile Diameter (mm)</th>
<th>Minimum No. of tubes</th>
<th>Tube spacing (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D \leq 750 )</td>
<td>2</td>
<td>180</td>
</tr>
<tr>
<td>( 750 &lt; D \leq 1000 )</td>
<td>3</td>
<td>120</td>
</tr>
<tr>
<td>( 1000 &lt; D \leq 1500 )</td>
<td>4</td>
<td>90</td>
</tr>
<tr>
<td>( 1500 &lt; D \leq 2500 )</td>
<td>6</td>
<td>60</td>
</tr>
<tr>
<td>( 2500 &lt; D )</td>
<td>8</td>
<td>45</td>
</tr>
</tbody>
</table>
4.2 Defect and Anomaly Classifications

Faiella and Superbo (1998) published defect classification criteria for piles monitored by different numbers of access tubes based on interpretation from tests on over 6,000 piles from 37 sites in Italy.

In Thailand, it was based on the experience of testing specialists who performed sonic logging tests on over 3,000 piles in the past 15 years, coupled with the analysis on both sonic logging test results and core samples taken from the zones detected with anomaly. The classification of anomalies was established with signal transmission time criteria. Figure 13 shows the sonic signal profiles of 4 categories classified in Table 2.

Table 2. Category of sonic logging signals and anomaly classification, Thasnanipan et al (2004).

<table>
<thead>
<tr>
<th>Category</th>
<th>Criteria</th>
<th>Integrity Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>No or slight increase in transmission time not more than 10%</td>
<td>Homogeneous concrete</td>
</tr>
<tr>
<td>B</td>
<td>Minor increase in transmission time &gt;10% but &lt;25%</td>
<td>Minor anomaly</td>
</tr>
<tr>
<td>C</td>
<td>Transmission time more than Category B but transmission time can still be defined</td>
<td>Major anomaly</td>
</tr>
<tr>
<td>D</td>
<td>Transmission time cannot be defined</td>
<td>Defect</td>
</tr>
</tbody>
</table>

A review of sonic logging test results and pile construction records of 148 large diameter bored piles (dia. 2.0m, length 64.0m) was carried out to study the detected anomalies and its causes. Anomalies were detected in 14% of the piles, with equal percentage of minor and major anomalies and defects as classified in Table 2. Construction records indicated these anomalies were associated with relatively high sand content in drilling slurry (the sand content was still under the limit of specification). Actually the high sand content was contributed by soil disturbance caused by removal of underground obstruction – the existing old pile at the project pile location. Another major cause of anomaly in pile was found to be due to congested reinforcement arrangement. As the reinforcement cage was considerably large, horizontal stiffener rings were included for lifting at levels of pile cutoff and every cage section lapping. Such reinforcement with spiral bars of the pile reduced the spacing of horizontal bars down to 32mm around the rebar cage at the above mentioned levels for some piles. Contaminated concrete and sediment were considered to be trapped at these congested areas during concrete pouring. This was later confirmed when piles were trimmed to cut-off level and trapping of slurry contaminated concrete was found below the horizontal bars.

Based on the review of both records of pile construction and sonic logging tests, a summary of causes of anomaly is presented in Table 3.

Table 3. Summary of causes of anomaly in concrete of large diameter bored pile found in a typical project.

<table>
<thead>
<tr>
<th>Causes</th>
<th>Probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement bar congestion</td>
<td>X</td>
</tr>
<tr>
<td>Relatively high sand content in slurry</td>
<td>X</td>
</tr>
<tr>
<td>Concrete pouring time longer than usual</td>
<td></td>
</tr>
<tr>
<td>Interruption and concrete pouring</td>
<td>X</td>
</tr>
<tr>
<td>Localized collapse of borehole surface</td>
<td>X</td>
</tr>
</tbody>
</table>

5 PARALLEL SEISMIC TEST

Parallel seismic test is used in Thailand for checking the length and integrity of old foundation piles where both construction records and pile head were not accessible for investigation. Available records indicate that the earliest parallel seismic test in Thailand was performed in 2004.

The test is adopted for quality assurance of tunnel design alignment since the tunnels are usually planned along roads and canals. Bangkok has many canals with bridges crossing over and flyovers at road intersections. Frequency of parallel seismic tests is increasing, as tunnels for subway, water supply, storm-water drainage system and power
transmission lines are being designed and constructed to pass adjacent to the foundation piles within the available corridor and due to subsoil conditions. To date parallel seismic tests were performed on a total of 190 piles for 50 projects.

Table 4. List of projects using parallel seismic test for pile integrity and length determination

<table>
<thead>
<tr>
<th>Description of Site</th>
<th>Number of Project</th>
<th>Number of Piles Tested</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge</td>
<td>2</td>
<td>15</td>
</tr>
<tr>
<td>Old building</td>
<td>47</td>
<td>145</td>
</tr>
<tr>
<td>Building near Pile Driving</td>
<td>1</td>
<td>30</td>
</tr>
</tbody>
</table>

Figure 14. Parallel seismic test (after Williams and Stain, 1987)

Figure 15. A typical test record of parallel seismic test (after Williams and Stain, 1987)

Information on old pile length and integrity is also used in carrying out geotechnical analysis for studying the impact of tunneling, deep excavation and pile driving works on structure supported by old piles. To be cost effective, the parallel seismic test is carried out using the same borehole which is drilled for investigation of the subsoil condition in the close vicinity of the pile to be tested.

6 CONCLUSIONS

Use of low-strained integrity tests in Thailand for quality assurance of deep foundation piles has been presented. Low-strain integrity tests are effective methods to assess the pile integrity. Signal interpretations need to be compared with soil investigation results and pile construction records.

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REFERENCES


Urban Tunnel Construction for Protection of Environment

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ABSTRACT

This paper presents the methods adopted in prediction of excavation and tunneling induced ground movement and building damage risk-assessment carried out for the Contract No. 1 of the M.R.T Chaloem Ratchamongkhon Line, the first underground mass transit system project of Bangkok. The measured ground and associated movement of the buildings and structures were within predicted values. All deep excavation works and tunneling were successfully completed without any significant damages to the adjacent buildings and structures.

1. PROJECT OVERVIEW

The M.R.T Chaloem Ratchamongkhon Line is the first underground mass transit system project in Bangkok developed by the Mass Rapid Transit Authority (MRTA) of Thailand. Total length of the underground structure is 21.5km, comprises of 16 km of twin single-track bored-tunnel, a total of 4 km long 18 cut-and-cover stations, a twin 1.5km long cut-and-cover approach tunnel to Depot and other associated structures. The project was commenced in late 1996 and opened for the public in 2004. The Contract No. 1, Underground Structure South - southern portion of this initial system comprising 9 km of twin 6m outside diameter bored tunnels, nine underground stations and cut-and-cover depot approach tunnel, was awarded to the Joint Venture BCKT consisting of Bilfinger + Berger Bauaktiengesellschaft, Ch Karnchang Public Company Limited, Kumagai Gumi Company Limited and Tokyu Construction Company Limited. Figure 1 shows the full length of the Contract No. 1 in simple schematic layout.

One of the major challenges of the Contract No. 1 was the need of stacked-alignment for a major portion of twin bored-tunnel underneath Rama IV road, one of the busiest roads of Bangkok, which called for a stacked configuration at 3 stations, Lumpini, Si Lom and Sam Yan. This requirement led the contractor to design and construct the deepest underground structures of Bangkok at Si Lom Station - four-floor stacked-station of 32m deep right under the existing flyover and adjacent to a number of sensitive structures. Deep excavation and bored tunneling works in congested urban environment of Metropolitan Bangkok requires a systematic process of ground movement prediction, building risk damage assessment and protection. This paper presents the methods adopted in prediction of excavation and tunneling induced ground movement and building damage risk-assessment carried out for the Contract No. 1.
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2. **SUBSOIL CONDITIONS**

Similar to other localities in Bangkok, a typical subsoil profile along the tunnel alignment is characterized by the alternating layers of clay and sand deposits as shown in Figure 1. Weathered crust of 2 m thick is commonly found as the top layer. In urban areas of Bangkok, this layer is covered by fill material. Soft to very soft, highly compressible dark gray marine clay lies beneath weathered crust or fill. Depending on the location, this layer extends up to 12-16 m. About 2 m thick medium clay layer can be observed between soft clay and underlying stiff clay. Generally stiff Clay layer occurs directly underneath medium clay and its depth goes up to 22 m. Below stiff clay layer, first sand layer of 5-8 m thickness can be found. This first sand layer, however, is absent in some areas. Stiff to hard clay layer underlies first sand and it is found to be about 5 m thick. Second sand layer generally occurs at depths between 45 to 65 m.

![Figure 1. Generalized soil profile along the tunnel alignment and stations](image)

3. **OVERVIEW OF GROUND MOVEMENT PREDICTION AND BUILDING DAMAGE ASSESSMENT PROCESS APPLIED IN THE PROJECT**

Staged Assessment for each station and tunnel sections was carried out according to the requirement stipulated in the Outline Contract Design Specification (OCDS). Specified references were used as basic literatures in ground movement prediction and building damage assessment. For cut-and-cover excavation zone, the work of Peck (1969) and Clough and O’Rourke (1990) were used whereas published papers of Burland et. al. (1977) and Boscardin & Cording (1989) were applied for bored tunnels. Building damage assessment was carried out in 3 stages; Stage 1 Assessment, Stage 2 Assessment, and Stage 3 Assessment.
"Greenfield Site" assumption was adopted in Stage 1 and 2 Assessment ignoring any influence of the stiffness of the structure and type of supporting foundation. Stage 2 Assessment was further extended by Additional Stage 2 Assessment. In Additional Stage 2 assessment, the variation of settlement trough shape and magnitude with depth were considered in identifying the risk of building damage. In Stage 3 assessment, a detailed qualitative assessment of structure and vulnerability of each building or structure to damage were taken into account, coupled with detailed structural analysis in assessing the buildings falling in risk category 3 classified by Mair et. al. (1996).

Table 1: Building Damage Classification (after Burland et. al., 1977 and Boscarding and Cording, 1989)

<table>
<thead>
<tr>
<th>Risk Category</th>
<th>Degree of Damage</th>
<th>Description of Typical Damage</th>
<th>Approximate Crack Width (mm)</th>
<th>Max. Tensile Strain %</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>Hairline cracks</td>
<td></td>
<td>Less than 0.05</td>
</tr>
<tr>
<td>1</td>
<td>Very Slight</td>
<td>Fine cracks easily treated during normal decoration</td>
<td>0.1 to 1</td>
<td>0.05 to 0.075</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>Cracks easily filled. Several slight fractures inside building. Exterior cracks visible.</td>
<td>1 to 5</td>
<td>0.075 to 0.15</td>
</tr>
<tr>
<td>3</td>
<td>Moderate</td>
<td>Cracks may require cutting out and patching. Door and windows sticking.</td>
<td>5 to 15 or a number of cracks greater than 3</td>
<td>0.15 to 0.3</td>
</tr>
<tr>
<td>4</td>
<td>Severe</td>
<td>Extensive repair involving removal and replacement of walls, especially over doors and windows. Windows and door frames distorted. Floor slopes noticeably.</td>
<td>15 to 25 but also depends on number of cracks</td>
<td>Greater than 0.3</td>
</tr>
<tr>
<td>5</td>
<td>Very Severe</td>
<td>Major repair required involving partial or complete reconstruction. Danger of instability.</td>
<td>Greater than 25 but depends on number of cracks</td>
<td></td>
</tr>
</tbody>
</table>

4. SUMMARY OF GEOTECHNICAL INSTRUMENTATIONS

Instrumentation played a major role in the construction of this first underground mass transit system project in congestive urban area with difficult soil condition. The primary objective of the instrumentation program was to monitor the performance of the deep excavation and tunneling operation to ensure safe execution of the construction works and that adjacent structures were not adversely affected. Furthermore, the instrumentation program was established to provide feedback in application of the observational method. Extensive sets of instrumentation were installed around the predefined influence zones of station box excavation, tunneling and associated building, structures and utilities. In addition to other geotechnical instruments, inclinometers, extensometers, various types of settlement points, tile-meter, crack meter and crack gauge were installed to monitor the ground and associated building and structures movement.

5. BUILDING CONDITION SURVEY
Prior to commencement of the major works, all the existing buildings and structures within the predefined influence zone were systematically surveyed. The primary objectives of the surveys were:

- To record pre-construction status of all buildings and structures within the influence zone. The records included were age of building, building type, storey height, expected foundation types, visible defects supported by sketches and photographs as appropriate.
- To utilize the information collected from the building / structure condition survey in the assessment of the buildings and evaluate sensitive structures with respect to construction aspects and monitoring.
- To establish a benchmark to monitor the possible effect of construction.

Expected foundation types of different buildings were classified based on the height, number of floors and age of the buildings. Building survey information was also used to identify the types of instrumentation required.

### 6. Prediction of ground movement induced by station excavation

#### 6.1 Prediction of ground movement induced by station excavation

Surface settlement induced by station excavation was predicted based on the dimensionless design settlement profile behind the earth-retaining structures. The dimensionless design settlement profile was developed from the results of various prediction methods such as numerical analysis carried out by CRISP program, method proposed by Bowels (1990) and Clough & O’Rourke (1990).

#### 6.2 Stage 1 Assessment

Based on the calculated surface settlement described above, contours of settlement around the station and associated underground structures were plotted in the topographic plan with buildings and other structures. Figure 2 depicts the surface settlement contours prepared for one of the stations.
Figure 2. Settlement contour in station excavation zone prepared for stage 1 assessment

Buildings falling outside 10mm settlement contour were eliminated from further assessment. Remaining buildings with settlement greater than 10mm were short-listed for Stage 2 Assessment.

6.3 Stage 2 Assessment

The dimensionless settlement profile for the station box construction were fitted using a polynomial curve to identify the point of inflexion and divide the settlement profile into sagging and hogging zone and calculated the tensile strain induced in the buildings from excavation works using the method proposed by Mair et. al. (1996). The results were then used to identify the category of risk to damage according to the classification of Building Damage Classification presented in Table 1. Buildings classified within “Slight” risk category of damage were eliminated from further assessment. Buildings falling into “Moderate” or higher risk category of damage were further evaluated.

6.4 Additional Stage 2 Assessment

The settlement estimated in Second Stage Assessment presented above was based on surface movement. For the buildings located in close proximity to the excavation founded on piles, it was considered that subsurface settlement may be more critical. Stage 2 Assessment was then further extended by Additional Stage 2 Assessment in which the variation of settlement trough shape and magnitude with depth were considered in identifying the risk of building damage. Settlement of the buildings due to the excavation works were predicted assuming the “green filed” condition. Neglecting restraints from foundation and structure, it was assumed that buildings follow the ground settlement trough at foundation level (estimated pile tip level).

Available methods were further reviewed for predicting the surface settlement with the consideration of two main factors; (1) simple and practical in application (2) enable to correlate with the predicted and measured diaphragm wall deflection. The method proposed by Bowels (1988) was selected as it meets the required criteria mentioned above. Bowels suggested the ground settlement induced by excavation as a function of ground loss due to the deflection of the retaining wall. Bowel demonstrated the calculation of settlements at specified distance by assuming parabolic variations of settlement within the influence distance. Using predicted diaphragm wall deflection, surface settlement behind the wall was computed by empirical formulas proposed by Bowels.

6.4.1 Prediction of vertical sub-surface settlement

A simplified prediction of subsurface settlement was carried out based on the calculated surface settlement described above. First, subsurface settlement influence line was constructed. As shown in Figure 3, settlement influence zone is assumed to decrease with depth from “Dc” at the surface and zero at the wall toe. With the assumption of linear relationship between the volume of deflected wall shape and the volume of settlement trough at any depth within settlement influence zone, subsurface settlement at different depths were calculated.
Based on the polynomial curve developed in Stage 2 Assessment, subsurface settlement can be predicted by the similar approach described above.

6.4.2 Prediction of subsurface horizontal ground movement

Assuming a simple linear distribution of horizontal ground movement with distance, prediction method for sub-surface horizontal ground movement was developed. The influence distance of horizontal displacement behind the diaphragm wall was assumed to be 2.5 $H_g$ at ground level as shown in Figure 4. The influence depth of horizontal displacement is a depth at zero diaphragm wall deflection, $H_w$, which can be obtained by extrapolating the diaphragm wall deflection profile. Horizontal ground movement influence zone was constructed by connecting the end points of $D_o$ and $H_w$ as shown in the figure. Horizontal displacement is assumed to decrease linearly with distance, maximum at wall face and zero at end of influence zone.

$$D_o = 2.5 \, H_g$$

$$S_{WO} = \frac{4V_O}{D_o}$$

$$S_{oi} = S_{WO} \cdot \left( \frac{x}{D_o} \right)^2$$

$$D_y = Y \cdot \frac{D_o}{H_w}$$

$$V_{Ty} = V_T \cdot \frac{V_{zo}}{V_o}$$

Figure 3. Demonstration of subsurface settlement prediction from diaphragm wall deflection values

Figure 4. Demonstration of subsurface horizontal movement prediction from diaphragm wall deflection values

<table>
<thead>
<tr>
<th>Type</th>
<th>Building</th>
<th>Estimated Pile Length</th>
<th>No. of Stories</th>
<th>Risk category by pile</th>
<th>Risk category by chart of Boscarding and Cording (1989)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1 Concrete</td>
<td>Block</td>
<td>3m</td>
<td>2</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>A1 Concrete</td>
<td>Block</td>
<td>3m</td>
<td>3</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>A1 Concrete</td>
<td>Block</td>
<td>3m</td>
<td>4</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>A1 Concrete</td>
<td>Block</td>
<td>6m</td>
<td>2</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>A1 Concrete</td>
<td>Block</td>
<td>6m</td>
<td>3</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>A1 Concrete</td>
<td>Block</td>
<td>6m</td>
<td>4</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>A2 Masonry</td>
<td>Concrete</td>
<td>6m</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>A2 Masonry</td>
<td>Concrete</td>
<td>6m</td>
<td>3</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>A2 Masonry</td>
<td>Concrete</td>
<td>6m</td>
<td>4</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>A2 Masonry</td>
<td>Concrete</td>
<td>12m</td>
<td>2</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>A2 Masonry</td>
<td>Concrete</td>
<td>12m</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>A2 Masonry</td>
<td>Concrete</td>
<td>12m</td>
<td>4</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>A3 Concrete</td>
<td>Masonry</td>
<td>6m</td>
<td>2</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>A3 Concrete</td>
<td>Masonry</td>
<td>6m</td>
<td>3</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>A3 Concrete</td>
<td>Masonry</td>
<td>6m</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>A3 Concrete</td>
<td>Masonry</td>
<td>14m</td>
<td>2</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>A3 Concrete</td>
<td>Masonry</td>
<td>14m</td>
<td>3</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>A3 Concrete</td>
<td>Masonry</td>
<td>14m</td>
<td>4</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>B1 Masonry</td>
<td>Concrete</td>
<td>6m</td>
<td>2</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>B1 Masonry</td>
<td>Concrete</td>
<td>6m</td>
<td>3</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>B1 Masonry</td>
<td>Concrete</td>
<td>6m</td>
<td>4</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>B1 Masonry</td>
<td>Concrete</td>
<td>12m</td>
<td>2</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>B1 Masonry</td>
<td>Concrete</td>
<td>12m</td>
<td>3</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>B1 Masonry</td>
<td>Concrete</td>
<td>12m</td>
<td>4</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>B2 Masonry</td>
<td>Masonry</td>
<td>6m</td>
<td>2</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>B2 Masonry</td>
<td>Masonry</td>
<td>6m</td>
<td>3</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>B2 Masonry</td>
<td>Masonry</td>
<td>6m</td>
<td>4</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>B2 Masonry</td>
<td>Masonry</td>
<td>14m</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>B2 Masonry</td>
<td>Masonry</td>
<td>14m</td>
<td>3</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>B2 Masonry</td>
<td>Masonry</td>
<td>14m</td>
<td>4</td>
<td>3</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 2. Summary of risk category obtained from two different methods and foundation type

480
6.4.2 Prediction of subsurface horizontal ground movement

Based on the polynomial curve developed in Stage 2 Assessment and predicted subsurface ground movement using method described in section 6.3.1, maximum tensile strain of the building was predicted applying the method proposed by Mair et. al. (1996). Risk category of to the building was then classified according to Table 1.

6.4.2.2 Risk of damage specified by angular distortion and horizontal strain

The chart proposed by Boscarding and Cording (1989), was used to determine the potential damage of the building. The chart is a simple tow-dimensional plot of horizontal strains versus angular distortion in which zones of potential damage to buildings are mapped. Angular distortion ($\beta$) was calculated based on the difference between predicted settlement of building one edge to another or two adjacent foundation supports. Subsurface settlement at pile toe level was used in determining angular distortion.

$$\beta = \frac{(S_{v1} - S_{v2})}{L}$$

where, $S_{v1}$ = vertical displacement at one end of the building

$S_{v2}$ = vertical displacement at other end of the building

$L$ = building length

Horizontal strain induced by excavation was predicted assuming horizontal ground movements at surface level decrease linearly with distance, maximum at excavation wall and zero at end of influence zone. Average horizontal strain of the building can be computed by ;

$$\varepsilon_h = \frac{(S_{h1} - S_{h2})}{L}$$

where,

$S_{h1}$ = horizontal displacement at surface at one end of the building

$S_{h2}$ = horizontal displacement at surface at other end of the building

$L$ = building length

Table 2 shows the summarized results of risk category obtained from two different methods described above and assumed foundation types (without and with pile)

<table>
<thead>
<tr>
<th>Building Block</th>
<th>No. of stories</th>
<th>Building Type</th>
<th>Estimated Pile Length</th>
<th>Distance to edge of wall</th>
<th>Risk category by maximum tensile strain</th>
<th>Risk category by chart of Boscarding and Cording (1989)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Without pile</td>
<td>With pile</td>
</tr>
<tr>
<td>A</td>
<td>2</td>
<td>Masonry / Concrete</td>
<td>7-14m</td>
<td>18m</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>B</td>
<td>2</td>
<td>Masonry / Concrete</td>
<td>7-14m</td>
<td>17.5m</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>C</td>
<td>2</td>
<td>Masonry / Concrete</td>
<td>7-14m</td>
<td>32m</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>D</td>
<td>1</td>
<td>Masonry / Concrete</td>
<td>&lt; 6m</td>
<td>2m</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>E</td>
<td>37</td>
<td>RC</td>
<td>50m</td>
<td>8m</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 2 Summary of risk category obtained from two different methods and foundation type
The predicted degree of damage of the buildings using simplified chart is more critical than that of maximum tensile strain method as can be seen in above table. According to the results obtained from of simplified chart, predicted degree of damage was higher if buildings were assumed to be with piled foundation rather than without piled foundation.

6.5 Third Stage Assessment

Examples of the buildings and structures that required the stage 3 assessment as a result of Stage 2 assessment are listed below.

Table 3. The buildings required Stage 3 Assessment in station excavation zones

<table>
<thead>
<tr>
<th>No. of Storey / Building Type</th>
<th>Preliminary Estimated Pile Length</th>
<th>Stage 2 Calculated Total Tensile Strain (%)</th>
<th>Risk Category / Degree of Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 / RC*</td>
<td>8m</td>
<td>0.121 %</td>
<td>2 / Slight</td>
</tr>
<tr>
<td>22 / RC</td>
<td>&gt;22m</td>
<td>0.159 %</td>
<td>3 / Moderate</td>
</tr>
<tr>
<td>4 / RC</td>
<td>7-14m</td>
<td>0.197 %</td>
<td>3 / Moderate</td>
</tr>
<tr>
<td>2 / Steel Frame</td>
<td>15-22m</td>
<td>0.288 %</td>
<td>3 / Moderate</td>
</tr>
<tr>
<td>5 / RC, Masonry</td>
<td>15-22m</td>
<td>0.166 %</td>
<td>3 / Moderate</td>
</tr>
<tr>
<td>2 / RC, Masonry</td>
<td>&lt;6m</td>
<td>0.29 %</td>
<td>3 / Moderate</td>
</tr>
<tr>
<td>3 / RC, Masonry</td>
<td>7-14m</td>
<td>0.242 %</td>
<td>3 / Moderate</td>
</tr>
<tr>
<td>1 / RC, Masonry</td>
<td>≤6m</td>
<td>0.191 %</td>
<td>3 / Moderate</td>
</tr>
</tbody>
</table>

* Stage 3 Assessment of this building was required due to the pre-existing damage of the building.

A process of Stage 3 Damage Assessment involved;
- Collection of the record of pre-construction defects/damages of the building from Building Condition Survey Reports
- Conducting detailed structural Survey and assessing the current state of the building
- Conducting damage assessment

6.5.1 Collection of building condition survey

The building condition survey records were collected and primary review of the state of the building prior to commencement of construction works.

6.5.2 Detailed Structure Survey

In order to understand the building response to the ground movement and to carry out the qualitative damage assessment of the building, details of the structure should be known in advance. A structural survey was carried out by experienced structural engineer to obtain the detailed information of structural members.

6.5.3 Damage Assessment

Structural survey information was the key factor in the selection of methodology for the Stage 3 Assessment. Based on the pre-construction state building condition survey and detailed structural survey, the method for damage assessment was selected. Two main approaches were applied.
• Conservative approach, mainly applied in additional stage 2 assessment with some refinements
• Frame analysis using FEM structural software

6.5.3.1 Stage 3 Assessment by conservative approach

Conservative approach applied in Stage 3 Assessment involved consideration of two separate criteria, namely:
• The risk of damage to the building specified by angular distortion and horizontal strain.
• The risk of damage to the building in terms of magnitude of building slope with reference to the limiting values specified by various researchers.

The following assumptions were made in the conservative approach of Stage 3 assessment.
• Building movement is influence by excavation induced settlement of both main station box and associated underground structures (e.g. Entrance Building)
• Building settlement was casued by subsurface settlement at foundation level as buildings are supported by piled foundation.
• No allowance was made for structural stiffness of the building.
• No allowance was made for possible effects of soil-structure interaction.

A summary of risk category determination in Stage 3 Assessment based on the predicted and monitored settlement of the building adjacent to station excavation work is shown in the Table 4 below.

Table 4  Summary of risk category determination based on Boscarding and Cording (1989)

<table>
<thead>
<tr>
<th>Data Taken From</th>
<th>Horizontal Strain</th>
<th>Angular Distortion</th>
<th>Risk Category</th>
<th>Degree of Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sh1 (mm)</td>
<td>Sh2 (mm)</td>
<td>L (m)</td>
<td>εh (mm)</td>
</tr>
<tr>
<td>Settlement Prediction</td>
<td>31</td>
<td>24</td>
<td>18.6</td>
<td>0.0004</td>
</tr>
<tr>
<td>Actual Monitoring Data</td>
<td>31*</td>
<td>24*</td>
<td>18.6</td>
<td>0.0004*</td>
</tr>
</tbody>
</table>

*Predicted horizontal movement was used in computing horizontal strain for actual monitoring data.

From the above table, following conclusions can be made.
• Predicted angular distortion broadly agrees with that of actual monitoring data. Hence, settlement prediction and assumed effective depth of piled foundation is fairly reasonable.
• The settlement of actual monitoring data is slightly higher than that of prediction. This may be due to the additional settlement caused by diaphragm wall panel construction which influence zone might have extended to 15m from the diaphragm wall (equivalent distance to bottom of soft clay level).

Limiting values of the building slopes at two critical conditions specified by various researchers as shown in the table below were also used in determining the potential building damage of stage 3 assessment.
6.5.3.2 Stage 3 Assessment by structural frame analysis

Some buildings of critical condition were assessed by structural analysis using detailed structural survey information as basic data. By simulating the differential settlement of the building predicted from Additional Stage 2 Assessment, development of moment and shear in the main structural members were determined with the use of structural analysis software. Computed moment and shear were assessed in comparison with the capacity of the structural members estimated from collected building condition survey and detailed structural survey data. The process involved in structural assessment can be summarized below.

- Factor of safety of each structural members were determined base on predicted differential settlement of the buildings.
- A series of assessment were performed to determine the critical differential settlement causing structural instability of the members or differential settlement at limiting factor of safety.
- Trigger values were established based on above results.
- Based on the trigger values and the actual building condition and response in conjunction with monitoring data, the most sensitive structural members were protected, mainly by strengthening approach.

6.5.4 Requirement of Stage 3 Assessment for bridge structures

Due to the close proximity to the adjacent station, the predicted maximum settlement in the Stage 1 Assessment of the flyover and elevated expressway structures was greater than 10mm. Therefore, further assessment using Stage 2 approach was required. However, since flyovers and elevated expressways are bridge structure with piled foundations, the approach of strain calculations used in Stage 2 Assessment was considered inappropriate and they were placed under the requirement of the Stage 3 Assessment as listed in Table 6.

Table 6 List of Flyovers / Bridges required for Stage 3 Assessment

<table>
<thead>
<tr>
<th>Location</th>
<th>Name of the Bridge / Flyover</th>
<th>Type of bridge</th>
<th>Foundation Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hua Lamphong</td>
<td>Second Stage Expressway</td>
<td>Steel</td>
<td>Deep-seated bored pile</td>
</tr>
<tr>
<td>Sam Yan</td>
<td>Thai-Japan Flyover</td>
<td>Steel</td>
<td>Deep-seated bored pile</td>
</tr>
<tr>
<td>Si Lom</td>
<td>Thai-Japan Flyover</td>
<td>Steel</td>
<td>Deep-seated bored pile</td>
</tr>
<tr>
<td>Lumphini</td>
<td>Thai-Belgium Flyover</td>
<td>Steel</td>
<td>Deep-seated bored pile</td>
</tr>
<tr>
<td>Phetchaburi</td>
<td>Asok Flyover</td>
<td>Steel</td>
<td>Pre-cast pile</td>
</tr>
</tbody>
</table>
Stage 3 Assessments of above listed bridges were carried out by soil-structure interaction analysis using finite element method. FEM Analyses were mainly carried out by in-house design team consisted of experience geotechnical engineers of Tokyu Construction Company Limited. Two-dimensional non-linear soil model applying Duncan-Chang parameters was employed. The process involved in soil-structure analysis is presented in diagram as shown in Figure 5.

![Figure 5 Diagram showing process of stage 3 assessment for bridge structure](image)

Among the bridges listed in Table 6, impact of excavation work on the structure of Petchaburi flyover was considered the most critical due to its close proximity to the diaphragm wall and having relatively less rigid and shallower foundation of pre-cast piles. Detailed structural assessment of the existing pre-cast piles of the flyover was performed using results of FEM and verified with structural capacities of the piles calculated from the available as-built drawings. Predicted differential settlement of two adjacent piers (pier close to diaphragm wall and pier away from diaphragm wall) caused by subsurface settlement at pile toe level was used to calculate the angular distortion of the flyover superstructure and compared with the tolerable angular distortion specified by for simple span bridge. Predicted angular distortion (0.0003) was much lower than specified tolerable value of ASSHTO (0.005). Predicted horizontal movement of flyover piers were 10mm and 12mm for left and right pier (of Figure 6) respectively. Tolerable horizontal movement of flyover structure based on the observations of actual bridge performance from the works of various researchers (extracted from Xanthakos, 1995) are shown in Table 7.
### Table 7: Tolerable horizontal movement of bridge recommended by various researchers

<table>
<thead>
<tr>
<th>Horizontal Movement (mm)</th>
<th>Recommendation</th>
<th>Recommended by</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>Not harmful</td>
<td>Bozozuk (1978)</td>
</tr>
<tr>
<td>38</td>
<td>Tolerable in most cases</td>
<td>Moulton et. al. (1985)</td>
</tr>
<tr>
<td>51</td>
<td>Harmful but tolerable</td>
<td>Bozozuk (1978)</td>
</tr>
</tbody>
</table>

Extensive sets of instrumentation were installed including tilt meter, settlement points and inclinometers. Monitoring was carried out in high frequency during critical stage of excavation and compared with established trigger values.

![Cross-section of Petchaburi Station diaphragm wall showing the position of adjacent flyover foundations](image)

**Figure 6** Cross-section of Petchaburi Station diaphragm wall showing the position of adjacent flyover foundations

### 7. PREDICTION OF GROUND MOVEMENT INDUCED BY TUNNELING AND RISK ASSESSMENT

#### 7.1 Prediction of surface settlement

The Gaussian normal distribution curve developed by Peck (1969), O’Reilly and New (1982) was used to estimate the ground settlement induced by a single tunnel.

#### 7.2 Stage 1 Assessment

First stage assessment of the risk of damage to buildings and structures in tunneling zones involved preparation of settlement profile induced by tunnel excavation. Settlement was computed at every 100m interval along tunnel alignment using the method based on Gaussian normal distribution curve presented above. Settlement induced by each tunnel (north and
south bound) was predicted independently and then superimposed to determine the total settlement induced by the twin tunnels. 10mm and 25mm settlement contours were plotted in the topographic plan with buildings and other structures including flyover and khlong bridges in the tunnel alignment. Buildings falling outside 10mm settlement contours were eliminated from further assessment. Remaining buildings with settlement greater than 10mm were short-listed for stage 2 assessment.

7.3 Stage 2 Assessment

The buildings and structures short-listed in stage 1 were assessed in this stage. Stage 2 assessment involves calculation of maximum tensile strain induced in the building and structures due to tunnel boring and classification of risk category. Risk category of the buildings was determined applying the method proposed by Mair et. al. (1996), similar to the building damage assessment made for excavation induced settlement presented in earlier sections.

7.4 Additional Stage 2 Assessment in tunnel zones

As most of the buildings and structures in Bangkok along tunnel alignment are supported by piled foundations, it was considered that Stage 2 Assessment using surface settlement profile might be insufficient. Hence, Additional Stage 2 Assessment considering the variation of settlement trough shape and magnitude with depth (subsurface settlement) was carried out for the critical buildings.

In general, subsurface settlement trough will be narrower and steeper than those of the surface as shown in the sketch below. A simple approach was used in predicting the subsurface settlement profile with following assumptions.

(1) The shape of the subsurface settlement profile caused by the tunnel construction is characterized by a Gaussian distribution similar to surface settlement profile.
(2) The ground settlement influence line for tunnel in clay is the same for subsurface as shown in the figure below:

![Form of Surface and Subsurface Settlement Profiles](image)

Figure 7 Distribution of surface and subsurface settlement induced by tunnel boring

Therefore, substituting the distance above the tunnel axis $Z_0-Z$ for $Z_0$, equations derived from Gaussian normal distribution curve subsurface settlement profile was determined. In addition to this method following equation can also be used to predict the subsurface settlement.
K’ = \[0.14 + 0.26(1 - z/z_0)\] / \((1 - z/z_0)\)

By substituting K’ values for K in equation derived from Gaussian normal distribution curve the subsurface settlement at required depth can be calculated.

7. 5 Stage 3 Assessment

7.5.1 Stage 3 Assessment for the buildings

The method used in Stage 3 Assessment of the buildings in tunnel zone was similar to that of station excavation zone based on the superimposed predicted subsurface settlement trough induced by twin tunnels.

7.5.2 Structures Required for Stage 3 Assessment

The most critical structure required for Stage 3 Assessment were MWA main water tunnel as tunnel alignment crosses the water tunnels at two critical locations – the first near Rama IV and Rachadaphisek road intersection and the second at Rama IV and Siphraya road intersection near Sam Yan station. The position of water tunnels in relation to MRT tunnels drawn in the subsoil profile at relevant locations are presented in Figure 8 and 9.

Figure 8  Position of MWA water tunnel in relation to MRT tunnels (clear distance are 3.38m and 3.18m for North and South bound respectively) at the first crossing

According to the available information, MWA tunnel at the first crossing is of 2.5m internal diameter steel lining with 150mm segmental ring and at the second crossing MWA tunnel is of 2.5m diameter segmental ring with lightly reinforced concrete lining.

In preliminary stage 3 assessment, based on the predicted subsurface profile using the method described in earlier section, structural assessment of water main was carried out and results were compared with structural capacity of water tunnel calculated from the material properties indicated in the construction drawings provided by MWA. As bending moment induced in the water tunnel was considered critical parameter in assessing the damage which is governed by both maximum settlement and curvature of the water tunnel further assessment was proposed.
• To carry out back-calculation using monitoring data obtained from the settlement arrays (settlement points, extensometer data) at first 3 locations before reaching to MWAA water tunnel near Rama IV – Rachadapisek intersection, to determine the volume loss and relationship between surface and subsurface settlement as well as best estimation of subsurface K value and subsurface settlement profile at water tunnel level.

• To perform beam-on-elastic foundation analysis using the subsurface settlement profile obtained from above item (1) to determine bending moment likely to occur in MWAA water tunnel.

• To compare the result obtained from above item with allowable values.

• To establish the final trigger values based on above comparison.

• To confirm the settlement profile of last array is within the alert level of final trigger value.

Figure 9 Position of MWA water tunnel in relation to MRT tunnels (clear distance are 1.5m and 2m for North and South bound respectively) at the second crossing

For the first location of MRT-MWA tunnel crossing, target ground loss from tunnel operation was set at 0.5%, whereas ground loss based on allowable movement curvature of water tunnel derived from predicted pattern of subsurface settlement was 0.65%. Back-calculated ground loss derived from the monitoring data of rod-extensometer placed at the crown of MWA water tunnel was within target value. In addition to the preliminary assessment based on prediction of subsurface settlement using empirical formulas, a detailed soil-structure interaction was analyzed by experience geotechnical engineers of Tokyu Construction Company Limited using 2-dimensional FEM considering behavior of tunnel boring by earth pressure balance (EPB) method. Stress relief from the effect of cutter-head, tail void and over-cut from EPB tunnel boring was considered in the analyses. Adding ground movement due to consolidation caused by disturbance from tunnel operation surface and subsurface settlement were predicted by finite element analysis. Detailed parameters and assumption made in analysis was presented in the published paper of Sakai & Sugden (2000).

After the successful passing of both tunnels at the first MWA crossing, another trial area tunneling was planned to further determine the ground movement associated with various tunneling parameters well before reaching to more critical crossing after Sam Yan station. Tunnel section between Silom and Samyan station, in which soil profile similar to that of second crossing was selected and comprehensive instrumentation program was established to
monitor surface and subsurface settlement as well as horizontal movement of the ground at depth.

8. COMPARISON OF PREDICTION AND ACTUAL MONITORING RESULTS

The monitored ground deformation due to station box construction and bored tunneling were compared with the predicted values for one of the station and tunnel sections to evaluate the performance of the actual operation.

8.1 Ground Movement Induced by Excavation Works

The monitored wall deflections and ground settlement induced due to station constructions works were within the design prediction. Adopted top-down method in excavation works of station boxes utilizing rigid diaphragm wall extensively propped by stiff permanent slabs, was the key factor contributed in minimizing associated ground movement and disturbance to adjacent buildings and structures. Figure 10 shows the comparison of monitored wall deflections with predicted values at final stages of station construction.

![Figure 10: Diaphragm wall deflection showing predicted trigger levels and monitored data for final excavation stages to concourse and base slab level](image)

The comparison of monitored and predicted ground settlement profile resulting from station excavation is presented in Figure 11.

![Figure 11: The comparison of monitored and predicted ground settlement profile resulting from station excavation works](image)
8.2 Ground Movement Induced by Tunneling Works

The observed ground movement induced by tunneling works were within predicted values. Well organized plan, systematic approach of observational method to achieve control values backup by extensive instrumentation and close co-operation among relevant personnel were the key factors contributed to successful completion of EPB tunneling particularly in the most sensitive points at MWA water tunnel crossings. The monitored ground settlement profile in the most critical section of parallel twin tunnel between Petchaburi and Sukhumvit station is presented in Figure 12. As can be seen in the figure monitored settlement profile was within that of prediction. The volume loss due to tunneling works in this tunnel zone was back calculated using the monitored profile and found less than 2% and is typically in the range of 1.1% to 1.7%.

![Figure 12](image1.png)

Figure 12: Predicted and observed ground settlement profile due to tunnel boring.
9. CONCLUSION

The M.R.T Chaloem Ratchamongkhon Line, one of the most modern underground mass transit systems in the region was successfully completed and has been in operation for over 1 year. Ground movement prediction and the detailed stage assessment carried out in the M.R.T Chaloem Ratchamongkhon Line are presented. The measured ground and associated movement of the buildings and structures were within predicted values. All deep excavation works and tunneling were successfully completed without any significant damages to the adjacent buildings and structures. Well organized plan, systematic approach of observational method to achieve control values backup by extensive instrumentation and close co-operation among relevant personnel were the key factors contributed to successful completion of the project. The purpose of this paper however is not to extol success but to provide a set of technical guidelines for ground movement prediction and building damage risk assessment of deep excavation and tunneling works in Bangkok subsoil which may partly serve as a reference for the future underground mass transit system projects in urban area of Bangkok. Successful completion of the first MRT project in Bangkok marked the practicality of well-executed deep excavation works and tunneling by EPB method in protecting the sensitive urban environment.

10. ACKNOWLEDGEMENT

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11. REFERENCES

Clough, G.W., and O’Rourke, T.D., (1990), Construction Induced Movements of In-situ Walls, ASCE Geotechnical Special Publication No. 25, pp. 439-470
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9. CONCLUSION

The M.R.T Chaloem Ratchamongkhon Line, one of the most modern underground mass transit systems in the region was successfully completed and has been in operation for over 1 year. Ground movement prediction and the detailed stage assessment carried out in the M.R.T Chaloem Ratchamongkhon Line are presented. The measured ground and associated movement of the buildings and structures were within predicted values. All deep excavation works and tunneling were successfully completed without any significant damages to the adjacent buildings and structures. Well organized plan, systematic approach of observational method to achieve control values backup by extensive instrumentation and close cooperation among relevant personnel were the key factors contributed to successful completion of the project. The purpose of this paper however is not to extol success but to provide a set of technical guidelines for ground movement prediction and building damage risk assessment of deep excavation and tunneling works in Bangkok subsoil which may partly serve as a reference for the future underground mass transit system projects in urban area of Bangkok.

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JV BCKT, (1997) Geotechnical Interpretation Report (GIR) for MRTA Initial System - UGS, Bangkok


O’Reilly M.P and New B.M (1982), Settlements above tunnels in United Kingdom – their magnitude and prediction, Tunneling ’82, pp. 173-181, London, IMM


Observational Method and its Application in Hong Kong

Key Factors in Application of Observation Method – Bangkok Experience

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Application of observational method in two different projects is presented in this paper. Intensive modification of construction sequence in actual work execution with "value engineering options" different from tender stage design along with application of observational method in Diaphragm-wall-support two level underground car park building located in the historically and culturally significant area of Bangkok is firstly demonstrated. Simple approach of observational method in building damage assessment backed by extensive instrumentation in tunneling and deep excavation works of Contract No. 1 of the M.R.T Chaloem Ratchamongkhon Line, the first underground mass transit system project of Bangkok (MRTA) is also presented. Key Factors contributed in successful application of observational method are discussed.

INTRODUCTION

Observational Method is not a very popular term among geotechnical engineers in Thailand, though the method has been applied in some major projects. The main reason of unpopularity of the method in Thailand may be due to the fact that traditionally there is less option opened for design modification as the project progresses. Geotechnical engineers may be more concerned on misused and limitation of the method and do not prefer to take risk. It is important to discuss among geotechnical engineers with regard to overall philosophy, key requirement, limitation and practical approach in application of observational method. In this paper, application of observation method in two different projects is presented.

CONSTRUCTION OF UNDERGROUND CAR PARK IN HISTORICAL AREA OF BANGKOK

The project is a two-level underground car park located in the center of Rattanakosin Island, the heart of an old established, historically and culturally significant area of Bangkok. The project owner, Bangkok Metropolitan Authority (BMA) awarded the semi-turnkey basis construction contract "Lam Kon Muang Underground Car Park" to SEAFCO Co., Ltd. as a contractor. The contract consists of 3 major scope of works: (1) Construction of building foundation and retaining structure - diaphragm wall, barrette and bored piles (2) Excavation works including temporary bracing design and installation (3) Construction of the entire two-level underground car park building having car park area of 18,552 m² and roof-level park of 10,936 m² plus cut-and-cover tunnel, underpass access to the City Hall. Geotechnical
KEY FACTORS IN APPLICATION OF OBSERVATION METHOD
– BANGKOK EXPERIENCE

Noppadol Phienwej¹ and Zaw Zaw Aye²

Abstract: Application of observational method in two different projects is presented in this paper. Intensive modification of construction sequence in actual work execution with “value engineering options” different from tender stage design along with application of observational method in Diaphragm-wall-support two level underground car park building located in the historically and culturally significant area of Bangkok is firstly demonstrated. Simple approach of observational method in building damage assessment backup by extensive instrumentation in tunneling and deep excavation works of Contract No. 1 of the M.R.T Chaloem Ratchamongkhon Line, the first underground mass transit system project of Bangkok (MRTA) is also presented. Key Factors contributed in successful application of observational method are discussed.

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aspects highlighting the application of observational method and performance of buttress-support diaphragm wall of 0.60m width for two level underground car park building is discussed in this paper.

**Project Requirement and Major Constraints**

Since there is a limited availability of car parking space in the surrounding congested neighborhood and the project site was being used as a grade-level car parking space prior to the award of the contract, the key requirement was to construct the underground car park in two phases – to construct Phase 1 while leaving space for car parking in Phase 2, and to utilize semi-finished underground car park of Phase 1 during construction of Phase 2. This requirement posed the need of temporary retaining wall between Phase 1 and 2.

Construction site is surrounded by numbers of sensitive structures as shown in Fig. 1, in the south, Wat Suthat, one of Thailand’s most important temples and the Historical Giant-swing, in the north, the City Hall and at South-east corner, the Historical Brahmin Temple. Rows of old shop-house buildings are closely located in the east and west boundaries of the project. Location of the project site itself in the vicinity of sensitive structures and buildings therefore posed some constraints, which called for the need of careful consideration in establishing the design principles and sequence of construction.

Fig. 1. Layout plan of the project showing adjacent buildings
Aspects highlighting the application of observational method and performance of buttress-support diaphragm wall of 0.60m width for two level underground car park building is discussed in this paper.

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Fig. 1. Layout plan of the project showing adjacent buildings

Fig. 2 (a) Layout of diaphragm wall and piles (b) plan of diaphragm wall and buttress (c) sectional view of the structures

Location of the project site itself in the vicinity of sensitive structures and buildings therefore posed some constraints, which called for the need of careful consideration in
establishing the design principles and sequence of construction. The architectural and utility aspects of the project called for the design of the basement with a number of openings from the ground surface to the final basement slab level to facilitate the ventilation system as shown in figure 2.

Hence the roof slab can not be physically utilized as bracing in most of the area where the diaphragm wall is to be acting as a cantilever retaining wall in the permanent stage. It was analyzed in the preliminary analyses that the deflection of the diaphragm wall of 0.60 m width would be large if it was to be fully cantilevered. As the project is located in a sensitive area, ground movement induced by large deflection was unfavorable. It was therefore decided to use buttress to minimize the diaphragm wall deflection as shown in figure 2.

Diaphragm Walls Barrettes and Bored Piles Construction

Diaphragm wall having 600mm width founded at 16m below ground level (B.G.L) was constructed simultaneously with dry-processed bored piles of diameter 600mm with toe depth 20m below ground level. Barrettes having same toe depth as bored piles were installed at 8m spacing along with diaphragm wall panels. Sheet pile wall (14m deep) was used as a temporary retaining wall at the boundary of Phase 1 and 2 as shown in Fig. 3.

Subsoil Condition and Design Parameters

Typical subsoil profile at the site is characterized by thick Bangkok soft clay layer at the top followed by thin layer of medium clay, and stiff clay layers.

![Subsoil Profile](image)

![Plot of Su and SPT N-value with design lines](image)

Fig. 3. Plotted Su and SPT N-value with design lines
Undrained shear strength (Su) and SPT N-value obtained from 3 SI boreholes were plotted and design line was derived as shown in Fig. 3. The design soil parameters are tabulated in Table 1.

### Table 1. Design soil parameters

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Depth (m)</th>
<th>$\gamma_s$ (kN/m$^3$)</th>
<th>$S_u$ (kN/m$^2$)</th>
<th>$E_u$ (kN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay</td>
<td>0 - 9</td>
<td>16.50</td>
<td>18</td>
<td>7000</td>
</tr>
<tr>
<td>Medium clay</td>
<td>9 – 12</td>
<td>17.50</td>
<td>30</td>
<td>19250</td>
</tr>
<tr>
<td>Stiff Clay</td>
<td>12 – 15</td>
<td>19.00</td>
<td>60</td>
<td>45000</td>
</tr>
<tr>
<td>Very stiff clay</td>
<td>15 - 26</td>
<td>19.50</td>
<td>80</td>
<td>60000</td>
</tr>
</tbody>
</table>

**Original Tender Stage Design**

The designers involved in the tender stage design made a fairly conservative design with two levels temporary bracing as shown in Fig. 4. Uncertainty of the performance of a thin diaphragm wall in soft clay layer was the likely reason to adopt the conservative design in the tender stage. It is not unreasonable to adopt the conservative design considering time-dependent consolidation property of soft marine clay and likely long elapsed time of un-strutted diaphragm wall (relatively long un-strutted span of about 6.0 m between first strut and final excavation level for 600mm diaphragm wall) due to large volume of excavation work involved.

![Fig. 4. Tender stage bracing system – diaphragm wall was designed with soil-berm and 2 struts support (horizontal bracing and raker)](image)

**Value Engineering Option for Phase 1**

Value engineering review of the temporary works was undertaken by the contractor’s new in-house design engineering team prior to the commencement of Phase 1 excavation works. Rigorous attention to detail of the design concept and constructability was made in the pre-construction discussions between design engineers and construction team.
The main objectives for value engineering options were to minimize the material and construction sequence involved in temporary works so as to accelerate the excavation time thereby saving overall costs, without compromising the safety aspect. After conducting a series of re-analyses with different conditions major modifications were made: (1) To lower the first strut level to –1.8m from the original tender stage design level –1.0m (2) To use only 1 temporary strut, omitting second level raking strut with the provision of sloping soil berm against diaphragm walls. Soil berm was to remove after completion of base slab construction in the majority of area – minimizing the elapsed time of partially un-strutted diaphragm wall between temporary strut and the final excavation level. Modified bracing system of Phase 1 as an outcome of value engineering review is illustrated in Fig. 5.

Fig. 5. Value engineering option - Modified Bracing system for Phase 1

Implementation of the Observational Method

Professor R. B. Peck set out procedures for the observational method (OM) as applied in soil mechanics in the Ninth Rankine Lecture (Peck, 1969). Peck described the limitation and drawbacks of observational method. Powderham (1996) reviewed the main features of the observational method of Peck and summarized the key requirements as follows:

1. It must be possible to alter the design during construction
2. The contractual condition must be compatible and allow design to be directly related to actual construction method
3. An acceptable level of risk must be identified and controlled. In particular this requires a planned course of action for every foreseeable eventuality
4. Critical observation must be identified and obtained

During the review of tender stage design for Phase 1, it was recognized that two-phase excavation works in this project was ideally suitable for application of the observation method. The flexibility of the contractual requirement in temporary design which allowed the contractor to modify the design and construction method also provided the favor for the observational method.

In order to assess the most probable condition assumed in “value engineering design” and to take necessary actions if monitoring results reveal most unfavorable conditions (i.e.
actual deflection of diaphragm wall reaches maximum acceptable limit), the observational
method was implemented on the followings basis.

- Reviewed the design parameters together with critical conditions posed on sites as well as
  most critical stage in excavation and basement construction work
- Predicted the performance of diaphragm wall with “most probable” as well as “most
  unfavorable” conditions and parameters.
- Established the trigger criteria based on predicted diaphragm wall deflection
- Predefined the practical contingency plan for “most unfavorable” conditions where wall
  deflection reaches trigger levels
- Set out the instrumentation program with the consideration of above factors
- Monitored the performance of diaphragm wall. Compared the monitoring results with the
  predicted and trigger values and reassessed
- Implemented the contingency measures if monitoring result reaches action level of trigger
  values

Figure 6 shows the predicted diaphragm wall lateral displacement or deflection of
Phase 1 (east, west and south diaphragm wall) at two conditions together with trigger levels
and tender stage prediction. It should be noted that the diaphragm wall deflection was
predicted to be maximum or most critical after removing the horizontal temporary strut.

![Figure 6](image_url)

**Fig. 6.** Prediction of diaphragm wall performance in pre-construction stage with trigger
criteria in comparison with tender stage prediction

Most probable condition was established for the predicted deflection of diaphragm
wall with full influence of buttress support – assuming buttress effectively supports as
permanent strut in diaphragm wall analysis model. Most unfavorable condition was set out for
the predicted deflection of diaphragm wall without considering influence of buttress –
buttress was excluded in the model. Diaphragm wall reinforcement was designed based on the
most unfavorable condition.

In establishing the criteria for trigger values, it was necessary to consider the broad context in which diaphragm wall exists, design assumption and concept, likely behaviour of the wall itself or its predicted performance and effectiveness of selected temporary bracing system. Trigger levels were established to provide the design team and construction team, an opportunity for early review and resetting of the monitoring frequency as well as for implementing the contingency measure as necessary.

In general, exceeding alert trigger levels must initiate a review of design data, construction progress and monitoring frequencies with the consideration of possible measures to limit further deflection. Exceeding action trigger levels must initiate further review of above mentioned points and if necessary to initiate a planned course of action or contingency measures.

Effective and good communications between the design team and construction crew were made all along the excavation stages with clear responsibilities in construction control. The contingency measures included immediate backing filling of excavated soil and installing of temporary struts in the critical area.

Monitoring Program

In planning the monitoring system and program, it is important to consider the parameters to be measured which reflect the actual performance of the diaphragm wall support excavation. It is also necessary to take account the practical measurement applicable for established trigger criteria, number and frequency of measurement required to carry out a meaningful interpretation of wall behaviour which would be integrated in the implementation of observational method.

Comprehensive and robust monitoring program was set up as a key element in application of the observational method and to ensure that modified construction sequence would not have adverse effect in temporary stage and on permanent design. A total of 6 inclinometers (3 in each phase) were installed in diaphragm wall together with some survey points. In order to make effective use of the established trigger levels, an adequate number of measurement were carried out at appropriate frequency. Typical monitoring frequencies for inclinometer set up as guideline for the project is outlined below.

• Measured immediately before commencing excavation in the vicinity of instrument
• Minimum readings of 2 times a week while excavation in progress
• Minimum readings of 1 time a week when no excavation the vicinity of instrument
• Minimum readings of 3 times a week when measured deflection values exceeded alert trigger levels
• Minimum reading 1 time a day when measured deflection values reached action trigger levels

As the most critical stage was predicted at the time horizontal temporary bracings were removed, a full attention was paid to the inclinometer monitoring with the following special criteria and frequencies.

• First temporary strut removal was to carry out at the diaphragm wall panel where inclinometer was located
• Measured immediately before removal of temporary strut at the closest distance to the instrument.
• Measured every 6-8 hours immediately after removing the first strut
• Second strut to be removed must be the one located immediately adjacent to first strut
which had removed
- Not to remove the second strut until inclinometer measured deflection values had stabilized
- Not to remove more struts unless measured deflection values were stabilized and within alarm trigger levels

In addition to inclinometer measurement, diaphragm wall movement was also monitored by the survey points strategically marked on the wall panels. Ground settlement and surface cracks behind the diaphragm wall were also visually checked by the construction team as daily basic.

Performance of Phase 1 Diaphragm Wall

After carrying out the comprehensive desk studies and establishment of systematic monitoring program presented above, Phase 1 excavation work was carefully commenced. Figure 7 shows the maximum accumulated diaphragm wall deflection at different stages of excavation monitored by inclinometer No.1 (I-1 at East wall of Phase 1) together with trigger levels and predicted maximum deflection profile of 3 different conditions - tender stage design (2 temporary struts), modified design with buttress and modified design without buttress. It can be observed from figure that measured lateral movement pattern of diaphragm wall agreed well with that of prediction for modified design with buttress - meaning buttress-support has significant influence on wall deflection.

Fig. 7. Phase 1 East Wall-Monitored diaphragm wall deflection at different stages with trigger levels
Deflection profile of South diaphragm wall which braced against temporary sheet pile wall at Phase 1 and 2 boundaries is presented in Fig. 8. As can be seen in Fig. 8, South diaphragm wall deflection is significantly higher than that of east wall, which is likely to be caused by the fact that South diaphragm wall is braced with more flexible sheet pile wall. Description of stages shown in the legend of Fig. 7 and 8 is summarized in Table 2.

Table 2. Description of stages shown in Fig. 7 & 8

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Excavate to –2.2m and installed temporary strut</td>
</tr>
<tr>
<td>2</td>
<td>Excavate to –4m</td>
</tr>
<tr>
<td>3</td>
<td>Excavate to –6.6m with berm</td>
</tr>
<tr>
<td>4</td>
<td>Removal of berm</td>
</tr>
<tr>
<td>5</td>
<td>Removal of temporary strut after completion of buttress</td>
</tr>
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</table>
Modification of Phase 2 Bracing System

Monitoring results of the Phase 1 excavation work provided an ample opportunity to review the design assumption, fine tune the parameters used in the analysis of the diaphragm wall for the Phase 2 and made modification of construction sequence. The major modifications are: (1) Removal of soil-berm at East and West diaphragm wall in shorter duration than that of Phase 1, and (2) Using raking struts instead of horizontal strut for North diaphragm wall as shown in Fig. 10.

Fig. 9. Phase 1 excavation work in progress with historical Buddhist temple Wat Suthat in background (Phase 2 area was being used as car parking space)

Fig. 10. Bracing system of Phase 2 diaphragm walls
Performance of Phase 2 Diaphragm Wall

Figure 11 depicts the measured deflection of east diaphragm wall. As can be observed in Fig. 11 in comparison with Fig. 7, the maximum deflection of east diaphragm wall in Phase 2 is larger than that of Phase 1.

The likely reasons of this observation are;
- Un-strutted elapsed time for first temporary bracing in Phase 2 was longer than that of Phase 1.
- In Phase 1, horizontal struts were installed in north-south direction which temporary kingpost columns and strut were integrated in crisscross pattern with east-west struts - providing complete-support more rigid bracing system. Whereas in Phase 2, horizontal struts were installed only in east-west direction without having crisscross pattern with north wall – having less rigid bracing system than that of Phase 1.

![Phase 2 East Wall - Monitored diaphragm wall deflection at different stages with trigger levels](image)

**Table 3. Description of stages shown in Fig. 11 & 12**

<table>
<thead>
<tr>
<th>Stage</th>
<th>East Diaphragm wall</th>
<th>North Diaphragm wall</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>Excavate to –2.2m and installed temporary strut</td>
<td>Excavate to –2.2m at d-wall, and to –6.6m with sloping berm</td>
</tr>
<tr>
<td>2</td>
<td>Excavate to –4m</td>
<td>Installed raking strut</td>
</tr>
<tr>
<td>3</td>
<td>Excavate to –6.6m with berm</td>
<td>Removal of berm</td>
</tr>
<tr>
<td>4</td>
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</tr>
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</table>

With assurance of diaphragm wall performance from monitoring results of Phase 1, original plan of using horizontal struts for North diaphragm wall was modified by using raking struts instead. As can be seen in Fig. 12, deflection of North diaphragm wall (with raking strut support) is significantly higher than that of East diaphragm wall (with horizontal struts).
strut support). The main reason of larger movement of north diaphragm wall is due to the fact that it was supported only by the berm for the long period (about 52 days) before completion of raking struts so that soil-berm became soften during the long elapsed un-strutted period. Time-dependent deflection pattern due to softening and deformation of soft clay can be observed in North diaphragm wall as illustrated in Fig. 13. North diaphragm wall moved progressively toward excavation before completion of raking struts (at 52 days) as can be seen in Fig. 14.

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Fig. 12. Phase 2 North Wall - Monitored diaphragm wall deflection at different stages with trigger levels
Fig. 13. View of buttress-support diaphragm wall prior to temporary bracing removal

Fig. 14. Phase 2 – Time dependent wall deflection of North diaphragm wall – diaphragm wall was supported only by soil-berm for 52 days before completion of raker installation.

Implementation of Contingency Plan

Since deflection of North diaphragm wall (Phase 2) approached action trigger levels, monitoring frequency was increased and the following contingency measures were
implemented on site.

- Poured 15cm thick 1m wide lean concrete on the top of the berm along North diaphragm wall to provide bearing-effect
- Installed additional king-post and diagonal struts attached to the raking struts to provide more rigid support against diaphragm wall
- Soil-berm was removed locally in bays followed by construction of wale beam, tie beam and buttress as shown in Fig. 15.

Fig. 15. Perspective of North diaphragm wall constructed in bay

Movement of diaphragm wall was observed to be decreased and eventually stabilized by the above actions. No significant ground settlement was observed in the vicinity of the North diaphragm wall.

Fig. 16. View of raker and soil-berm support – soil-berm was removed locally bay by bay
Time and Cost Saving from Value Engineering Options and the Application of Observational Method

Significant cost and time saving were achieved from the value engineering option coupled with observational method implemented for both Phase 1 and 2. The major savings were achieved by less operation and material utilized in the following elements of temporary works.

- Cancellation of 2\textsuperscript{nd} level raking struts against diaphragm wall for both phases
- Modification of bracing system – using raking struts with soil-berm support instead of horizontal struts for North diaphragm wall in Phase 2

Key Factors Contributed in Successful Application of Observational Method

Outcome of a through desk study at post-tender stage provided an effective value engineering option which offered significant cost and time saving for overall construction program. Effective and good communications between the design team and construction crew played a key role in successful application of observational method to completion of the project. Systematic monitoring program with clear defined trigger criteria was also the
important element in implementing the observational method. This case study reveals that a thin permanent diaphragm wall coupled with effective design and construction method supplemented by the observational method and robust monitoring program could offer a logistically and financially attractive solution in construction of underground car park without disturbing the environment in the prominent historical area of Bangkok.

CONTRACT 1 OF MRTA CHALOEM RATCHAMONGKHON LINE PROJECT

Observational method was applied mainly in building and structure damage assessment in this Bangkok very first mass transit system project. The M.R.T Chaloem Ratchamongkhon Line is the first underground mass transit system project in Bangkok developed by the Mass Rapid Transit Authority (MRTA) of Thailand. Total length of the underground structure is 21.5km, comprises of 16 km of twin single-track bored-tunnel, a total of 4 km long 18 cut-and-cover stations, a twin 1.5km long cut-and-cover approach tunnel to Depot and other associated structures. The project was commenced in late 1996 and opened for the public in 2004. The Contract No. 1, Underground Structure South - southern portion of this initial system comprising 9 km of twin 6m outside diameter bored tunnels, nine underground stations and cut-and-cover depot approach tunnel, was awarded to the Joint Venture BCKT consisting of Bilfinger + Berger Bauaktiengesellschaft, Ch Karnchang Public Company Limited, Kumagai Gumi Company Limited and Tokyu Construction Company Limited. The project was commenced in late 1996, successfully completed and opened for the public in 2004.

One of the major challenges of the Contract No. 1 was the need of stacked-alignment for a major portion of twin bored-tunnel underneath Rama IV road, one of the busiest roads of Bangkok, which called for a stacked configuration at 3 stations, Lumpini, Si Lom and Sam Yan. This requirement led the contractor to design and construct the deepest underground structures of Bangkok at Si Lom Station - four-floor stacked-station of 32m deep right under the existing flyover and adjacent to a number of sensitive structures. Deep excavation and bored tunneling works in congested urban environment of Metropolitan Bangkok requires a systematic process of ground movement prediction, building risk damage assessment and protection.

Subsoil Condition along Contract 1 Alignment

Similar to other localities in Bangkok, a typical subsoil profile along the tunnel alignment is characterized by the alternating layers of clay and sand deposits as shown in Figure 18. Weathered crust of 2 m thick is commonly found as the top layer. In urban areas of Bangkok, this layer is covered by fill material. Soft to very soft, highly compressible dark gray marine clay lies beneath weathered crust or fill. Depending on the location, this layer extends up to 12-16 m. About 2 m thick medium clay layer can be observed between soft clay and underlying stiff clay. Generally stiff Clay layer occurs directly underneath medium clay and its depth goes up to 22 m. Below stiff clay layer, first sand layer of 5-8 m thickness can be found. This first sand layer, however, is absent in some areas. Stiff to hard clay layer underlies first sand and it is found to be about 5 m thick. Second sand layer generally occurs at depths between 45 to 65 m.
Overview of Ground Movement Prediction and Building Damage Assessment Process
Applied in the Project

Well accepted published literatures were used as basic reference and method in ground movement prediction and building damage assessment. For the station boxes where cut-and-cover methods were used for excavation, the work of Peck (1969) and Clough and O’Rourke (1990) were used whereas published papers of Burland et. al. (1977) and Boscardin & Cording (1989) were applied for bored tunnels. Building damage assessment was carried out in 3 stages; Stage 1 Assessment, Stage 2 Assessment, and Stage 3 Assessment. “Greenfield Site” assumption was adopted in Stage 1 and 2 Assessment ignoring any influence of the stiffness of the structure and type of supporting foundation. Stage 2 Assessment was further extended by Additional Stage 2 Assessment. In Additional Stage 2 assessment, the variation of settlement trough shape and magnitude with depth were considered in identifying the risk of building damage. In Stage 3 assessment, a detailed qualitative assessment of structure and vulnerability of each building or structure to damage were taken into account, coupled with detailed structural analysis in assessing the buildings falling in risk category 3 classified by Mair et. al. (1996).
Summary of Instrumentation Program

Instrumentation played a major role in the construction of this first underground mass transit system project in congestive urban area with difficult soil condition. The primary objective of the instrumentation program was to monitor the performance of the deep excavation and tunneling operation to ensure safe execution of the construction works and that adjacent structures were not adversely affected. Furthermore, the instrumentation program was established to provide feedback in application of the observational method. Extensive sets of instrumentation were installed around the predefined influence zones of station box excavation, tunneling and associated building, structures and utilities. In addition to other geotechnical instruments, inclinometers, extensometers, various types of settlement points, tile-meter, crack meter and crack gauge were installed to monitor the ground and associated building and structures movement.

Building Condition Survey

Prior to commencement of the major works, all the existing buildings and structures within the predefined influence zone were systematically surveyed. The primary objectives of the surveys were;

- To record pre-construction status of all buildings and structures within the influence zone. The records included were age of building, building type, storey height, expected foundation types, visible defects supported by sketches and photographs as appropriate
- To utilize the information collected from the building / structure condition survey in the assessment of the buildings and evaluate sensitive structures with respect to construction aspects and monitoring
- To establish a benchmark to monitor the possible effect of construction and feedback information in applying observational method and if necessary to implement the building protection measure

Expected foundation types of different buildings were classified based on the height, number of floors and age of the buildings. Building survey information was also used to identify the types of instrumentation required.

Building Damage Assessment and Observational Method

Process of Stage-Assessment is shown in figure 20. After carrying out the initial assessment of potential building damage, it was recognized that without applying the
observational method, the result obtained from assessment process could be conservative for some buildings and could be underestimated for others. For example, the settlement estimated in Second Stage Assessment shown in above diagram was based on surface movement. For the buildings located in close proximity to the excavation founded on piles, it was considered that subsurface settlement may be more critical. Stage 2 Assessment was then further extended by Additional Stage 2 Assessment in which the variation of settlement trough shape and magnitude with depth were considered in identifying the risk of building damage. Settlement of the buildings due to the excavation works were predicted assuming the “green filed” condition. Neglecting restraints from foundation and structure, it was assumed that buildings follow the ground settlement trough at foundation level (estimated pile tip level). However, if the building is supported by deep foundations, assessment result could be too conservative.

Figure 19. Diagram showing process of Staged-Assessment

Available methods were further reviewed to predict the surface and subsurface settlement with the consideration of two main factors; (1) simple and practical in application (2) enable to correlate with the predicted and measured diaphragm wall deflection. The method proposed by Bowels (1988) was selected as it meets the required criteria mentioned above. Bowels suggested the ground settlement induced by excavation as a function of ground loss due to the deflection of the retaining wall. Bowl demonstrated the calculation of
settlements at specified distance by assuming parabolic variations of settlement within the influence distance. Using predicted diaphragm wall deflection, surface settlement behind the wall was computed by empirical formulas proposed by Bowels.

A simplified prediction of subsurface settlement was carried out based on the calculated surface settlement described above. First, subsurface settlement influence line was constructed. As shown in Figure 20, settlement influence zone is assumed to decrease with depth from “D_o” at the surface and zero at the wall toe. With the assumption of linear relationship between the volume of deflected wall shape and the volume of settlement trough at any depth within settlement influence zone, subsurface settlement at different depths were calculated.

\[ S_{WO} = \frac{4 V_o}{D_o} \]

\[ S_{wa} = S_{WO} \left( \frac{x}{D_o} \right)^2 \]

\[ D_y = y \frac{D_o}{H_W} \]

\[ V_{ty} = V_y \frac{V_{To}}{V_o} \]

Figure 20. Demonstration of subsurface settlement prediction from diaphragm wall deflection values

Based on the polynomial curve developed in Stage 2 Assessment, subsurface settlement can be predicted by the similar approach described above. With predicted surface and subsurface ground movement, risk of damage to the buildings were determined by two methods (i) Risk of damage based on maximum tensile strain, the method proposed by Mair et. al. (1996) and (ii) Risk of damage specified by angular distortion and horizontal strain using the chart proposed by Boscarding and Cording (1989). Table 4 shows the summary of building damage assessment, risk category obtained for different types of foundation determined by two different methods.
Table 4  Summary of risk category obtained from two different methods and foundation type

<table>
<thead>
<tr>
<th>Building Block</th>
<th>No. of stories</th>
<th>Building Type</th>
<th>Estimated Pile Length</th>
<th>Distance to edge of wall</th>
<th>Risk category by maximum tensile strain</th>
<th>Risk category by chart of Boscarding and Cording (1989)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Without pile</td>
<td>With pile</td>
</tr>
<tr>
<td>A</td>
<td>2</td>
<td>Masonry / Concrete</td>
<td>7-14m</td>
<td>18m</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>B</td>
<td>2</td>
<td>Masonry / Concrete</td>
<td>7-14m</td>
<td>17.5m</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>2</td>
<td>Masonry / Concrete</td>
<td>7-14m</td>
<td>32m</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>D</td>
<td>1</td>
<td>Masonry / Concrete</td>
<td>&lt; 6m</td>
<td>2m</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>E</td>
<td>37</td>
<td>RC</td>
<td>50m</td>
<td>8m</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

The predicted degree of damage of the buildings using simplified chart is more critical than that of maximum tensile strain method as can be seen in above table. According to the results obtained from of simplified chart, predicted degree of damage was higher if buildings were assumed to be with piled foundation rather than without piled foundation.

In order to justify the most probable condition assumed in building damage assessment and to take necessary actions if monitoring results reveal most unfavorable conditions (i.e. ground movement induced by deflection of diaphragm wall reaches maximum acceptable limit), observational method was implemented on the followings basis.

- Reviewed the building condition survey record paying particular attention to the type of foundation and existing visible defects
- Predicted the ground movement based on the calculated diaphragm wall deflection
- Performed structural assessment based on the predicted ground movement (for Stage 3 assessed buildings)
- Established the trigger criteria based on the magnitude of ground movement (maximum settlement and differential settlement between foundations or structural columns)
- Predefined the practical contingency plan for “most unfavorable” conditions where ground movement reaches trigger levels (ground movement was derived using simple approach based on diaphragm wall deflection described in earlier section)
- Set out the instrumentation program with the consideration of above factors.
- Strictly controlled the construction sequence assumed and defined in design and if necessary changed or modified the sequence
• Monitored the performance of retaining wall both diaphragm wall (for station boxes) and sheet pile walls (for station entrance structures). Compared the monitoring results with the predicted and trigger values and reassessed
• Monitored the building settlement. Compared the monitoring results with the predicted and trigger values and reassessed
• Implemented the contingency measures if monitoring result reaches action level of trigger values

Monitoring results obtained from early stage of the project were used to reassess the potential damage of the buildings and structures. Some buildings of critical condition were put in Stage 3 assessment requirement and assessed by structural analysis using detailed structural survey information as basic data. By simulating the differential settlement of the building predicted from Additional Stage 2 Assessment, development of moment and shear in the main structural members were determined with the use of structural analysis software. Computed moment and shear were assessed in comparison with the capacity of the structural members estimated from collected building condition survey and detailed structural survey data. The process involved in structural assessment can be summarized below.
• Factor of safety of each structural members were determined base on predicted differential settlement of the buildings.
• A series of assessment were performed to determine the critical differential settlement causing structural instability of the members or differential settlement at limiting factor of safety.
• Trigger values were established based on above results.
• Based on the trigger values and the actual building condition and response in conjunction with monitoring data, the most sensitive structural members were protected, mainly by strengthening approach.

The following assumptions were made in the conservative approach of Stage 3 assessment.
• Building movement is influence by excavation induced settlement of both main station box and associated underground structures (e.g. Entrance Building)
• Building settlement was caused by subsurface settlement at foundation level as buildings are supported by piled foundation.
• No allowance was made for structural stiffness of the building.
• No allowance was made for possible effects of soil-structure interaction.

A summary of risk category determination in Stage 3 Assessment based on the predicted and monitored settlement of the building adjacent to station excavation work is shown in the Table 4 below.
Table 4  Summary of risk category determination based on Boscarding and Cording (1989) – based on prediction and actual monitoring of settlement

<table>
<thead>
<tr>
<th>Data Taken From</th>
<th>Horizontal Strain</th>
<th>Angular Distortion</th>
<th>Risk Category</th>
<th>Degree of Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sh1 (mm)</td>
<td>Sh2 (mm)</td>
<td>L (m)</td>
<td>εh</td>
</tr>
<tr>
<td>Settlement Prediction</td>
<td>31</td>
<td>24</td>
<td>18.6</td>
<td>0.0004</td>
</tr>
<tr>
<td>Actual Monitoring Data</td>
<td>31*</td>
<td>24*</td>
<td>18.6</td>
<td>0.0004</td>
</tr>
</tbody>
</table>

*Predicted horizontal movement was used in computing horizontal strain for actual miniotring data.

From the above table, following conclusions may be made.

- Predicted angular distortion broadly agrees with that of actual monitoring data. Hence, settlement prediction and assumed effective depth of piled foundation is fairly reasonable.
- The settlement of actual monitoring data is slightly higher than that of prediction. This may be due to the additional settlement caused by diaphragm wall panel construction which influence zone might have extended to 15m from the diaphragm wall (equivalent distance to bottom of soft clay level).

**Damage Assessment on Sensitive Structures**

The most critical structure required for Stage 3 Assessment were MWA main water tunnel as tunnel alignment crosses the water tunnels at two critical locations – the first near Rama IV and Rachadaphisek road intersection and the second at Rama IV and Siphraya road intersection near Sam Yan station. The position of water tunnels in relation to MRT tunnels drawn in the subsoil profile at relevant locations are presented in Figure 21 and 22.

According to the available information, MWA tunnel at the first crossing is of 2.5m internal diameter steel lining with 150mm segmental ring and at the second crossing MWA tunnel is of 2.5m diameter segmental ring with lightly reinforced concrete lining.

In preliminary stage 3 assessment, based on the predicted subsurface profile using the method described in published paper “Ground Movement Prediction and Building Damage Risk-Assessment for the Deep Excavations and Tunneling Works in Bangkok Subsoil”, Zaw et. al (2006) e, structural assessment of water main was carried out and results were compared with structural capacity of water tunnel calculated from the material properties indicated in the construction drawings provided by MWA. As bending moment induced in the water tunnel was considered critical parameter in assessing the damage which is governed by both maximum settlement and curvature of the water tunnel further assessment was proposed coupled with observational approach.
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<table>
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<th>Settlement Prediction</th>
<th>Actual Monitoring Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sh1 (mm)</td>
<td>31</td>
<td>31*</td>
</tr>
<tr>
<td>Sh2 (mm)</td>
<td>24</td>
<td>24*</td>
</tr>
<tr>
<td>L (m)</td>
<td>18.6</td>
<td>18.6</td>
</tr>
<tr>
<td>εh</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
<tr>
<td>Sv1 (mm)</td>
<td>48</td>
<td>53*</td>
</tr>
<tr>
<td>Sv2 (mm)</td>
<td>23</td>
<td>14*</td>
</tr>
<tr>
<td>ΔS (mm)</td>
<td>25</td>
<td>39*</td>
</tr>
<tr>
<td>L (m)</td>
<td>18.6</td>
<td>18.6</td>
</tr>
<tr>
<td>β</td>
<td>0.0013</td>
<td>0.0020</td>
</tr>
</tbody>
</table>

*Predicted horizontal movement was used in computing horizontal strain for actual monitoring data.

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- To carry out back-calculation using monitoring data obtained from the settlement arrays (settlement points, extensometer data) at first 3 locations before reaching to MWWA water tunnel near Rama IV – Rachadapisek intersection, to determine the volume loss and relationship between surface and subsurface settlement as well as best estimation of subsurface K value and subsurface settlement profile at water tunnel level.
- To perform beam-on-elastic foundation analysis using the subsurface settlement profile obtained from predicted subsurface settlement bending moment likely to occur in MWWA water tunnel.
- To compare the result obtained from above item with allowable values.
- To establish the final trigger values based on above comparison.
- To confirm the settlement profile from actual monitoring of last array is within the alert level of final trigger value.

Figure 21.  Position of MWA water tunnel in relation to MRT tunnels (clear distance are 3.38m and 3.18m for North and South bound respectively) at the first crossing
Figure 22. Position of MWA water tunnel in relation to MRT tunnels (clear distance are 1.5m and 2m for North and South bound respectively) at the second crossing

For the first location of MRT-MWA tunnel crossing, target ground loss from tunnel operation was set at 0.5%, whereas ground loss based on allowable movement curvature of water tunnel derived from predicted pattern of subsurface settlement was 0.65%. Back-calculated ground loss derived from the monitoring data of rod-extensometer placed at the crown of MWA water tunnel was within target value. In addition to the preliminary assessment based on prediction of subsurface settlement using simple empirical formulas, a detailed soil-structure interaction was analyzed using 2-dimensional FEM considering behavior of tunnel boring by earth pressure balance (EPB) method. Stress relief from the effect of cutter-head, tail void and over-cut from EPB tunnel boring was considered in the analyses. Adding ground movement due to consolidation caused by disturbance from tunnel operation surface and subsurface settlement were predicted by finite element analysis. Detailed parameters and assumption made in analysis was presented in the published paper of Sakai & Sugden (2000).

After the successful passing of both tunnels at the first MWA crossing, another trial area tunneling was planned to further determine the ground movement associated with various tunneling parameters well before reaching to more critical crossing after Sam Yan station. Tunnel section between Silom and Samyan station, in which soil profile similar to that of second crossing was selected and comprehensive instrumentation program was established to monitor surface and subsurface settlement as well as horizontal movement of the ground at depth.

The observed ground movement induced by tunneling works were within predicted values. Well organized plan, systematic approach of observational method to achieve control
values backup by extensive instrumentation and close co-operation among relevant personnel were the key factors contributed to successful completion of EPB tunneling particularly in the most sensitive points at MWA water tunnel crossings. The monitored subsurface settlement data from extensometer and predicted settlement trough of assumed ground loss 1% ground settlement profile of the first MWA tunnel crossing is presented in Figure 23.

![Figure 23](image)

**Figure 23** Predicted subsurface settlement trough of assumed ground loss 1% calculated from empirical formula and actual subsurface settlement (indicated by dark-circle) observed from rod extensometer – first MWA tunnel crossing

**Key Factors in Successful Application of Observational Method**

The M.R.T Chaloem Ratchamongkhon Line, one of the most modern underground mass transit systems in the region was successfully completed and has been in operation for over 2 years. All deep excavation works and tunneling were successfully completed without any significant damages to the adjacent buildings and structures. Well organized plan, systematic approach of observational method to achieve control values backup by extensive instrumentation and close co-operation among relevant personnel were the key factors contributed to successful completion of the project. The purpose of this paper is to provide a set of technical guidelines for ground movement prediction and building damage risk assessment of deep excavation and tunneling works in Bangkok subsoil and application of observational method which may partly serve as a reference for the future underground mass transit system projects in urban area of Bangkok. Successful completion of the first MRT project in Bangkok marked the practicality of well-executed deep excavation works and tunneling by EPB method coupled with observational method in the sensitive urban environment.
REFERENCES


Clough, G.W., and O’Rourke, T.D., (1990), Construction Induced Movements of In-situ Walls, ASCE Geotechnical Special Publication No. 25, pp. 439-470

Clough C.W. and T.D. O’Rourke (1990), Deep Excavation and Tunneling, 7th ICSME, Mexico City

JV BCKT, (1997) Geotechnical Interpretation Report (GIR) for MRTA Initial System - UGS, Bangkok


O’Reilly M.P and New B.M (1982), Settlements above tunnels in United Kingdom – their magnitude and prediction, Tunneling ’82, pp. 173-181, London, IMM


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Ground Movements Associated with the Underground Excavations of the First Bangkok MRT Line

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Zaw Zaw Aye
SEAFCO Public Co., Ltd. Bangkok, Thailand
Abstract: Works of the first Bangkok Underground MRT project involved the largest diameter shield tunneling and the deepest diaphragm wall excavation ever attempted in Bangkok subsoils. The design-built contracts required the design and construction to avoid damages to existing buildings, thus the Observational Method was adopted and large amount of instrumentation for monitoring of ground movements was integrated as an important component of the works. The ground and building movement data associated with the shield tunneling and station excavation are interpreted and the findings are briefly presented. Tunnel volume loss in the EPB shield tunneling was typically in the range of 0.5-2.5% while the maximum ground surface settlement above the tunnel was typically 20-60 mm. The magnitude of ground movement varied with the face pressure used in the shield driving. The settlement trough width parameter was $i = 0.5-0.6 \, Z$ for tunneling in clay layers and $< 0.4Z$ for tunneling in the underlying sand layer. Maximum lateral deflection in braced mode of the diaphragm wall in the station excavation was in the range of 0.05-0.4% of the excavation depth. A similar range was observed for the maximum ground surface settlement adjacent to the wall. There were only limited cases of damages to adjacent buildings, which occurred only to small sized buildings founded on short piles.

1 INTRODUCTION

The Bangkok MRT Initial System Project (ISP), the Chaloem Ratchamongkol Line, is the first underground MRT of Bangkok. Total length of the line is 21.5 km, comprising 16 km of twin single-track tunnels, 18 cut-and-cover stations and twin 1.5-km-long cut-and-cover approach tunnels to Depot. The project was commenced in late 1996 and opened for public use in 2004. The alignment mostly followed congested city roads. Construction works of the underground structures were carried out in two fast track design-build contracts, the South and North Sections. The tunnel tubes were bored at 6.3-m diameter by 8 EPB TBMs while the stations were excavated with an aid of diaphragm walls following the top down construction method. The depth of the tunnels was between 8 and 25 m below ground level. The general layout of the twin tunnels is in side by side configuration (mostly with clearance between tunnel walls of 1.0-1.5 times the excavated tunnel diameter (5.0-9.0 m) but the smallest clearance was as smaller as 2.6 m). The vertically stacked configuration of the tunnels was adopted for the first 4.5 km of the alignment from the terminal station at the main intercity railway station owing to obstruction from foundations of elevated roadways and existence of water main tunnels. Consequently, the stations in the section were excavated at depth as deep as 30 m. Two stations are located beneath existing intersection overpasses that necessitated underpinning works. Other design and construction issues involve consideration of the tunnels passing close to concretelined water main tunnels at two locations, and removal and underpinning of piled foundations of four canal bridges.

Prior to the start of the MRT construction, there had been numerous shield tunneling and deep excavation works for both building construction and utilities development. However, underground works of the MRT project marked the largest tunnel diameter and the deepest excavation ever attempted in Bangkok subsoils. Thus great caution must be paid to the design and implementation of the works for safety assurance of the works and success of the project development.

Control of ground movement and preventing damages to adjacent structures from the excavation was the key for the success for the project. The Observation Method design was adopted for that purpose. A comprehensive instrumentation program for monitoring of ground movements and responses of concerned structures was carried out as an important component of the contracts. The monitoring data served as indices for control of works and safety assurance during construction as well as allowed for a better understanding on the ground movement behavior induced by EPB shield tunneling and deep excavation with diaphragm wall in the stratified Bangkok soils.

2 SUBSOIL CONDITIONS

Bangkok is situated on the low-lying plain. The subsoil within the first 20-30 m depth generally consists of layers of soft clay, stiff to hard clay, and sand. Below are alternating layers of stiff to
hard clay and dense sand existing to a great depth. The subsoil condition is relatively uniform throughout the city area. The soft clay layer generally extends from the ground surface to a depth of 12 to 15 m. The soft clay, which is known as “Bangkok soft clay,” has high water content (70-120%). The underlying stiff clay layer has much better engineering properties than the soft clay and is an ideal medium for tunnelling within. Below the stiff clay layer is a layer of fine to medium, dense to very dense, silty to clayey sand. The sand is normally saturated but the piezometric pressure has been considerably drawn down as a result of excessive deep well pumping in the past 60 years. Presently, the piezometric head in the sand layer is mostly at 21-24 m below the ground surface. The condition is beneficial to tunnelling and deep excavation works. Fig. 1 shows subsoil stratigraphy along the MRT alignment. Although most of the tunnel alignment is placed in the stiff clay layer, some shallow sections have roof in the overlying soft clay and some of the deep alignment was excavated in the underlying sand layer.

3 TUNNEL EXCAVATION

Eight earth-pressure-balanced (EPB) shields were used in the 16 km long twin tunnel excavation-4 Kawasaki shields in North Contract and 2 Kawasaki + 2 Herrenknecht shields in South Contract. The tunnels were supported with 300-mm-thick bolted concrete segmental lining ring. To achieve effective performance of the EPB shield operation, conditioning additives were added to excavated soils in the EPB chamber. The additives were bentonite slurry, polymer, or organic foam. The four shields in each of the contracts took about 14-15 months to excavate the tunnel length. The average advance rate of excavation of each shield is 60-70 m/week. Despite some major downtime due to breakdowns of main parts, all of the shields performed very satisfactorily in all types of soils. The rates of excavation of the EPB shields in the MRT project in all types of soil were essentially similar except for excavation in the stiff clay that the rate could be smaller due to higher cohesion and stickiness.

Subsurface Condition of North Section

Subsurface Condition of South Section

Fig. 1 Subsoil stratigraphy along the MRT alignment
4  STATION EXCAVATION

Excavation of the 18 stations was made by the top-down construction using 1-m-thick concrete diaphragm walls. Tips of the wall usually extend to the depth of 35 - 40 m below the ground surface (Fig. 2) to satisfy load bearing and uplift resistance requirements as well as to provide ground water cutoff in the first sand layer. The distinct difference in depth of embedment of the wall below the excavation level between the two contracts was partly due to the difference in the adopted design criteria. Construction period of each station from utility relocation to architectural finishes was usually around 45 months- 6 months of piling and diaphragm wall construction and 18 months of excavation and casting of station floors and walls.

5  OBSERVATIONAL METHOD

The contract required that the design and construction methods limit ground movements from tunneling and station excavation to avoid damage to adjacent structures and buildings. The Observational Method was then adopted for the control of work. It involved the followings.

- Three stages of assessment on the effects of the ground movement on structures depending upon the results of each stage.
- In stage one, prediction of the green filed ground surface settlement using approved empirical prediction methods was made.
- Building condition surveys of all structures within the zone of influence determined by the first stage. Any structures with the predicted ground settlement less than 10 mm need not be subject to further assessment.
- Structures located in the zone of surface settlement were checked for potential damages according to the limit tensile strain criteria (Boscardin & Cording, 1989, Burland, 1995). Structures falling in damage category within slight level should be acceptably safe thus need no further assessment.
- Stage 3 assessment of structures exceeding slight level of damage need stage assessment which involved detailed structure analysis considering soil structure interaction. Depending on the results of the analysis, pre-excavation protection or preventive measures as well as contingency plan to be implemented during construction may be needed.
- Monitoring of settlements arising from works must be carried out. Comprehensive instrumentation program was required as a part of the contract.

Relatively few incidents of damages to adjacent structures occurred. The most significant ones were damages to small buildings on short piles. No cases of damages to tall buildings were experienced.

6  GROUND MOVEMENTS IN TUNNELLING

6.1 Surface Settlement

Monitoring of ground surface settlements over the tunnel alignment was made at 50 m intervals. The data showed that the

maximum settlements were in the range of 5 mm to 120 mm, mostly between 20 mm and 60 mm (Fig. 3). Larger settlement tended to occur in the break-ins and break-outs areas of the tunneling near the walls of the station boxes. The largest magnitudes were recorded in the early leg of tunneling of the North Contract and in the areas of the vertically-stacked tunnels in South Contract where the tunnel were bored in sand layer below drawn-down ground water level.

Studies were made to examine influencing factors controlling the magnitude of the ground movement in the EPB shield tunneling (Suwansawat, 2002, Timpong, 2002, Tavanun, 2003, etc). It was found that the most important factor seems to be the applied face pressure in the front chamber of the shield machine. The face pressure played an important role in maintaining stabil-
ity of the excavation and minimizing ground movement. Fig. 4 shows the plot between the observed maximum surface settlement and the face pressure in the project. The plot clearly shows the influence of the applied face pressure on the magnitude of ground settlement. The lower the face pressure used, the larger the settlement occurred. In case of a very high face pressure, heave occurred at the ground surface. On the other hand, when the shields were operated at very low face pressures (<100 kPa), large ground movement occurred. The effect of the face pressure was observed in all soil settings except for the case of the full face in sand layer. No trend was found between the face pressure and surface settlements when the tunnels were excavated entirely in sand layers.

Out of the 39 locations of settlement trough monitoring arrays, 19 arrays showed the observed trough shapes reasonably fitted with the Gaussian curve (Peck, 1969), Figs. 5(a) and 5(b). The tunnel ground loss was mostly in the range of 0.5-2.5%. In difficult sections, the ground loss was up to 3.5%.

A plot is made between relationship between the settlement trough width parameter ($i$) and tunnel depth ($z$), Fig. 7. Most of the data points fall within the range of $i = 0.5Z$ to $0.6Z$ for tunnels located in both the stiff clay and soft clay layers and $i < 0.4Z$ for tunnels in sand layer. This is in agreement with the observation of O’Reilly & New (1982) for tunneling in UK.

6.2 Subsurface Ground Movement

Monitoring of subsurface ground movements was made by means of borehole extensometers and inclinometers installed at a number of sections along the tunnel alignment. The layouts of the instruments in the two contracts are shown in Fig. 8.
Fig. 8 Typical layouts of subsurface instruments for shield tunnelling

6.2.1 Subsurface settlement

For areas above the tunnel centerline, the magnitude of subsurface settlement after the tunnel passage generally increased with depth toward the tunnel roof (Fig. 9a). For the settlement to the side of the tunnel wall, it was generally observed that the largest settlement occurred in the soft clay layer near the interface with the underlying stiff clay (Fig. 9b). The settlement in the stiff clay along the instrument tube decreased with depth as the measurement points fall outside the movement zone. At some sections, a small heave was measured below the tunnel level.

Long term ground settlement behavior for the EPB shield tunnel excavation was examined. The subsurface settlement data showed that after the shield passage, significant increase in the settlement in long term was only observed in the soft clay layer. In some cases, the amount of the increase in the settlement 3-4 months after the shield passage was almost 100 percent of the short term settlement (Fig. 10). On the other hand, the long term increase in the settlement in the stiff clay layer near the tunnel roof was generally not significant.

6.2.2 Lateral displacement

The lateral displacements of ground beside the tunnel were observed at 12 locations along the alignment. The instruments were placed at distance of 0.20-2.6 m from the tunnel wall. Most of the measurements showed a similar pattern of the lateral ground movement which can be divided into two different depth zones, i.e. above and below the level of the tunnel arch. Above In the upper zone, the ground displaced towards the tunnel with the maximum displacement Fig. 6 Effect of presence of piled foundation on shape of settlement trough occurring in the lower area of the soft clay layer. The inward displacement within this depth zone continued to increase after the passage of the shield tail.

In the lower zone, two patterns of the lateral ground displacement were observed. In most cases, inward movement toward the tunnel was observed. The magnitude was smaller than those occurring above the tunnel roof. At few sections where high face pressure was used in the shield tunnelling, the ground showed outward movement (Fig. 11). The movement ceased after the passage of the shield tail. This clearly indicates the effect of the applied face pressure in the EPB shield tunnelling in Bangkok soils.

Studies were made to investigate applicable methods for prediction of the ground in shield tunnelling. Both the analytical/empirical approach (e.g. Verruijt & Booker, 1996; Loganathan & Poulos, 1998; etc.) and the numerical approach (e.g. FEM PLAXIS, FDA FLAC, etc.) were investigated (Tavaranum, 2003). For the first approach, it was found that the method of Loganathan & Poulos (1998) method generally can give reasonable fits to subsurface settlement pattern with field data for both single and twin tunnels cases (8 out of 12 sections). The gap parameter, g, from back calculation was in the range of 20-60 mm (Timpong, 2002). Reasonable prediction was also offered by the Verruijt & Booker (1996)’s method. However, they could not sufficiently give reasonable prediction of lateral ground movement for cases of excavation with high face pressures, where outward ground movement occurred (6 out of 12 sections).
7 GROUND MOVEMENTS FROM STATION EXCAVATION

Ground movements associated with the excavation of the 18 stations of the project were closely monitored with systematic instrumentation programs. The monitoring data provided useful information to capture the typical characteristics of lateral ground movement and settlement in the areas adjacent to the controlled deep excavation using diaphragm wall in Bangkok subsoil. The station boxes were generally 230 m long by 25 m wide with excavation depths of 20-30 m. Hooi (2002) conducted comprehensive reviews and interpretation of the monitoring data. The results showed the ratio of maximum lateral wall displacement to the excavation depth at various stages of excavation as shown in Fig. 12. For the cantilever mode occurring in the first stage of excavation (H = 1.6 m – 4.0 m), the $d_{max}/H = 0.20 - 1.60 \%$. For the braced wall mode, the ratio $d_{max}/H = 0.05 - 0.60\%$ while excavating in soft clay layer (H = 6.5 m – 10.8 m). The wall deflection in the braced wall mode while excavating in the stiff clay or sand layers (H = 12.4 m – 32.6 m) showed $d_{max}/H = 0.05 - 0.40\%$. The observed ground settlement adjacent to the excavation walls is summarized in Fig. 13. The ratio of the maximum settlement to excavation depth is high in the first stage cantilever excavation mode. After the excavation advanced down to the braced mode in either the soft clay layer or the underlying stiff clay layer, the ratio of the maximum settlement to excavation depth ($d_{max}/H$) was in the range of 0-0.3\%. The observed width of settlement zone (D) is large, up to 7 D/H for braced mode with 4 D/H for braced mode with excavation base soft clay and reducing to 4 D/H in deeper stiff clay or sand. The influence zone of the diaphragm wall excavation is smaller than that suggested by Peck (1969) for braced excavation using soldier piles/sheet piles. Since the FEM design analyses used for all station excavations of the MRT followed linear elasto-plastic soil property, back-analyses based upon the monitoring data were made to determine the appropriate corresponding modulus values of the Bangkok subsoils for the analysis. The results show that for the soft/medium clay the modulus, $E_u$ should be around 500 Su, where Su is undrained shear strength. For other soil layers underlying the soft clay, the values are:

- 1st Stiff Clay $E_u = 700$ N
- Clayey Sand & Sandy/Silty Clay $E_u = 900$ N
- 2nd Hard Clay $E_u = 1600$ N
- 3rd Hard Clay $E_u = 2500$ N

8. BUILDING RESPONSES

Along the alignment, there were 339 buildings and structures located in the zone of influence. These structures were subjected to a detailed building condition survey. Instruments were installed to monitor movements and cracks that may be induced during and after the passages of the shields. With the existence of the soft clay layer, buildings constructed in Bangkok need to be founded on piled foundations. Different length and size of piles have been used depending on the size and age of the buildings. Generally, the depth of pile tips can be categorized as shown in Fig. 15. The monitoring data of all the instrumented buildings (mostly reinforced concrete frame types) were evaluated and the broad picture of the building response to the tunnel excavation could be depicted by the summary plots of building settlements versus number of storey of the building and distance from the tunnel excavation (Figs. 15 and 16).

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It clearly shows that buildings on deep piles were less influenced by the tunnel excavation (Types 4 Buildings: settlements < 10 mm). For smaller buildings (Types 2 & 3), the settlement could be larger (up to 30 mm). For buildings on short piles (Type
1), settlement could be excessive. Fig 15 shows that significant building settlement (> 20 mm) only recorded for buildings located within distance of 30 m from the tunnel centerline.

The measured differential settlements of the buildings were calculated and expressed in term of settlement ratio or tilt (differential settlement/distance between the two reference points). The observed tilts of these buildings were mostly smaller than 1:1000 (0.001) which was the alert level set forth for the control of work in the Bangkok MRT project. According to Burland (1995) a (masonry) building experiencing a maximum tilt of 1:500 (0.002) and a settlement of less than 10 mm has negligible risk of any damage. In fact, very few cases of damages of the existing buildings due to shield tunneling occurred in the project. These were limited to old 1-3 story buildings founded on short piles (probably with tips within soft clay layers). All of the tall buildings were not damaged by the tunnel excavation even though the tunnel was excavated right next to the piles. A study is currently made to evaluate the influence of the shield tunneling on the load carrying capacity and integrity of the piles.

9 CONCLUSIONS

1. EPB shield tunneling for the first Bangkok MRT was successfully completed with the average advance rate of each shield of 60-70 m/week. Similarly, the deep excavation using diaphragm wall of the 18 stations were completed without major difficulties.

2. Maximum ground surface settlement above the tunnels ranged from 5 mm to 120 mm, mostly 20-60 mm. The corresponding volume loss in the tunnel excavation was 0.5-2.50%. Tunneling at in water bearing sand layer tended to give the largest ground movement.

3. Most of the surface settlement trough could be fitted by Gaussian function. The trough width parameter \( i \) is approximately between 0.4\( Z \) and 0.6\( Z \) for tunnels in clays and less than 0.4\( Z \) for tunnels in sands.

4. The long-term settlements in soft clay layer above the tunnel could be significant in some sections but not for the stiff clay layer.

5. The observed subsurface settlements and lateral displacements could generally be reasonably fitted with the prediction by the analytical methods of Verruijt & Booker (1996) and Loganathan & Poulos (1998). However, the prediction was poor for lateral ground movements in case where lateral outward ground movement occurred.

6. Both maximum lateral wall deflection and maximum ground surface settlement near the station excavation using braced diaphragm walls showed similar ranges of movement. The upper bound magnitude is around 0.3-0.4% of the excavation depth.

7. Settlement of adjacent buildings induced by ground movement from the MRT excavation showed clear correlation with building heights which reflects depth of piled foundations.

REFERENCES


Geotechnical Risk Assessment and Management for Tunnelling Project in Congestive Urban Area – Bangkok Experience

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Abstract: The M.R.T Chaloem Ratchamongkhon Line is the first underground mass transit system project in Bangkok developed by the Mass Rapid Transit Authority (MRTA) of Thailand. This paper presents the methods adopted in prediction of tunneling induced ground movement and associated building damage risk-assessment carried out for the Contract No. 1 of the M.R.T Chaloem Ratchamongkhon Line constructed in congestive urban area. Application of observational method in managing risk of damage to the existing buildings and structures are also reported.

1 INTRODUCTION

Geotechnical risk is one of the key elements of the technical and financial risk in most of the major construction projects. Though engineers use adequate factor of safety and taking account of other risk factors in design and construction process, systematic approach of risk assessment and management is not commonly applied in most of the projects.

Tunneling works and deep excavation works pose a high degree of geotechnical risk that needs to be identified, assessed, minimized and managed. Deep excavation and tunneling works generally interact with natural ground and incorporating their characteristics as structural elements of their own stability. This interaction is more significant in tunneling projects than in other civil engineering projects particularly in congestive urban area. In this paper, process of geotechnical risk assessment in Bangkok first underground mass transit project is presented.

2 GEOTECHNICAL RISK MANAGEMENT

Geotechnical risk is the risk to construction project originated from the ground condition. In general case, once risk have been identified and assessed, to manage the risk fall into one or more of the following major categories.

• Eliminate or avoid
• Transfer
• Mitigate
• Tolerate

Unfortunately, ground-related risks are not often practical to completely avoid or transfer. However, systematic approach, control and management can minimize the risk if not entirely eliminate. CIRIA Report 185 outlined the four fundamental stages of risk management as follows:

• Hazard identification: where hazards are identified and documented in a register via experience of similar projects, brainstorming, structured interviews etc.
• Risk assessment: where the likelihood and consequence of each hazards are evaluated and combined to estimate the risk corresponding to each hazard. A risk register is produced at this stage for systematic assessment of the risk.
• Risk reduction: where the hazards are eliminated if possible and the risks are reduced by a combination of design changes, procedural changes and additional monitoring
• Risk control: where the risks are monitored, controlled and managed throughout the project. The risks normally diminish as the construction progresses.

In general, risk management is the process of managing response and policies to reduce and control risks. It is an integral part of the observational method, which concentrates on technical risk (CIRIA Report 185). The merits of using the OM to manage ground-related risks have been discussed by Whitman (1984), Blockely (1994), Chowdhury (1994) and Godfrey (1996). Roles of different parties involved in construction project namely the client, the designer and constructor are described in details by Clayton (2001).

3 CONTRACT 1 OF MRTA CHALOEM RATCHAMONGKHON LINE PROJECT

The M.R.T Chaloem Ratchamongkhon Line is the first underground mass transit system project in Bangkok developed by the Mass Rapid Transit Authority (MRTA) of Thailand. Total length of the underground structure is 21.5km, comprises of 16 km of twin single-track bored-tunnel, a total of 4 km long 18 cut-and-cover stations, a twin 1.5km long cut-and-cover approach tunnel to Depot and other associated structures. The project was commenced in late 1996 and opened for the public in 2004. The Contract No. 1, Underground Structure Southern portion of this initial system comprising 9 km of twin 6m outside diameter bored tunnels, nine underground stations and cut-and-cover depot approach tunnel, was awarded to the Joint Venture BCKT...
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One of the major challenges of the Contract No. 1, was the need of stacked-alignment for a major portion of twin bored-tunnel underneath Rama IV road, one of the busiest roads of Bangkok, which called for a stacked configuration at 3 stations, Lumpini, Si Lom and Sam Yan as shown in figure 2. This requirement led the contractor to design and construct the deepest underground structures of Bangkok at Si Lom Station - four-floor stacked-station of 32m deep right under the existing flyover and adjacent to a number of sensitive structures. Moreover, as alignment of the lower tunnel had to place in the deeper depth within first sand layers for the last four sections of the Contract 1, the process involved in tunnelling works were more complicated.

Regional settlement due to active consolidation of top soft clay layer and piezometric drawdown caused by deep well pumping was another key factor for which design and construction of both tunnels and station boxes had have taken into consideration.

Deep excavation and bored tunneling works in congested urban environment of Metropolitan Bangkok requires a systematic process of ground movement prediction, building risk damage assessment and called for the stringent management of the risks. In this paper focus is limited to prediction of tunnelling induced settlement and assessment of potential damage to existing adjacent building and structures though rigorous assessment was also made for ground movement caused by deep excavations by diaphragm wall support cut-and-cover construction method.

Consisting of Bilfinger + Berger Bauaktiengesellschaft, Ch Karnchang Public Company Limited, Kumagai Gumi Company Limited and Tokyu Construction Company Limited. The project was commenced in late 1996, successfully completed and opened for the public in 2004.
4. SUBSOIL CONDITION ALONG CONTRACT 1 ALIGMENT

Similar to other localities in Bangkok, a typical subsoil profile along the tunnel alignment is characterized by the alternating layers of clay and sand deposits. Figure 3 shows the generalized soil profile along the tunnel alignment of the Contract No.1. Weathered crust of 2 m thick is commonly found as the top layer. In urban areas of Bangkok, this layer is covered by fill material. Soft to very soft, highly compressible dark gray marine clay lies beneath weathered crust or fill. Depending on the location, this layer extends up to 12-16 m. About 2 m thick medium clay layer can be observed between soft clay and underlying stiff clay. Generally stiff Clay layer occurs directly underneath medium clay and its depth goes up to 22 m. Below stiff clay layer, first sand layer of 5-8 m thickness can be found. This first sand layer, however, is absent in some areas. Stiff to hard clay layer underlies first sand and it is found to be about 5 m thick. Second sand layer generally occurs at depths between 45 to 65 m. As can be seen in Figure 3, tunnel alignment is located mainly in stiff clay layer. Tunnels were bored in the first sand layer before reaching to Queen Sirikit Center station and deeper sections between Bon Kai (Khlong Toey station) and Hua Lamphone stations.

5. STATION BOX CONSTRUCTION AND TUNNELLING WORKS

Cast-in-place rigid diaphragm walls were used as permanent retaining structures for all stations and some entrance structures and ventilation buildings. Most of the entrance and ventilation buildings were excavated by sheet pile walls. Tip levels of diaphragm wall ranges from 19m (at Hua Lamphong station) to 39m (at Sam Yan and Silom stations) for the final excavation depths of 15m to 31m. Excavated down to 31 below existing ground level at Silom station marked the deepest excavation supported by diaphragm wall in Bangkok. Top-down construction method was used for all stations having stanchions for vertical support and skeleton slabs as bracing during bulk excavation of station boxes (Figure 4).

Fig. 3. Generalized soil profile along the tunnel alignment and stations of Contract No. 1 (Phienwej et. al. 2006)

Fig. 4 View of diaphragm wall, stanchions and skeleton slabs

Earth-pressure-balanced (EPB) tunneling method using shield tunnel boring machines (TBM) was applied for tunnel excavation except at the cut-and-cover section of tunnel approach to the depot. Japan made two sets of Kawasaki TBM were used from Rama 9 station to Queen Sirikit Convention Center (QSCC) station. Tunnel boring from QSCC station to Hulamphong station was carried out by two sets of Herrenknecht TBM. Kawasaki TBMs were designed to drill through the concrete diaphragm wall – having specially manufactured Fiber Reinforcement (FRP) installed at tunnel eyes of diaphragm walls. Pre-opening of diaphragm wall concrete and removal of steel reinforcement were necessary at tunnel eyes for Herrenknecht TBM. Tunnels were bored with overcut by cutter-head at front (total cut diameter during boring was 6.48m) with backfill grouting. At permanent stage, tunnel was supported by 300mm thick precast segmental concrete rings having 6.30m and 5.70m outer and inner diameter respectively. The loads from existing and future structures within influence zone were taken into account in tunnel lining design. Movement joints (Omega Joint) were provided at the connection at station diaphragm walls and tunnels to accommodate the diffe-
rential settlement between two structures. Figure 5 shows the tunnel configuration at permanent stage.

Tunnel alignment generally follows the existing main Bangkok roads, having single-bored twin tunnels for north and south bound tunnels at operation stage. From Rama 9 station to initial section of Klong Toey station, twin tunnels were bored at same level (about 21m below existing ground level). Due to the presence of existing flyovers located in the middle, two tunnels were required to change the alignment having stacked one above the other.

Fig. 5. Typical sectional configuration of tunnel at permanent stage

Fig. 6. View of EPB Tunnel Boring Machine after cutting through diaphragm wall

Fig. 7. View of Bangkok major road where twin tunnels were bored underneath at approximately 21m below existing ground level

6. OVERVIEW OF GROUND MOVEMENT PREDICTION AND BUILDING DAMAGE ASSESSMENT PROCESS APPLIED IN THE PROJECT

Well accepted published literatures were used as basic reference and method in ground movement prediction and building damage assessment. For the station boxes where cut-and-cover methods were used for excavation, the work of Peck (1969) and Clough and O’Rourke (1990) were used whereas published papers of Burland et. al. (1977) and Boscardin & Cording (1989) were applied for bored tunnels. Building damage assessment was carried out in 3 stages; Stage 1 Assessment, Stage 2 Assessment, and Stage 3 Assessment. “Greenfield Site” assumption was adopted in Stage 1 and 2 Assessment ignoring any influence of the stiffness of the structure and type of supporting foundation. Stage 2 Assessment was further extended by Additional Stage 2 Assessment. In Additional Stage 2 assessment, the variation of settlement trough shape and magnitude with depth were considered in identifying the risk of building damage. In Stage 3 assessment, a detailed qualitative assessment of structure and vulnerability of each building or structure to damage were taken into account, coupled with detailed structural analysis in assessing the buildings falling in risk category 3 classified by Mair et. al. (1996).

7. SUMMARY OF INSTRUMENTATION PROGRAM

Instrumentation played a major role in the construction of this first underground mass transit system project in congestive urban area with difficult soil condition. The primary objective of the instrumentation program was to monitor the performance of the deep excavation and tunneling operation to ensure safe execution of the construction works and that adjacent structures were not adversely affected. Furthermore, the instrumentation program was established to provide feedback in application of the observational method. Extensive sets of instrumentation were installed around the predefined influence zones of station box excavation, tunneling and associated building, structures and utilities. In addition to other geotechnical instruments, inclinometers, extensometers, various types of settlement points, tile-meter, crack meter and crack gauge were installed to monitor the ground and associated building and structures movement.
8. BUILDING CONDITION SURVEY

Prior to commencement of the major works, all the existing buildings and structures within the predefined influence zone were systematically surveyed. The primary objectives of the surveys were:

- To record pre-construction status of all buildings and structures within the influence zone. The records included were age of building, building type, storey height, expected foundation types, visible defects supported by sketches and photographs as appropriate.

- To utilize the information collected from the building / structure condition survey in the assessment of the buildings and evaluate sensitive structures with respect to construction aspects and monitoring.

- To establish a benchmark to monitor the possible effect of construction and feedback information in applying observational method and if necessary to implement the building protection measure.

Expected foundation types of different buildings were classified based on the height, number of floors and age of the buildings. Building survey information was also used to identify the types of instrumentation required.

9. BUILDING DAMAGE ASSESSMENT AND APPLICATION OF OBSERVATIONAL METHOD

The main reasons of applying observational method in risk assessment were:

- Uncertainty in Ground Movement Prediction and Staged Assessment of Buildings.

- Due to the different types of buildings and structures along MRTA alignment it was recognized that results obtained from Building Damage Risk Assessment might be conservative for some buildings / structures and could be underestimated for others.

- To minimize unnecessary implementation of building protection measures without compromising safety.

Figure 8 shows the link between observational method and geotechnical risk management, particularly risk of damage to existing buildings and structures. As mentioned in earlier sections of the paper, controlling and managing the risk is one of the most important factors in successful completion of the tunneling works in urban environment and hence all parties involved had to pay special attention on the issue.

![Fig. 8. Flowchart showing the OM process involved in Building Damage Risk Assessment](image)

![Fig. 9. Diagram showing process of Staged-Assessment](image)
Process of Stage-Assessment is shown in figure 9. After carrying out the initial assessment of potential building damage, it was recognized that without applying the observational method, the result obtained from assessment process could be conservative for some buildings and could be underestimated for others. For example, the settlement estimated in Second Stage Assessment shown in above diagram was based on surface movement. For the buildings located in close proximity to the excavation founded on piles, it was considered that subsurface settlement may be more critical. Stage 2 Assessment was then further extended by Additional Stage 2 Assessment in which the variation of settlement trough shape and magnitude with depth were considered in identifying the risk of building damage. Settlement of the buildings due to the excavation works were predicted assuming the "green filed" condition. Neglecting restraints from foundation and structure, it was assumed that buildings follow the ground settlement trough at foundation level (estimated pile tip level). However, if the building is supported by deep foundations, assessment result could be too conservative.

10. PREDICTION OF GROUND MOVEMENT INDUCED BY TUNNELLING WORK AND RISK ASSESSMENT

10.1 Prediction of surface settlement

The Gaussian normal distribution curve developed by Peck (1969), O’Reilly and New (1982) was used to estimate the ground settlement induced by a single tunnel.

10.2 Stage 1 Assessment

First stage assessment of the risk of damage to buildings and structures in tunneling zones involved preparation of settlement profile induced by tunnel excavation. Settlement was computed at every 100m interval along tunnel alignment using the method based on Gaussian normal distribution curve presented above. Settlement induced by each tunnel (north and south bound) was predicted independently and then superimposed to determine the total settlement induced by the twin tunnels. 10mm and 25mm settlement contours were plotted in the topographic plan with buildings and other structures including flyover and khlong bridges in the tunnel alignment. Buildings falling outside 10mm settlement contours were eliminated from further assessment. Remaining buildings with settlement greater than 10mm were short-listed for stage 2 assessment.

10.3 Stage 2 Assessment

The buildings and structures short-listed in stage 1 were assessed in this stage. Stage 2 assessment involves calculation of maximum tensile strain induced in the building and structures due to tunnel boring and classification of risk category. Risk category of the buildings was determined applying the method proposed by Mair et. al. (1996).

10.4 Additional Stage 2 Assessment in tunnel zones

As most of the buildings and structures in Bangkok along tunnel alignment are supported by piled foundations, it was considered that Stage 2 Assessment using surface settlement profile might be insufficient. Hence, Additional Stage 2 Assessment considering the variation of settlement trough shape and magnitude with depth (subsurface settlement) was carried out for the critical buildings.

In general, subsurface settlement trough will be narrower and steeper than those of the surface as shown in the sketch below. A simple approach was used in predicting the subsurface settlement profile with following assumptions.

(1) The shape of the subsurface settlement profile caused by the tunnel construction is characterized by a Gaussian distribution similar to surface settlement profile.

(2) The ground settlement influence line for tunnel in clay is the same for subsurface as shown in the figure 10 below:

Form of Surface and Subsurface Settlement Profiles

Fig. 10. Distribution of surface and subsurface settlement induced by tunnel boring.
Therefore, substituting the distance above the tunnel axis \(Z_0-Z\)
for \(Z_0\), equations derived from Gaussian normal distribution curve subsurface settlement profile was determined. In addition to this method following equation can also be used to predict the subsurface settlement.

\[
K' = [0.14+0.26(1-z/z_0)] / (1-z/z_0)
\]

By substituting \(K'\) values for \(K\) in equation derived from Gaussian normal distribution curve the subsurface settlement at required depth can be calculated.

- The risk of damage to the building in terms of magnitude of building slope with reference to the limiting values specified by various researchers.

The following assumptions were made in the conservative approach of Stage 3 assessment.

- Building movement is influence by excavation induced settlement of both main station box and associated underground structures (e.g. Entrance Building)
- Building settlement was caused by subsurface settlement at foundation level as buildings are supported by piled foundation.
- No allowance was made for structural stiffness of the building.
- No allowance was made for possible effects of soil-structure interaction.

A summary of risk category determination in Stage 3 Assessment based on the predicted and monitored settlement of the building adjacent to station excavation work is shown in the Table 1 below.

Table 1. Summary of risk category determination based on Boscarding and Cording (1989)

<table>
<thead>
<tr>
<th>Data Taken From</th>
<th>Angular Distortion</th>
<th>Risk category / degree of damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Settlement Prediction</td>
<td>48 23 25 18.6 0.0013</td>
<td>2 / slight</td>
</tr>
<tr>
<td>Actual Monitoring Data</td>
<td>53 14 39 18.6 0.0020</td>
<td>2 / slight</td>
</tr>
</tbody>
</table>

*Predicted horizontal movement was used in computing horizontal strain for actual monitoring data.

From the above table, following conclusions can be made.

- Predicted angular distortion broadly agrees with that of actual monitoring data. Hence, settlement prediction and assumed effective depth of piled foundation is fairly reasonable.
- The settlement of actual monitoring data is slightly higher than that of prediction. This may be due to the additional settlement caused by diaphragm wall panel construction which influence zone might have extended to 15m from the diaphragm wall (equivalent distance to bottom of soft clay level).

Limiting values of the building slopes at two critical conditions specified by various researchers as shown in the table 2 below were also used in determining the potential building damage of stage 3 assessment.

Table 2. Summary of Limiting Building Slope Specified by Various Researchers

<table>
<thead>
<tr>
<th>Limiting Values by Various Researchers</th>
<th>Type of Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking in walls and partitions</td>
<td>Structural Damage</td>
</tr>
<tr>
<td>1. Skempton and Macdonald</td>
<td>1/300 to 1/500</td>
</tr>
<tr>
<td>2. Meyerhof</td>
<td>1/500</td>
</tr>
<tr>
<td>3. Polshin and Tokar</td>
<td>1/500</td>
</tr>
<tr>
<td>4. Bjerrum</td>
<td>1/500</td>
</tr>
</tbody>
</table>
**10.7 Stage 3 Assessment by structural frame analysis**

Some buildings of critical condition were assessed by structural analysis using detailed structural survey information as basic data. By simulating the differential settlement of the building predicted from Additional Stage 2 Assessment, development of moment and shear in the main structural members were determined with the use of structural analysis software. Computed moment and shear were assessed in comparison with the capacity of the structural members estimated from collected building condition survey and detailed structural survey data. The process involved in structural assessment can be summarized below.

- Factor of safety of each structural member was determined based on predicted differential settlement of the buildings.
- A series of assessment were performed to determine the critical differential settlement causing structural instability of the members or differential settlement at limiting factor of safety.
- Trigger values were established based on above results.
- Based on the trigger values and the actual building condition and response in conjunction with monitoring data, the most sensitive structural members were protected, mainly by strengthening approach.

**10.8 Damage Assessment on Sensitive Buried Structures**

Apart from the flyovers (bridges), the most critical structure required for Stage 3 Assessment were MWA main water tunnel as tunnel alignment crosses the water tunnels at two critical locations – the first near Rama IV and Rachadaphisek road intersection and the second at Rama IV and Siphraya road intersection near Sam Yan station. The position of water tunnels in relation to MRT tunnels drawn in the subsoil profile at relevant locations are shown in Figure 13 and 14.

According to the available information, MWA tunnel at the first crossing is of 2.5m internal diameter steel lining with 150mm segmental ring and at the second crossing MWA tunnel is of 2.5m diameter segmental ring with lightly reinforced concrete lining.

In preliminary stage 3 assessment, based on the predicted subsurface profile using the method described in published paper “Ground Movement Prediction and Building Damage Risk Assessment for the Deep Excavations and Tunneling Works in Bangkok Subsoil”, Zaw et. al (2006) e, structural assessment of water main was carried out and results were compared with structural capacity of water tunnel calculated from the material properties indicated in the construction drawings provided by MWA. As bending moment induced in the water tunnel was considered critical parameter in assessing the damage which is governed by both maximum settlement and curvature of the water tunnel further assessment was proposed coupled with observational approach.

- To carry out back-calculation using monitoring data obtained from the settlement arrays (settlement points, extensometer data) at first 3 locations before reaching to MWWA water tunnel near Rama IV – Rachadaphisek intersection, to determine the volume loss and relationship between surface and subsurface settlement as well as best estimation of subsurface $K$ value and subsurface settlement profile at water tunnel level.
- To perform beam-on-elastic foundation analysis using the subsurface settlement profile obtained from predicted subsurface settlement bending moment likely to occur in MWWA water tunnel.

- To compare the result obtained from above item with allowable values.
- To establish the final trigger values based on above comparison.
- To confirm the settlement profile from actual monitoring of last array is within the alert level of final trigger value.

![Fig. 13. Position of MWA water tunnel in relation to MRT tunnels (clear distance are 3.38m and 3.18m for North and South bound respectively) at the first crossing.](image)

![Fig. 14. Position of MWA water tunnel in relation to MRT tunnels (clear distance are 1.5m and 2m for North and South bound respectively) at the second crossing.](image)

For the first location of MRT-MWA tunnel crossing, target ground loss from tunnel operation was set at 0.5%, whereas ground loss based on allowable movement curvature of water tunnel derived from predicted pattern of subsurface settlement was 0.65%. Back-calculated ground loss derived from the monitoring data of rod-extensometer placed at the crown of MWA water tunnel was within target value. In addition to the preliminary assessment based on prediction of subsurface settlement using simple empirical formulas, a detailed soil-structure interaction was analyzed using 2-dimensional FEM considering behavior of tunnel boring by earth pressure balance (EPB) method. Stress relief from the effect of cutter-head, tail void and over-cut from EPB tunnel boring was considered in the analyses. Adding ground movement due to consolidation caused by disturbance from tunnel operation surface and subsurface settlement were predicted by finite element analysis. Detailed parameters and assumption made in analysis was presented in the published paper of Sakai & Sugden (2000).
After the successful passing of both tunnels at the first MWA crossing, another trial area tunneling was planned to further determine the ground movement associated with various tunneling parameters well before reaching to more critical crossing after Sam Yan station. Tunnel section between Silom and Samyan station, in which soil profile similar to that of second crossing was selected and comprehensive instrumentation program was established to monitor surface and subsurface settlement as well as horizontal movement of the ground at depth.

The observed ground movement induced by tunneling works were within predicted values. Well organized plan, systematic approach of observational method to achieve control values backup by extensive instrumentation and close co-operation among relevant personnel were the key factors contributed to successful completion of EPB tunneling particularly in the most sensitive points at MWA water tunnel crossings. The monitored subsurface settlement data from extensometer and predicted settlement trough of assumed ground loss 1% ground settlement profile of the first MWA tunnel crossing is presented in figure 15.

10. KEY FACTORS IN SUCCESSFUL COMPLETION OF THE PROJECT

The M.R.T Chaloem Ratchanatamon Line, one of the most modern underground mass transit systems in the region was successfully completed and has been in operation for over 6 years. All deep excavation works and tunneling were successfully completed without any significant damages to the adjacent buildings and structures. Well organized plan, systematic approach of observational method to achieve control values backup by extensive instrumentation and close co-operation among relevant personnel were the key factors contributed to successful completion of the project. The purpose of this paper is to provide a set of technical guidelines for ground movement prediction and building damage risk assessment of deep excavation and tunneling works in Bangkok subsoil and application of observational method which may partly serve as a reference for the future underground mass transit system projects in urban area of Bangkok. Successful completion of the first MRT project in Bangkok marked the practicality of well-executed deep excavation works and tunneling by EPB method coupled with observational method in the sensitive urban environment.

REFERENCES


Clayton C. R. I (2001), Managing Geotechnical Risk, Improving Productivity in UK Building and Construction, Institute of Civil Engineers and Thomas Telford Ltd. UK

Clough, G.W., and O’Rourke, T.D., (1990), Construction Induced Movements of In-situ Walls, ASCE Geotechnical Special Publication No. 25, pp. 439-470

Clough C.W. and T.D. O’Rourke (1990), Deep Excavation and Tunneling, 7th ICSEME, Mexico City


JV BCKT, (1997) Geotechnical Interpretation Report (GIR) for MRTA Initial System - UGS, Bangkok


O’Reilly M.P and New B.M (1982), Settlements above tunnels in United Kingdom – their magnitude and prediction, Tunneling ’82, pp. 173-181, London, IMM


Tunneling effects on pile group response in Bangkok
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ABSTRACT
Tunneling adjacent to an existing pile group may induce additional force, bending moment and settlement of a pile. Moreover, it may cause load redistribution and differential settlement in a pile group. In this paper, the construction of a water diversion tunnel with an outside diameter (D) of 5.55 m constructed in Bangkok is reported. The tunnel was constructed in very stiff clay using an earth pressure balance shield with a cover-to-diameter ratio of 4.5 and very close to a pile group. The pile group was located within a distance of 1D from the tunnel springline. Tunneling induced ground responses were measured by using surface settlement points, extensometers and inclinometers. To investigate the influence of tunnelling on the pile group, a three-dimensional, elasto-plastic, coupled-consolidation numerical analysis was also conducted to simulate the pile group responses. Observed ground responses and computed changes of horizontal effective stress and excess pore water pressure around piles are reported. Redistribution of pile shaft resistance, load transfer in the pile group and pile head settlements are investigated.

INTRODUCTION
Tunneling adjacent to an existing pile group may induce additional bending moment, cause redistribution of axial force in each pile and load transfer in a pile group. Any stress change in the ground due to tunnelling is the main concern to engineers in evaluating capacity and additional settlement of a pile group.

Chen et al. (1999) and Lee & Ng (2005) carried out numerical analyses to study the influence of tunneling on adjacent pile. They found that negative skin friction was induced along the pile shaft as a result of stress relief. Tunneling adjacent to a pile group was also studied by Pang et al. (2005) and Cheng et al. (2007). It is reported that the maximum bending moment induced in a pile occurs at the tunnel axis in the transverse direction of tunnel advancement while no significant bending moment occurs in the longitudinal direction.

In this paper, monitored soil displacements due to a 5.5m diameter shield tunnelling adjacent to a pile group are reported and compared with computed finite element results. The capacity and settlement of the pile group due to the tunneling are also investigated.
Tunneling effects on pile group response in Bangkok

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In this paper, monitored soil displacements due to a 5.5m diameter shield tunnelling adjacent to a pile group are reported and compared with computed finite element results. The capacity and settlement of the pile group due to the tunneling are also investigated.
THE SITE

Figure 1a shows plan of tunnel alignment and adjacent pile groups of elevated railways. The layout of instrumentation is also shown in the Figure. Various instruments were installed to observe tunneling induced ground displacement and structural response. Three surface settlement points (S1, S2 and S3), two extensometers (Ex1 and Ex2) and one inclinometer tube (I1) were installed to measure ground response. Two pier settlement points (C1 and C2) were mounted on pier to measure pier settlement. The monitored data is interpreted from “BMA flood diversion tunnel project: Sansaeb Canal and Ladprao Canal Section” (Lim et. al, 2009). The total length of tunnel in this project is about 5 km.

Figure 1b shows a typical soil profile of the study area. The first upper layer is 12 m thick soft clay. Below it is the succession of medium stiff clay of 3 m, stiff clay of 5 m, very stiff clay of 14 m and stiff clay of 14 m thick layers. A 12 m very dense sand layer underlies the clay layer. A water diversion tunnel with an outer diameter (D) of 5.55 m was constructed in very stiff clay at cover-to-diameter ratio (C/D) of 4.5 using an earth pressure balance shield machine. The tunnel alignment is very close to a (2×2) pile group with center to center of pile spacing 3 m. The outer diameter and length of the shield are 5.70 m and 8.05 m, respectively. The diameter of each concrete bored pile is 1.0 m. The length of piles is 60 m and pile toes are embedded in very dense sand layer. All four piles are rigidly connected with 1.8 m thick pile cap. The clear distance between the nearest pile and the tunnel springline is 3.65 m. The tunnel axis depth is approximate at the middle of pile length.

Figure 1. (a) Plan of study area and lay out of instrumentation (b) sectional view A-A and soil profile

NUMERICAL ANALYSIS

To investigate the influence of tunnelling on the pile group, a three-dimensional, elasto-plastic, coupled-consolidation numerical analysis was carried out using Finite Element program ABAQUS (Hibbitt et al., 2008).
Figure 2a shows the geometry of a 3D finite element mesh. A plane of symmetry is identified at the tunnel axis, so only half of the domain is modeled. The mesh is 40 m wide, 50 m long and 70 m high. Eight-node brick elements, four-node shell elements and two-node beam elements are used to model the soil, pile cap and piles, respectively. Roller and pin supports are assigned on the four vertical sides and bottom of the mesh as boundary conditions, respectively.

Figure 2b illustrates tunnel excavation simulation technique adopted in the finite element analysis (FEA). This technique is known as Displacement Controlled Method (DCM), modified from Cheng et al. (2007). During shield advancement over-excavation due to pitching angle is often encountered at the site. Therefore, it is assumed that soil displacement at the upper part of the shield is larger than the lower part of the shield and the shield’s invert is in contact with soil element. To simulate each tunnel advancing step, displacement boundary is specified to each node around the excavated perimeter. Consequently, soil element around the shield is pulled to a final tunnel diameter of 5.55 m. Also, soil inside the excavated section is removed simultaneously. A volume loss of 2.5% was imposed on tunnel boundary. By using this technique, tunnel lining is not required. In order to simulate an earth pressure balance shield, a roller support is assigned to the shield face to restrain horizontal soil movement in front of the shield. Shield advancing speed of 5 m per day is modeled in the analysis. Tunnel alignment is simplified from curve in actual condition to be straight in FEA. The presence of pile foundations of surrounding buildings are ignored in the analysis.

In Figure 2c piles and pile cap are arranged in rotated square. Total load on the pile group is assumed to be 18,000 kN. Excessive pore water pressure generated in response of the applied load on pile group is allowed to dissipate before tunnel advancement in the numerical analysis.
An elasto-plastic soil model with Mohr-Coulomb failure criterion is used to simulate soil behaviour. Soil parameters are selected based on a soil investigation report and adopted from previous studies [Tanaka et. al (2001), Seah et. al (2000)]. Permeability of clay layers and sand layer is assumed to be $1 \times 10^{-9}$ and $1 \times 10^{-3}$ m/s, respectively. Soil parameters are summarized in Table 1. The concrete piles and pile cap are modeled as linear elastic with Young’s modulus and Poisson’s ratio as 30 GPa and 0.3, respectively. The unit weight of concrete is assumed as 24 kN/m$^3$. The water table is assumed to be located at the ground surface with a hydrostatic initial pore water pressure profile.

Table 1. Soil parameters in finite element analysis

<table>
<thead>
<tr>
<th>Material</th>
<th>$\gamma_f$ (kN/m$^3$)</th>
<th>$S_u$ (kPa)</th>
<th>SPT (N/ft)</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ (°)</th>
<th>$\psi'$ (°)</th>
<th>$\nu'$</th>
<th>$E'$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay</td>
<td>16</td>
<td>18-20</td>
<td>N/A</td>
<td>1.5</td>
<td>18</td>
<td>-</td>
<td>0.15</td>
<td>1,200</td>
</tr>
<tr>
<td>Stiff clay 1</td>
<td>19</td>
<td>90-120</td>
<td>12-15</td>
<td>2.0</td>
<td>19</td>
<td>-</td>
<td>0.15</td>
<td>5,000</td>
</tr>
<tr>
<td>Very stiff clay</td>
<td>20</td>
<td>160-200</td>
<td>25-30</td>
<td>3.0</td>
<td>20</td>
<td>-</td>
<td>0.15</td>
<td>7,000</td>
</tr>
<tr>
<td>Stiff clay 2</td>
<td>19</td>
<td>100-130</td>
<td>13-18</td>
<td>2.0</td>
<td>19.5</td>
<td>-</td>
<td>0.15</td>
<td>6,000</td>
</tr>
<tr>
<td>Very dense sand</td>
<td>20</td>
<td>-</td>
<td>50-60</td>
<td>0.1</td>
<td>35</td>
<td>6</td>
<td>0.15</td>
<td>40,000</td>
</tr>
</tbody>
</table>

MONITORED DATA AND COMPUTED RESULTS

In this paper, only measured ground horizontal displacement is reported. Reference point of the shield distance used in this paper is taken at the shield face. A monitoring section is selected at the center of the pile group (i.e., at $Y=0$) as reference point for tunnel advancement.

Ground horizontal displacement. Measured horizontal ground displacement in transverse direction of tunnel advancement is shown in Figure 3. When the shield was more than 12 m away from the monitoring section, measured horizontal ground displacement was insignificant. When the shield reached at 10 m behind the monitoring section (i.e., $Y=-10$ m), soil moved outward from the shield. It indicates that stress redistribution of soil due to soil arching may occur. More discussion and explanation is given in the later section. Maximum outward movement of 3 mm from the tunnel axis was observed at this stage.

When the shield reached the monitoring section (i.e., $Y=0$ m), soil abruptly moved inward towards the shield and caused maximum inward displacement. Large displacement occurred from depth 25 to 32 m which is considered about 1D (tunnel diameter) from the tunnel axis. Measured horizontal displacement at the tunnel axis depth was about 15 mm inward to the shield. When the shield reached at 25 m ahead of the monitoring section (i.e., $Y=25$ m), measured horizontal displacement reduced by 3 to 5 mm. The reduction may be caused by stress redistribution. No significant horizontal ground displacement was observed when the shield advanced further.

Computed ground horizontal displacements show a similar trend as compared with the measured data. When the shield reaches at 10 m away from the monitoring section (i.e., $Y=-10$ m), computed outward soil movement from the shield occurs
from depth 23 m to 40 m. When the shield reaches the monitoring section (i.e., Y=0 m), computed results show an abrupt soil displacement towards the shield. In contrary to the measured data, when the shield passed at 25 m (i.e., Y=25 m) beyond the monitoring section, a slight additional horizontal displacement towards the shield occurs. The differences between measured and computed results may be caused by tail void grouting is not simulated in the FEA. Therefore, outward movement in FEA as the shield advances beyond the monitoring section does not occur. It can be noticed that, the computed ground horizontal movement from FEA can reasonably capture the measured ground response at the depth of tunnel axis. However, there is a significant difference between computed and measured ground response near the ground surface. Large measured horizontal ground displacement may be triggered by other activities rather than tunneling such as surcharge induced by heavy traffic load near the inclinometer tube.

**Figure 3** Comparison of measured and computed horizontal ground displacement of I1 (measured data from Lim et al, 2009)

**COMPUTED PILE GROUP RESPONSE**

**Load transfer in the pile group.** Figure 4a shows changes of computed load at pile head with respect to the shield advancement. The changes of load at the pile head are noticeable when the shield advances from Y = -5 to -2 m. Load on P2 significantly increases while load on P1, P3 and P4 decreases. Since P2 is the farthest pile from the shield, it is the least affected by tunneling. This explanation is verified with computed mobilized shaft resistance in the following section. Consequently, load at pile head from P1, P3 and P4 is transferred to P2. Steady state of load transfer in pile group is achieved when shield passes at Y=10 m ahead of the monitoring section.
Maximum reduction and increment in load on pile head of P1 and P2 is 0.8% and 1.2%, respectively.

**Pile head settlement and pier tilting.** Figure 4b illustrates the computed results of pile head settlement against shield advancement. In addition to effects of tunneling, the observed pier settlement was highly influenced by other factors such as vibration induced by heavy traffic and the elevated train. Furthermore, pier settlement was barely noticeable by conventional surveying instrument. Therefore, the measured pier settlement result is not reported in this paper. Maximum computed pile head settlement occurs at P3 since significant shaft resistance reduction in this pile occurs when the shield is approaching to the monitoring section. Moreover, load at pile head on P3 is almost constant (see Figure 4a). Pile head settlement in P1 and P2 is almost equal although different mechanism is developed. Pile head settlement in P1 would be mainly caused by reduction of horizontal effective stress and development of negative shaft resistance which is discussed in following section. Elastic shortening due to increase of load at pile head would be the major cause of pile head settlement in P2. The reduction of load at pile head in P4 is higher than in P3 (see Figure 4a). Therefore, minimum pile head settlement is induced on P4. Tunneling induced pier settlement is about 1.3 mm. The differential settlement in the pile group causes the pier to tilt in the longitudinal direction of the tunnel axis toward P3. The maximum inclination of the pier becomes 1:12,500 when the shield is 5 m away from the monitoring section (Y = -5 m). In addition, the computed differential settlement between two piers is 1:27,000 at the end of tunnel driving (the distance between two piers is 35 m). Those computed results are considered very small for the serviceability limit state criteria.

![Figure 4](image)

**Figure 4** (a) Changes of computed load at pile head (b) induce pile head settlement

**Mobilized shaft resistance.** Figure 5 shows computed mobilized unit shaft resistance of P1, P2, P3 and P4, during shield advancement. The significant change of shaft resistance occurs in P1 as shown in Figure 5a. Mobilized shaft resistance decreases significantly at depth 13 to 24 m. Negative shaft resistance develops in P1 when the shield reaches at the monitoring section and advances further, indicating that soil settles more than pile at depth 12 to 25 m. To maintain load equilibrium in a pile, mobilized shaft resistance in P1 below the tunnel axis increases. The change of mobilized shaft resistance in P2 is the lowest as shown in Figure 5b. Negative shaft
resistance did not develop in P2, which suggests that impact of tunneling on P2 is the smallest. It is suggested that tunneling influence zone is about 1.5D from the tunnel axis. P3 and P4 respond to the shield advancement in the same trend as shown in Figure 5c and 5d, respectively. The only difference that can be noticed is significant change occurs in P3 when the shield is at 5 m away from the monitoring section. The significant change in P4 occurs when the shield is at 5 m beyond the monitoring section. In other words, the maximum reduction of shaft resistance on each pile occurs when the shield face is slightly beyond the pile.

**Figure 5** Computed mobilized unit shaft resistance of (a) P1 (b) P2 (c) P3 (d) P4

**Induced axial force and bending moment in pile.** Figure 6a illustrates computed tunneling induced axial force in each pile when shield is at 5 m ahead of monitoring section. The maximum induced axial force occurs in P1 at the depth of tunnel axis. The trend of induced axial force distribution of P2, P3 and P4 is similar to P1. The differences of magnitude of induced axial force are due to development of negative shaft resistance in P1 (see Figure 5a). It can be noticed that minimum induced axial force occurs in P2 as a result of minimum reduction of unit shaft resistance (see Figure 5b).

Figure 6b shows the distribution of computed bending moment of each pile in longitudinal direction of the tunnel axis ($M_x$). Maximum $M_x$ occurs in P1 when the shield is at 5 m beyond the pile ($Y = 5$ m). The bending moment ($M_x$) causes every pile to bulge in the same direction as the shield advancement at the depth of tunnel axis. Bending moment in each pile extends to depth 50 m, which is about 4D below the tunnel axis.

Figure 6c illustrates the distribution of computed bending moment of each pile in transverse direction of tunnel axis ($M_y$). Maximum $M_y$ occurs in P1 when the shield is at 5 m beyond the pile. The bending moment ($M_y$) causes the pile to bulge towards the shield due to stress relief around the shield. By comparing bending moment of a pile in 2 directions, $M_y$ is significantly larger than $M_x$ due to higher
stress reduction around the shield in transverse direction than in the longitudinal direction.

The ultimate bending moment capacity of a reinforced concrete bored pile with diameter 1.0 m is approximate between 1,000 and 1,400 kN.m, given the reinforcement ratio is 0.5% of the pile section. Therefore, maximum computed induced bending moment in P1 is only about 20% of its capacity.

![Figure 6 Computed (a) induced axial load distribution (b) bending moment in longitudinal direction, Mx (c) bending moment in transverse direction, My in each pile due to tunneling](image)

**Changes of horizontal effective stress and induced excess pore water pressure.** Figure 7a shows changes of computed normalized horizontal effective stress ($\Delta \sigma_h'/\sigma_h^{*0}$) with respect to shield advancement. In this study, coupled-consolidation analysis is conducted which allows both volume and pore water pressure to change during tunnel advancement. Soil elements around piles at the depth of tunnel axis are selected to be investigated. As the shield reaches between $Y = -10$ to $-5$ m, $\Delta \sigma_h'/\sigma_h^{*0}$ of P1, P2 and P4 increases. This is caused by horizontal effective stress relief around the shield. Subsequently, arching effect or stress redistribution occurs on soil element in front of the shield and causes outward movement from the shield. The measured horizontal ground displacement confirmed the explanation of stress redistribution. When the shield advances further and reaches at $Y = -5$ to 3 m, $\Delta \sigma_h'/\sigma_h^{*0}$ suddenly drops due to stress relief around the shield. Horizontal effective stress around piles slightly increases when the shield advances beyond the monitoring section ($Y = 3$ to 25 m). This is due to stress redistribution behind the shield causing a slight outward movement at the piles. The highest change of $\Delta \sigma_h'/\sigma_h^{*0}$ occurs at P1 with the highest increment $\Delta \sigma_h'/\sigma_h^{*0}$ of 1% and the maximum reduction $\Delta \sigma_h'/\sigma_h^{*0}$ of -3%. The tunneling impact in terms of changes of horizontal effective stress on P2 is the lowest as expected.
The highest change of 25 m). This is due to stress redistribution behind the shield causing a slight outward movement away from the shield. The measured horizontal ground displacement confirmed the explanation of stress redistribution.

Based on field observations and computed results, the following conclusions may be drawn. Within the pile group, P1, P2, P3 and P4 denote the closest, the farthest, the front and the rear pile to the tunnel, respectively.

1. The maximum measured transverse horizontal ground displacement was 0.2%D (i.e., 15 mm) at depth of the tunnel axis when the shield was at the monitoring section. When the shield was approaching to the monitoring section, outward soil movement away from the shield was observed due to stress redistribution. As the shield reached the monitoring section, an abrupt change of inward horizontal ground displacement toward the shield was observed as a result of stress relief around the shield.

2. The influence zone of load transfer and pile head settlement are computed when the shield advances from 1D (tunnel diameter) away from the center of the pile group to 1D beyond the center of pile group. As the shield advances towards the pier, computed pile load at P2 increases because it is the least affected one by shaft resistance reduction due to tunneling. On the other hand, loads at pile head of P1, P2 and P3 decrease because of load re-distribution within the pile group. The maximum pile head settlement occurs on P3 causing the pier to tilt in the longitudinal direction of tunnel towards P3.
3. Based on computed results, tunneling causes a significant reduction of shaft resistance in P1 at depth above the tunnel axis. To maintain equilibrium, shaft resistance below the tunnel axis increases. The influence zone of shaft resistance reduction is estimated to be 1.5D from the tunnel axis.

4. The maximum computed induced axial force, longitudinal and transverse bending moments occur in P1 at the depth of tunnel axis when the shield advances at 1D beyond P1. The largest induced axial force occurs due to the development of negative shaft resistance. The bending moment of the pile in the longitudinal direction of tunnel advancement (M_x) causes the pile to bulge in the same direction as the shield advancement. The bending moment of the pile in the transverse direction of tunnel advancement (M_y) causes the pile to bulge towards the shield due to stress relief around the shield.

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