SRI LANKAN GEOTECHNICAL SOCIETY

A Member Society of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)

ANNUAL CONFERENCE 2025

CHALLENGES IN DEEP FOUNDATIONS AND BASEMENT CONSTRUCTIONS

22nd October 2025

At Galle Face Hotel, Colombo, Sri Lanka

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PRESIDENT'S MESSAGE

It is with great pleasure and honour that I convey this message to the **Annual Conference – 2025** of the **Sri Lankan Geotechnical Society (SLGS)**, held under the theme **"Challenges in Deep Foundations and Basement Constructions."** This event, as is customary, is organized preceding the Annual General Meeting of the Society.

The Sri Lankan Geotechnical Society (SLGS) was established nearly three and a half decades ago under the visionary leadership of Professor A. Thurairajah, who, after an illustrious and inspiring academic career at the University of Cambridge, returned to his motherland and became rightfully recognized as the Father of Soil Mechanics in Sri Lanka.

Since its inception, SLGS has played a vital role in advancing the discipline of **Geotechnical Engineering** in Sri Lanka, serving as the principal professional body for both practicing engineers and academia in the field. The Society's commitment and global recognition were further strengthened through its affiliation with the **International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)**.

Over the years, SLGS has organized a wide range of professional development activities — including **conferences, seminars, and workshops** — facilitated by internationally and nationally acclaimed experts. The monthly **Geotechnical Forum** has been a flagship initiative, providing a platform for local and regional geotechnical professionals to exchange knowledge and learn from the experiences of both academia and industry experts. In addition, **technical visits** to major infrastructure and geotechnically significant project sites have enhanced practical exposure among our members.

The Society has proudly hosted **three major international conferences** in 2007, 2015, and 2021, each attracting a large number of globally renowned academics and practitioners. Continuing this tradition, **SLGS will host the next International Conference on Geotechnical Engineering (ICGE–Colombo 2026)** on **24th and 25th August 2026** at **Cinnamon Grand Colombo**, under the theme **"Resilient Geotechnics for a Sustainable Future,"** in celebration of the **40th Anniversary of SLGS**.

During the current tenure, we successfully conducted **six Geotechnical Forums** on contemporary topics of national and regional importance. As the final event of this tenure, the **Annual Conference 2025** has been organized in **hybrid mode**, enabling the participation of both in-person and online attendees to maximize the reach and benefit of this knowledge-sharing platform.

Today's conference brings together a distinguished panel of **resource persons** from both academia and industry, who will share their expertise and insights on the theme **"Challenges in Deep Foundations and Basement Constructions."** We are deeply grateful to our eminent speakers for accepting our invitation despite their demanding schedules. Their contributions will undoubtedly make this conference a thought-provoking and enriching experience for all participants in the geotechnical community.

On behalf of the Society, it is my distinct privilege to warmly welcome and express our sincere appreciation to:

- **Prof. Saman Thilakasiri**, Senior Professor, Sri Lanka Institute of Information Technology
- **Eng. R.M. Abeysinghe**, Principal Geotechnical Engineer, Pile Test Consultants (Pvt) Ltd.
- **Prof. Nalin de Silva**, Professor, Department of Civil Engineering, University of Moratuwa
- **Eng. Madan Kumar Annam**, Engineering Director, Keller Asia (nominated by IGS)
- **Dr. Asiri Karunawardena**, Director General, National Building Research Organisation

for serving as **Resource Personnel** and sharing their valuable knowledge and experience with our members and professionals across Sri Lanka and the region.

I also extend my gratitude to **Emeritus Prof. Athula Kulathilaka** of the University of Moratuwa for graciously agreeing to **co-chair the sessions** alongside me at this conference.

In addition, I wish to place on record our heartfelt appreciation to **Dr. Anil Joseph**, President of the **Indian Geotechnical Society**, for his kind assistance and cooperation in organizing this event.

With immense gratitude, I warmly welcome all **distinguished invitees**, including **Past Presidents**, and both **past and present Executive Committee Members of SLGS**, whose presence greatly honours this occasion.

Our sincere thanks are also extended to our **Platinum Sponsors, Prime Residencies and Home Lands, Gold Sponsors, Maga Engineering (Pvt) Ltd, San Piling (Pvt) Ltd, Silver Sponsors, Access Engineering, PileTest Consultants (PVT) LTD, ELS Construction (Pvt.) Ltd, K D Piling (PVT) LTD, Bitumix (Pvt) Ltd. and Bronze sponsor, D P Jayasinghe Piling Co – Pvt. for their invaluable support and generous sponsorship, which have been instrumental in making this event a success.**

I am particularly grateful to the members of the **Organizing and Executive Committees**, especially **Eng. R.M. Abeysinghe**, **Dr. K. H. S. M. Sampath**, **Eng. Mahinda Rathnasiri**, **Eng. (Ms.) Lasanda Padmasiri**, and **Eng. Janaka Priyantha**, for their unwavering commitment and dedication in ensuring the successful organization of this conference.

Finally, I extend a warm welcome to all participants who have joined us in this significant event. Your presence and engagement make this gathering truly meaningful.

Undoubtedly, this conference will be yet another **landmark achievement** in the journey of the Sri Lankan Geotechnical Society — an event that will inspire, inform, and strengthen our professional community.

I wish you all a most **productive and rewarding conference experience**.

Thank you.

Eng. K.L.S. Sahabandu
B.Sc.Eng.(Hons.), Pg. Dip.(Hyd.Eng.), M.Sc.(Struct. Eng.), C.Eng., M.I.C.E.(London), M.Cons.E.(S.L.), F.I.E.(S.L.), Hon. F.S.S.E.(S.L.)
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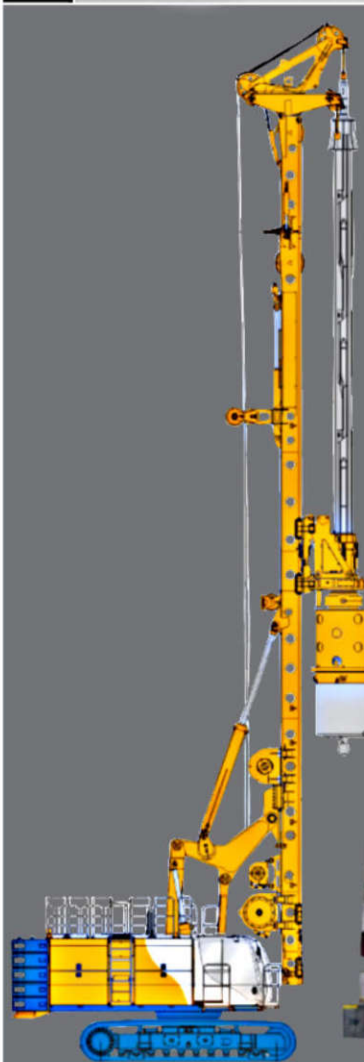

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SLGS ANNUAL CONFERENCE, 2025

"CHALLENGES IN DEEP FOUNDATIONS AND BASEMENT CONSTRUCTIONS"

PROGRAMME

8:30 – 9:00	Registration of Participants / Morning Tea	
9:00 – 9:15	Ceremonial Opening	
9:15 – 9:30	Welcome Address	**Eng. K.L.S. Sahabandu** President - SLGS
Morning Session: Session Chair: Eng. K.L.S. Sahabandu		
9:30 – 10:30	Estimation of the ultimate skin friction and allowable end bearing capacities of bored and cast-in situ piles in the rock socket using Rock Mass Classification System (RMR)	**Prof. Saman Thilakasiri** Senior Professor, Sri Lanka Institute of Information Technology (SLIIT)
10:30 – 11:30	Quality assurance methods for enhancing structural reliability and optimization of pile foundations & deep retaining walls	**Eng. R.M. Abeysinghe** Principal Geotechnical Engineer, Pile Test Consultants (PVT) LTD
11.30 – 12.30	Challenges encountered during piling through cavernous and weak rock formations	**Prof. Nalin De. Silva** Professor, Department of Civil Engineering, University of Moratuwa
12:30 – 13:30	**Lunch Break**	
Afternoon Session: Session Chair: Prof. Athula Kulathilaka		
13:30 – 14:30	Ensuring safety in deep excavations through No-Go rules	**Eng. Madan Kumar Annam** Engineering Director, Keller ASIA
14:30 – 15:30	Best practices to mitigate damages to properties adjacent to construction in urban areas	**Dr. Asiri Karunawardena** Director General, National Building Research Organisation (NBRO)
15:30 – 15:40	Vote of Thanks & Closing Remarks	**Dr. K.H.S.M. Sampath** Hony. Secretary - SLGS
15:40 – 16:00	Evening Tea	
16.00 – 16.45	29th Annual General Meeting of SLGS	

* Each presentation will have 50 minutes for presenting and 10 minutes for Q&A.

SLGS ANNUAL CONFERENCE, 2025

"CHALLENGES IN DEEP FOUNDATIONS AND BASEMENT CONSTRUCTIONS"

CONTENT

Changing landscapes...
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Eng. K. L. S. Sahabandu

Session Chair – Morning Session

SLGS Annual Conference, 2025

Eng. K. L. S. Sahabandu is a distinguished engineer with over 40 years of experience in the engineering consultancy sector. He has served leading organizations in Sri Lanka, most notably the Central Engineering Consultancy Bureau (CECB), the country's largest multidisciplinary consultancy firm, where he retired as General Manager in 2019. He currently serves as Head of the Engineering Consultancy Division at the Urban Development Authority (UDA).

His expertise spans structural and geotechnical design for major national projects in hydropower, high-rise buildings, highways, railways, harbours, and airports. He played a key role in Sri Lanka's expressway and railway development, contributing to the Southern Expressway Project and several subsequent initiatives as Team Leader, Technical Committee Member, and Investigator etc.

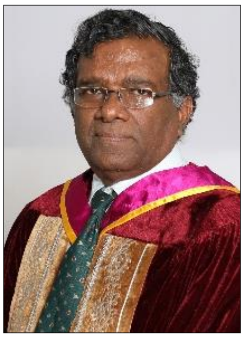
Eng. Sahabandu presently serves as President of both the Sri Lankan Geotechnical Society (SLGS) and the Association of Consulting Engineers – Sri Lanka (ACESL). He is also a former President of the Society of Structural Engineers, Sri Lanka (SSESL) and Immediate Past Vice President of the Sri Lanka National Committee of large Dams (SLNCOLD).

He has been recognized with several national honours, including the CIDA Award of Eminence (2016), the Chartered Engineer Excellence Award (2018), and the Eng. (Dr.) A. C. Visvalingam Award for the Most Outstanding Engineer (2023) from the Institution of Engineers, Sri Lanka (IESL), acknowledging his exceptional contribution to the engineering profession and the construction industry in Sri Lanka.

Prof. Athula Kulathilaka

Session Chair – Afternoon Session

SLGS Annual Conference, 2025



Prof. Athula Kulathilaka is an emeritus professor at the University of Moratuwa, renowned for his significant contributions to geotechnical engineering and the Sri Lankan Geotechnical Society (SLGS). As a past president and secretary of the SLGS, he has played a pivotal role in organizing various activities and has actively represented society on international platforms, including conferences and knowledge-sharing workshops. His efforts have effectively showcased the achievements of the Sri Lankan geotechnical community, fostering collaborations and knowledge exchange with global experts.

Prof. Kulathilaka holds a B.Sc. from the University of Moratuwa and earned his PhD from Monash University in 1990, focusing on the finite element analysis of earth-retaining structures. His key research areas include internally stabilized earth-retaining structures, finite element analysis of earth-retaining structures, slope stability analysis, rain-induced landslides, characteristics of unsaturated soils, and ground improvement techniques.

A committed member of several professional organizations, he has been affiliated with the Institution of Engineers Sri Lanka (IESL) since 2004 and the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) since 1992. Prof. Kulathilaka's enduring contributions to the field have established him as a leading figure in geotechnical engineering in Sri Lanka and beyond.

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Estimation of the ultimate skin friction and allowable end bearing capacities of bored and cast-in situ piles in the rock socket using Rock Mass Classification System (RMR)

By

Prof. Saman Thilakasiri, Senior Professor, Sri Lanka Institute of Information Technology (SLIIT)

Prof. Saman Thilakasiri is a Senior Professor at the Faculty of Engineering, Sri Lanka Institute of Information Technology (SLIIT). He holds a First Class Honors Engineering Degree from the University of Moratuwa, Sri Lanka, a Master's Degree with Distinction from Imperial College, London, UK, and a PhD from the University of South Florida, USA. With a robust academic career spanning over three decades since 1990, Prof. Thilakasiri has served in various academic capacities.

In addition to his academic pursuits, he has extensive experience as a senior engineer on numerous national-level projects. He obtained chartered status from the Institution of Engineers Sri Lanka (IESL) in 2003 and became a Fellow of the same institution in 2007. Prof. Thilakasiri has also gained valuable experience from prestigious educational institutions in the USA, UK, and Australia, serving as a research student, adjunct faculty, and research fellow.

Before joining SLIIT in 2014, he held significant administrative roles at the University of Moratuwa, including Director of Undergraduate Studies and Postgraduate Research Coordinator for the Faculty of Engineering. His diverse expertise in engineering and education continues to make a substantial impact on the field in Sri Lanka and beyond.

Grades of rock based on RMR (Rock Mass Rating) of End Bearing Bored Piles in Sri Lanka

H. S. Thilakasiri
Senior professor, Sri Lanka Institute of Information Technology, Malabe

ABSTRACT: As a developing country, Sri Lanka is undergoing massive construction transformation, with numerous skyscrapers and highway bridges being built across the country. Most of these constructions are progressing mainly in capital Colombo and its suburbs. For these large-scale buildings and bridges, pile foundations are fundamental requirement. In Sri Lanka primarily use bored and rock sockted cast in situ bored piles. However, the current design methods for these piles design of rock socket for these pile foundations are not very cost effective. The Hong Kong guidelines which is based on the modified the RMR method proposed by Bieniawski (1989) to was adopted in Sri Lanka to develop a more efficient approach. This is because the RMR method is not only more cost effective but also more reliable compared to other methods currently used in Sri Lanka (Thilskasiri et al. (2015)).

Sri Lankan foundation engineers, pile experts, and rock experts, based on their experience, believe that the country's existing rock can bear a higher load capacity than currently assumed. Therefore, most believe that pile foundation in Sri Lanka are under designed. However, Bieniawski (1989) has developed RMR method to suit tunnel construction and in Hong Kong Geo guideline (2006) the RMR values for the rock mass beneath the test piles are calculated using the data given in the modification to the Geo mechanics classification proposed by Bieniawski in 1989. To address these issues, based on the instrumented pile load tests done in the Sri Lanka is about 20, based on 10 of these instrumented pile load tests, it is suggested to grade the rock in the view of determining skin friction & end bearing with the view of developing new rock grade based on the RMR method based on the Bieniawski (1989) as modified by Geo guidelines (2006). These rock grades and the corresponding capacities can be modified as and when new data is made available.

Keywords: Bored piles, Instrumented Pile Load Tests, Skin friction in bedrock, end bearing in bedrock

1. INTRODUCTION

Instrumented Pile load test (IMLT) is an accurate method of determining the mobilized end bearing capacity and the mobilized skin frictional capacity of the bedrock. (1) Terzarghi Rock Mass Classification System (rarely used), (2) Rock Structure Rating (RSR) method (3) Rock Mass Rating System (RMR) by Bieniawski (1976), (4) Rock Tunneling Quality Index by Barton (1973). (better known as the Q system).

CIRIA (1995) and Clayton et al. (1995) both argue that, among the established classification systems, RMR is more suitable for foundation engineering, especially in the context of piling, compared to Barton's Q-system proposed in 1983 (Littlechild et al 2000). ASCE (1999) also recognized the application of RMR in the design of rock slopes and foundations, including its role in estimating the in-situ modulus of deformation and rock mass strength. Moreover, RMR has proven particularly advantageous for foundation design work since it can be determined from the borehole investigation data from a vertical borehole. Therefore, in this paper use of RMR value using vertical borehole is described. Latter, using the instrumented pile load tests results, the used values are justified.

Bieniawski identified the selection of parameters to classify soil as an important issue ,because there is no single parameter or index that can fully and quantitatively describe a jointed rock mass for engineering purposes. There are different significances for different parameters and to classify a rock mass satisfactorily all parameters must be taken together.

The following six parameters are used to classify a rock mass using the RMR system:

1. Uniaxial compressive strength of rock material. Rock Quality Designation (RQD).
2. Spacing of discontinuities.
3. Condition of discontinuities.
4. Groundwater conditions.
5. Orientation of discontinuities

In Hong Kong Geo guideline the RMR values for the rock mass beneath the test pile are calculated using the data given in the Table 1 below. It is a modification to the Geo mechanics classification proposed by Bieniawski in 1989.

Table 1- Rating Assigned to Individual Parameters using RMR Classification System - Based on Bieniawski, 1989 (HK Geoguideline, 2006)

(A) Strength of Intact Rock

Uniaxial compressive strength, σ_c (MPa)	> 250	250 – 100	100 – 50	50 – 25	25 – 5	5 – 1	< 1
Point load strength index, PLI_{50} (MPa)	> 10	10 – 4	4 – 2	2 – 1	σ_c is preferred		
Rating	15	12	7	4	2	1	0

(B) Rock Quality Designation (RQD)

RQD (%)	100 – 90	90 – 75	75 – 50	50 – 25	< 25
Rating	20	17	13	8	3

(C) Spacing of Joints

Spacing	> 2 m	2 m – 0.6 m	0.6 m – 0.2 m	200 – 60 mm	< 60 mm
Rating	20	15	10	8	5

(D) Conditions of Joints

Discontinuity length[(1)]					
Rating	2				
Separation	None	< 0.1 mm	0.1 – 1 mm	1 – 5 mm	> 5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickenside
Rating	6	5	3	1	0
Infilling (gouge)	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Rating	6	5	3	1	0

(E) Groundwater

Rating[(1)]	7

Notes :

(1) Rating is fixed as the parameter is considered not relevant to the evaluation of allowable bearing pressure of rock mass.

(2) RMR is the sum of individual ratings assigned to parameters (A) to (E).

1.1. DETERMINATION OF RMR

For examples CR = 93%, RQD = 80%, Fractures are slightly rough un-weathered ends, hard infilling, UCS = 24 kPa

According to Tomlinson (1994)

Table 2-RQD vs. Fracture spacing (Tomlinson, 1994)

RQD	Fracture frequency in meters
0 - 25	15
25 - 50	15 - 8
50 - 75	8 - 5
75 - 90	5 - 1
90 - 100	1

Fracture spacing is 1 when RQD = 90% and Fracture per meter is 5 when RQD = 75%. What is the fracture per meter when RQD = 80%? By linear interpolation, Average Fracture per meter =3.66; infilling gouge = (100 - 80) / 3.66 = 5.46 cm/m

A. Rating = 4
B. Rating = 17
C. Rating = 10
D.
  d.1. Rating = 2

  d.2. 20 cm/m > 5 mm > Rating = 0

  d.3. – Slightly rough = Rating = 3

  d.4. – infilling gauge (100 – 80)/3.66 = 5.46 cm/m > 5mm (Hard filling), Rating = 2

  d.5. – weathering Rating = 6 (un-weathered ends)

E. Groundwater, Rating = 7

RMR = 51

For example, CR = 100%, RQD = 15%, Fractures are slightly rough, un-weathered ends, hard infilling, UCS = 24 kPa

Fracture per meter is 15, when RQD = 25% and Fracture per meter is 0 when RQD = 0%. What is the fracture per meter when RQD = 15%? By linear interpolation, Fracture frequency per meter = 9.0; Average Fracture spacing = 1/9.0 = 0.111 per meter. By linear interpolation Fracture frequency per meter =9.0; Average infilling gouge = (100 - 15)/9.0 = 9.44 cm/m

Separation of the fractures = (100% - RQD%)/9 = (100 - 15)/9 cm/m = 98.44 cm/m. Based on Table 1 below

A. Rating = 4
B. Rating = 3
C. Rating = 8
D.
  d.1. Rating = 2

d.2. 9.44 cm/m > 5 mm > Rating = 0

d.3 – Slightly rough = Rating = 3

d.4 – infilling gouge (length > 5mm (Hard filling), Rating = 2

d.5 – weathering Rating = 6 (un-weathered ends)

E. Groundwater, Rating = 7

RMR = 37

1.2. INSTRUMENTED STATIC LOAD TEST

Normal static load test is done measuring the pile top and the force applied at top of the settlement. In an instrumented pile load test also we perform the static pile load test on an instrumented pile. The pile is instrumented with strain gauges at intervals the pile enabling it to determine the mobilized skin friction between strain gages using the following equation. The strain gauges are tied to the reinforcement cage of the pile during installation of the pile as shown in Figure 1.

The skin friction is measured using the following relationship.

$$F_{Z+\Delta Z} - F_Z = \Delta F_S$$

Where, $F_{z+\Delta c}$ is the axial force measured at the strain gauge installed at z + Δz and $F_z$is the axial force from the strain gauge installed at depth z.

The mobilized ski friction and end Bearing are measured directly, the method of getting the distribution of stresses in a pile is more accurate than the getting the distribution of stresses using CAPWAP analysis using PDA test as this is a direct measuring method.

Figure 1 – Sacrificial strain gauge tied to a main reinforcement during instrumented pile load test.

2. INSTRUMENTED PILE LOAD TEST RESULTS

2.1. INSTRUMENTED PILE LOAD TEST 1

This IMLT was done in the subsurface and the bedrock condition which is highly weathered with no reported RQD up to 10 m into the bedrock coring and CR is less than about 45% and the UCS about 6 MPa. The mobilisation of the skin friction given is by Figure 2 and mobilized end bearing given by Figure 3. Table 1 gives pile head movement and applied load on the pile.

Table 3- Summary of Pile Head Movement and the maximum load on the pile at each cycle

	Cycle 1 Max. Test Load 4756 kN	Cycle 2 Max. Test Load 7,134 kN	Cycle 3 Max. Test Load 14268 kN
Gross Settlement (mm)	5.46	7.71	19.79
Permanent Settlement (mm)	2.87	3.72	11.12
Recovery Rebound (mm)	2.59	3.99	8.67

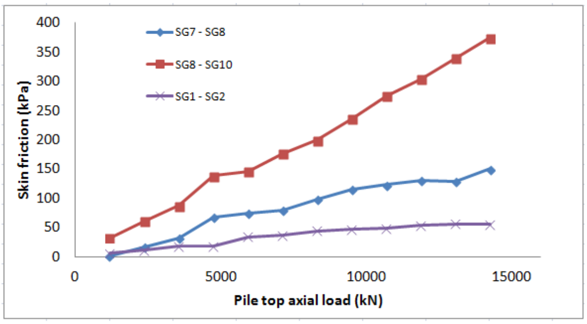


Figure 2a – Mobilised skin friction

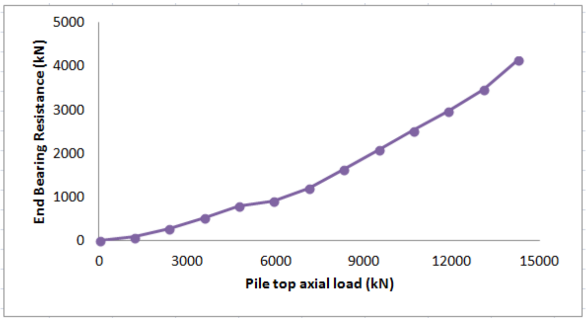


Figure 2b – mobilized end bearing

Table 4 – Mobilized capacities

Load as per Load Cell reading (kN)	Pile top settlement mm	Total shaft resistance (kN)	Base resistance (kN)	Mobilized Unit Shaft friction, f_s (kN/m^2)			Mobilized Unit End Bearing, (kN/mm^2)
				SG1 to SG2	SG7 to SG8	SG8 to SG10	
1189	0.74	1108	81	6	1	32	46
2378	1.93	2100	278	11	17	61	157
3567	3.11	3043	524	17	32	87	297
4756	5.46	3961	795	18	68	138	450
5945	6.14	5034	911	34	74	146	516
7134	7.71	5924	1210	37	80	177	685
8323	8.46	6697	1626	44	98	200	920
9512	10.70	7434	2078	47	115	236	1176
10701	12.82	8172	2529	49	123	275	1431
11890	14.93	8919	2971	53	130	304	1681
13079	16.86	9607	3472	56	129	340	1965
14268	19.79	9579	4139	55	150	375	2342

Table 5- Recommendations

Layer	Skin Friction, f_s (kN/m^2)		End Bearing, f_b (kN/m^2)	
Elevation MSL (m)	Ultimate	Allowable	Mobilized	Allowable
-1.137 to -14.37 (SG1 – SG2)	56	20	-	-
-21.87 to -23.37 (SG7 – SG8)	150	75	-	-
-23.37 to -26.37 (SG8 – SG10)	375	125	-	-
Pile Tip	-	-	2342	1500

The skin friction mobilisation in the strain gauges (SG 8 to SG 10) is linearly increasing but the skin friction mobilization between SG 7 –SG 8 (in the weathered rock) has come close to ultimate value of 150kPa. The mobilized end bearing is increasing. But the mobilized ultimate end bearing is 2342kpa can be increasing trend. Therefore, the allowable end bearing of the mobilised completely weathered rock is taken as 1500 kpa and ultimate skin friction as 125 kPa even though the mobilised SF is 375 kPa.

2.2. INSTRUMENTED PILE LOAD TEST 2

This IMLT was done in the subsurface and the bedrock condition which is reported RQD and CR varying 06% to 60% and 75% to 100% respectively and the UCS between about 2.70MPa to 48.09 MPa in the nearest borehole. In the nearest borehole rock is encountered at about 17m depth and during the drilling for the pile bedrock was encounter at about 19.5m depth. The mobilisation of the skin friction given is by Figure 4 and mobilized end bearing given by Figure 5. Weathering varied from slightly weathered to fresh.

Table 6 gives pile head movement and applied load on the pile.

Table 6- Summary of Pile Head Movement and the maximum load on the pile at each cycle

	Cycle 1 Max. Test Load 6687 kN	Cycle 2 Max. Test Load 10030 kN	Cycle 3 Max. Test Load 20061 kN	Cycle 4 Max. Test Load 26748 kN
Gross Settlement (mm)	2.56	3.86	9.57	13.80
Permanent Settlement (mm)	0.30	0.34	1.37	2.31
Recovery Rebound (mm)	2.26 (88%)	3.52 (91%)	8.2 (87%)	11.49 (83%)

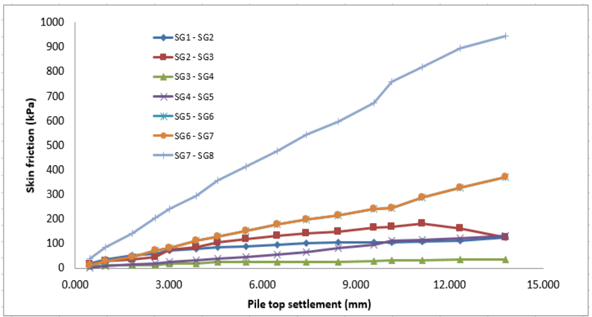


Figure 3a – mobilized skin friction

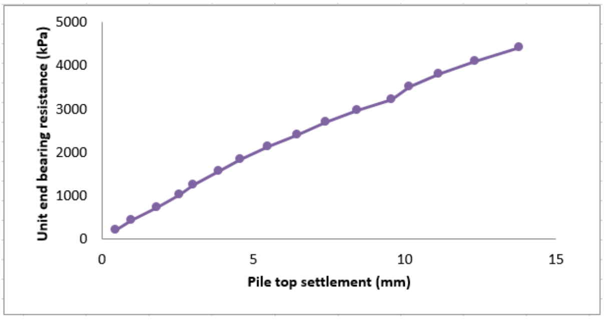


Figure 3b – Mobilised end bearing

Table 7- Mobilized skin friction in the socketed region

Load (kN)	SG5 to SG6	SG6 to SG7	SG7 to SG8
1671	13	13	39
3345	28	28	86
5015	45	45	143
6687	70	70	205
8358	82	82	241
10030	110	110	294
11702	127	127	357
13374	151	151	412
15045	178	178	476
16717	198	198	542
18389	215	215	596
20061	240	240	674
21732	244	244	760
23404	288	288	819
25076	326	326	895
26748	372	372	946

Table 8– Recommended capacities

Layer	Skin Friction, f_s (kN/m^2)		End Bearing, f_b (kN/m^2)	
Depth (m)	Mobilized	Allowable	Mobilized	Allowable
-19.50 to -20.80 (slightly weathered Rock)	240 (Ultimate)	100	-	-
-20.80 to -22.10 (slightly weathered Rock)	240 (Ultimate)	100	-	-
-22.10 to -23.40 (Fresh Rock)	946	300	-	-
Pile Tip	-	-	4431	3750

The skin friction mobilisation in the strain gauges (SG 07 to 08) is increasing with the deformation but the skin friction mobilization between SG 05 –SG 06 and SG 06 to SG 07 (in the slightly weathered rock) has come close to ultimate value of 240 Kpa. The mobilized end bearing is increasing. But the mobilized ultimate end bearing is 4431kpa can be increasing trend. Therefore, ultimate skin friction in the slightly weathered rock as 200 kPa and ultimate skin friction in the fresh unweathered rock as 300 kPa.

2.3. INSTRUMENTED PILE LOAD TEST3

Table 9 gives pile head movement and applied load on the pile.

Table 9- Summary of Pile Head Movement and the maximum load on the pile at each cycle

	Loading Cycle 1 Max. Test Load 3,055.60kN	Loading Cycle 2 Max. Test Load 6,111.2kN	Loading Cycle 2 Max. Test Load 7,639kN
Gross Settlement (mm)	2.13	5.62	7.34
Permanent Settlement (mm)	0.24	0.16	0.29
Recovery Rebound (%)	88.73%	97.15%	96.04%

Table 10: Summary of Mobilized Unit Skin Friction

Depth (m)		Load Transfer Zones	Mobilized Unit Skin Friction (kN/m²)			At toe	Layer Description
From	To		Loading Cycle 1	Loading Cycle2	Loading Cycle3		
22.300	26.275	L6-L7	113.44	291.21	378.28	40.93	Rock
26.275	29.275	L7-L8	29.66	32.21	36.48	57.25	Rock
29.275	31.275	L8-L9	13.20	33.63	20.22	90,61	Rock

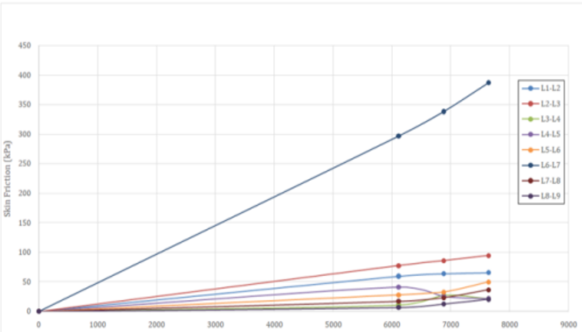


Figure 4a – Mobilized skin friction

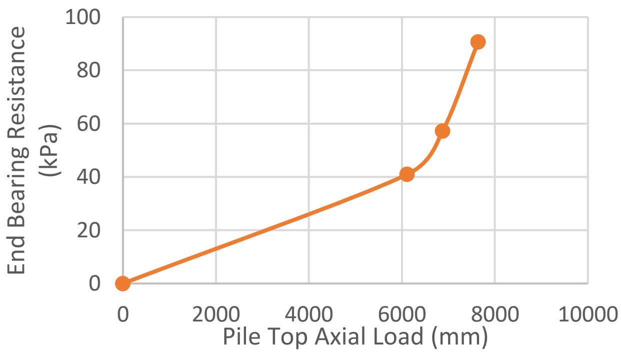


Figure 4b - Mobilized end bearing

It is also seen from the variation of the unit skin friction with the load applied at the pile top that the unit skin friction even in those rock layers are nearly varying linearly with increase in stain and has not clearly reached the ultimate skin friction of these rock layers. Further, the movement of this zone (less than 2mm) is not sufficient to activate full shaft resistance, therefore the mobilized shaft resistance shall not be considered as it has reached to its ultimate value.

2.4. INSTRUMENTED PILE LOAD TEST 4

Two 900 mm diameter piles are tested the results are given at sections 2.4 and 2.5.

Table 11- Summary of Pile Head Movement and the maximum load on the pile at each cycle

	Loading Cycle 1 Max. Test Load 11,300 kN	Loading Cycle 2 Max. Test Load 22,600 kN
Gross Settlement (mm)	5.78	18.12
Permanent Settlement (mm)	0.32	3.80
Recovery Rebound (%)	94.64%	79.02%

Table 12: Summary of Mobilized Unit Skin Friction and end bearing

Applied Load (kN)	L3-L4	L4-L5	At Pile Toe
0	0	0	0
11,300	255.74	324.41	1,704.47
14,125	316.54	438.95	2,503.63
16,950	365.76	556.96	3,471.17
19,775	418.72	671.60	4,623.81
22,600	440.53	787.55	5,692.33
Type of material	Rock	Rock	At pile toe

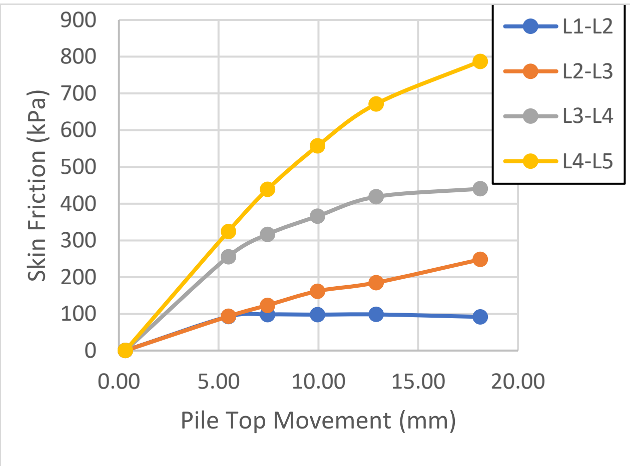


Figure 5a - Mobilized skin friction vs. Pile top movement

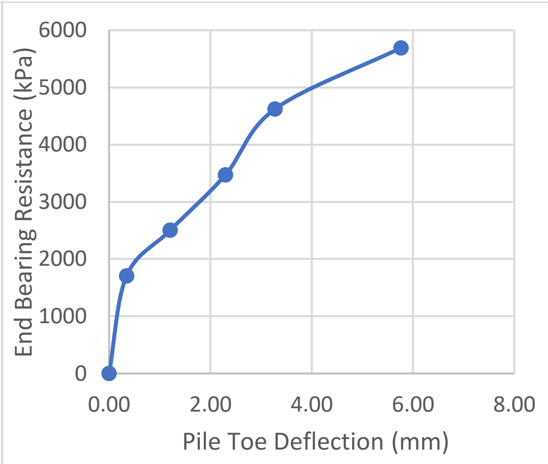


Figure 5b - Mobilized end bearing vs. pile toe settlement

Table 13 - Recommendations

Layer Description	Ultimate Shear Resistance (kPa)	Allowable End Bearing Resistance (kPa)
Completely Weathered Rock-	200	-
Highly Weathered Rock-	250	-
Slightly/Moderately Weathered Roc	350	-
Fresh	500	7,500

2.5. INSTRUMENTED PILE LOAD TEST 5

Table 14- Summary of Pile Head Movement and the maximum load on the pile at each cycle

	Loading Cycle 1 Max. Test Load 11,300kN	Loading Cycle 2 Max. Test Load 22,600kN
Gross Settlement (mm)	7.61	18.58
Permanent Settlement (mm)	1.17	3.93
Recovery Rebound (%)	84.62%	78.84%

Table 15 –Mobilized skin friction and unit end bearing

Applied Load (kN)	Pile Segment/Load Transfer Zone (kPa)					
	L1-L2	L2-L4	L4-L5	L5-L6	L6-L7	At Pile Toe
11,300	21.28	65.49	116.66	349.72	452.45	3224.8
14,125	47.14	126.55	136.29	375.69	498.65	3657.3
16,950	66.07	139.03	136.72	378.71	527.41	5330.4
19,775	69.35	146.04	216.19	368.13	537.98	6960.6
22,600	69.35	204.86	263.96	372.17	537.33	8217.4
Type of material	soil	soil	W. rock	Rock	Rock	At pile toe

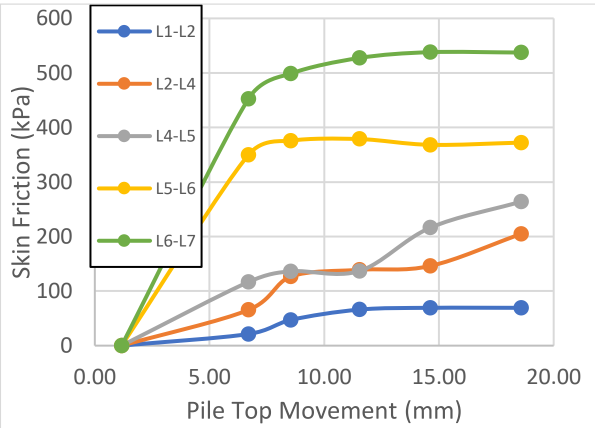


Figure 6a - Mobilized skin friction

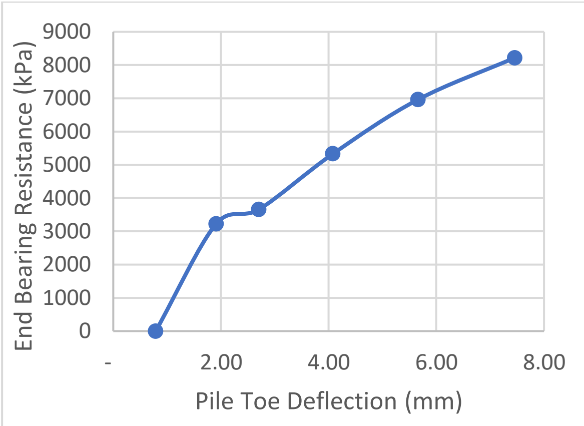


Figure 6b - Mobilized end bearing vs. pile toe sett

Table 16 – Final recommendations according to the final report

Layer Description	Ultimate Shear Resistance (kPa)	Allowable End Bearing Resistance (kPa)
Highly Weathered Rock-	200	
Slightly/Moderately Weathered Rock	350	-
Fresh Rock	500	7,500

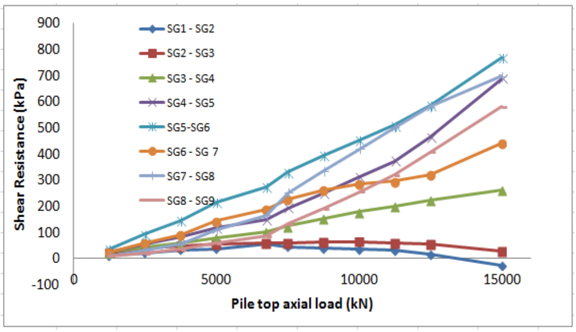
2.6. INSTRUMENTED PILE LOAD TEST 6

Table 17- Summary of Pile Head Movement and the maximum load on the pile at each cycle

	Cycle 1 Max. Test Load 5000 kN	Cycle 2 Max. Test Load 12500 kN	Cycle 3 Max. Test Load 15000 kN
Gross Settlement (mm)	5.44	19.03	26.07
Permanent Settlement (mm)	0.70	2.73	4.39
Recovery Rebound (mm)	4.74	16.30	21.68

Table 18 –Mobilized skin friction

Load as per Load Cell reading (kN)	Mobilized Unit Shaft friction, f_s (kN/m^2)							
	SG1 to SG2	SG2 to SG3	SG3 to SG4	SG4 to SG5	SG5 to SG6	SG6 to SG7	SG7 to SG8	SG8 to SG9
1250	12	22	23	26	38	27	11	10
2500	22	39	43	57	93	60	31	20
3750	34	49	59	83	146	92	56	35
5000	38	55	80	117	218	144	112	60
6750	56	61	102	149	275	188	167	86
7500	46	58	125	194	332	229	253	132
8750	41	65	155	251	396	264	337	191
10000	36	66	179	312	455	287	420	255
11250	32	61	201	374	511	296	501	321
12500	16	57	225	465	587	321	582	409
15000	-27	29	262	688	769	441	698	581



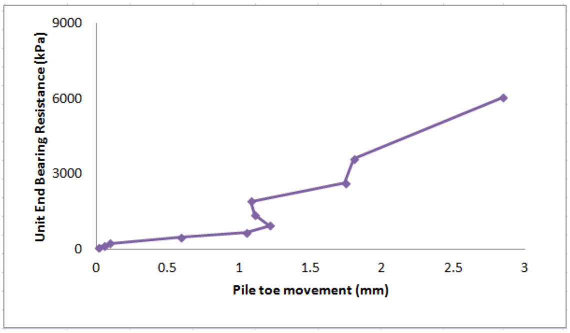


Figure 7a: Unit Skin Friction vs. pile top Load Figure 7b: Unit End Bearing vs. Pile Toe Sett.

Table19: Recommendations

Layer	Skin Friction, f_s (kN/m^2)		End Bearing, f_b (kN/m^2)	
Elevation MSL (m)	Mobilized	Allowable	Mobilized	Allowable
-22.70 to -24.80 (SG4 – SG5)	688	200		
-24.80 to -26.60 (SG5 – SG6)	769	200		
-26.60 to -27.60 (SG6 – SG7)	441	220		
-27.60 to -28.60 (SG7 – SG8)	698	325		
-28.60 to -29.60 (SG8 – SG9)	581	275		
Pile Tip	-	-	6028	>5000

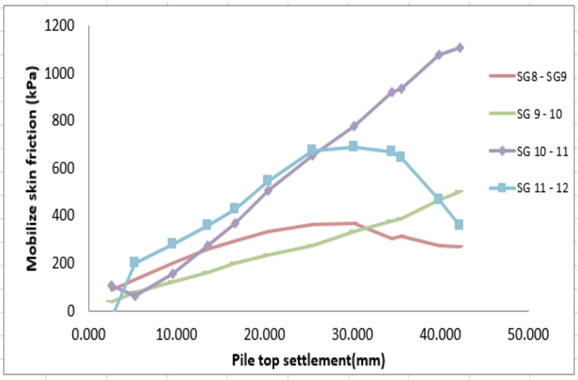
2.7. INSTRUMENTED PILE LOAD TEST 7

Table 20 - Summary of Pile Head Movement and the maximum load on the pile at each cycle

	Cycle 1 Max. Test Load 10600 kN	Cycle 2 Max. Test Load 21160 kN	Cycle 3 Max. Test Load 29920 kN
Gross Settlement (mm)	13.49	30.22	42.15
Permanent Settlement (mm)	7.29	12.50	16.93
Recovery Rebound (mm)	6.20	17.72	25.22

Table 21 – Mobilized unit skin friction in the socketed region

Load (kN)	SG8 to SG9	SG9 to SG10	SG10 to SG11	SG11 to SG12
2680	87	39	107	-31
5320	134	76	64	201
7960	202	123	156	277
10600	262	159	274	359
13240	293	202	367	430
15880	335	233	507	546
18520	363	274	655	675
21160	369	333	778	687
23800	304	379	921	669
26440	312	389	934	645
29080	276	466	1077	466
29920	271	503	1109	359



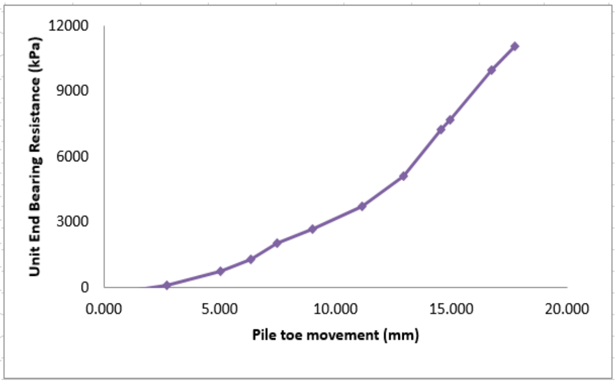


Figure 8a- Skin Friction vs. Pile top displace.   Figure 8b: End Bearing vs. Pile Toe Settlement

Table 22: Recommendations

Layer	Skin Friction, f_s (kN/m^2)		End Bearing, f_b (kN/m^2)	
Elevation MSL (m)	Mobilized	Allowable	Mobilized	Allowable
-34.50 to -36.30 (SG8 – SG9)	368 (Ultimate)	150		
-36.30 to -38.30 (SG9 – SG10)	503	160		
-38.30 to -39.30 (SG10 – SG11)	1108	250		
-39.30 to -40.30 (SG11 – SG12)	687	250		
Pile Tip	-	-	11057	1250

2.8. INSTRUMENTED PILE LOAD TEST 8

Table 23- Summary of Pile Head Movement and the maximum load on the pile at each cycle

	Cycle 1 Max. Test Load 10600 kN	Cycle 2 Max. Test Load 21160 kN	Cycle 3 Max. Test Load 29920 kN
Gross Settlement (mm)	9.522	22.153	35.183
Permanent Settlement (mm)	3.073	7.859	13.483
Recovery Rebound (mm)	6.449	14.294	21.700

Table 24 - Mobilized skin friction in the socketed region

	SG6 to SG7	SG7 to SG8	SG8 to SG9
2680	276	182	4
5320	485	310	0
7960	664	423	70
10600	773	533	92
13240	930	675	121
15880	1072	770	168
18520	1247	851	190
21160	1455	945	250
23800	1636	1059	275
26440	1834	1155	322
29080	1993	1264	367
29920	2088	1356	416

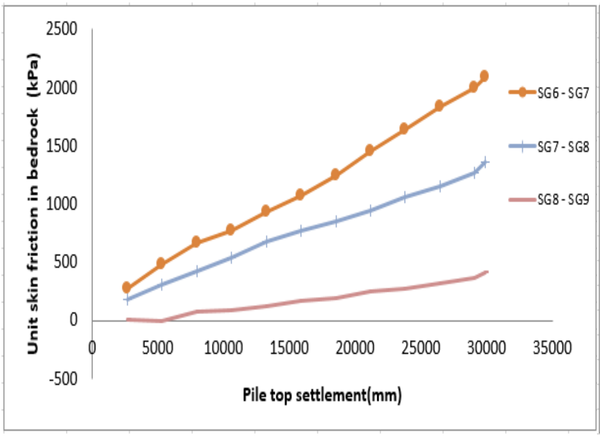


Figure 9a- Skin Friction vs. Pile top disp.

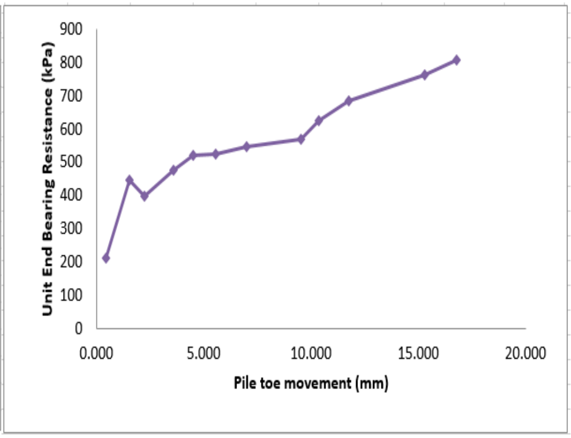


Figure 9b: End Bearing vs. Pile Toe Settlement

Table 25: Recommendations

Layer	Skin Friction, f_s (kN/m^2)		End Bearing, f_b (kN/m^2)	
Depth (m)	Mobilized	Allowable	Mobilized	Allowable
-26.60 to -27.70 (SG6 – SG7)	2008	600		
-27.70 to -28,70 (SG7 – SG8)	1356	400		
-28.70 to -29.70 (SG8 – SG9)	416	120		
Pile Tip	-	-	807	-

2.9. INSTRUMENTED PILE LOAD TEST 9

Table 26 - Summary of Pile Head Movement and the maximum load on the pile at each cycle

	Loading Cycle 1 Max Test Load 10,000kN	Loading Cycle 2 Max Test Load 20,000kN	Loading Cycle 3 Max Test Load 30,000kN
Gross Settlement (mm)	3.08	5.94	8.12
Permanent Settlement (mm)	0.91	1.78	2.14
Recovery (%)	70%	70%	73%

Table 27 - Mobilized skin friction in the socketed region and the end bearing

Applied Load (kN)	Pile Segment/Load Transfer Zone (kPa)			
	W. Rock	Rock	Rock	At Toe
0	0	0	0	0
10000	201	53	16	3774
15000	293	87	33	5505
20000	425	124	58	7285
22500	450	154	68	8328
25000	467	189	75	9398
27500	479	221	87	10421
30000	483	267	101	11868

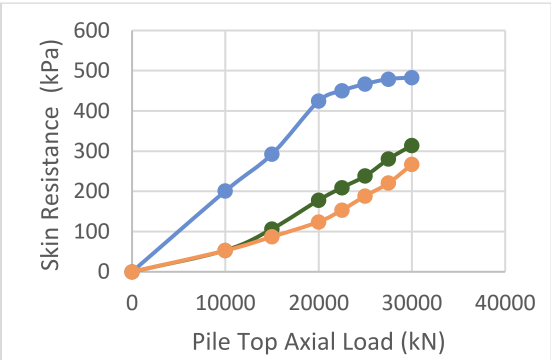


Figure 10a - Skin Friction vs. Pile top force

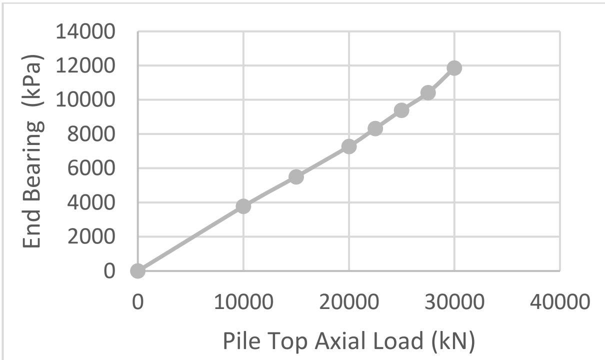


Figure 10b: End Bearing vs. Pile top force

Table 28: Recommendations

31.20	34.30	L13-L14	Moderately and slightly weathered rock	314	200		
34.30	37.40	L14-L15		483			
37.40	40.50	L15-L16		267			
40.50	44.50	L16-L17		101			
45.00	45.00	Pile Tip				11,868	4,000

2.10. INSTRUMENTED PILE LOAD TEST 10

Table 29- Summary of Pile Head Movement and the maximum load on the pile at each cycle

	Loading Cycle 1 Max Test Load 10,000kN	Loading Cycle 2 Max Test Load 20,000kN	Loading Cycle 3 Max Test Load 30,000kN
Gross Settlement (mm)	3.08	5.94	8.12
Permanent Settlement (mm)	0.91	1.78	2.14
Recovery (%)	70%	70%	73%

Table 30 - Mobilized unit skin friction in the socketed region and the unit end bearing

Pile Head load (kN)	W. rock	Rock	Rock	AT TOE
0	0	0	0	0
10000	299	139	350	2104
15000	411	269	569	2614
20000	411	534	767	3221
22500	434	612	868	3595
25000	447	683	966	4141
27500	458	701	1119	4699
30000	486	704	1182	6129

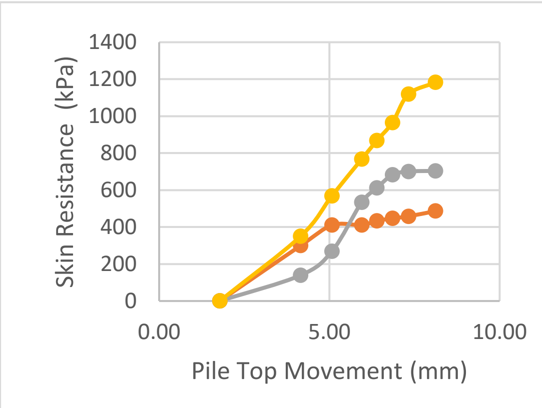


Figure 11a- Skin Friction vs. top movement

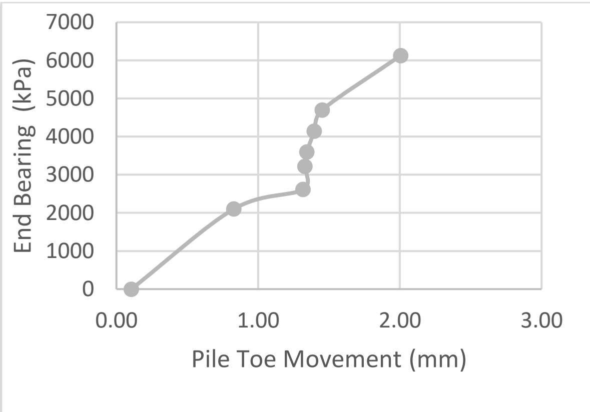


Figure11b - End Bearing vs. pile toe movement

Table 31- Recommendations

Depth (m)		Rock Description	Skin Friction, f_s (kN/m^2)		End Bearing, f_b (kN/m^2)	
From	To		Maximum Mobilized	Allowable	Mobilized	Allowable
6.50	9.50	Biotite Gneiss Fresh Rock	1182**	350		
9.50	10.00	Biotite Gneiss Fresh Rock	-	-	6129**	6000

*Ultimate skin friction had been reached.

** Ultimate condition had not been reached.

3. DATA ANALYSIS

Considering the instrumented load test presented above, the rocks are classified according to Table 32.

Table 32 – Rock grade and RMR

Grade	Description	Lithology	RMR value
Grade I	Fresh rock	Clean rock	<60
Grade II - I	Fresh rock/ Slightly weathered rock	Clean rock / increased fractures	50 - 59
Grade II	Slightly weathered rock	Increased fractures	50
Grade III - II	Slightly weathered rock/ Moderately weathered rock	Increased fractures / Partly changed to soil; rock > soil	40 - 49
Grade III	Moderately weathered rock	Partly changed to soil; rock > soil	35 - 39
Grade IV - III	Moderately weathered rock/Highly weathered rock	Partly changed to soil; rock > soil / Partly changed to soil; rock < soil	30 - 35
Grade IV	Highly weathered rock	Partly changed to soil; rock < soil	26 - 29
Grade IV - V	Highly weathered rock / Completely weathered rock	Partly changed to soil; rock < soil / Some remnant rock structure; completely weathered to soil	21 - 25
Grade V	Completely weathered rock	Some remnant rock structure; completely weathered to soil	18 - 21

Based on the RMR, (HK guideline (2006)) values and the ICTE/DEV/15 (Sri Lankan national standards) and the rock grades, the following ultimate skin frictional capacities and the allowable end bearing capacities can be suggested as given in Table 33 for the rock grades.

Table 33 – Ultimate skin frictional capacities and allowable end bearing capacities based on the above grades

Grade	RMR value	Ultimate skin frictional capacities (kPa)	Allowable end bearing capacities (kPa)	Reference Instrumented Pile load test	
				Skin Friction	End bearing
Grade I	<60	350 - 300	7000	Tests 2, 3, 4, 5	Tests 4,5
Grade II - I	55 - 59	300 - 250	6000 5500	Tests 7, 8,10	HK guideline (2006), Tests 10
Grade II	50 - 54	200	5000	ICTE/DEV/15	ICTE/DEV/15
Grade III - II	40 - 49	200	4000	ICTE/DEV/15, Test 9	ICTE/DEV/15, Test 9
Grade III	35 - 39	200	3000	ICTE/DEV/15	ICTE/DEV/15
Grade IV - III	30 - 35	150	2500		
Grade IV	26 - 29	150	2000	Test 2	Test 2
Grade IV - V	21 - 25	120	1500		
Grade V	18 - 21	120	1000 - 1500	Test 1	Test 1

4. CONCLUSIONS

Sri Lankan foundation engineers, pile experts, and rock experts, based on their experience, believe that the country's existing rock can bear a higher load capacity than currently assumed. Therefore, most believe that pile foundation in Sri Lanka are under designed.

It is shown that the Rock Mass Rating (RMR) can be assigned based on the bore hole investigation of the bedrock for any rock. Rock grades are assigned based on the instrumented pile load test results. Based on the RMR, (HK guideline (2006)) values and the ICTRE/DEV/15 (Sri Lankan national standards) and the rock grades, following ultimate skin frictional capacities and the allowable end bearing capacities can be suggested for the rock grades.

Based on the instrumented pile load tests done in the Sri Lanka is about 20, based on 10 of these instrumented pile load tests, it is suggested to grade the rock in the view of determining skin friction & end bearing with the view of developing new rock grade based on the RMR method based on the Bieniawski (1989) as modified by Geo guidelines (2006). These rock grades and the corresponding capacities can be modified as and when fresh data is made available.

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PiLETesT

Quality assurance methods for enhancing structural reliability and optimization of pile foundations & deep retaining walls

By

Eng. R.M. Abeysinghe, Principal Geotechnical Engineer, Pile Test Consultants (PVT) LTD

Eng. R.M. Abeysinghe is a highly experienced geotechnical engineer with over 25 years of expertise in the installation of bored piles, driven piles, pile testing, geotechnical instrumentation, geotechnical engineering designs and geotechnical investigations. He is a pile testing expert, especially in Dynamic pile testing and instrumented pile load testing. He graduated from the University of Peradeniya with a BSc (Eng) and obtained his MSc degree from the University of Moratuwa, specializing in Dynamic Pile Testing, and a Postgraduate Diploma in Structural Engineering. He is a Chartered Engineer and a corporate member of the Institution of Engineers, Sri Lanka (IESL).

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Quality Assurance Methods for Optimization of Pile Foundations & Deep Retaining Walls.

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Abstract: Quality assurance (QA) and optimization of pile foundations are two interrelated aspects that significantly influence the safety, performance, and economy of geotechnical structures. Despite advances in design methodologies, many foundation failures in practice are still attributed to inadequate QA/QC during installation, while overly conservative designs frequently lead to unnecessary material and cost burdens. This paper presents a consolidated review of QA/QC methods and optimization strategies for deep pile foundations. Key QA/QC measures are outlined across the project lifecycle, including soil investigation, material compliance, construction supervision, and post-installation testing using both destructive and non-destructive techniques. In parallel, design and construction optimization approaches are discussed, with emphasis on rational safety factors, pile–soil–structure interaction, load testing for design verification, and sustainability considerations. Two selected case histories are included to demonstrate how rigorous QA/QC has prevented costly remedial works, and how optimization has delivered significant savings in high-rise and infrastructure projects without compromising reliability. The paper aims to provide practicing engineers with a framework for integrating QA/QC and optimization to achieve safe, cost-effective, and sustainable pile foundation solutions in Sri Lanka's geotechnical practice.

1.0 INTRODUCTION

Pile foundations play a central role in supporting major infrastructure and building projects in Sri Lanka, particularly in urban areas with variable subsurface conditions. The design and construction of pile foundations have traditionally been governed by conservative approaches, with large safety margins applied to account for uncertainties in soil conditions and construction quality. While this conservatism provides assurance against failure, it often leads to unnecessary increases in rock socketing length, diameters, or pile quantities, inflating project costs and material use. At the same time, quality-related defects in pile construction, such as improper slurry management, inadequate cleaning of boreholes, segregation in tremie concreting, or undetected pile damage during driving, continue to pose serious risks to structural safety.

In this context, two complementary strategies are essential: quality assurance (QA) and optimization. QA/QC ensures that the piles constructed in the field meet the intended design assumptions through systematic monitoring, inspection, and testing. Optimization, on the other hand, seeks to refine design and construction practices to achieve cost efficiency, resource economy, and environmental sustainability, while maintaining the required levels of safety and performance. When combined, these approaches offer a pathway to both safer and more economical foundations.

This paper synthesizes lessons learned from over 25 years of practice in pile installation and testing, highlighting QA/QC methods across the project lifecycle and optimization opportunities in design and construction. The discussion is supported by two selected case studies from Sri Lanka demonstrating both the consequences of inadequate QA/QC and the benefits of well-executed optimization. The aim is to equip geotechnical practitioners and design engineers with practical insights that can be directly applied to ongoing and future projects within Sri Lanka's rapidly evolving construction landscape.

2.0 QUALITY ASSURANCE METHODS IN PILE FOUNDATIONS

Quality assurance begins before construction with reliable site investigation and material compliance. Geotechnical investigations must capture the variability the soil conditions, ensuring that design parameters reflect actual ground conditions. Similarly, concrete mix design, reinforcement quality, and casing or pile shoe inspections must be verified prior to work commencement. Establishing QA plans, checklists, and tolerances at this stage sets a strong foundation for subsequent phases.

During construction, QA requires continuous supervision and monitoring. For bored piles, slurry properties such as density, viscosity, and sand content must be checked, while boreholes must be cleaned thoroughly to avoid sediment accumulation at the base. Tremie concreting must be carried out with continuous flow to prevent segregation and cold joints. For driven piles, hammer energy must be monitored, blow counts recorded, and the risk of pile damage carefully observed. Across both methods, accurate setting out, pile alignment, and tolerance checks remain essential.

Post-construction QA focuses on verification through testing. Static load tests in compression, uplift, and lateral modes provide direct capacity validation, while dynamic load testing (PDA) offers a faster, cost-effective alternative that can be correlated with static results. Non-destructive testing methods, including Pile Integrity Testing (PIT), Crosshole Sonic Logging (CSL), and Thermal Integrity Profiling (TIP), are vital for detecting defects such as necking, bulging, or inclusions. Acceptance criteria must be clearly defined, and remedial actions, including pile replacement, strengthening, or redistribution of loads; must be promptly implemented when defects are identified.

2.1 STATIC LOAD TESTS (COMPRESSION, TENSION, LATERAL)

Static load testing remains the most reliable and widely accepted method of verifying the performance of deep foundations. It provides direct measurement of the load–settlement behavior of a pile under controlled conditions and is typically regarded as the benchmark against which other testing methods are calibrated. In compression tests, the pile is loaded incrementally using kentledge, reaction piles, or structural frames until either the design load or a pre-defined maximum test load is reached. The settlement of the pile head is measured at each load stage, and the resulting load–settlement curve is used to assess ultimate capacity and serviceability performance.

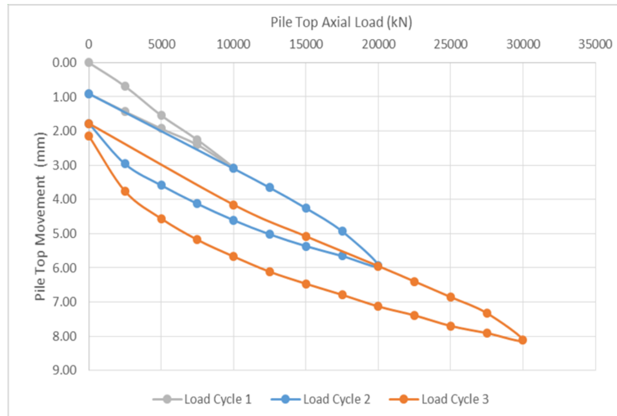

Fig 1. Typical load settlement curve of static load test

Tension or uplift tests are carried out to evaluate pile resistance against upward forces, which are critical in projects with significant wind, seismic, or buoyancy effects. In these tests, the pile head is connected to a reaction system that anchors to adjacent piles or ground beams, and the displacement under incremental tension loads is monitored. Lateral load tests, meanwhile, are performed to determine the pile's stiffness and resistance against horizontal forces. This is of particular importance in bridge abutments, towers, and marine structures, where lateral stability is a governing factor.

For all three types of static load tests, careful instrumentation and monitoring are required. Dial gauges, LVDTs, or electronic displacement transducers should be used to capture displacements with high accuracy. Test durations must comply with recognized standards (e.g., ASTM D1143, D3689, D3966) to ensure reliable results. In Sri Lankan practice, compression tests are more common, but the increasing prevalence of tall and slender structures makes uplift and lateral load testing equally essential for comprehensive QA/QC.

Fig. 2 Kentledge loading arrangement of static load test

2.2 INSTRUMENTED STATIC PILE LOAD TESTS

While conventional static load tests provide valuable information on the global load–settlement behavior of a pile, they do not reveal the distribution of load transfer along the pile shaft or the contribution of end bearing. Instrumented static load tests address this limitation by incorporating sensors within the test pile to capture detailed response characteristics. These tests allow engineers to separate shaft resistance from toe resistance, understand the load transfer mechanism, and verify design assumptions with high precision.

Instrumentation typically includes vibrating wire strain gauges or resistance type strain gauges installed at various depths along the reinforcement cage prior to concreting. In some cases, tell-tales or load cells are installed at discrete levels to monitor internal deformations. During load application, whether in compression or uplift modes, strain measurements are taken simultaneously with settlements. By interpreting strain profiles, the mobilized shaft resistance at different soil strata can be quantified, while the residual load at the pile base represents toe resistance.

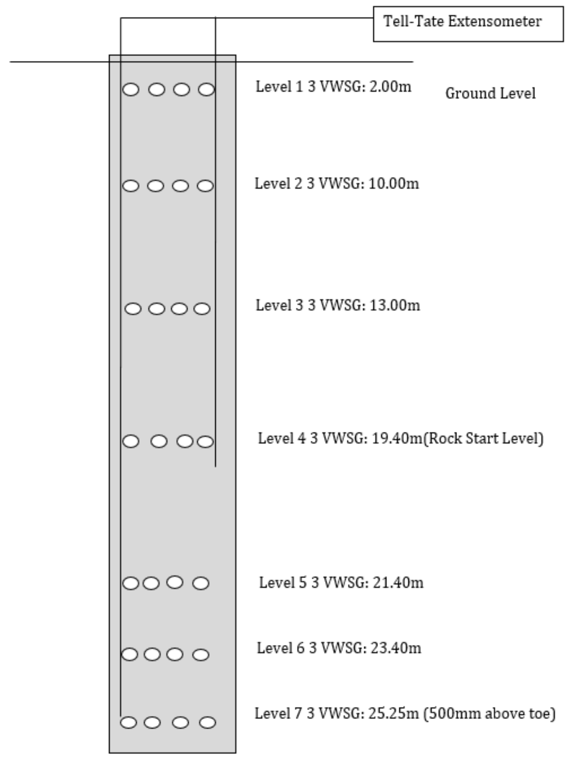

Fig. 3 Typical strain gauges installation locations of instrumented static load test

The results of instrumented load tests are particularly valuable in heterogeneous ground conditions where different soil layers contribute unevenly to pile capacity. For example, in soft clay overlain by dense sand, the majority of load may be transferred through shaft resistance in sand, with limited contribution from the clay or toe. Instrumented data provides evidence of this distribution.

In Sri Lankan practice, instrumented static load tests have proven especially beneficial in high-rise building foundations where settlement control is critical. By calibrating design parameters against actual load transfer data, engineers have been able to optimize pile socketing lengths and diameters, often leading to significant material and cost savings without compromising safety. Furthermore, such tests provide invaluable validation of advanced design approaches such as finite element modeling of pile groups and pile–raft foundations.

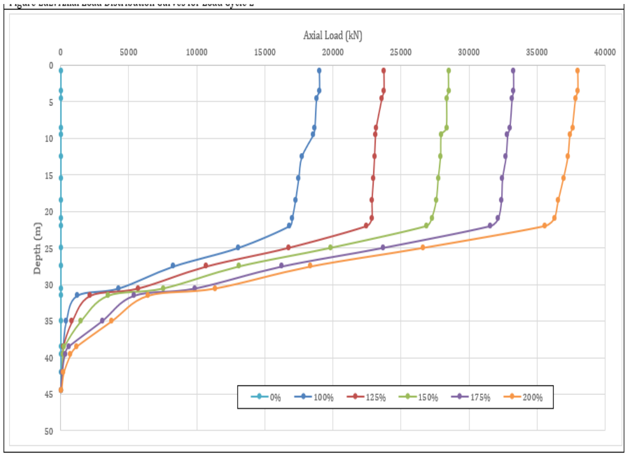

Fig. 4 Axial loads variation along the pile shaft for different loading steps

Although instrumented tests are more complex and costly than conventional static load tests, their diagnostic value far outweighs the additional investment, particularly for landmark or sensitive projects. The ability to directly observe the soil–structure interaction at depth provides confidence to both designers and clients, while also contributing to the growing body of local geotechnical performance data in Sri Lanka.

2.3 DYNAMIC LOAD TESTING (PDA), CAPWAP ANALYSIS AND CORRELATION TO STATIC CAPACITY

Dynamic load testing, performed using the Pile Driving Analyzer (PDA), provides a rapid and cost-effective means of evaluating pile capacity, particularly in large projects where testing every pile with static methods is impractical. The PDA measures strain and acceleration at the pile head during impact from a hammer or drop weight. Using wave equation theory, the pile's capacity, integrity, and driving stresses are estimated in real time.

A key feature of PDA testing is its ability to assess multiple piles in a short duration, making it highly efficient for production control. However, the accuracy of PDA results depends heavily on proper data acquisition and interpretation. To enhance reliability, CAPWAP (Case Pile Wave Analysis Program) is employed as a signal-matching procedure. CAPWAP refines the interpretation of dynamic test data by matching measured signals with theoretical wave propagation models, yielding estimates of ultimate pile capacity, shaft resistance, toe resistance, and load–displacement behavior.

Fig. 5 A guide frame and drop weight for PDA testing

Correlation with static load tests is an essential step in validating PDA results. In well-executed studies, PDA/CAPWAP results correlate with static test outcomes within ±10–15% of ultimate capacity. In Sri Lanka, dynamic testing has been increasingly adopted for large infrastructure projects where cost and time savings are critical. However, the practice of performing at least one or two confirmatory static load tests remains vital for calibration and validation of dynamic methods. When used together, static and dynamic testing provide a comprehensive QA framework that balances accuracy with efficiency.

2.3.1 CASE METHOD

The Case Method, developed at Case Western Reserve University in the 1960s, provides a relatively simple way of estimating pile capacity from measured force and velocity records at the pile head. Using one-dimensional wave equation theory, the method assumes a uniform pile and linear soil resistance over the short time duration of impact. The measured signals are analyzed to calculate an equivalent resistance (Case resistance), which is separated into static and dynamic components. The static portion is taken as an estimate of pile capacity. The bearing capacity, defect assessment and hammer performance given by following equations.

Bearing capacity

$RMX = (1 - J)\ Fd(tx) + (1 + J)\ Fu(tx + 2L/c)$ -(01)

Pile Integrity

$\beta = [Fd(t1) - 1.5\ R + Fu(t3)] / [Fd(t1) + 0.5\ R - Fu(t3)]$ -(02)

Hammer performance

$E(t) = \int F(t)\ v(t)\ dt$ -(03)

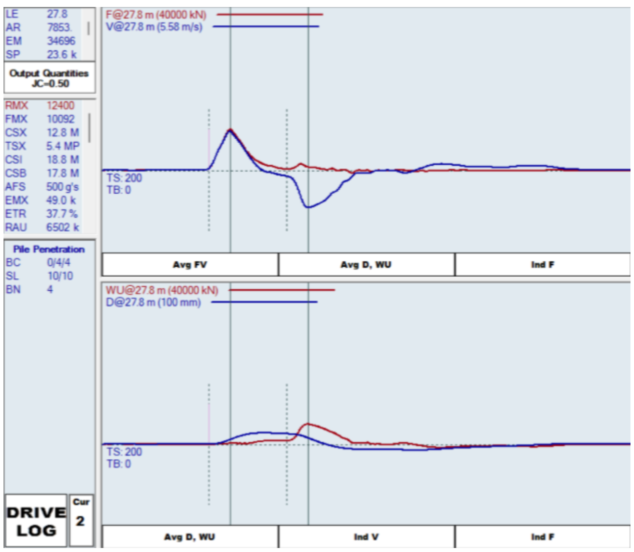

Fig. 6 Force and velocity measurement of 1000mm diameter bored pile by Mode 8G Pile Driving Analyzer

The Case Method is quick and practical for field use, allowing on-site evaluation of pile capacity immediately after driving or re-striking. It has been widely adopted in Sri Lanka and internationally for driven piles, especially where rapid assessments are needed to confirm driving criteria or verify pile termination depths. However, accuracy is influenced by soil damping assumptions and signal quality, and it is generally recommended to use Case Method results for preliminary estimates only. For final acceptance, correlation with static load tests or more refined signal matching analysis is necessary.

2.3.2 CAPWAP METHOD

The Case Pile Wave Analysis Program (CAPWAP) is a signal-matching procedure that builds upon the Case Method by providing a more detailed interpretation of dynamic test results. CAPWAP iteratively adjusts a numerical soil–pile model until calculated force and velocity signals at the pile head closely match the measured PDA signals. Through this process, the program separates static resistance into shaft and toe components, estimates damping characteristics, and produces a simulated load–displacement curve that can be directly compared to static load test results.

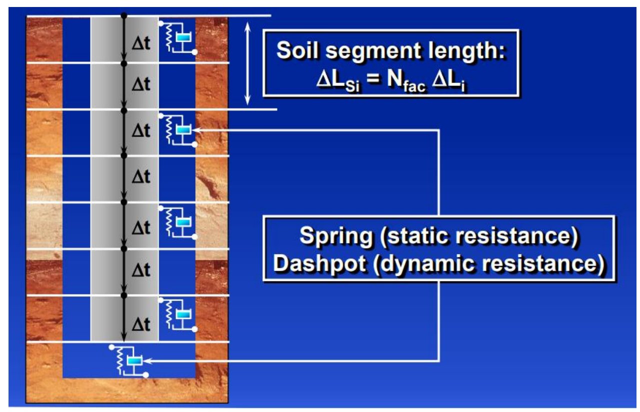

Fig. 7 CAPWAP Pile and soil model.

The key advantage of CAPWAP is its ability to provide a more realistic representation of soil–pile interaction under dynamic loading. It allows identification of the relative contribution of different soil strata, assessment of pile integrity, and evaluation of pile head mobility. CAPWAP results typically correlate within ±10–15% of static load test capacities when the testing is properly executed and calibrated. This makes it an invaluable tool for projects involving large numbers of piles, where performing static load tests on every pile is impractical.

In Sri Lankan practice, CAPWAP has been effectively used for all PDA testing on bridge, port, and high-rise building projects to confirm design assumptions, and validate PDA results against static testing. While CAPWAP analysis requires experienced operators and careful interpretation, its ability to bridge the gap between dynamic and static testing makes it an essential component of modern pile QA/QC frameworks

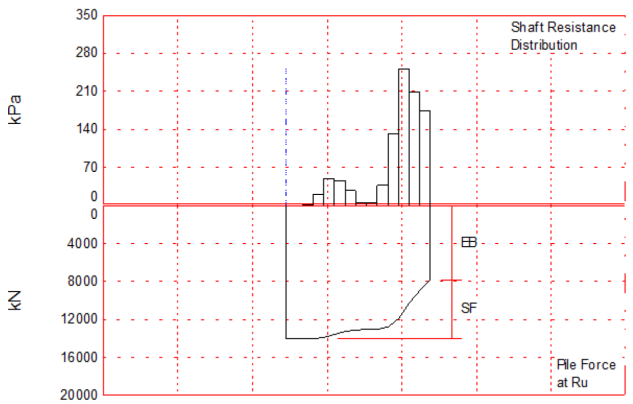

Fig.8 Skin friction distribution and end bearing estimated by CAWAP

2.4 NON-DESTRUCTIVE TESTING

Non-destructive testing (NDT) of piles has become an essential component of modern QA/QC practices, particularly in projects where large numbers of piles must be assessed within limited timeframes. Unlike static or dynamic load tests, which evaluate performance directly under load, NDT techniques are used to verify the structural integrity of piles and detect construction-related defects without imposing significant loads. Three widely adopted methods in Sri Lankan and international practice are the Low Strain Pile Integrity Test (PIT), Cross-hole Sonic Logging (CSL), and Thermal Integrity Profiling (TIP).

2.4.1 LOW STRAIN PILE INTEGRITY TEST (PIT)

The Low Strain Pile Integrity Test, often referred to simply as PIT, is one of the most common and economical methods of screening pile quality. The test is performed by applying a light hammer impact to the exposed pile head while recording the resulting stress wave with an accelerometer or geophone. The principle is based on one-dimensional wave propagation theory: the stress wave travels down the pile, reflects at changes in impedance, and is recorded back at the pile head.

In practice, anomalies such as necking, bulging, voids, or incomplete pile length manifest as reflections occurring earlier than the expected toe reflection. By comparing the measured signal to theoretical expectations for a sound pile, engineers can infer the presence and approximate location of defects. PIT is rapid and requires minimal equipment, allowing large numbers of piles to be screened efficiently at relatively low cost.

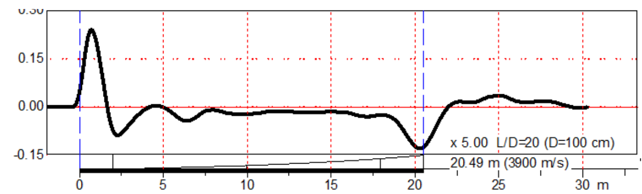

Fig. 9 Velocity record of 1000mm diameter rock socketed bored pile

However, PIT has limitations. The resolution diminishes with pile depth, making it less reliable for very long, large-diameter or piles having high skin friction. It is also less sensitive to minor inclusions, gradual variation or defects that do not cause significant impedance changes. Interpretation requires experience, as soil effects and testing conditions can produce misleading signals. In Sri Lankan practice, PIT is widely used for bored piles in building foundations as a first-level integrity check, with suspect piles referred for more advanced testing such as CSL or TIP.

2.4.2 CROSS-HOLE SONIC LOGGING (CSL)

Cross-hole Sonic Logging is a higher-resolution NDT method, particularly suited for large-diameter bored piles and diaphragm walls. During pile construction, access tubes (typically four or more) are installed within the reinforcement cage and filled with water. A transmitter probe in one tube sends ultrasonic pulses to a receiver in an adjacent tube, while both probes are raised or lowered together along the pile length.

In sound concrete, the ultrasonic signal travels at a consistent velocity and amplitude. Defects such as honeycombing, voids, inclusions, or poorly compacted zones reduce wave velocity and signal amplitude. By analyzing signal loss or velocity reduction between tube pairs, engineers can identify both the presence and depth of defects. CSL has become an industry standard for high-rise buildings, bridges, and critical infrastructure where pile quality must be verified with a high degree of confidence.

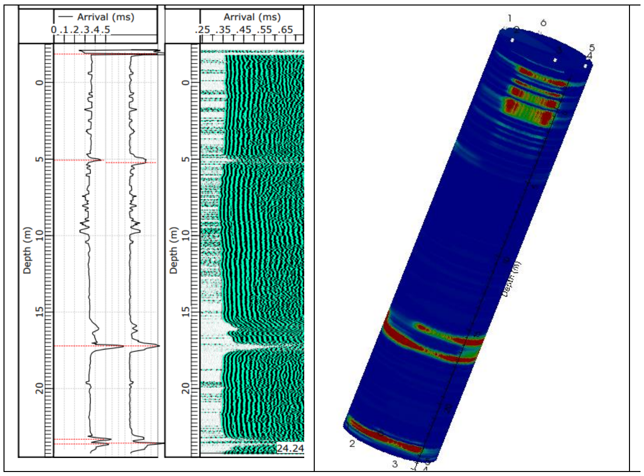

Fig 10. CSL and Tomography profiles of a defective bored pile.

The main advantages of CSL are its high sensitivity to defects and its ability to provide continuous profiles along the pile. The data can be used to Tomography analysis which provide 3D visualization of defective zones. Limitations include the requirement for access tubes (which must be properly installed and maintained during concreting) and the inability to provide information outside the cage area. In Sri Lankan projects, CSL has been used successfully to detect deep-seated defects in bored piles where PIT results were inconclusive, thereby preventing premature acceptance of defective foundations.

2.4.3 THERMAL INTEGRITY PROFILING (TIP)

Thermal Integrity Profiling (TIP) is a more recent innovation in pile integrity testing, based on monitoring the heat generated during cement hydration. Temperature sensors, typically in the form of thermal wires are attached to the reinforcement cage before concreting. As the concrete cures, hydration generates heat, and the distribution of this heat is monitored over time.

In a uniform, defect-free pile, the temperature profile is relatively smooth and consistent. Anomalies such as necking, bulging, inclusions, or variations in cross-section alter the local heat signature. For example, a necked pile section shows reduced heat because of smaller concrete volume, while a bulged section shows elevated temperatures due to increased volume. Unlike CSL, TIP does not require access tubes, and it provides a full cross-sectional view of the pile, including areas outside the reinforcement cage. Thermal probing is also possible by inserting an infrared probe through dry CSL tubes.

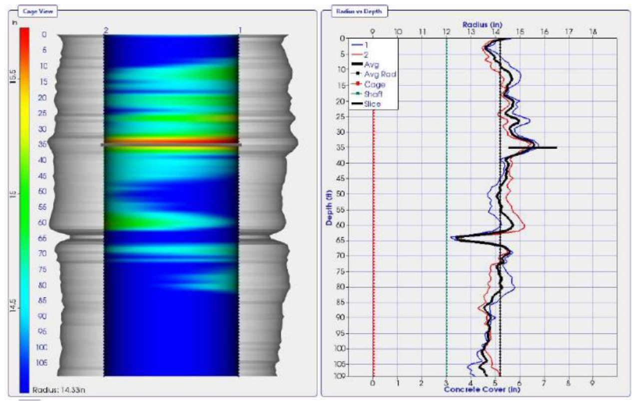

Fig 11. Temperature variation along the pile shaft

TIP has proven particularly effective for very large-diameter bored piles and secant pile walls where conventional PIT resolution is poor and CSL is limited by tube spacing. It also has the advantage of early defect detection, since data can be obtained within a few days of concreting, long before the pile is loaded. Its main limitation is higher initial cost and the need for sensor installation before concreting. However, for critical projects, the added assurance and diagnostic capability often outweigh the investment. In Sri Lanka, TIP is rarely used because of high initial cost involvement, compared with other NDT methods. However, its ability to provide full-volume information has been especially valuable in cases where conventional testing methods offered only partial insights.

3.0 OPTIMIZATION OF PILE FOUNDATIONS

Optimization of pile foundations has become increasingly critical in modern geotechnical practice, where engineers must balance safety, economy, and sustainability. Traditionally, pile design has relied on conservative assumptions, with large global safety factors applied to account for uncertainties in soil parameters and construction quality. While this approach ensures structural safety, it often results in excessive rock socketing, large-diameter piles, or large numbers of piles, leading to inflated costs and unnecessary material consumption. Optimization addresses this by using data-driven and performance-based design to refine foundation solutions without compromising reliability.

3.1 DESIGN OPTIMIZATION

Design optimization involves rationalizing safety factors and adopting reliability-based methods. Instead of applying blanket global safety margins, modern practice utilizes partial factors for loads and resistances, consistent with Eurocode 7 and other international standards. Incorporating site-specific geotechnical variability into probabilistic models allows engineers to design foundations that achieve the desired reliability index without excessive conservatism. Furthermore, soil–pile–structure interaction analysis, using p–y and t–z curves or finite element modeling, provides more realistic predictions of settlement and group behavior, leading to more efficient foundation systems.

3.2 CONSTRUCTION OPTIMIZATION

Construction optimization focuses on selecting the appropriate pile type for site conditions and refining pile dimensions and installation procedures. Driven piles, bored piles, and other types (Micro, Mini or CFA) of piles each have distinct advantages depending on soil profile, logistics, and project scale. For example, driven piles offer faster installation and quality control for relatively light structures in non-residential areas, while bored piles are better suited to dense urban environments with variable subsurface conditions for high-rise buildings. Optimization also extends to termination criteria: rather than driving or drilling piles to arbitrary depths, energy monitoring and dynamic testing can confirm that sufficient resistance has been mobilized, often reducing unnecessary penetration. Similarly, reinforcement and concrete volumes can be minimized by adopting performance-based design standards rather than prescriptive detailing.

3.3 TESTING FOR OPTIMIZATION

Testing plays a pivotal role in optimization. Preliminary test piles provide vital information on actual soil–pile performance, enabling validation of design assumptions, contractors' workmanship and the capability of machinery and tools. Load–settlement data from static load tests, interpreted through instrumented piles or combined with dynamic testing, allows engineers to refine working pile designs with greater accuracy. In addition, compiling and analyzing databases of past pile test results offers predictive insights, enabling more efficient design for future projects in similar soil conditions.

Finally, sustainability is an increasingly important driver of optimization. Reducing embodied carbon in concrete and steel, minimizing excavation spoil, and exploring reuse of piles in redevelopment projects all contribute to more sustainable foundation practices. In Sri Lanka, where infrastructure expansion is rapid and material costs are rising, optimization offers not only cost savings but also long-term environmental benefits.

4.0 CASE STUDIES

The following two cases provide real evidence of the value of rigorous QA/QC and optimization in pile foundation practice.

4.1 QA CASE STUDY – FAILURE AVOIDED

In one Colombo high-rise project, routine Low Strain Pile Integrity Testing (PIT) identified irregular reflections in a significant number of piles. Subsequent Dynamic Load Testing (PDA) on all the piles confirmed anomalies in the pile shaft caused by poor slurry control and segregation during tremie concreting and inadequate base cleaning. Early detection allowed the contractor to rectify the defective piles. However, rectification was not successful because of the presence of multiple defects along shaft and at toe regions. Finally, supplementary piles were installed for all the piles that poorly performed during the PDA tests, preventing a potentially catastrophic failure. Although this caused a significant delay and additional cost, the proactive QA approach saved the project from much larger remedial costs that would have arisen if defects were discovered after superstructure construction.

4.1.2 GEOTECHNICAL DESIGN PARAMETERS

10 nos of borehole investigations had been carried out about 17,500m^2 land area to cover building footprint coverage around 7,500m^2. The rock level was encountered between 16m to 23m depths and a minimum of one meter coring was done for each borehole to identify rock properties.

Table 1 Details of the bore holes and rock strengths

BH No	Depth (m)	CR (%)	RQD (%)	UCS (Mpa)
BH 01	16.0-17.0	100	100	21.94
BH 02	19.8-20.8	00	00	26.95
	20.8-21.8	90	65	20.51
BH 03	16.0-17.0	100	90	29.98
BH 04	17.5-18.5	75	00	33.18
	18.5-19.5	100	75	21.94
BH 05	19.8-20.8	100	90	26.95
BH 06	19.5-20.5	00	00	-
	20.5-21.5	40	00	-
	21.5-22.5	100	58	24.88
BH 07	22.5-23.5	100	37	20.13
BH 08	21.3-22.3	00	00	-
	22.3-23.3	95	42	15.07
BH 09	18.0-19.0	00	00	-
	19.0-20.0	100	62	13.18
BH 10	19.3-20.3	90	50	11.83

Table. 2 Recommended ultimate skin frictions for Zone A for each soil layer

Layer No.	Layer description	Average SPT	f_u (kN/m^2)
2	Soft Peat	0-8	-
3a	Residual Soils-I	15-20	22
3b(i)	Residual Soils-II (i)	27-43	45
3b(ii)	Residual Soils-II (ii)	>50	65
4a	Completely Weathered Rock-I	34-38	70
4b	Completely Weathered Rock-II	>50	100
5	Basement Rock	-	200

Table. 3 Shear strength parameter and ultimate bearing capacities

Layer No.	Layer Description	Shear strength parameters	Ultimate bearing capacity (kN/m^2)	Elastic Modulus E (kN/m^2)
2	Soft Peat	-	<100	1000
3a	Residual Soils-I	c' = 6 kPa, φ' = 26	270	8500
3b(i)	Residual Soils-II (i)	c' = 8 kPa, φ' = 29	420	15000
3b(ii)	Residual Soils-II (ii)	c' = 10 kPa, φ' = 35	1000	25000
4a	Completely Weathered Rock-I	c' = 10 kPa, φ' = 30	530	20000
4b	Completely Weathered Rock-II	c' = 10 kPa, φ' = 38	1500	25000

Recommended net allowable end bearing capacity of 2.0-3.0 MN/m^2.

Based on the above recommendations, the initial foundation design was done, and the design team was in the opinion that the original design could be optimized based on the results of a preliminary test pile.

Table. 4 Allowable pile capacities based on GI report recommendations

Pile Diameter (mm)	End Bearing (kN)	Skin Friction (kN)	Allowable Capacity (kN)
600	707	1,358	2,065
750	1,105	1,697	2,802
800	1,257	1,810	3,067
900	1,591	2,037	3,628
1000	1,964	2,263	4,227
1200	2,829	2,715	5,544
1500	4,420	3,394	7,814

4.1.3 PRELIMINARY TEST PILE.

At the most critical location of the project, a 1000 mm diameter test pile was installed near borehole BH 10, in accordance with the recommendations of the geotechnical expert. The pile was drilled to a depth of 23.80 m, with a 1.0 m socket into the underlying rock. Construction followed the anticipated quality control procedures and used the same materials and machinery specified for the working piles, ensuring the test conditions were fully representative

A combination of low-strain integrity testing (PIT) and high-strain dynamic testing (PDA) was performed on the test pile to evaluate both its integrity and its ultimate load-carrying capacity.

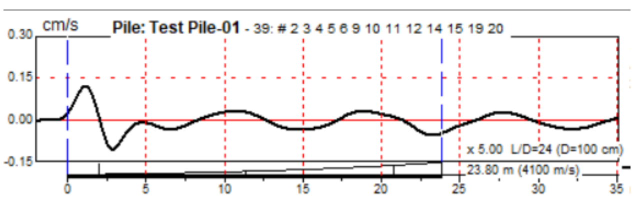

Fig. 11 PIT velocity records of test pile

The PIT records indicated minor impedance increases at depths of approximately 6.6 m and 15.0 m, but these were not significant enough to compromise structural integrity. Overall, the pile was classified as sound, and no geotechnical failure was observed.

Table. 5 Summary PDA test of the preliminary Test pile

Description	Values
Mobilized static pile capacity (KN)	15,156
Mobilized Skin Resistance (kN)	3,889
Mobilized Toe Resistance (kN)	11,267
Pile integrity (Min Beta Values in PDA)	(100) OK
Gross Settlement at working load (mm)	6.30
Gross Settlement at test load (mm)	12.50
Gross Settlement at mobilized load (mm)	21.70
Ultimate shaft friction up to 12m depth (negative skin friction) kN	708
Effective Skin Friction (kN)	3,181
Allowable Skin Friction (FOS is 2.5 for skin friction) kN	1,272
Mobilized End Bearing Resistance (kPa)	14,345
Allowable End Bearing Resistance (FOS is 2.00 for End Bearing)(kPa)	7,172
Net allowable capacity (Considering effective skin friction) (kPa)	8,444

PDA testing confirmed that the ultimate pile capacity was governed by the structural strength of the pile itself, rather than geotechnical failure. On this basis, a net allowable bearing pressure of 7,500kPa was recommended. By revising the design using actual performance data, the design team was able to optimize the foundation scheme, reducing average pile size by more than 50%. This modification resulted major savings in concrete and reinforcement, reduced construction time, and a substantial overall cost benefit to the project.

Table.6 Revised pile design capacities

Pile Diameter (mm)	Allowable Capacity as GI Recommendation (kN)	Revised Design Load (kN)
600	2,065	1,075
900	3,628	4,617
1000	4,227	5,757
1200	5,544	7,967
1500	7,814	13,245

4.1.4 FAILURE OF WORKING PILES

Following the installation of the full set of 176 working piles, integrity assessment was carried out using the PIT method. To the project team's concern, a significant proportion of piles exhibited abnormal wave reflections indicative of potential structural defects. The suspected piles were categorized into groups based on the severity of anomalies observed in the PIT records. Representative piles from each group were then selected for PDA testing as an initial verification step. The PDA results were alarming: none of the tested piles demonstrated the ability to safely sustain the specified design load.

As a remedial measure, several defective piles were cored and subjected to pressure grouting in an attempt to restore integrity. However, the rectification process was complicated by the presence of multiple, distributed defects, and the improvement achieved was limited. Even after grouting, repeat PDA testing showed that pile capacities remained below the required capacity level. Recognizing that continued grouting was unlikely to deliver the desired performance, the decision was made to suspend further remedial work.

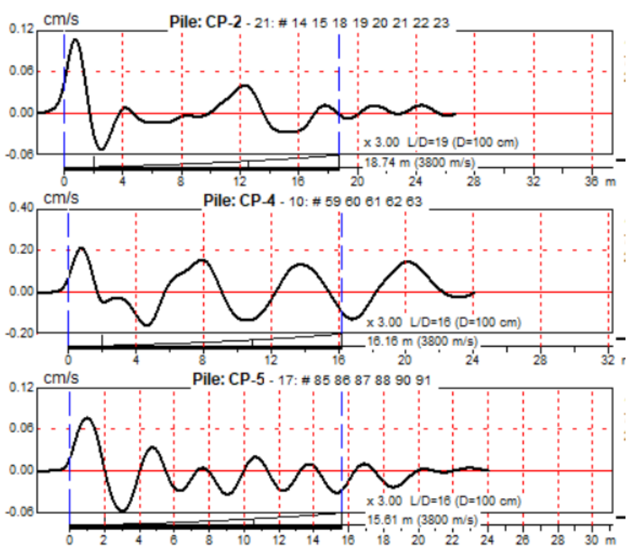

Fig. 13 PIT Velocity records of some suspected piles.

Subsequently, comprehensive PDA testing was performed on all piles flagged by PIT as suspect and the piles had poor workmanship concerns during construction. The results confirmed widespread deficiencies: more than 70% of the piles failed to meet the required capacity. Given the scale of defects and the inadequacy of remedial options, the only viable solution was to construct additional replacement piles to ensure the safety and stability of the foundation system.

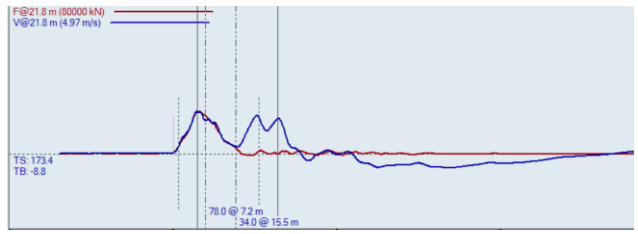

Fig. 14 PDA output showing a toe defect

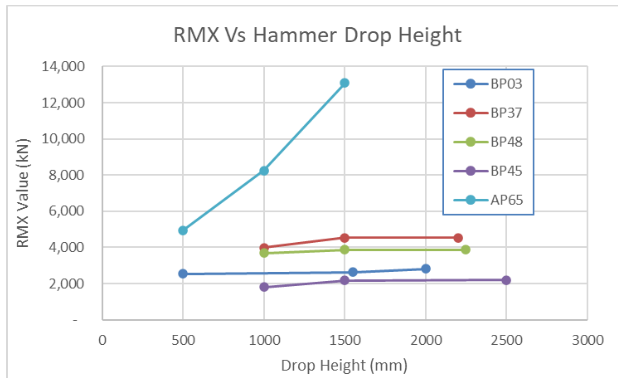

Fig. 15. Variation RMX values with the increase of hammer energy.

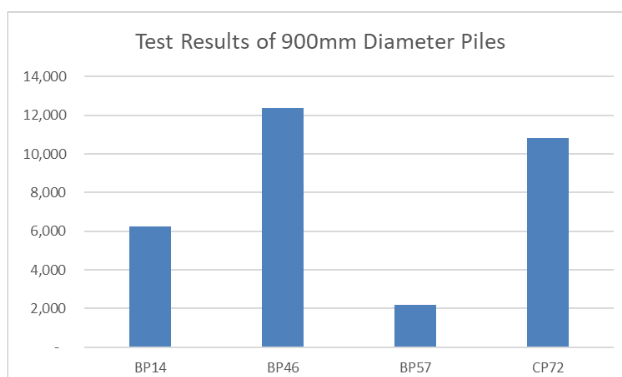

Fig. 13 Mobilyzed capacity of 900mm diameter piles

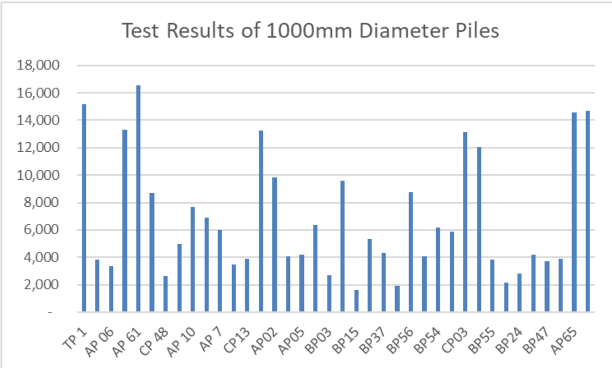

Fig. 16 Mobilyzed capacity of 1000mm diameter piles

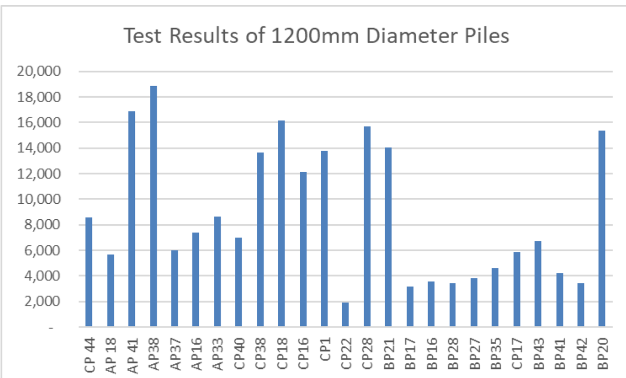

Fig. 17 Mobilyzed capacity of 1200mm diameter piles

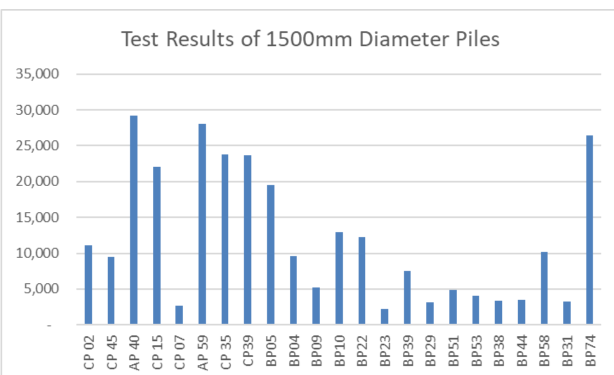

Fig. 18 Mobilyzed capacity of 1500mm diameter piles

Total 86 nos piles were selected for PDA testing and 59 nos piles could not meet the required threshold limits mainly because of insufficient end resistance and high pile top movement. All 93 nos PDA test have carried out with eight repeated test. Additional piles were installed for all unsatisfactorily performed piles.

This case study illustrates the critical role of systematic QA/QC in preventing foundation failure. Early reliance on PIT and PDA testing exposed serious deficiencies that would otherwise have gone undetected until structural distress occurred. By identifying and addressing these issues before superstructure loading, the project team avoided a potential foundation failure and its associated economic and safety consequences.

4.2 OPTIMIZATION CASE STUDY – COST SAVINGS

In contrast, a 27-story luxury apartment building project in Kalaniya demonstrated the benefits of optimization through testing. Initial designs specified piles with conservative lengths based on desk-study soil data. A series of instrumented static load tests and dynamic load tests showed that ultimate capacity was mobilized at shorter embedment lengths than originally specified in the soil investigation report. By calibrating design values with actual performance, the design team was able to reduce average pile rock socketing lengths and pile sizes by 10% and 60% respectively. This adjustment resulted in significant savings in concrete and reinforcement, heavy machinery involvement as well as shortened construction time, all while maintaining required safety margins.

4.2.1 THE GROUND CONDITION AND ORIGINAL FOUNDATION DESIGN PARAMETERS.

The site was located at Thorana Junction, Kalaniya, bordering to Kandy -Colombo Road. The land extent is around 30,000m^2 and three building blocks spread covering around 25 of the land area. The land is relatively flat and in low-lying very close to Kalani River. Fourteen (14) number of boreholes with SPT test, rock coring, and standard laboratory tests had been carried out. It was revealed that very soft organic peat is present up to 9-12m depth and thereafter a clayey sand layers followed by 2.4m thick highly weathered rock layers before the moderately weathered Biotite Gneiss rock.

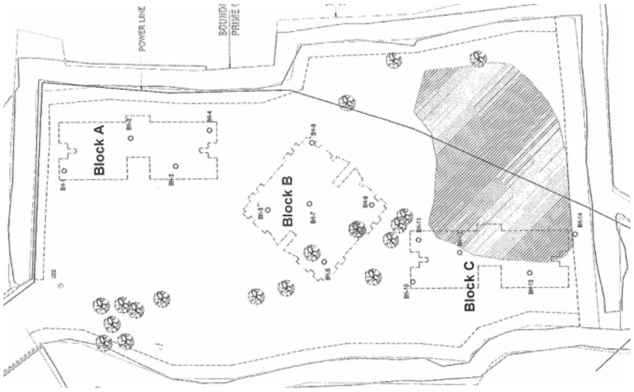

Fig 19. Borehole location layout

Table 7 Rock coring information

BH No.	Depth (m)	CR %	RQD %	UCS (Mpa)
BH-01	20.6-22.1	91	41	19.54
	22.1-24.1	97	83	9.96
BH-02	21.1-22.3	76	33	11.71
	22.3-24.1	56	20	-
	24.1-25.1	90	44	8.81
	25.1-26.1	82	46	-
BH-03	21.3-22.3	85	00	-
	22.3-24.3	76	34	5.88
	24.3-26.3	99	62	9.53
BH-04	21.3-24.3	86	36	6.80
	24.3-26.3	100	20	20.86
	26.3-27.8	100	63	-
BH-05	18.0-19.5	80	15	10.32
	19.5-21.3	97	70	
	21.3-23.3	35	08	
	23.3-24.8	88	41	10.89
	24.8-26.6	97	81	
BH-06	19.6-21.25	97	27	3.77
	21.25-23.25	100	65	6.75
BH-07	18.0-20.0	92	42	8.96
	20.0-21.0	100	40	10.27
BH-08	19.3-21.3	100	71	2.15
	21.3-22.4	82	20	7.78
	22.4-24.4	81	45	-
	24.4-25.4	97	91	-
BH-09	18.3-19.35	90	33	-
	19.35-20.85	93	37	6.80
	20.85-21.8	100	89	-
	21.8-23.8	98	88	14.34
BH-10	19.5-22.1	75	17	6.21
	22.1-23.6	100	50	-
	23.6-24.2	100	92	12.56
BH-11	18.7-19.7	97	42	8.03
	19.7-21.2	93	70	-
	21.2-22.2	75	30	-
	22.2-23.3	100	89	19.09
BH-12	18.5-20.1	63	16	13.57
	20.1-21.7	38	00	-
	21.7-23.2	26	00	-
	23.2-24.2	100	37	15.57
BH-13	22.1-24.1	42	11	-
	24.1-25.1	75	14	-
	25.1-26.3	78	12	-
	26.3-27.3	99	31	6.00
	27.3-28.6	97	68	10.48
BH-14	19.6-21.6	46	07	-
	21.6-22.6	70	00	-
	22.6-24.1	60	00	-
	24.1-25.0	67	00	-
	25.0-26.5	58	27	12.57
	26.5-27.9	99	53	4.59

Table 8. Geotechnical design parameters of sub-surface layers at Block A

Layer No.	Layer Description	Avg. SPT No.	Shear strength parameters	Elastic Modulus E (kN/m²)
2a	Very Soft Organic Clay	1	—	1000
2b	Very Soft Clay	1	—	1000
3a (i)	Very Loose Clayey Sand	3	c' = 0 kPa, φ' = 23	3000
3a (ii)	Soft Clay	2	c' = 2 kPa, φ' = 22	2000
3b (i)	Loose Clayey Sand	9	c' = 0 kPa, φ' = 25	7500
3b (ii)	Firm Lateritic Clay	—	c' = 6 kPa, φ' = 25	7000
3b (iii)	Very Stiff Lateritic Clay	20	c' = 8 kPa, φ' = 29	15000
3c	Very Dense Clayey Sand	>50	c' = 0 kPa, φ' = 35	25000

4a (i)	Firm to Stiff Clay of CWR-I	8	c' = 6 kPa, φ' = 28	7000
4a(ii)	Stiff to Very Stiff Clay of CWR-I	15	c' = 8 kPa, φ' = 29	13000
4a (ii)	Very Stiff Clay of CWR-I	20	c' = 8 kPa, φ' = 29	15000
4b	Very Dense Clayey Sand of CWR-II	>50	c' = 5 kPa, φ' = 38	25000

Table 9 Ultimate skin friction coefficient (fu) at Block A

Layer No.	Layer description	Average SPT	fu (kN/m²)
3a(i)	Very Loose Clayey Sand	3	—
3a(ii)	Soft Clay	2	—
3b(i)	Loose Clayey Sand	9	—
3b(ii)	Firm Lateritic Clay	8	—
3b(iii)	Very Stiff Lateritic Clay	20	26
3c	Very Dense Clayey Sand	>50	65
4a(i)	Firm to Stiff Clay of CWR-I	8	20
4a(i)	Stiff to Very Stiff Clay of CWR-I	15	30
4a(ii)	Very Stiff Clay of CWR-I	20	40
4b	Very Dense Clayey Sand of CWR-II	>50	100
5	Basement Rock	—	200

The initial foundation design was done based on the parameters and recommendations for bearing capacities for soil and rock layers.

Table. 10 Allowable pile capacities based on GI report recommendations.

Pile Diameter (mm)	End Bearing (kN)	Skin Friction (kN)	Total Capacity (kN)
1500	4,420	1,414	5,834
1200	2,829	1,131	3,960
1000	1,964	943	2,907
900	1,591	849	2,440
800	1,257	754	2,011
750	1,105	707	1,812
600	707	566	1,273

It soon became evident that the overall cost of the foundation was exceeding the allocated budget, prompting the need to explore alternative foundation solutions. On the advice of the design team, a comprehensive pile testing program was initiated. This program was carefully planned after a detailed review of the borehole investigation data and information by the team's prior experience with similar ground conditions and projects.

4.2.2 PILE TESTING PROGRAM

It was proposed to implement a comprehensive testing program consisting of one instrumented static pile load test (IMLT), one conventional static pile load test (MLT) carried out up to a predetermined load, and high-strain dynamic pile load testing (PDA) on both test piles, each loaded to three times the anticipated design load. A 1000 mm diameter pile was selected as the representative test pile size, consistent with the final pile arrangement envisaged for the foundation system. The test location was strategically chosen near boreholes BH 3 and around BH 8 and BH 9, as this zone was considered representative of the overall site conditions and would also support the tallest tower within the development.

Table. 11 Details of the selected piles for testing

Pile Details	Test Pile 1	Test Pile 2
Pile ID	P3 (02)	P3 (01)
Location	BH 3	BH 5
Pile Diameter (mm)	1,000	1,000
Depth to Weathered (m)	18.50	18.50
Depth to sound rock (m)	19.00	24.50
Terminated Depth (m)	25.09	29.00
Service Load (kN)	6,400.00	6,400.00
Test Load (kN)	9,600.00	9,600.00
Test Type	MLT, PDA	IMLT, PDA

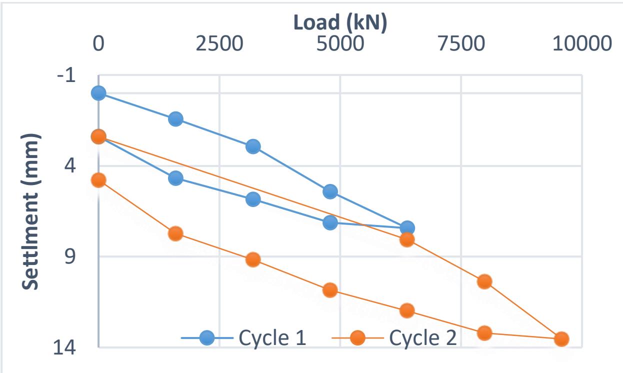

Fig 20. Load settlement curves of TP 01 (ID 02)

Table. 12 Results of conventional static load test

Applied Load		Average Settlement (mm)	
(kN)	(% of WL)	Gross Settlement	Net Settlement
6,390	100	7.43	2.39
9,585	150	13.53	4.79

Table. 13 Results of IMLT- Skin Friction

Depth (m)		Load Transfer Zones	Mobilized Unit Skin Friction (kN/m^2)	
From	To		Loading Cycle 1	Loading Cycle 2
2.00	6.00	L1-L2	71.57	80.10
6.00	14.00	L2-L3	8.94	61.97
14.00	24.50	L3-L4	17.07	18.29
24.50	26.40	L4-L5	235.22	244.48
26.40	27.40	L5-L6	105.04	154.73
27.40	28.40	L6-L7	76.97	148.42
28.40	29.40	L7-L8	88.08	116.58

Table 14 Results of IMLT-End Bearing

Depth (m)		Load Transfer Zones	Mobilized Unit End Resistance (kN/m^2)	
From	To		Loading Cycle 1	Loading Cycle 2
29.40	Toe	Pile Tip	3,129.17	4,648.31

Table 15 Pile top movement

Applied Load		Average Settlement (mm)	
(kN)	(% WL)	Gross Settlement (mm)	Net Settlement (mm)
6,400	100	6.61	2.66
9,600	150	11.62	4.25

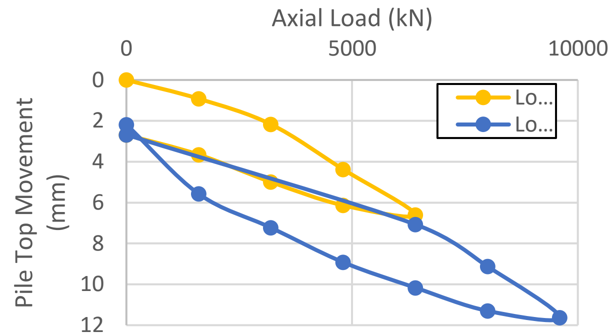

Fig 21. Load settlement curves of TP 02 (ID 01)

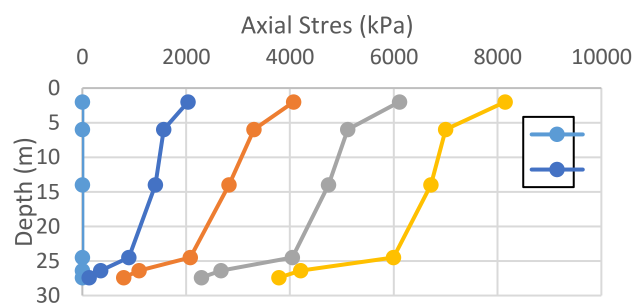

Fig 22. Variation of stress along the pile shaft

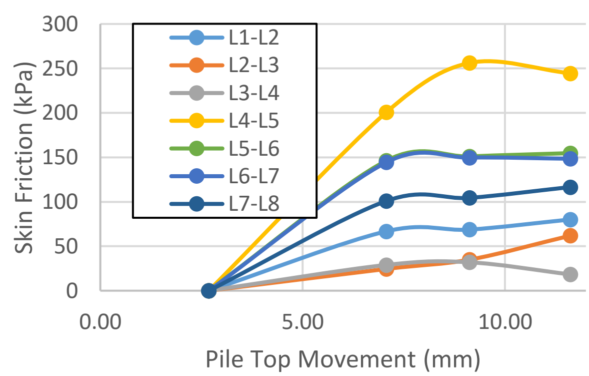

Fig 23. Variation of stress along the pile shaft

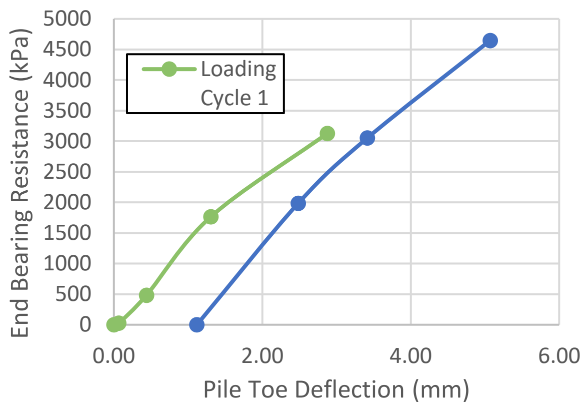

Fig 24 Pile toe movement with load

Table. 16 PDA Field Resus

Pile No	Blow No	Drop Height (mm)	CSX (Mpa)	RMX (kN)	Observed Set (mm)
02 (P3) TP 01	1	500	5.00	5,100	<1.0
	2	1000	9.20	9,639	<1.0
	3	1000	10.30	10,385	<1.0
	4	1500	12.30	12,808	<1.0
	5	1750	15.70	14,286	<1.0
01 (P3) TP 02	1	500	2.40	2,566	<1.0
	2	1000	8.20	9,070	<1.0
	3	1500	11.00	12,652	<1.0
	4	1750	12.80	13,391	<1.0

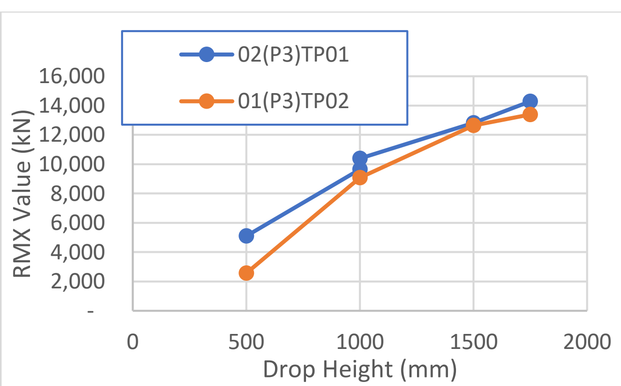

Fig. 25 Variation of RMX values with drop heights

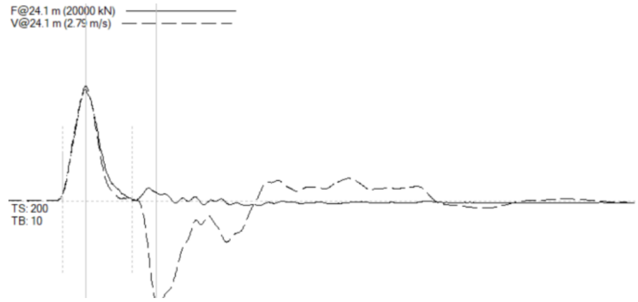

Fig. 26 PDA Force and velocity curves

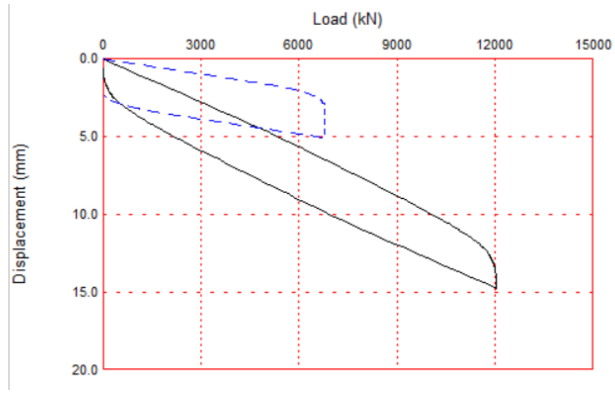

Fig. 27 Pile top and toe movement estimated by CAPWAP

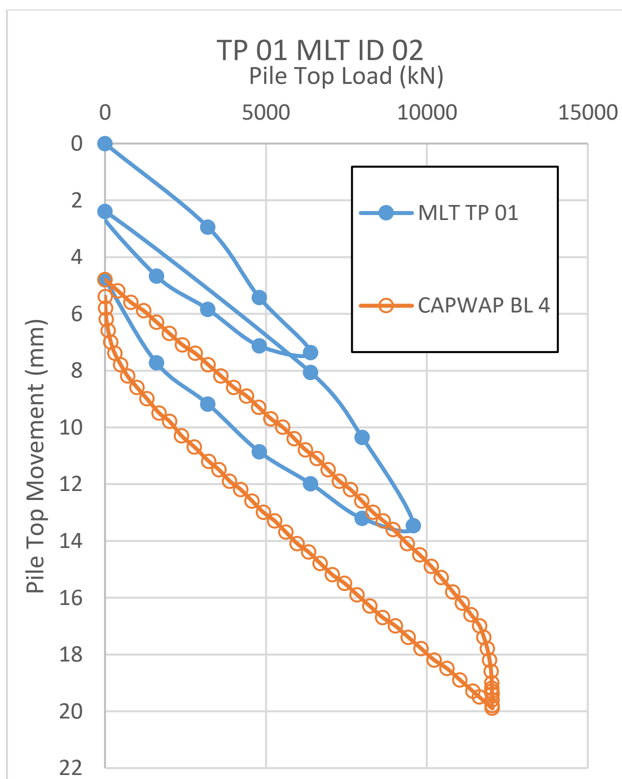

Fig. 28 Pile top movement MLT VS CAPWAP

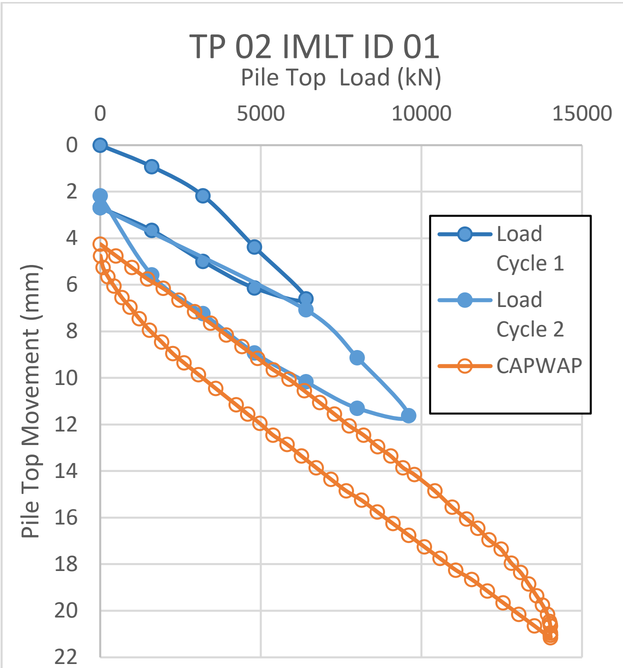

Fig. 29 Pile top movement IMLT VS CAPWAP

Table 17. Revised design loads and pile diameters

Pile Diameter (mm)	Allowable Capacity as GI Recommendation (kN)	Revised Design Load (kN)
1000	2,907	6,400
1200	3,960	10,000
1500	5,834	17,000

5.0 DISCUSSION

The Sri Lankan construction industry operates in a context of rising material costs, increasing urban density, and demand for rapid project delivery. Within this environment, the importance of combining QA/QC with optimization cannot be overstated. QA ensures that design assumptions are achieved in practice, while optimization prevents waste of resources and enables more sustainable construction.

Despite the availability of advanced testing methods such as PDA analysis, instrumented static load testing, CSL and Thermal Integrity Profiling, their adoption in Sri Lankan projects has been inconsistent. Many projects continue to rely solely on only conventional static tests, which, although reliable, are expensive and impractical for large-scale foundations. At the same time, optimization is sometimes resisted due to lack of familiarity with reliability-based design approaches or a preference for conservative specifications. This conservative approach often results in over-designed foundations that unnecessarily inflate costs.

The discussion also extends to sustainability. With Sri Lanka’s growing emphasis on green building practices, optimized pile foundations offer a path to reduce embodied carbon and material usage. Encouragingly, several landmark projects in Colombo have already begun adopting IMLT and advanced QA/QC tools, demonstrating a gradual shift towards international best practice. However, challenges remain in terms of training, specification updates, and convincing stakeholders of the long-term benefits of these methods.

6.0 CONCLUSIONS

This paper has highlighted the critical roles of quality assurance and optimization in pile foundation engineering. Quality assurance ensures that piles are constructed as designed, safeguarding against structural failures and costly remedial works. Optimization, by contrast, seeks to refine design and construction practices to achieve safety, economy, and sustainability simultaneously. When applied together, QA and optimization form complementary pillars of a robust foundation engineering strategy.

Key QA/QC methods. including instrumented static load testing and dynamic load testing, advanced CAPWAP signal analysis, and non-destructive methods such as PIT, CSL, and TIP — provide engineers with powerful tools to detect defects, verify design assumptions, and ensure the long-term reliability of foundations. Instrumented static load tests in particular offer valuable insights into load transfer mechanisms, supporting more rational design. Case histories demonstrate how QA/QC has prevented failures, while optimization has delivered substantial cost and time savings in Sri Lankan projects.

Looking forward, the Sri Lankan geotechnical community must continue to move toward performance-based design and adopt international best practices in testing and optimization. Building a national database of pile load test results would further reduce reliance on conservative assumptions and support data-driven design. In addition, sustainability considerations — reducing embodied carbon and reusing piles in redevelopment — should be embedded into foundation engineering practice.

Ultimately, the integration of rigorous QA/QC and rational optimization will not only enhance the safety and economy of pile foundations but also contribute to a more sustainable and competitive construction industry in Sri Lanka.

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Challenges encountered during piling through cavernous and weak rock formations

By

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Prof. L. I. N. de Silva joined the Department of Civil Engineering in 2009 as a senior lecturer Grade II. He got his bachelor's degree from the University of Moratuwa in 2001 and obtained his masters and PhD degrees from the University of Tokyo in 2004 and 2008, respectively. He has over 17 years' post-doctoral experience as a researcher and a consultant in geotechnical engineering and published over 65 research papers and three book chapters based on his work related to various aspects of geotechnical engineering.

Further to his scholastic work, Prof. de Silva has contributed significantly to knowledge dissemination by conducting well-attended public lectures and workshops at the Institution of Engineers Sri Lanka, Sri Lankan Geotechnical Society, Society of Structural Engineers Sri Lanka and various government, semi-government and private organizations in Sri Lanka.

Prof. de Silva is well known by the construction industry as a geotechnical expert, and he has been providing his services as a geotechnical consultant for major construction projects in Sri Lanka and overseas, including Central Expressway, Ruwanpura Expressway and Lotus Tower projects.

Challenges Encountered During Piling in Cavernous and Weak Rock Formations

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ABSTRACT: Piling through cavernous and weak rock formations poses considerable challenges due to sinkholes, socketing failure, loss of drilling fluid, and concrete placement difficulties. This paper presents two Sri Lankan case studies: Kandy, underlain by marble and calc-gneiss with large cavities, and Kelaniya, characterized by weak rock with vertical fractures. Investigations in Kandy included over 30 boreholes, seismic refraction surveys, and load testing (IMLT, High strain dynamic load tests (PDA), MLT). Investigations in Kelaniya included 14 boreholes and load testing (IMLT, PDA, MLT). The results indicate mobilized skin friction values of 284–344 kPa and end bearing capacities of 8–12 MPa at Kandy, while Kelaniya piles achieved 8600–9930 kPa mobilized end bearing despite low UCS values. Pile top displacement vs axial load curves obtained from IMLT, MLT reasonably agree with that obtained through CAPWAP analysis of PDA tests. Site-specific correlations between UCS, point load index, and penetration rate were established to develop site specific pile termination criteria. Conservative design values (f_u < 400 kN/m², q_{all} < 4000 kN/m²) are recommended based on the test results in Kandy. The study concludes that careful geotechnical investigation and pile load testing, real-time monitoring of mobilized capacity and appropriate quality assurance measures are essential in constructing pile foundations in cavernous and weak rocks.

1 INTRODUCTION

Pile foundations are often required to transfer structural loads to competent strata, where subsurface geology is highly variable. Cavernous and weak rocks, including marble, limestone, and gneiss formations, present unpredictable challenges such as sudden ground collapse, loss of drilling fluid, concrete placement issues, drill deviations, equipment damage and inadequate socketing. These conditions increase costs and risks, necessitating specialized investigations and testing. This paper summarizes the test results and challenges encountered during piling in cavernous and weak rocks, highlighting lessons from case studies in Kandy and Kelaniya.

2 GEOLOGICAL CONTEXT

Sri Lanka's geology includes widespread occurrences of marble, carbonatites, and limestones that are prone to karstification. At the Kandy site, marble, calc-gneiss, and biotite gneiss dominate, while the Kelaniya site consists of basement rock with relatively high CR and RQD with vertical fractures. Subsurface variability in Kandy includes cavities, boulders, sloping bedrock, and fractured weak rock zones, creating high uncertainty for pile design and construction.

3 GEOTECHNICAL INVESTIGATIONS

At the Kandy site, over 30 boreholes were drilled, supplemented by a Seismic Refraction Survey (SRS) conducted along eight lines. Results revealed highly variable bedrock depths, steep bed rock slopes, core losses, and solution cavities (refer to Fig. 1 and Fig, 2). Core recovery (CR), Rock Quality Designation (RQD), and Unconfined Compressive Strength (UCS) and point load index ($I_{s(50)}$)values varied widely, with significant weak zones. The SRS identified cavity zones and boulder terrains, confirmed by borehole records.

Fig. 1 Photo of a core box showing significant core losses in Kandy.

Fig. 2 presence of solution cavities in marble rock in Kandy.

Geotechnical investigations in Kelaniya includes drilling 14 number of boreholes. Almost all the boreholes encounter rock with vertical fractures. However, the CR, RQD values of rock is high (CR = 85%-99% and RQD = Nil- 62%). The presence of vertical fractures yields very low UCS values (UCS = 5.9-9.5 MPa) indicating very weak rock.

Fig. 3 Presence of vertical fractures in Kelaniya,

4 CHALLENGES DURING PILING AND CASE STUDIES

4.1 Kandy Site

A 1.0 m diameter test pile was subjected to an instrumented maintained load test (IMLT) and high strain dynamic load testing (PDA) both. Load-settlement behavior showed good agreement between IMLT and PDA as shown in Fig. 4. CAPWAP analysis was employed in establishing the Load settlement behavior during PDA testing. In addition to the above IMLT and PDA test, three other test piles have been subjected to PDA testing. The results of IMLT and PDA testing confirmed mobilized average unit skin friction of 284–344 kPa and end bearing of 8072–12609 kPa through rock.

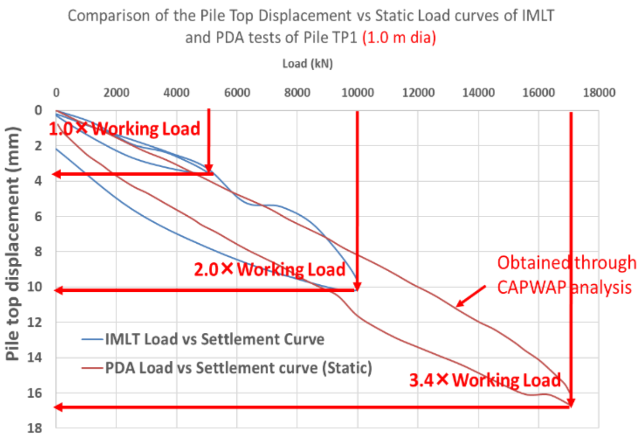

Fig. 4 Comparison of load settlement behavior of IMLT and PDA tests.in Kandy,

A detailed comparison of mobilized unit skin friction values of each soil/rock layer during the IMLT and PDA testing of the test pile is presented in Fig.5. It can be observed from the mobilized skin friction versus vs pile top displacement curves that almost the ultimate values are mobilized during the IMLT. A very similar skin friction values are mobilized during both IMLT and PDA testing. A maximum unit skin friction value of about 608 kPa has been mobilized in slightly weathered rock with CR > 90%, RQD > 80% and UCS = 16.6-18.4 MPa.

Depth (m)		Instrumentation zones	Layer description	Approximate SPT N/CR,RQD of rock	Mobilized skin friction (as per IMLT) (kN/m²)	Mobilized skin friction (as per PDA) (kN/m²)	Remarks
from	to						
2	10	L1-L2	Dark Brown Sandy Clay	18-25	45	30-51	Almost the ultimate
10	13	L2-L3	Dark Brown Sandy Clay	<10	13.5	14.8	Almost the ultimate
13	19.4	L3-L4	Dark Brown Sandy Clay	30->50	95	98 - 279	Not yet the ultimate as per the IMLT. However, 100 kN/m² is considered as the ultimate
19.4	21.4	L4-L5	Moderately Weathered fractured basement rock (MARBLE)	CR=50%-65% RQD=35%-60%	290	295	Not yet the ultimate as per the IMLT. However, 300 kN/m² is considered as the ultimate
21.4	23.4	L5-L6	Highly to Moderately Weathered fractured basement rock (MARBLE)	CR < 40%	150	197	Almost the ultimate as per the IMLT
23.4	25.25	L6-L7	Slightly Weathered fractured basement rock (MARBLE)	CR>90% RQD>80%	600	608	Not yet the ultimate as per the IMLT but 600 kN/m² is considered as the ultimate

Fig. 5 A comparison of mobilized skin friction values during IMLT and PDA testing in Kandy.

At several locations, the presence of large cavities required pre-concreting the cavity with Grade 15 concrete and subsequent reboring. The concrete volume of each truck was used to measure the average pile diameter within the cavity zone during the placement of Grade 15 concrete.

The skin friction mobilized through concrete-concrete interface after reboring was estimated as 150 kPa using PDA testing. This value may be close to the allowable skin friction value since the pile top settlement recorded at the mobilized capacity during the PDA is just 11 mm.

Further to the above, a very steep bed rock slope (>75^0) was encountered at another pile location. A PDA test conducted on the above pile shows very low mobilization of skin friction (29-55 kPa) and end bearing (957 kPa) through sloping bed rock. However, these values may be less than the allowable values since the pile top settlement at the mobilized load during the PDA test is just 6 mm.

4.2 Kelaniya Site

Here, piles were socketed a minimum of 4.0 m into very weak rock (UCS 5.9–9.5 MPa) but with high CR and RQD values. Most of the rock specimens show vertical fractures. As result, very low UCS values have been reported. A 1.0 m diameter working pile was subjected to an IMLT and PDA test both. The load settlement behavior between the IMLT and the PDA is very similar as presented in Fig. 6.

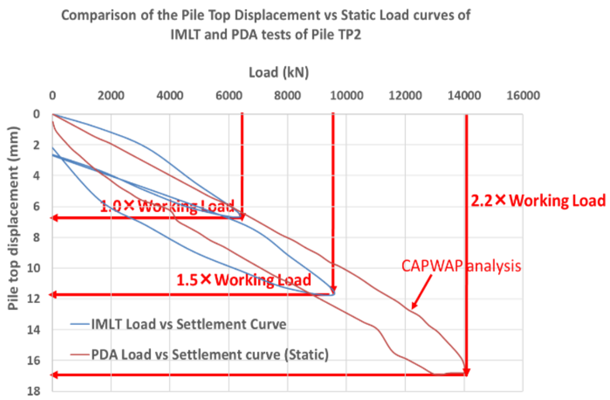

Fig. 6 Comparison of load settlement behavior of IMLT and PDA tests.in Kelaniya,

Despite vertical fractures, IMLT and PDA tests showed mobilized end bearing capacities of 4650–9930 kPa, respectively at 1.5 times and 2.2 times the working load. It can be observed from the mobilized skin friction versus vs pile top displacement curves that almost the ultimate values are mobilized during the IMLT. A relatively low skin friction values of 117-155 kPa has been mobilized through the rock socket. These low skin friction values may be attributed to the presence of vertical fractures in rock.

Another 1.0 m pile was subjected to a maintained load test (MLT) and a PDA test both. Similar load settlement behavior was observed between the MLT and PDA. The PDA test showed mobilized end bearing capacity of 8620 kPa at 1.9 times the working load.

Since high lateral confinement may be required for the mobilization of high end bearing values in rock with vertical fractures, a minimum socket length of 4D or 4.0 m whichever larger was adopted to ensure stability. With reference to the test results, an allowable end bearing of 4000 kPa has been adopted for slightly weathered to fresh rock in the pile design despite the very low UCS values.

5 QUALITY ASSURANCE AND DESIGN IMPLICATIONS

In Kandy site, a site specific correlation (i.e. UCS = K×$I_{s(50)}$) has been developed between the UCS and Point Load Index ($I_{s(50)}$) as shown in Fig. 7. The results of UCS and point load tests conducted on similar rock specimens during geotechnical investigations has been utilized in developing the correlation. It can be observed that the K value is about 10, which is significantly lower than the theoretical value of 24.

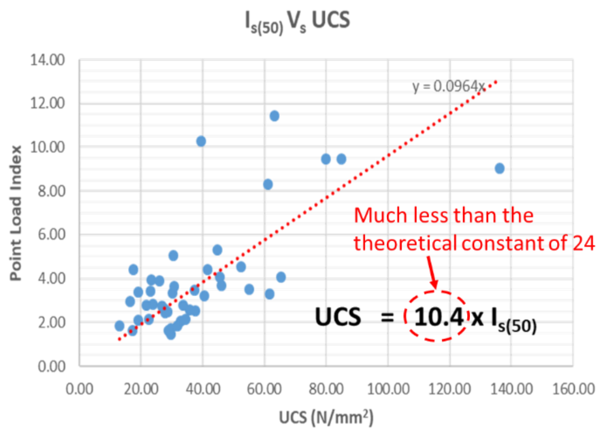

Fig. 7 A site specific correlation between $I_{s(50)}$ and UCS at Kandy project site.

Quality assurance during the construction of working piles in Kandy involved real-time monitoring of point load index ($I_{s(50)}$) (ASTM D 5731), penetration rate and the visual manual observation of rock. The measured average $I_{s(50)}$ (three rock specimens were tested for $I_{s(50)}$ and the average was taken) was converted to the UCS value using the site specific correlation. Then the following equations were used in the estimation of real-tile allowable skin friction and the allowable end bearing. Wyllie (1991) suggested that the use of Bentonite during drilling reduces the skin friction. Therefore, the ultimate skin friction is taken as 25% of the ultimate skin friction obtained from the following equation.

Allowable Unit Skin Friction (f_a) = $0.375x\,(UCS)^{0.515}/(4\times2)$ (Rosenberg and Journeaux,1976)

Allowable End Bearing Resistance (q_{all_l}) = 0.33 x UCS/3 (Kulhawy and Goodman, 1980)

A site-specific pile termination criterion was developed based on the above. Conservative limits were applied ($f_u \leq 400$ kN/m², $q_{all} \leq 4000$ kN/m²) considering the uncertainties in the subsurface conditions at the Kandy site. Drilling continues until 20% additional capacity is mobilized (to account for the uncertainties in the subsurface conditions at Kandy site). Further to the above, the following additional measures were also taken during the construction of working piles.

- Pile shall not be terminated at a depth where a bentonite loss is observed (unless it is designed as a friction pile).
- No bentonite loss shall be observed during and after toe cleaning (if the end bearing is considered in the design).
- In general, a minimum of 7.50 m continuous rock socket shall be maintained.

More than 15 PDA tests have been conducted on working piles in Kandy. Mobilized unit skin friction values obtained from the CAPWAP analysis of PDA tests and pile drilling records such as the Point Load Index ($I_{s(50)}$) and the Penetration rate (PR) have been analyzed to identify any correlations as presented in Fig. 8 and Fig. 9 below.

It can be from Fig. 8 that more than 100 kN/m² unit skin friction is mobilized when $I_{s(50)} > 1$. If the pile top settlement is less than 10 mm, the mobilized skin friction values were considered as less than allowable values. Further, the allowable skin friction values estimated from the measured $I_{s(50)}$ values as per the proposed pile termination criteria is also presented in the same Figure. However, no clear correlation could be observed between the mobilized unit skin friction and $I_{s(50)}$.

It can be from Fig. 9 that more than 100 kN/m² unit skin friction is mobilized when the penetration rate > 3000 mm/hr. However, no clear correlation could be observed between the mobilized unit skin friction values and the penetration rates.

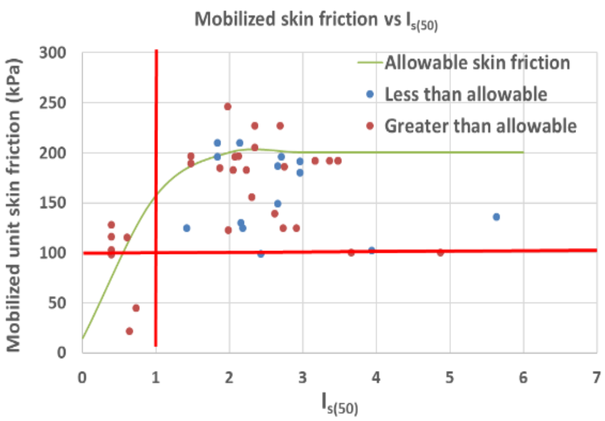

Fig. 8 Mobilized skin friction values versus $I_{s(50)}$ at Kandy.

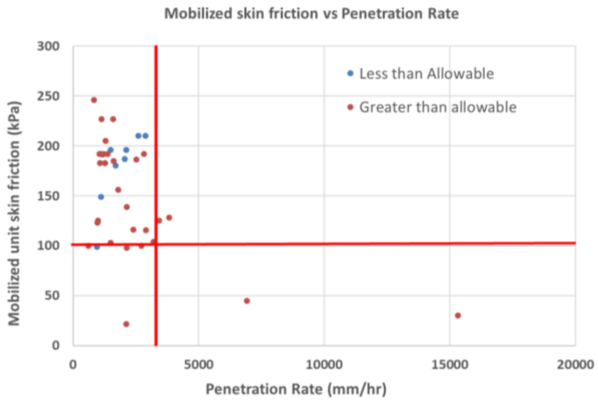

Fig. 9 Mobilized skin friction values versus the penetration rate at Kandy.

6 CONCLUSIONS

1. Piling in cavernous and weak rocks is challenging which requires proper investigation and quality assurance procedures.
2. Cavities and fractured zones may require specialized treatment (pre-concreting, reboring).
3. IMLT and PDA tests provide consistent assessment of mobilized skin friction and end bearing and the load settlement behavior obtained from both IMLT/MLT and PDA are quite similar for the cases studied.
4. Site-specific termination criteria based on $I_{s(50)}$, penetration rate and visual manual observation is useful in terminating piles in uncertain geology.
5. Qualitative correlations are evident between the mobilized skin friction values and the $I_{s(50)}$ or penetration rate.
6. With sufficient lateral confinement, piles achieved 8600–9930 kPa mobilized end bearing despite low UCS values of rock due to the presence of vertical fractures.

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Ensuring safety in deep excavations through No-Go rules

By

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In the past, Madan Kumar has worked in leading Indian and international firms, including L&T, L&T-Ramboll, DAR Group (Dubai), and COWI (India and Denmark). He handled several complex Geotechnical Engineering challenges throughout his career. His area of interest includes the design and construction of Heavy Foundations, esp. "Bored Piles, Diaphragm Walls and Deep Excavations”. Madan Kumar successfully accomplished his role as "Design Team Leader of Civil Works" for the prestigious "Underground LPG Storage Cavern" at Vizag. This project is still acclaimed as a Ground Engineering Marvel, 300,000 m^3 of rock excavated 200 m below the seabed.

He has authored over 80 technical papers in reputed national and international journals and conferences. Beyond engineering, Madan Kumar is a passionate motivational and theme speaker, known for delivering impactful lectures across academic institutions and industry forums throughout India, sharing knowledge and inspiring the next generation of engineers.

Ensuring safety in deep excavations through 'No-Go' rules

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ABSTRACT: Deep excavation projects in urban environment demand robust earth retention systems where even small design oversights can escalate into safety risks and costly failures. This paper sets out critical "No-Go" rules that must never be compromised in the design and construction of retention systems. Core principles include comprehensive ground investigation, correct distinction between drained and undrained soil behavior, realistic assessment of groundwater and surcharge effects, consideration of soil stiffness, and mandatory sensitivity analyses. These elements are illustrated through three case studies. The cases include a deep commercial basement excavation, an intake well structure, and a complex metro infrastructure project. Each case demonstrates how adherence to the most disciplined design checks ensured serviceability, constructability, and safety under challenging ground and loading conditions. This paper concludes that sound engineering practices are essential for achieving effective retention solutions. Their application ensures safety, economy, and constructability under challenging ground conditions.

Keywords: Deep excavations, basement construction, earth retention system, drained & undrained conditions, surcharge, groundwater, soil stiffness.

1. INTRODUCTION

Infrastructure expansion in urban regions increasingly depends on deep excavations and multi-level basements to accommodate underground metro stations, commercial facilities, and high-rise developments within dense city zones. These projects are executed under severe spatial and geotechnical constraints, where soil–structure interaction, groundwater control, and construction sequencing play important roles in safety and its performance of any deep excavation. Failures of retention systems, even when localized, can trigger significant serviceability issues, project delays, or catastrophic structural and impacts on society.

Designing and constructing retention systems for such environments requires a well-disciplined application of geotechnical principles supported by reliable ground investigation, realistic characterization of soil behavior, and an appreciation of construction challenges. Despite advances in numerical modeling and monitoring data, industry experience continues to show that many failures stem not from lack of analytical sophistication, but from neglecting fundamental checks and misjudging governing conditions. This highlights a persistent 'knowledge-to-practice gap' in ensuring essential design safeguards are never compromised.

This paper addresses a set of "No-Go" rules that insists non-negotiable principles in earth retention system design and execution. These include: (i) comprehensive ground investigation and characterization, (ii) correct identification of drained versus undrained behavior, (iii) robust groundwater and surcharge assessment, (iv) appropriate consideration of soil stiffness in deformation analyses, and (v) mandatory sensitivity checks to evaluate uncertainty. The framework is illustrated through three case studies; a deep commercial basement, an intake well structure, and a metro infrastructure excavation each demonstrating how adherence to these principles ensured serviceability, constructability, and safety under challenging conditions. The findings reaffirm that success in engineering of deep excavation is not achieved by complexity alone but by consistently applying sound geotechnical practice respecting ground realities and embedding disciplined checks throughout the project lifecycle.

2. NO-GO” RULES FOR RETENTION SYSTEM DESIGN AND CONSTRUCTION

2.1. COMPREHENSIVE GROUND INVESTIGATION AND CHARACTERIZATION

A well-planned and executed ground investigation forms the cornerstone of any geotechnical project which directly influencing safety, economy and constructability. The investigation planning must extend beyond the routine boreholes to integrate in-situ and laboratory testing and hydrogeological assessment related to the project complexity. Investigations should not only capture soil strength but also cover stiffness, compressibility and time-dependent behavior under varying stress conditions (excavation sequences i.e. unloading or loading conditions). Characterization further requires geological interpretation including soil formation history, stratigraphic variability and discontinuities such as fissures or cavities within the rock mass. Recent advances in seismic CPT, pressuremeter testing and high-resolution geophysical surveys allow practicing engineers to obtain realistic input for deformation and stability analyses. Importantly, investigations should anticipate construction effects such as dewatering, unloading or stress redistribution. Lunne, Robertson, and Powell (1997) provides foundational guidance on comprehensive ground investigation and soil characterization essential for reliable earth retention system design. Many case studies show that incomplete or misinterpreted site characterization often leads to redesign, delays, or failures. Experience emphasizes that thorough investigation is not just data collection but the translation of ground response into reliable design parameters.

2.2. DISTINCTION BETWEEN DRAINED AND UNDRAINED SOIL BEHAVIOR

Correctly distinguishing drained and undrained soil behavior is critical for both short-term construction stability and long-term serviceability. Soils of low permeability, such as clays, initially respond in an undrained manner where pore pressures carry a significant portion of applied stresses. Over time, as dissipation occurs, effective stress governs the soil’s strength, deformation, and stability. Misapplication of parameters across these states leads to unsafe or overly conservative designs. Short-term excavation stability in low permeability soils should be assessed using undrained shear strength (Fig. 1).

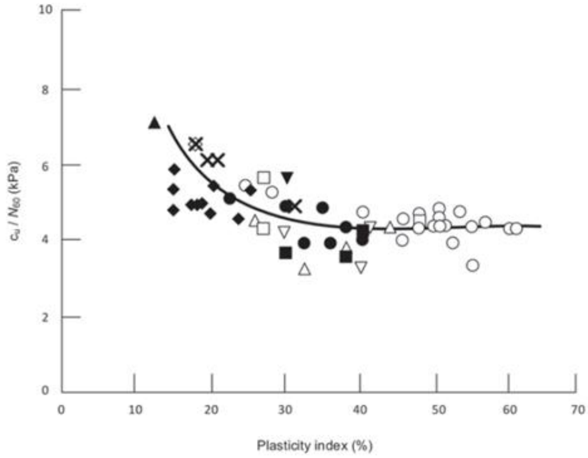

Fig. 1: SPT N vs cu correlation (Stroud, 1974)

While long-term effects like final design conditions, wall creep, and base heave require drained parameters (cohesion c′ and friction angle ϕ′, Ref. Fig. 2). For example, retaining wall analyses in stiff clays must distinguish short-term basal heave (undrained) from long-term creep or deflection (drained).

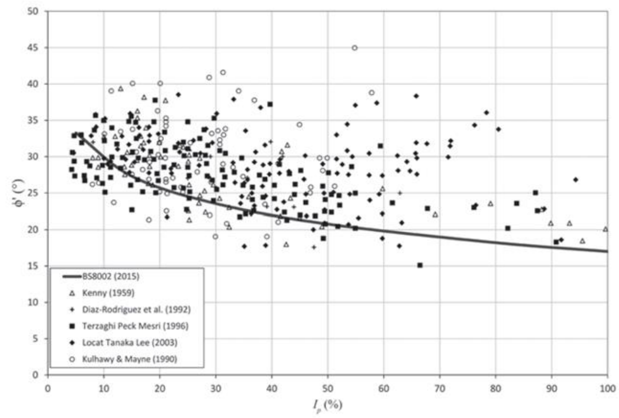

Fig. 2: Suggested phi values for Drained Case (CIRIA C760)

A landmark case often cited in this regard is the Nicoll Highway collapse in Singapore (2004). The failure occurred during a deep excavation supported by diaphragm walls and struts, where wall deflections and strut loads were significantly underestimated. The Committee of Inquiry attributed the collapse to shortcomings in soil parameter selection, inadequate consideration of staged undrained and drained responses, and insufficient design verification and safety margins. These issues were compounded by flawed modelling, underestimation of deformations, and inadequate site monitoring. Critical lesson from this tragedy is that deep excavation design must explicitly evaluate both short-term and long-term soil behaviours, ensuring robust checks across all construction stages. Failure to distinguish these conditions can compromise not only serviceability but ultimately structural stability and life safety.

2.3. GROUNDWATER AND SURCHARGE ASSESSMENT

Groundwater exerts a profound influence on excavation performance, altering effective stress conditions, pore pressure response, and seepage forces. Seasonal variations, construction-induced drawdowns and perched water lenses affect stability of the retaining wall. Elevated water pressures may trigger basal heave, hydraulic uplift, or piping failures if not mitigated by dewatering or cut-off systems. Equally important are surcharge effects from adjacent buildings, traffic, or stockpiles, which intensify wall bending moments and settlements. The timing and magnitude of surcharges during both temporary and permanent phases often govern the most critical design conditions. Extensive case histories document failures where either groundwater or surcharge loads were underappreciated during design (CIRIA C760). The report insists integrating transient hydrogeological modelling and realistic surcharge scenarios into both serviceability and ultimate limit state assessments. Ignoring these can produce unconservative predictions of deformation and stability. Groundwater and surcharge assessment must therefore be treated not as secondary checks but as primary design drivers.

2.4. ROLE OF SOIL STIFFNESS IN DEFORMATION AND STABILITY ANALYSES

While soil strength governs ultimate stability, soil stiffness dictates deformation behavior of the wall which is critical for serviceability in urban excavations. Soil stiffness is strain-dependent, reducing drastically from initial (small-strain) modulus to working strain conditions typically mobilized in excavations. Overestimating stiffness underpredicts wall deflections, settlements, and support loads, risking structural distress to nearby structures. In contrast, underestimating stiffness inflates design conservatism and costs. Laboratory triaxial, pressuremeter and seismic CPT tests, alongside empirical correlations with in-situ penetration values offer calibration points. Importantly, staged excavation analyses require stiffness degradation curves to realistically simulate soil-structure interaction. Extensive field observations and numerical simulations consistently indicate that soil strains mobilized in engineering projects generally fall within the small-strain range, typically between 0.01% and 0.3% of depth of excavation (Fig. 3). Atkinson and Sallfors (1991) demonstrated how soil stiffness varies logarithmically with strain and stress history, providing frameworks which is widely adopted in finite element analyses.

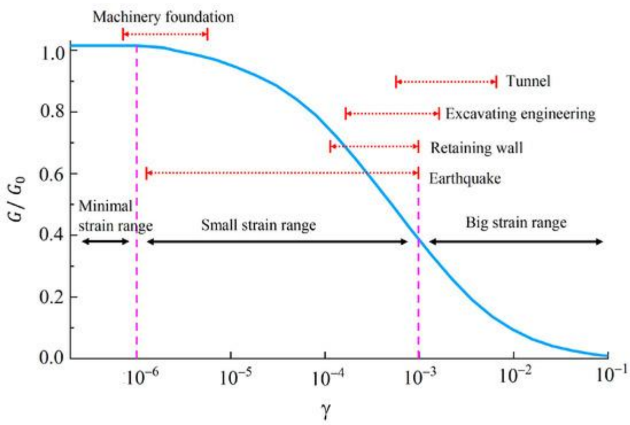

Fig. 3: Soil shear stiffness curve and strain division range (Atkinson and Sallfors, 1991)

Engineers must differentiate between initial, secant and tangent stiffness values applying them judiciously within the chosen constitutive model, for reliable outcomes. Without this rigor, deformation predictions remain speculative and risk unsafe or uneconomical solutions.

2.5. SENSITIVITY ANALYSES AND UNCERTAINTY EVALUATION

Uncertainty is inherent in geotechnical engineering due to natural soil variability, limitations in testing, and construction deviations. Sensitivity analysis provides a structured approach to evaluate how changes in soil strength, stiffness, or groundwater assumptions influence performance outcomes. By systematically varying parameters, engineers identify critical drivers and quantify robustness of design. This is particularly valuable for finite element analyses, where soil models and input parameters strongly influence predicted displacements and forces. Duncan (2000) highlighted that designs developed without systematic sensitivity checks risk being either unconservative or uneconomical. Geotechnical variability is more appropriately addressed through probabilistic approaches or partial safety factor frameworks, thereby maintaining reliability even under unfavourable conditions. Sensitivity analyses provide a rational basis for defining instrumentation and monitoring strategies by identifying the parameters with the greatest influence on performance. Incorporating these practices enhances design confidence, balances safety with economy, and ensures preparedness for unforeseen ground behavior.

2.6. CONSTRUCTION CONTROL AND MONITORING AS PART OF DESIGN DISCIPLINE

The design of underground structures does not end with drawings and calculations; it extends into construction control and performance monitoring. Excavation-induced ground movements are time-dependent and heavily influenced by workmanship, sequencing and dewatering efficiency. Instrumentation such as inclinometers, piezometers and settlement markers provide feedback on real-time soil-structure response, allowing early detection of deviations from predicted behavior. Monitoring must be interpreted within the design framework, enabling timely adjustments to strut loads, excavation sequences, or dewatering measures. The Observational Method, as formalized by Peck (1969) in the Rankine Lecture, illustrates this integration of design and monitoring. Design assumptions in uncertain ground conditions should be verified in the field, with predefined contingency actions guiding safe adaptation. These principles remain the backbone of modern risk-based geotechnical practice. Construction control and monitoring are thus not optional add-ons but intrinsic elements of the design discipline. They transform design from a static calculation into a living process that ensures both safety and constructability throughout execution.

3. CASE STUDY 1: DEEP COMMERCIAL BASEMENT EXCAVATION

This case presents a comprehensive case study on the analysis, design, and performance of a deep and complex cantilever retention system implemented for a high-rise commercial development in Chennai, India. The project involved the construction of a 200-feet tall IT park with three basement levels, necessitating a vertical excavation of up to 12.5 meters in challenging urban and geotechnical conditions. The site constraints, particularly limited setback space along a 110-meter stretch, precluded the use of conventional ground anchors, prompting the adoption of innovative T-shaped diaphragm wall panels to ensure excavation stability.

The subsoil profile at the site was highly variable and complex, comprising medium stiff to stiff clays, clayey sands, and dense sandy clays, underlain by weathered granitic gneiss. The variability in soil strength and stiffness posed significant challenges for both geotechnical and structural design. Soil layers exhibited SPT N-values ranging from 7 to 200, with saturated densities between 16.5 and 21.5 kN/m^3. The presence of weak clay layers and jointed rock required careful consideration in modeling soil-structure interaction and designing appropriate embedment depths and anchorage systems.

To address these challenges, the retention system was analyzed using PLAXIS 2D finite element software. The clay layers were modeled using Mohr-Coulomb and hardening soil models, with interface reduction factors ranging from 0.70 to 0.90. The T-shaped diaphragm wall panels, inherently three-dimensional in behavior, were approximated in 2D analysis using embedded square sections to simulate the web component. Structural analysis was performed using STAAD Pro, employing a classical stick model approach with soil springs derived from actual site conditions. The panels were designed for Ultimate Limit State (ULS) and checked against Serviceability Limit State (SLS) criteria.

Construction activities were executed in multiple stages, including diaphragm wall installation, dewatering, anchor installation and pre-stressing, and phased excavation. In the sump area, where anchor installation was not feasible, additional piles were constructed to retain weak clay layers. Field instrumentation included eight inclinometers installed within the diaphragm wall to monitor lateral displacements during excavation.

Performance monitoring revealed that the T-shaped panels exhibited satisfactory behavior under excavation-induced loads. The maximum measured lateral deflection at the wall top was approximately 44 mm, closely matching the theoretical predictions from PLAXIS analysis. The deflection profile suggested a rigid body rotation rather than elastic bending, indicating the effectiveness of the T-panel geometry in resisting lateral earth pressures. In contrast, straight diaphragm wall sections showed higher deflections (~60 mm), underscoring the superior performance of the T-shaped configuration (see Fig. 4).

This study demonstrates the successful implementation of a deep retention system using T-shaped diaphragm wall panels in constrained urban settings with complex subsoil conditions. The integration of advanced numerical modeling with appropriate design soil parameters, analysis with drained & undrained behavior tailored with structural design, and rigorous field monitoring enabled the safe and efficient execution of deep excavation works. The key outcome also highlights that T-shaped diaphragm walls offer enhanced rigidity and reduced lateral displacement, making them a viable alternative to anchored systems in setback-limited environments (Refer Madan Kumar 2019).

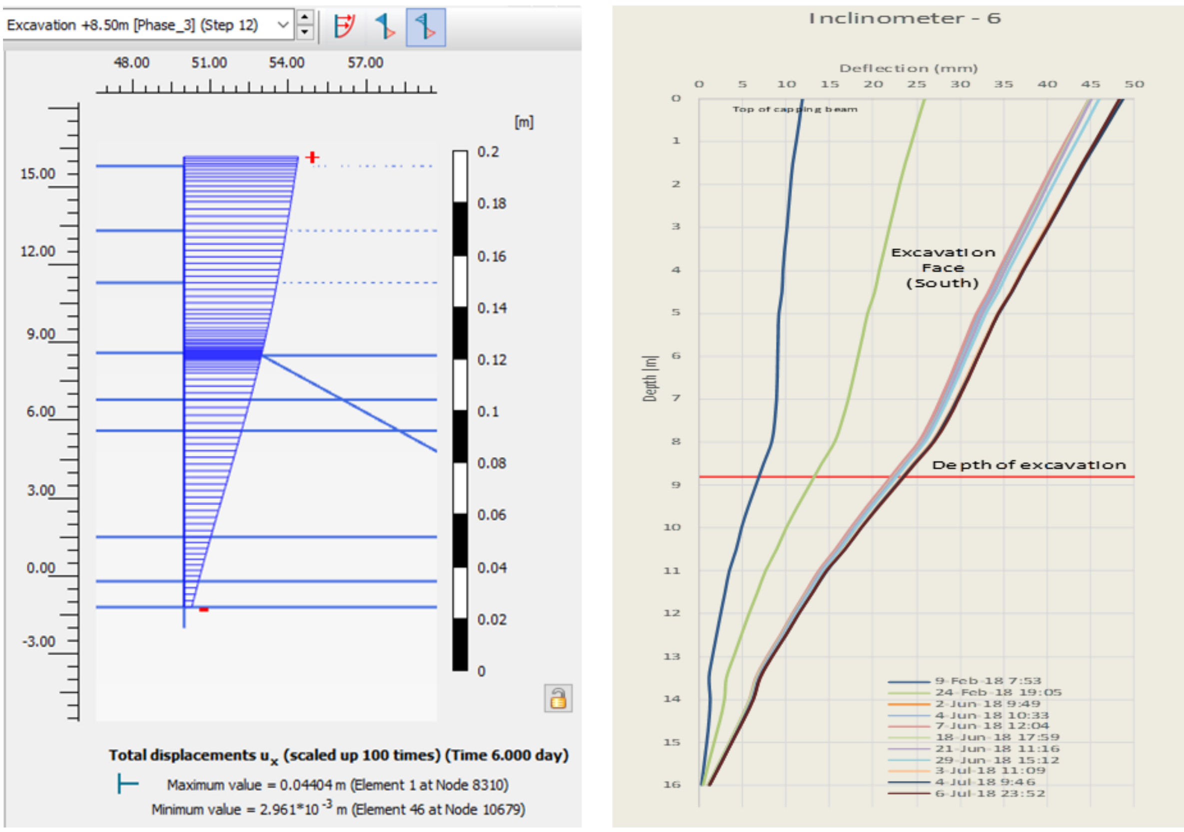

Fig. 4: Theoretical and actual deflection of T Panels.

4. CASE STUDY 2: INTAKE WELL STRUCTURE IN COMPLEX GROUND CONDITIONS

A diaphragm wall system was adopted as a permanent retaining structure for the seawater intake pump house forming part of a major desalination project on the West Coast of India. The facility required a watertight underground sump to ensure the integrity and functionality of the seawater intake system. Given the proximity to the shoreline (about 200 m inland) and the need for an 18.5 m deep excavation below the existing ground level, the project demanded a robust and water-impermeable retaining system. The diaphragm wall, 1.2 m thick and extending to a depth of 28 m, was designed to function both as a temporary excavation support and as a permanent structural wall, forming an integral part of the intake structure (Refer Fig. 5).

Geotechnical investigations revealed stratified subsoil conditions with stiff to very stiff clay extending to approximately 12.5 m below ground level, underlain by very dense silty sand. The groundwater table was shallow, encountered at 2–3 m below ground level, and subject to tidal variations. These conditions posed significant challenges in ensuring excavation stability, watertightness, and control of wall deflection. The combination of highwater pressure, mixed soil strata, and deep excavation demanded careful design of both wall section and support system.

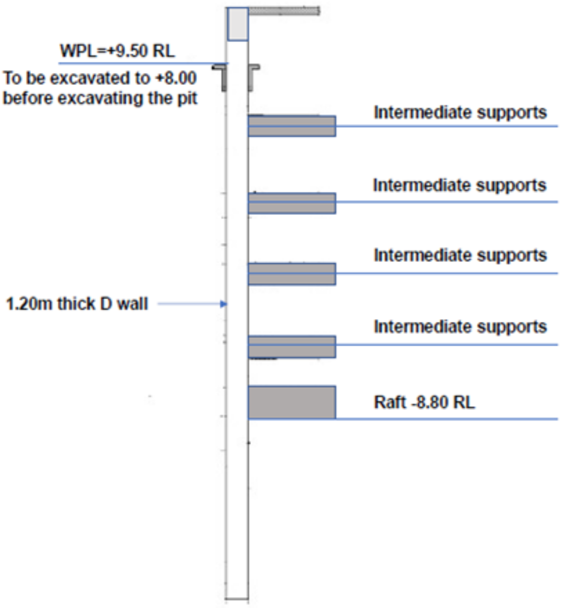

Fig. 5: Typical Retaining Wall Section.

The diaphragm wall was modeled as a multi-propped system with four intermediate RCC support beams, a top capping beam, and a bottom raft providing full enclosure. A detailed numerical analysis was undertaken using three design approaches viz. Limit Equilibrium, Subgrade Reaction, and Finite Element Methods (FEM). Comparative studies indicated that while FEM and Subgrade Reaction approaches produced consistent results, the Limit Equilibrium Method significantly underestimated bending moments and prop loads due to its inability to capture soil–structure interaction. Parametric analyses revealed the sensitivity of wall behavior to variations in groundwater level, over-excavation, and drained versus undrained soil conditions. A 3 m rise in groundwater increased wall bending moments by 12%, and an over-dig of 1 m led to a 6% rise. Transitioning from undrained to drained analysis conditions resulted in a 40% increase in bending moment, emphasizing the importance of long-term pore pressure dissipation effects in design, presented in Table 1. The study also highlighted the role of toe movements as an indicator of wall stability in multi-propped configurations, aligning with Eurocode 7 design combinations.

Execution challenges included ensuring a stable working platform under heavy machinery loads (15–18 t/m²), managed using a 0.75 m granular mat following BRE 470 guidelines. Pre-excavation from RL +9.5 m to +8.0 m optimized bending moments and wall embedment, reducing costs by about 10%. Reinforcement cages weighing up to 35 t were successfully lifted in a single operation using twin cranes, ensuring alignment and safety under restricted site conditions. The study demonstrated that accurate modeling of soil–structure interaction and groundwater influences is crucial in diaphragm wall design for deep marine-related structures.

Table 1: Comparison of results of various Methods of Analyses

Item	Equilibrium Method	Subgrade Reaction Method	Finite Element Method
SLS Bending Moment kNm/m	1652	2486	2270
SLS Shear Force (kN/m)	677	738	686
SLS Prop force (kN/m)	990	806	726
Deflection (mm)	-	43	54

Limit Equilibrium methods are unsuitable for multi-propped systems, whereas Subgrade Reaction and Finite Element analyses yield reliable predictions (Govind, 2021). Design must consider both drained and undrained conditions in cohesive soils. Toe movement assessment provides an effective criterion for verifying embedment adequacy. Overall, the project achieved a safe, durable, and watertight diaphragm wall system that met structural and hydraulic performance requirements, contributing valuable insights to embedded wall design practices in coastal environments.

5. CASE STUDY 3: METRO INFRASTRUCTURE PROJECT WITH ADJACENT STRUCTURES

This case study presents comprehensive work on the behavior of a diaphragm wall used in a semi top-down construction method for a two-level basement project with a 9.0 m deep excavation. The project, located in a densely developed urban area, involved the construction of an underground amenity center comprising parking and retail spaces, with a surface-level public park. The site was bounded by busy roads, residential buildings, and a sensitive underground metro line, necessitating a safe and deformation-controlled excavation system to prevent distress to nearby structures. Conventional support systems such as ground anchors or steel struts were deemed unsuitable due to space limitations, urban congestion, and proximity to existing infrastructure. Hence, a semi top-down construction technique was adopted, which combined the structural and cost advantages of the top-down method with the construction flexibility of the bottom-up approach. In this hybrid system, a diaphragm wall (800 mm thick) was used as the main retaining element, supported laterally by a passive berm formed in front of the wall. This berm, with a top width of 6–8 m, bottom width of 17–19 m, and height of 5 m, served as an effective temporary lateral support, minimizing wall deflection and eliminating the need for steel struts or anchors.

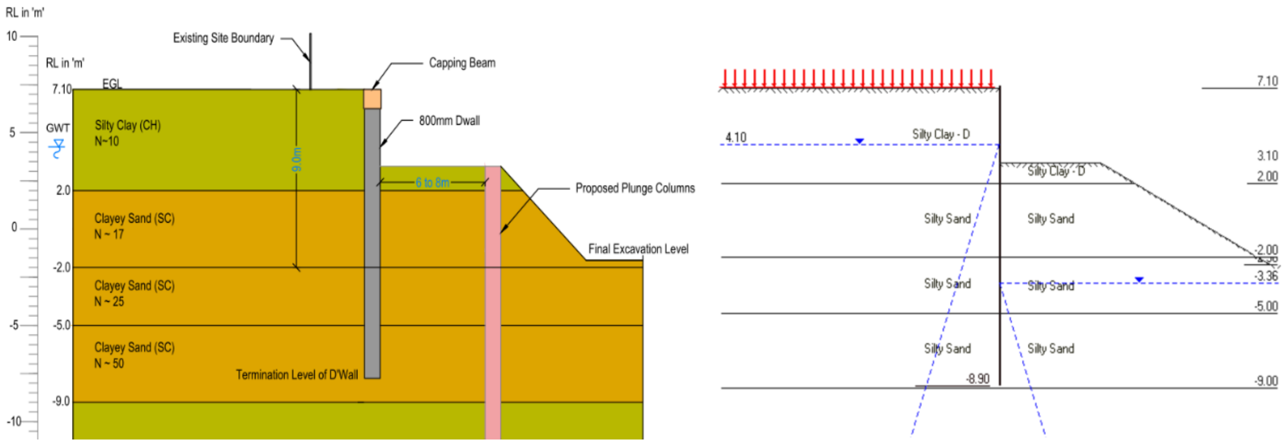

Fig. 6: Typical cross section & WALLAP model

The subsoil profile, established from boreholes up to 30 m depth, comprised stiff silty clay at the top, underlain by medium dense to dense silty sand and hard silty clay, resting on weathered rock. The groundwater table was encountered between 2.1 m and 4.3 m below ground level. The retaining wall design was analyzed using WALLAP software for temporary (excavation) stages to evaluate bending moments, shear forces, and lateral deflections, while the permanent condition was modeled using ETABS for 3D finite element structural analysis, considering raft, slabs, and columns interaction. The construction sequence was carefully designed and executed to maintain stability throughout the excavation stages. It included guide wall construction, diaphragm wall installation, plunge columns (later converted into structural columns), partial excavation with passive berm formation, raft casting at the central zone, slab construction providing intermediate lateral support, and final raft connection with the diaphragm wall. The semi top-down approach enabled faster excavation, reduced temporary works, and optimized material usage.

A systematic instrumentation and monitoring program was implemented, consisting of 16 inclinometers and 4 piezometers strategically placed to observe wall behavior during excavation and basement construction. The monitoring data revealed that the maximum measured lateral wall deflection (~18 mm) was significantly lower than the predicted deflection (~40 mm) from design analysis. The measured deflection profile exhibited a similar trend to the analytical predictions, confirming the validity of design assumptions and the efficiency of the semi top-down system. The study confirmed that semi top-down construction with a diaphragm wall and passive berm effectively controls lateral movements, ensures wall stability, and shortens construction duration in deep urban excavations. Moreover, it demonstrated that permanent slabs can be successfully integrated as lateral supports, further optimizing cost and time.

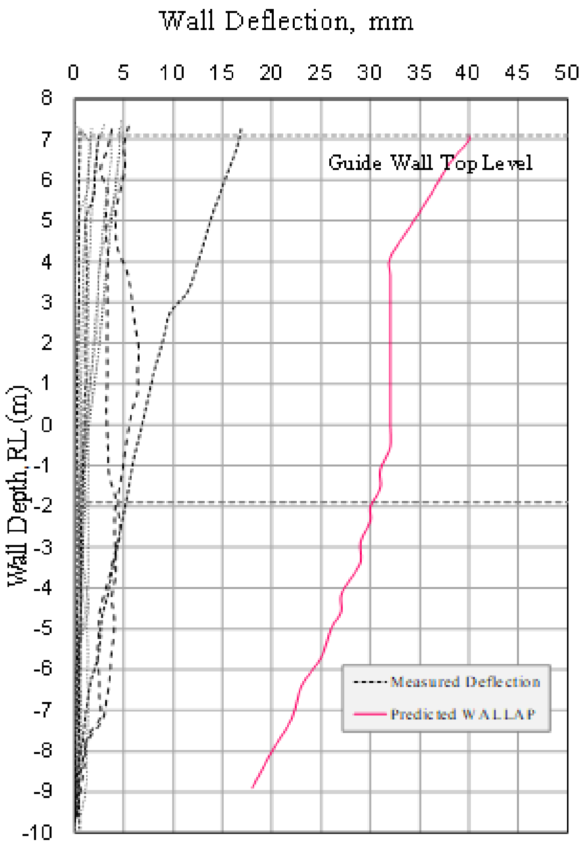

Fig. 7: Predicted and measured wall deflection.

In conclusion, the project validates the semi top-down diaphragm wall system as a technically robust and economical solution for deep excavations in constrained environments. The favorable correlation between predicted and monitored wall behavior underscores the reliability of analytical models and the practical viability of this hybrid construction approach for future urban infrastructure developments.

6. CONCLUSIONS

This study establishes a framework of essential "No-Go" rules that underpin safe and constructible deep excavation practices in urban environments. Through rigorous case studies and design principles, it demonstrates the success in retention system design is not driven by analytical complexity alone, but by disciplined adherence to geotechnical fundamentals. Comprehensive ground investigation, correct identification of drained versus undrained soil behavior, realistic groundwater and surcharge assessment, and strain-dependent stiffness modeling are shown to be critical for reliable performance. Sensitivity analyses and construction monitoring further enhance design robustness and adaptability. The case studies validate innovative solutions such as T-shaped diaphragm walls and semi top-down construction under challenging spatial and geotechnical constraints. These findings reinforce that deep excavation safety is achieved through a holistic integration of soil behavior understanding, numerical modeling and field verification. The proposed framework offers a practical and scalable approach for engineers to mitigate risk, optimize design and ensure serviceability in complex ground conditions. Future developments should focus on embedding these principles into design standards and training to bridge the persistent gap between geotechnical theory and construction practice.

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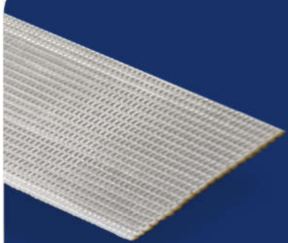

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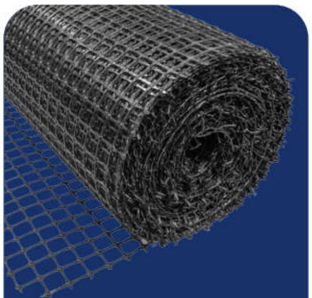

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Best practices to mitigate damages to properties adjacent to construction in urban areas

By

Dr. Asiri Karunawardena, Director General, National Building Research Organisation (NBRO)

Eng. (Dr.) Asiri Karunawardena is the Director General/ Chief Executive Officer of the National Building Research Organization (NBRO) of Sri Lanka. NBRO is the main focal point for landslide disaster risk reduction in Sri Lanka, and it is taking a leading role in promoting resilient and safe infrastructure construction in the country.

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As the Director General of the institute, he leads a team of multidisciplinary professionals to reduce the losses due to disasters by integrating disaster risk reduction into the planning and development process in the country to achieve the vision of a safer built environment and sustainable development gains.

Best Practices to Mitigate Damages to Properties Adjacent to Construction in Urban Areas

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ABSTRACT: Construction in densely built urban environments has considerable challenges due to existing structures and infrastructure that are often sensitive to ground movements and vibrations. Activities such as deep excavations, pile driving, dewatering, and heavy equipment operations can induce soil displacement, settlement, and structural distress in adjacent properties if they not carefully managed. This paper presents best practices for mitigating such risks through a systematic and engineering-based approach. Key aspects include detailed pre-construction condition surveys, geotechnical investigation, and numerical modeling to predict soil–structure interaction. The adoption of appropriate excavation support systems, ground improvement methods, and vibration control measures are discussed, together with instrumentation and real-time monitoring techniques for ground and structural responses. Case examples from urban projects are used to demonstrate the effectiveness of these practices in minimizing damage, ensuring structural safety, and supporting sustainable urban development.

1 INTRODUCTON

Rapid urbanization has intensified the demand for new infrastructure, high-rise buildings, and deep basements in densely populated cities. These developments are often carried out in close proximity to existing buildings, utilities, and transport corridors, many of which were not originally designed to withstand additional stresses induced by modern construction activities. As a result, adjacent property damage has become a critical concern in urban construction projects, leading to structural distress, costly disputes, project delays, and, in some cases, threats to public safety.

Construction activities such as deep excavation, pile boring and dewatering alter the in-situ stress conditions and groundwater regimes, potentially causing ground settlement, lateral soil movement, and vibration propagation. The magnitude of these effects depends on local geotechnical conditions, construction methods employed, and the sensitivity of adjacent structures. Historical case records worldwide have shown that the absence of adequate risk management and monitoring often results in avoidable structural damages. For example, the *Nicoll Highway collapse in Singapore (2004)*, caused by inadequate excavation support during deep tunneling works, highlighted the consequences of design and monitoring deficiencies in urban environment.

To address these challenges, best practices must be adopted across the planning, design, and execution stages of projects implemented in urban areas. Key measures include comprehensive pre-construction surveys to establish baseline conditions, advanced geotechnical investigation and numerical modeling to anticipate soil–structure interactions, deployment of real-time monitoring and instrumentation and quality control.

The above identified factors are further elaborated in several international codes and standards that emphasize the protection of adjoining properties during construction activities. The *International Building Code* (IBC, 2021) mandates that adequate measures be taken to safeguard adjoining public and private properties throughout the phases of construction, demolition, and remodeling. In addition, publications by the *Construction Industry Research and Information Association* (CIRIA) provide comprehensive guidance on managing ground movements, temporary works, drainage, and other environmental impacts that can adversely affect neighboring properties.

This paper consolidates current best practices and lessons learnt from case histories to provide a guidance for mitigating damages to adjacent properties due to the substructure construction works at urban areas.

2 MECHANISMS OF DAMAGES TO ADJACENT PROPERTIES

Damage to structures adjacent to construction sites arises from a variety of geotechnical and structural reasons. Excavation works alter the in-situ stress conditions of the soil mass, which can result in ground settlement at the surface or heave at the excavation base. Where support systems are insufficiently stiff or improperly designed, lateral ground movements occur, often leading to tilting or differential settlement of nearby foundations. Dewatering activities further complicate this scenario by lowering pore water pressures and inducing consolidation settlements in compressible soils. In sandy soils, groundwater drawdown may trigger instability or piping, threatening both the excavation and surrounding properties. Construction vibrations, particularly from pile driving represent another critical hazard. If vibration levels exceed peak particle velocity limits established by standards, cracking of masonry walls and disruption of sensitive equipment may occur. Finally, soil–structure interaction effects must also be considered, as the redistribution of loads during excavation may inadvertently transfer stresses to adjacent retaining walls, tunnels, or shallow foundations

3 BEST PRACTICES TO MITIGATE THE DAMAGES

3.1 *Pre – Construction Risk Assessment*

Actions for mitigating construction-induced damage begins long before the actual construction begins at site. A rigorous risk assessment framework provides the basis for identifying potential hazards and implementing suitable controls. Pre-condition surveys are essential to establish a baseline of existing structural conditions, documenting cracks, tilts, and defects through visual inspection, high-resolution photography, and increasingly, three-dimensional laser scanning or photogrammetry. This baseline provides a defensible record for future claims while also assisting in vulnerability assessment.

During assessment risk maps have to be developed, which identify vulnerable structures, utilities, and heritage assets within the area likely to be affected by the proposed construction activities. Once these risks are identified, acceptable trigger levels for settlement, displacement, and vibration are defined.

3.2 *Geotechnical investigation*

A comprehensive site investigation is a fundamental prerequisite prior to commencing the design of substructure works. The primary objective of the investigation is to evaluate the conditions of the site location and its subsurface profile, ensuring that reliable geotechnical parameters are available for design.

The investigation typically begins with preliminary desk studies and reconnaissance surveys, during which relevant data concerning the site and its surrounding environment are compiled. These initial stages are followed by detailed geotechnical investigations aimed at identifying the subsurface properties beneath the existing ground level. This phase is particularly critical, as the obtained soil parameters govern the feasibility and safety of the proposed construction. To ensure accuracy and reliability, the investigations should be conducted under the supervision and guidance of an experienced geotechnical engineer.

Geotechnical investigation techniques may include direct methods, such as trial pits and boreholes, or indirect methods, such as electrical resistivity, seismic surveys, and ground-penetrating radar (GPR). The selection of appropriate methods depends largely on the geological condition of the site and the required level of precision for the design.

The selection of foundation system, the depth of retaining structures, and the excavation sequence are strongly influenced by the nature of the soil strata and their thicknesses. Accordingly, the design of substructures and retaining systems must be carried out in accordance with the recommendations and parameters presented in the geotechnical investigation report. It is imperative that this report is prepared and endorsed by a qualified geotechnical engineer, as the reliability of the design is directly depends on the accuracy of the soil parameters provided. Furthermore, the parameters recommended in the geotechnical report must not be altered during subsequent stages of design and construction without any valid technical reason.

3.3 *Numerical Modelling*

Ensuring a safe built environment requires proper geotechnical design of substructures, based on reliable data obtained from site investigations and the corresponding soil investigation reports. In the case of deep excavations, the design process must incorporate not only the selection of suitable retaining systems but also the specification of excavation methods, construction sequences, and the provision of lateral support during each stage of excavation. These details should be clearly documented within the design drawings and specifications to follow during the construction.

In modern practice, finite element analysis (FEA)-based software tools are widely used in conjunction with conventional analytical methods to enhance the accuracy and reliability of design outcomes. Despite these advancements, several case histories documented in the literature highlight that deep excavation failures have often been attributed to inadequacies in geotechnical design. Lessons learned from such failures emphasize the necessity of involving both geotechnical and structural engineering specialists at the design stage, to safeguard the stability of the excavation and minimize adverse impacts on adjacent structures.

Failures of excavation support systems have, in some instances, been linked to the yielding of structural materials used in retaining systems and lateral bracing. Therefore, careful selection of structural sections and materials that can withstand anticipated stresses without yielding is critical to prevent such incidents. Moreover, the design process must incorporate a thorough assessment of potential impacts on nearby foundations, with the objective of preventing damage to existing structures during excavation.

When excavation activities are planned in areas underlain by sensitive soils, additional precautions should be incorporated into the design to mitigate risks associated with soil instability. Similarly, groundwater conditions required special consideration; excessive drawdown during dewatering operations can induce ground settlement and adversely affect surrounding structures. Consequently, groundwater control strategies should be carefully integrated into the design to avoid problems associated with fluctuations in the water table.

3.4 *Instrumentation and Monitoring*

Instrumentation and monitoring are integral components of substructure construction projects, particularly in urban environments where adjacent structures and underground services are highly sensitive to ground movements. Geotechnical instrumentation may broadly be classified into two categories: **deformation measuring instruments** and **stress measuring instruments**. Deformation measuring instruments are used to evaluate ground and structural displacements, while stress measuring instruments provide measurements of pore water pressures, soil stresses, and stresses induced within structures.

The implementation of a well-prepared **risk mitigation plan**, incorporating monitoring strategies and clearly defined trigger levels, is essential for managing potential hazards during construction. Particularly. in deep excavations, the movement of retaining systems towards the excavation often induces settlement in the ground immediately adjacent to the excavation, while heave may occur at locations far from the site due to stress redistribution. Such behavior necessitates continuous observation of both ground and structural responses.

Monitoring data should be collected at predetermined intervals and systematically compared with baseline measurements and design predictions. Deviations from the anticipated performance may indicate that the actual behavior of the system differs from design assumptions. In such cases, immediate evaluation and, if necessary, remedial actions must be undertaken to avoid potential failures. If monitoring reveals intolerable deformations, corrective measures may include modifications to the excavation sequence, enhancement of the support system, or even redesign of critical structural elements.

It must be acknowledged, however, that monitoring approaches have limitations. Monitoring may not provide sufficient early warning in cases of brittle structural behaviour or rapid soil deterioration, where the time available for intervention is inadequate. Examples include non-ductile failures of strut-to-waling connections in multi-propped basements, or sudden strength losses in soils triggered by groundwater-related instability. In such scenarios, preventive measures and conservative design assumptions remain the primary safeguards, as instrumentation alone cannot mitigate the risks of abrupt failure mechanisms.

3.5 *Quality Control and Assurance*

A robust quality control (QC) and quality assurance (QA) system must be established and maintained on-site to ensure that design requirements are fully met during the construction. Such measures are essential not only for safety but also for the sustainability of the constructed facility. Construction activities should strictly follow design drawings, technical specifications, approved method statements, and the designated construction sequence. Furthermore, all materials used must comply with relevant standards and specifications outlined in the design documentation.

In case of deep excavations, prior to commence the excavation, it is essential to verify the quality and integrity of the retaining structure to ensure full compliance with the design requirements. Without such verification, undetected construction defects can lead to severe risks during excavation.

4 CASE HISTORIES IN SRI LANKA

In a recently reported case in Sri Lanka, a major defect has been occurred in a diaphragm wall which has used as the retaining structure of an excavation carried out for the construction of basements. Due to the sand-boiling through the earth retaining system water level of the adjacent land has suddenly dropped and structure was damaged. The quality of the construction (integrity of the diaphragm wall) has not been verified prior to commencing the excavation which is very important. The damages to the structure and adjacent structures could have been minimized, if proper quality control and assurance has established during the construction.

Another similar incident also reported in Sri Lanka during an excavation supported by a **secant pile wall system**. In this case, seepage paths developed between piles due to inadequate construction tolerances and poor joint integrity, allowing groundwater ingress and soil boiling. This led to ground loss, rapid drawdown of the adjacent groundwater table, and subsequent damage to nearby structures. Once again, the absence of rigorous QA & QC procedures, such as pile integrity testing and water-tightness verification, was identified as a key contributing factor for the failure.

5 CONCLUSIONS

Substructure constructions such as deep excavations, pile construction, dewatering, etc. in urban environments inherently poses risks to adjacent properties due to complex soil–structure interactions, groundwater effects, and construction-induced vibrations. However, these risks can be substantially mitigated through the application of best practices. Pre-construction risk assessment, supported by condition surveys, geotechnical investigations, and predictive modeling, forms the basis of effective mitigation. Engineering measures such as robust excavation support systems, ground improvement, vibration reduction, and groundwater control are essential in managing specific hazards. Continuous monitoring and in case of abnormal behaviour is identified, the application of pre decided actions in the contingency plan is very important to mitigate the damages to the adjacent built environment. Ultimately, the adoption of best practices not only minimizes damage to adjacent properties but also contributes to safer, more resilient, and sustainable urban development.

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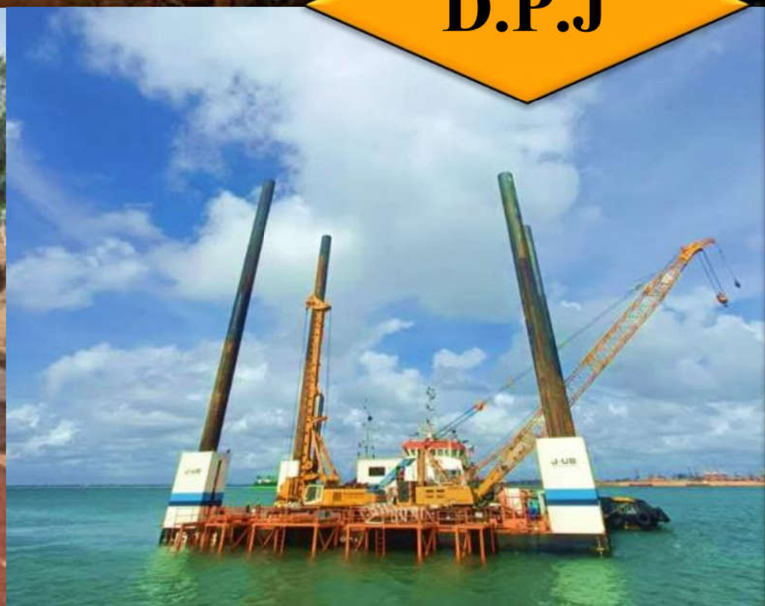

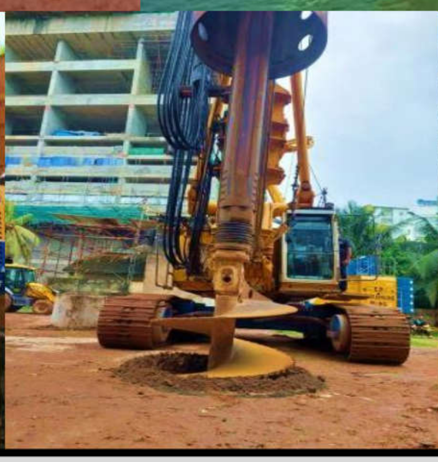

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