Micropile Foundation Systems for Railway Bridge Structures

K L S Sahabandu and J S M Fowze
Central Engineering Consultancy Bureau, Sri Lanka

ABSTRACT: The Government of Sri Lanka undertook to rehabilitate and newly construct railway bridges to reinstate and develop country's railway infrastructure. The Program included eight railway bridges of which five were new ones. Subsurface conditions at new bridge sites demanded deep foundation systems. Given a wide range of options, Micropile foundation systems were selected as the appropriate solution. Cement grouted, single steel tubes were generally adopted as micropiles. However, where weak organic and/or peat encountered at shallow depths, double steel tubes were proposed to avoid buckling failure, which was governing the design criterion. Geometric and mechanical properties of the adopted micro piles and establishment of their load carrying capacity are presented in detail.

1 INTRODUCTION

During the colonial rule, Sri Lanka had a 1,438 km long railway network, which in the recent past got reduced in length, partly due to the reason that some structures exceeded their life span. However, the demand for rail mode of transportation notably increased with the increased number of commuters to Colombo during peak hours. Responsively, the Government of Sri Lanka, while reinstating the deteriorated lines, launched a new project in 2003 to expand the railway network wherein five new bridges in Colombo suburb were to be constructed in the Main line (Kelaniya), Coastal line (Kalutara South and North) and the Puttalam line (JaEla, Seeduwa) as detailed below in Table 1.

Table 1.Basic Details of Bridges under this project

Location	Lane #	Substructure Span(m)
Kelaniya	Double	2x25+3x52+2x25
Kalurata South	Single	4x50
Kalutara North	Single	4x50
Seeduwa	Single	2x40
Ja-Ela	Single	40

This paper describes the foundation system adopted for the above mentioned new bridges with particular reference to their choice and design.

2 SUBSURFACE CONDITION AND THE CHOICE OF FOUNDATION TYPE

2.1 *Subsurface Condition*

All new bridge sites were located in the coastal belt, where thick alluvial deposits of very soft to soft and loose material were present above the basement rock as given in Table 2.

Table 2. Subsoil Condition

Layer No	SPT	Description
I	<10	Peat, soft organic clay or loose or very loose sand / silt.
II,III	>10	Completely to Moderately Weathered Rock
IV	CR>60% & RQD>50%	Slightly weathered to fresh Rock

Such a condition warranted that the loads and load effects from the structure be transferred safely via a deep foundation system. The task was subcontracted to M/s PORR Grundbau GmbH of Austria by the Main Contractor M/s Waagner Biro Brueckenbau AG of Austria. The Central Engineering Consultancy Bureau (CECB) was the Engineer /Consultant of the Project.

2.2 *Choice of foundation type*

The main factor affecting the choice of foundation was that all new bridges were to be constructed right next to the existing Railway bridges giving rise to a constricted working space and accordingly that the construction activities shall not cause any adverse effect on the performance of the existing structures. Bored and cast in-situ piles and driven piles which require a large working space and heavy / noisy equipment lost their prospects of being the foundation solution. Accordingly, Ductile Iron pipe piles, Deep Cement Mixing (DCM) piles with jet grouting and micro piles were considered. The Ductile Iron pipe option was assessed unfit due to tension and buckling capacities, drivability and its use as raked piles. The second option did not also fare well as the stabilization of peaty soils with cement still being a subject of research. The micro pile foundation system which could be installed in virtually any type of ground condition including karstic areas, uncontrolled fills, areas with rock boulders was chosen as appropriate for the intended load transfer. The equipment being light weight required only a small working space; and the ability to penetrate any obstacle with low noise / vibration were considered for selecting this option.

3 PROJECT MICROPILES

A 168 mm diameter, drilled and grouted non-displacement pile that is reinforced with a steel tube possessing a characteristic yield strength, f_{yk}, of 560 MPa was generally adopted as the micropile.

However, when a weak surface (SPT N<3) layer was encountered, an external steel tube with a diameter/thickness of 219.1mm/10.0mm was used to avoid buckling of the micropile..

Details of four different types of composite micro piles resulted thereof to cater to the anticipated loading are given in Table 3.

Table 3. Composite pile details

Type	Inner tube (mm)	Outer tube (mm)	Outer tube Material
1	114.3x10	219.1x10	N80(f_{yk}=560MPa)
2	114.3x10	219.1x10	S355(f_{yk}=355MPa)
3	127.0x12	219.1x10	N80
4	127.0x12	219.1x10	S355

A typical installation sequence of such a pile is illustrated in Fig. 1.

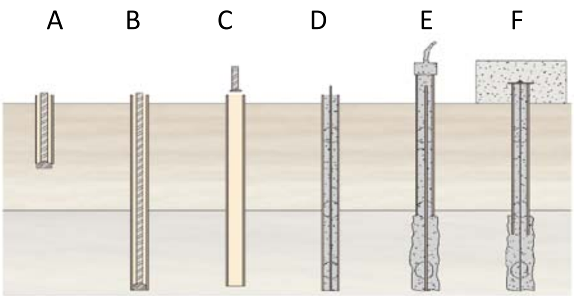

Fig. 1 Micropile installation sequence (Courtesy: US Department of Transportation publication: SA-97-070)

Constructions stage A to F are as follows;
A & B: drilling & / or installation of temporary casing
C: Removing inner drill bit and rod (if used)
D: Place reinforcement and grout (by tremie)
E: Remove temporary casing and inject further grout under pressure
F: Compile pile casting

4 LOAD CARRYING CAPACITY OF MICROPILES

4.1 Geotechnical capacity of micro pile

Load carrying capacity by the shaft of a micro pile, in compression, was estimated using;

$$A_s \times f_{s,all} \tag{1}$$

where A_s: pile surface area
$f_{s,all}$: allowable ground-grout resistance

when calculating A_s , the diameter of grouted column was assumed to be 1.4 times the diameter of the borehole, for piles in clay and sand although the diameter of the grout column in the latter would be more than that in the clay. This is due to the reason that pressure grouting causes re-compaction and re-densification of surrounding soil increasing the effective diameter.

Table 4 gives the allowable ground-grout resistances attributed to different layers encountered and detailed in Table 2.

Table 4.Allowable Ground-Grout Resistances

Layer No	Description	$f_{s,all}$
I	Peat, soft organic clay or loose or very loose sand / silt.	None
II,III	Completely to Moderately Weathered Rock	90 kPa
IV	Slightly Weathered to Fresh Rock	300 kPa

The end bearing capacity of the pile was determined as:

$$A_{end} \times p_{end,all} \tag{2}$$

where, A_{end}: area of pile toe,
P_{endall}: allowable end bearing resistance,

An allowable end bearing resistance of 3MPa was attributed to Layer IV based on the author's experience on allowable bearing capacity of similar rocks.
When the pile is in tension, the allowable ground-grout resistance was assumed to be 50% of resistance provided under compression in absence of prior field test data.

4.2 Structural capacities of micro pile

Design was carried out in accordance with the German Code of Practice DIN 18800 Part 1 & II. The partial safety factors considered for load (γ_F) and material (γ_M) were 1.4 and 1.1, respectively.

When the pile is not subjected to buckling, allowable structural capacity (P_k) of the pile in compression was determined using equation- 3 below;

$$P_k \leq \frac{A \times f_{yk}}{\gamma_F \times \gamma_M} \qquad (3)$$

where, A-Area of the tube, f_{yk}- material strength

When piles were subjected to tension, the structural capacity under tension was taken as 50% of the compression capacity considering the reduction of steel area by approximately 50%.

When the pile is subjected to buckling, the ultimate structural capacity (N_d) was determined as:

$$\frac{N_d}{\chi \times N_{pl,d}} \leq 1$$
$$\max N_d = \chi \times N_{pl,d} \qquad (4)$$

Thus,
Permissible structural capacity;

$$perm\ N = \max N_d / \gamma_F$$

Where; $N_{pl,d}$: design axial force in plastic state, χ = Reduction factor according to the standard buckling curves.
γ_F = Factor of safety
Reduction factor χ was determined based on equations (5a to 5c);

$$\chi = 1 when \bar{\lambda}_K \leq 0.2 \qquad (5a)$$

$$\chi = \frac{1}{k + \sqrt{k^2 - \bar{\lambda}^2_k}} when 0.2 < \bar{\lambda}_K < 3.0 \qquad (5b)$$

$$\chi = \frac{1}{\bar{\lambda}_k (\bar{\lambda}_K + \alpha)} when \bar{\lambda}_K > 3.0: \qquad (5c)$$

where,

$$k = 0.5[1 + \alpha(\bar{\lambda}_k - 0.2) + \bar{\lambda}_k^2]$$

$$\bar{\lambda}_K = \frac{\lambda_K}{\lambda_a} \qquad \lambda_k = \frac{L}{i} \qquad i = \sqrt{\frac{I}{A}} \qquad \lambda_a = \pi\sqrt{\frac{E}{f_{y,k}}}$$

I = Second moment of area,
i = Radius of gyration L= effective length = pile length within the very weak layer with SPT N<3,
λ_k = slenderness ratio,
λ_a = reference slenderness ratio,

where, *l*= thickness of weak soil layer in metre

The parameter α was taken as 0.76 pertaining to 'd' type member in Table 5 & 6, which associated with the worst imperfection buckling curve type 'd', as given in Table 4 of DIN18800 Part2.

Table 5. Bow imperfection

		Type of Member	Bow imperfection, ω_0 , υ_0
1	a	Solid member	l/300
2	b	Solid member	l/250
3	c	Solid member	l/200
4	d	Solid member	l/150
5	e	Build up member	l/500

Table 6. α values (Table 4 of DIN1880)

Type of Member	a	b	c	d
Reduction factor, α	0.21	0.34	0.49	0.76

Accordingly, structural capacities for piles of 114.3mm diameter were determined as per Table 7;

Table 7. Permissible Structural capacity (PermN) for 114.3 mm diameter tube piles

L	λk	k	χ	max N_d (kN)	Perm N (kN)
0.75	0.33	0.61	0.90	1500	1000
1.00	0.44	0.69	0.82	1366	976
1.50	0.67	0.90	0.67	1111	793
2.50	1.11	1.46	0.41	692	494

Similarly, the permissible structural capacity of composite piles was determined and the results are given in Table 8;

Table 8. Resulted permissible structural capacity of composite piles

L	Perm N (kN)			
	Type1	Type2	Type3	Type4
4.5	1000	1000	1000	1000
5.0	1000	901	1000	988
5.5	933	804	1000	880
6.0	821	719	895	786

The foundation system of Pier 5 consists of a four-pile group, which internally comprise a group of vertical and battered piles as illustrated in Fig.2

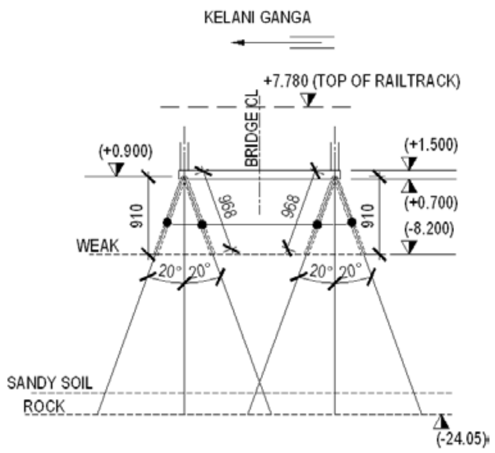

Fig. 2 Cross section of Pile foundation of Pier 5

4.3 Verification of capacity of micropiles

A static compression pile load test was carried out to verify the load bearing capacity of a micro pile (estimated incorporating the various adoptions made as detailed in the preceding sections

The test was carried out at Land Pier 5 of Kelaniya Bridge in accordance with ASTM D1143, with a kentledge arrangement. At this location, the working load considered was 900 kN. The thicknesses of layers I, II, III, and IV were considered to be 8.3 m, 11.85 m, 11.85 m, and 1 m, respectively. The thickness of the very weak layer, L (i.e. SPT N < 3) was 5.5m. Accordingly, the geotechnical and structural capacities of 1140kN and 933kN were estimated for a 21.15 m long Type 1 composite pile.

The test was planned for three loading cycles corresponding to 1 time, 1.5 times, and 2 times the working load. Fig. 3 shows the variation of settlement with load.

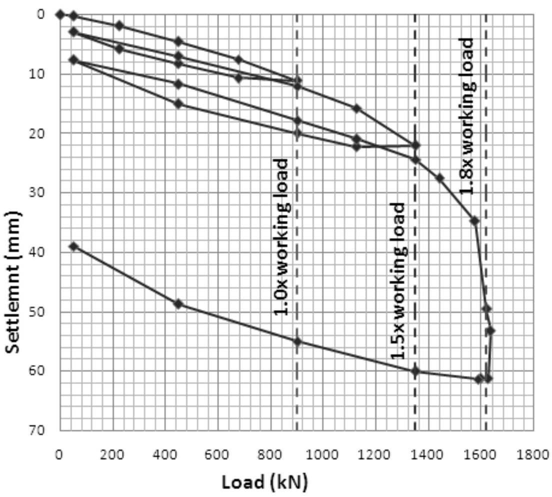

Fig. 3 Static load test result.

For the load cycle up to 1.0 times the working load, a gross settlement of 11.11 mm and a net settlement of 2.93mm at 42kN were observed. For the load cycle up to 1.5 times those were 22.07 mm and 7.69 mm, respectively. The respective gross and net settlement values obtained for the two loading cycles described above were within the performance specification of 12 mm and 6mm; 25mm and 12mm.

While loading beyond 1.8 times the working load, the ultimate condition with a sudden and sharp increase in settlement without any further increase in load was reached resembling structural failure of the micropile.

Based on the above observations which were accompanied by a lateral movement of the loading jack, it was inferred that the micropile had failed by buckling.

The failure load is more than 25% of the predicted maximum structural capacity, yet it is less than the ultimate geotechnical capacity. This indicates that the structural capacity of the micropile governs the load carrying ability.

5 CONCLUSIONS

A drilled and grouted non-displacement 168 mm diameter micropile reinforced with steel tubes with a characteristic yield strength, f_{yk}, of 560 MPa, is generally adopted for new railway bridge structures.

An external steel tube with a diameter/thickness of 219.1mm/10mm was used to avoid buckling of the micropile when weak surface layers were encountered

Theoretical predictions validated by a full scale static load test indicated that the load carrying ability of the micropile is governed by the structural capacity of the micropile.

ACKNOWLEDGMENT

The Authors wish to thank the Sri Lanka Railways for providing an opportunity to the Central Engineering Consultancy Bureau to take part in the Project as the Consultant towards ensuring safe and sound railway infrastructure.

REFERENCES

US Department of Transportation (2000), Micro pile design and construction guild lines; Pages379, Publication No:FHWA-SA-97-070 ,pp1-4

ASTM D1143 (2007), Standard Test Method for Deep Foundation Under Static Axial Compressive Load, Annual Book of ASTM Standard.