COMPARISON OF MALAYSIAN PRACTICE WITH EC7 ON THE DESIGN OF DRIVEN PILE AND BORED PILE FOUNDATIONS UNDER AXIAL COMPRESSION LOAD

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ABSTRACT: This paper presents commonly used design methodologies for driven pile and bored pile foundation in Malaysia. In Malaysia, empirical equations to predict ultimate shaft resistance (f_{su}) and the ultimate base resistance (f_{bu}) are commonly correlated to Standard Penetration Tests (SPT) 'N' values as they are extensively carried during subsurface investigation (S.I.) works at the site. Comparisons of the current Malaysian practice are made with EC7 methodologies with some suggested values of partial factors for development of EC7 Malaysian National Annex (MY-NA) for pile foundation under axial compression load, are presented. Finally results of pile load tests are also presented for verification of the suggested EC7 Malaysian National Annex values.

1.0 Introduction

Displacement driven piles, namely spun piles and RC square piles and cast-in-situ bored piles are commonly used in Malaysia as foundation to support for heavily loaded structures such as high-rise buildings and bridges in view of their flexibility of sizes to suit different loads, subsoil conditions and availability of many experienced foundation contractors to carry out the works. This paper presents commonly used design methodologies for driven pile and bored pile foundations in Malaysia. Comparisons are also made with the EC7 methodologies especially on the partial factors to be adopted together with some suggested values for development of EC7 Malaysian National Annex for pile foundations under axial compression loads. Finally results of pile load tests are also presented for verification of suggested EC7 Malaysian National Annex (MY-NA) values.

2.0 Malaysian Conventional Design Practice for Geotechnical Capacity of Piles

2.1 Factor of Safety

In Malaysia, the Factors of Safety (FOS) normally used in static calculation of pile geotechnical capacity are partial FOS on shaft (F_s) and base (F_b) respectively; and the global FOS (F_g) on total capacity. The lower geotechnical capacity obtained from both methods using the following equations is adopted as allowable geotechnical capacity

$$Q_{ag} = \frac{Q_{su}}{F_s} + \frac{Q_{bu}}{F_b} \qquad \text{(eq.1)}$$

$$Q_{ag} = \frac{Q_{su} + Q_{bu}}{F_g} \qquad \text{(eq.2)}$$

Note: Use the lower of Q_{ag} obtained from eq. 1 and eq. 2 above.

Where:

Q_{ag} = Allowable geotechnical capacity

Q_{su} = Ultimate shaft capacity = $\sum_i (f_{su} \times A_S)$

i = Number of soil layers

Q_{bu} = Ultimate base capacity = $f_{bu} A_b$

f_s = Unit shaft resistance for each layer of embedded soil

f_b = Unit base resistance for the bearing layer of soil

A_s = Pile shaft area

A_b = Pile base area

F_s = Partial Factor of Safety for Shaft Resistance (generally 1.5)

F_b = Partial Factor of Safety for Base Resistance (generally 3.0)

F_g = Global Factor of Safety for Total Resistance (Base + Shaft) generally 2.0

For the general practice in Malaysia, contribution of base resistance in bored piles is ignored due to the difficulty of proper base cleaning especially in wet holes (with drilling fluid). The contribution of base resistance can only be used if it is constructed in shallow dry holes where proper inspection of the base can be carried out, or base grouting is implemented or with fully instrumented preliminary pile loaded to failure and ultimate base capacity verified on site. Therefore, special attention should be given when designing base resistance for bored piles.

2.2 Design of Geotechnical Capacity in Soil

The design of pile geotechnical capacity is divided into two major categories namely:

a) Semi-empirical Method
b) Simplified Soil Mechanics Method

2.2.1 Semi-empirical Method

Piles installed in tropical residual soils are generally complex in soil characteristics. The complexity of these founding mediums with significant changes in ground properties over short distance and the friable nature of the materials make undisturbed sampling and laboratory strength and stiffness testing of the material difficult. Furthermore current theoretically based formulae also do not consider the effect of soil disturbance, stress relief and partial reestablishment of ground stresses that occur during the construction of piles; therefore, the sophistication involved in using such formulae may not be necessary.

Semi-empirical correlations have been extensively developed relating both shaft resistance and base resistance of piles to N-values from Standard Penetration Tests (SPT'N' values) (Tan & Chow, 2003). In the correlations established, the SPT'N' values generally refer to uncorrected values before pile installation.

The commonly used correlations for piles are as follows:

$$f_{su} = K_{su} \times \text{SPT'N'} \text{ (in kPa)}$$
$$f_{bu} = K_{bu} \times \text{SPT'N'} \text{ (in kPa)}$$

Where:

K_{su} = Ultimate shaft resistance factor
K_{bu} = Ultimate base resistance factor
SPT'N' = Standard Penetration Tests blow counts (blows/300mm)

For shaft resistance of bored piles, Tan et al. (1998), used the results of 13 fully instrumented bored piles in residual soils, presented K_{su} of 2.6 but limiting the f_{su} values to 200kPa. Toh et al. (1989) also reported that the average K_{su} obtained varies from 5 at SPT'N' 20 to as low as 1.5 at SPT'N'=220. Chang & Broms (1991) suggested K_{su} of 2 for bored piles in residual soils of Singapore with SPT'N'<150.

For base resistance, K_{bu} values reported by many researchers vary significantly indicating difficulty in obtaining proper and consistent base cleaning during construction of bored piles. It is very dangerous if the base resistance is relied upon when the proper cleaning of the base cannot be assured. From back-analyses of test piles, Chang & Broms (1991) showed that K_{bu} was 30 to 45 and Toh et al. (1989) reports that K_{bu} ranged between 27 and 60 as obtained from the two piles that were tested to failure.

Lower values of K_{bu} between 7 and 10 were reported by Tan et al. (1998). The relatively low K_{bu} values are most probably due to the soft toe effect which is very much dependent on the type of soil, workmanship and pile geometry. This is even more significant in long pile. Furthermore, a relatively larger base movement is required to mobilise the maximum base resistance as compared to the displacement needed to fully mobilise shaft resistance. The base displacement of approximately 5% to 10% of the pile diameter is generally required to mobilise the ultimate base resistance provided that the base is properly cleaned. However in the last few years, there has been a trend of increasing base and shaft resistance factors due to the improvement of machinery used and shorter construction times for each pile.

In view of the large movement required to mobilise the base resistance of bored piles and the difficulty in base cleaning, the authors strongly recommend to ignore the base contribution in the bored pile design unless proper base cleaning can be assured and verified by load tests.

For driven piles, the ultimate shaft resistance factor, K_{su} generally ranges from 2.0 to 3.0 depending on the size of piles, materials of pile, soil strength/stiffness (e.g. SPT'N' values) and soil type. Commonly K_{su} of 2.5 is used for preliminary design prior to load tests. Ultimate base resistance factors, K_{bu} for driven piles are tabulated in Table 1.

Table 1 Correlation between ultimate base resistance factor with soil type.

SOIL TYPE	K_{bu}	REFERENCES
Gravels	500 to 600	Authors local experiences
Sand	400(1) to 450(2)	(1)Decourt (1982) (2)Martin et al.(1987)
Silt, Sandy Silt	250(1) to 350(2)	(1)Decourt (1982) for residual sandy silts (2)Martin et al.(1987) for silt & sandy silt
Clayey Silt	200	Decourt (1982) for residual clayey silt
Clay	120(1) to 200(2)	(1)Decourt (1982) (2)Martin et al.(1987)
Note : $f_{bu} = K_{bu}$ x SPT'N' (in kPa)		

2.2.2 Simplified Soil Mechanics Methods

Generally the simplified soil mechanics methods for pile design can be classified into cohesive soils (e.g. clays, silts) and cohesionless soils (e.g. sands and gravels).

Cohesive Soils

The ultimate shaft resistance (f_{su}) of piles in cohesive soils can be estimated based on the undrained shear strength method as follows:

$$f_{su} = \alpha \text{ x } s_u$$

Where :

α = adhesion factor

s_u = undrained shear strength (kPa)

Whitaker & Cooke (1966) reported that the α value lies in the range of 0.3 to 0.6 for stiff over-consolidated clays, while Tomlinson (1994) and Reese & O'Neill (1988) reported α values in the range of 0.4 to 0.9. The α values for residual soils of Malaysia are also within this range as shown in Figure 1. Where soft clay is encountered, a preliminary α value of 0.8 to 1.0 is usually adopted together with the corrected undrained shear strength from the vane shear test. This α method is useful if the piles are to be constructed on soft clay near rivers or at coastal areas. The value of α to be used should be verified by preliminary pile load test.

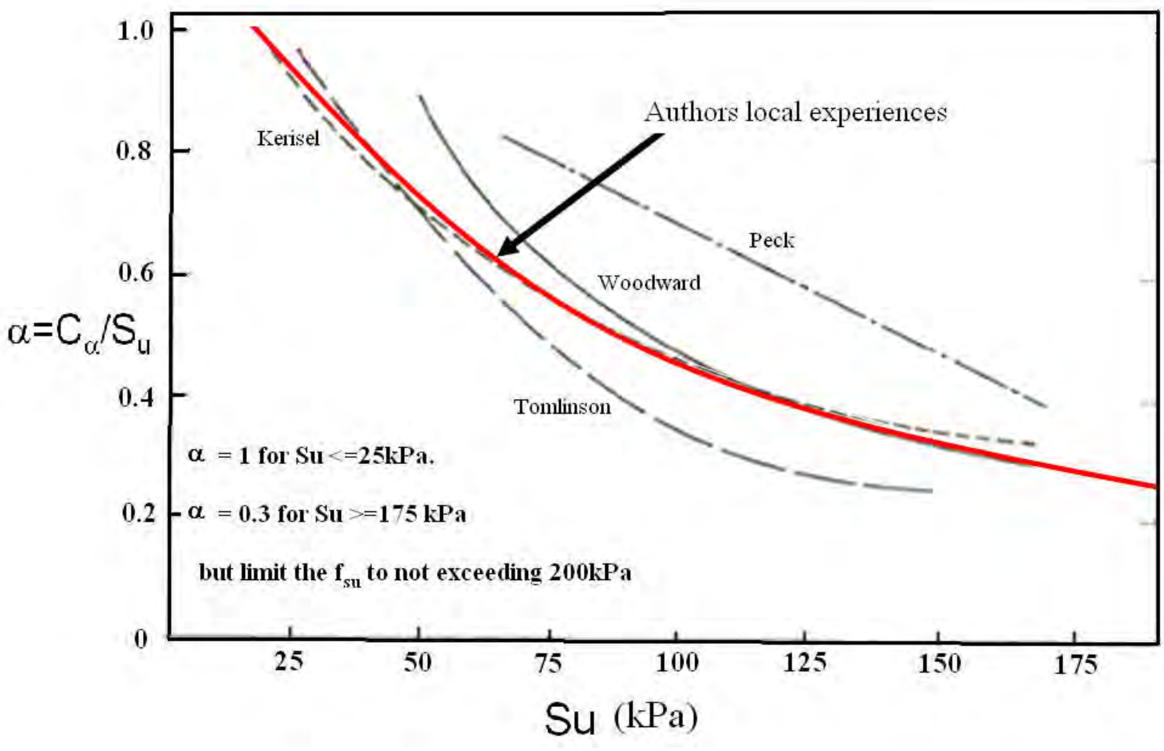

Figure 1 Adhesion factors for driven piles in clay (modified from McClelland, 1974)

Meanwhile, ultimate base resistance for piles in cohesive soil can be related to undrained shear strength as follows;

$$f_{bu} = N_c \text{ x } s_u$$

Where:

N_c = bearing capacity factor = 9

However, it is not recommended to include base resistance in the calculation of the bored pile geotechnical capacity due to difficulty and uncertainty in base cleaning.

In the case where piles are subjected to significant variations in stress levels after installation (e.g. excavation for basements, rise in groundwater table) the use of the effective stress method is more representative as compared to the undrained method. This is because the effective stress can take account of the effect of effective stress change on the K_{se} values. The value of ultimate shaft resistance can be estimated from the following expression:

$$f_{su} = (K_{se} \tan \phi') \sigma_v'$$

Where :

K_{se} = Effective Stress Shaft Resistance Factor = [can be assumed as K_o]

σ_v' = Vertical Effective Stress (kPa)

ϕ' = Effective Angle of Friction (degree) of cohesive soils.

However, this method is not popular in Malaysia and limited case histories of back-analysed K_{se} values are available for reliable and practical usage by the design engineer.

Cohesionless soils

The ultimate shaft resistance (f_{su}) of bored piles in cohesionless soils can be expressed in terms of effective stresses as follows:

$$f_{su} = \beta \text{ x } \sigma_v'$$

Where:

β = shaft resistance factor for cohesionless soils.

The β values can be obtained from back-analyses of pile load tests. The typical β values of bored piles in loose sand and dense sand are 0.15 to 0.3 and 0.25 to 0.6 respectively based on Davies & Chan (1981).

Meanwhile, the theoretical ultimate base resistance for piles in cohesionless soil can be related to effective stresses as follows;

$$f_{bu} = N_q \times \sigma_b'$$

Where:

N_q = bearing capacity factor

σ_b' = Effective overburden pressure at pile base (kPa)

Although the theoretical ultimate base resistance for bored piles in cohesionless soil can be related to plasticity theories, it is not recommended to be included in the calculation of the bored pile geotechnical capacity due to difficulty and uncertainty in base cleaning.

2.3 Design of Geotechnical Capacity in Rock for Bored Piles

The three major rock formations, namely sedimentary, igneous and metamorphic rocks, are commonly encountered in Malaysia. When designing structures over these formations using bored pile, the design approaches vary significantly depending on the formations and the local experience established on a particular formation.

In Malaysia, bored pile design in rocks is mostly based on the semi-empirical method. Generally, the design rock socket friction is the function of surface roughness of the rock socket, unconfined compressive strength of intact rock, confining stiffness around the socket in relation to fractures of rock mass and socket diameter, and the geometry ratio of socket length-to-diameter. Roughness is an important factor in rock socket pile design as it has significant effect on the normal contact stress at the socket interface during shearing. The normal contact stress increases due to dilation resulting increase of socket friction. The level of dilation is mostly governed by the socket roughness. The second factor on the intact rock strength governs the ability of the irregular asperity of the socket interface transferring the shear force, otherwise shearing through the irregular asperity will occur due to highly concentrated shear forces from the socket. The third factor will govern the overall performance of strength and stiffness of the rock socket in jointed or fractured rock mass and the last factor is controlled by the profile of socket friction distribution.

Table 2 Summary of Rock Socket Friction Design Values (updated from Tan & Chow, 2003)

Rock Formation	Working Rock Socket Friction*	Source
Limestone	300kPa for RQD < 30% 400kPa for RQD = 30 – 40% 500kPa for RQD =40 – 55% 600kPa for RQD =55 – 70% 700kPa for RQD =70 – 85% 800kPa for RQD > 85% The above design values are subject to 0.05x minimum of {q_{uc}, f_{cu}} whichever is smaller.	Authors
Sandstone	$0.10 \times q_{uc}$	Thorne (1977)
Shale	$0.05 \times q_{uc}$	Thorne (1977)
Granite	1000 – 1500kPa for q_{uc} > 30N/mm^2	Tan & Chow (2003)

Where:

RQD = Rock Quality Designation

q_{uc} = Unconfined Compressive Strength of rock

f_{cu} = Concrete grade

It is very complicated to quantify all these aspects in the rock socket pile design. Therefore, based on the conservative approach and local experience, some semi-empirical methods have evolved to facilitate the quick socket design with consideration to all these aspects. In most cases, roughness of socket is qualitatively considered as a result of a lack of systematic assessing methods. Whereas the other three factors can be quantified through strength tests on the rock cores and point load tests on the recovered fragments, the RQD values of the core samples and some analytical method of assessing the socket friction distribution. It is also customary to perform working load tests to verify the rock socket design using such semi-empirical method. Safety factor of two is the common requirement for rock socket pile design. Table 2 summarises the typical design socket friction values for various rock formations in Malaysia.

Sometimes, the borehole is a dry hole and at shallow depth, then base resistance will be considered if the base cleaning and inspection of the base condition can be carried out. Very often, the movement to mobilise base resistance is a few times higher than that to mobilise the socket friction, despite the ultimate base resistance possibly being very high. As such, with consideration of compatibility of the pile movement in mobilising both the socket and base, appropriate mobilising factors to both the socket and base should be applied to the foundation design after verification from the fully instrumented pile load test. Such mobilising factors should be at least 3, but finally subjected to verification by instrumented load tests prior to production of working piles if there is a large number of piles for value engineering. The assessment of ultimate end bearing capacity of bored piles in rock can be carried out using the following expression.

$$Q_{ub} = cN_c + \gamma BN_\gamma/2 + \gamma DN_q$$

Where:

c = Cohesion

B = Pile diameter

D = Depth of pile base below rock surface

γ = Effective density of rock mass

N_c, N_γ & N_q = Bearing capacity factors related to friction angle, ϕ (Table 3, for circular case, multipliers of 1.2 & 0.7 shall be applied to N_c & N_γ respectively)

N_c = $2N_\phi^{1/2}(N_\phi+1)$

N_γ = $N_\phi^{1/2}(N_\phi^2-1)$

N_q = N_ϕ^2

N_ϕ = $\mathrm{Tan}^2(45°+\phi/2)$

Table 3 Typical Friction Angle for Intact Rock (Wyllie, 1991)

Classification	Type	Friction Angle
Low Friction	Schist (with high mica content), Shale	20° - 27°
Medium Friction	Sandstone, Siltstone, Gneiss	27° - 34°
High Friction	Granite	34° - 40°

Alternatively, the allowable rock bearing pressure can be estimated from the empirical correlation recommended by the Canadian Foundation Engineering Manual (Canadian, 1992):

$$q_a = K_{sp} \times q_{u\text{-}core}$$

where

q_a = Allowable bearing pressure

$q_{u\text{-}core}$ = Average unconfined compressive strength of rock

K_{sp} = Empirical coefficient, which includes a factor of 3 and ranges from 0.1 to 0.4 (Table 4 for K_{sp} value at respective spacing of discontinuities)

Table 4 Coefficients of Discontinuity Spacing (Canadian, 1992)

Spacing of Discontinuities	Spacing Width (m)	K_{sp}
Moderately close	0.3 - 1	0.1
Wide	1 - 3	0.25
Very wide	> 3	0.4

If the pile length is significant, the contribution of the shaft resistance in the soil embedment above the rock socket should also be considered in the overall pile resistance assessment. In most cases of rock socket piles, the settlement performance is usually governed by the elastic shortening of the pile shaft. The socket displacement is usually insignificant. However, load transfer analyses would provide the overall settlement performance.

Construction method is another important aspect to be considered in the bored pile design on rock. In Malaysia, there are two most commonly-used methods for forming the rock socket, namely rock coring with rock cutting bits and chiselling by mechanical impact. Both methods have their own merits and need skilful operators to form a proper rock sockets. In general, the rock coring method forms smoother, but intact, socket surfaces. On the other hand, the chiselling method forms relatively rougher sockets, which could be more fracture due to disturbance to the inherent discontinuities in bedrock. Chiselling is usually used as a supplementary technique in drilling through hard rock.

There are also other inherent problems associated with some of the aforementioned rock formations such as:

a. Limestone: The existence of erratic karst features will need further consideration in the foundation pile design. Downgrading of pile capacity for piles founded on these karst features or installing the pile at deeper depth to penetrate these features or treatment to strengthen them can be considered depending on the cost-benefit analyses of the viable options. Another problem in limestone formation is the existence of slime made of very soft soil materials immediately above the bedrock, which can cause frequent cave-ins and pose difficulties in cleaning up the rock socket. Gue (1999) presented some solutions to overcome the abovementioned problems and the construction controls.
b. Degradable sedimentary formations: These formations are easily subject to rapid degradation in terms of strength and stiffness as a result of stress relief and ingression of drilling fluid. Slow progress in drilling operations due to inefficient coring methods or inter-layered hard and soft rocks and delay in concreting the piles will further aggravate the capacity of the socket. The solutions to these problems are to use powerful drilling equipment and avoid delays in concreting.
c. Granite: Core boulders are common features in this formation. This feature can be easily observed from the outcrops or along river. Therefore, it is important to identify proper founding stratum for the foundation piles during the subsurface investigation. This can be overcome by careful assessment of the weathering profile interpreted from the deep boring exploratory holes.

3.0 Concept for the Application of EC7 for Geotechnical Design of Pile Foundation under Compression Load in Malaysia

The application of EC7 for pile design in Malaysia needs rationalization and harmonization with current established local practices that have been serving the construction industry well as there was no reported catastrophic failure of buildings or bridges due to the failure of pile foundations under compression.

Following are the main criteria that require rationalization and harmonization for application of EC7 in Malaysia for geotechnical design of pile foundations under compression load:

a) An understanding of the indirect comparison of load factors, partial factors and other model factors used in EC7 with conventional Factor of Safety which local engineers are familiar with.
b) The transformation of current Factor of Safety (FOS) on shaft (F_s) and base (F_b) and global FOS (F_g) on total capacity to partial factors and other model factors to be used in the Malaysian National Annex (MY-NA). The suggested Malaysian National Annex will be compared with EC7 Annex A (normative) and UK National Annex to EC7 (UK-NA).
c) A clear distinction between the partial factors on resistance for shaft and base which are mobilized at different magnitudes of displacement respectively.
d) Requirements of pile testing, especially static load tests and dynamic load tests on preliminary piles (sacrificial pile) that are to be loaded to failure and also on working piles which are to be loaded to a designed test load.

3.1 Concept of Different Partial Factors of Safety for Shaft and Base

In conventional design practice in Malaysia, the Factors of Safety (FOS) normally used in static evaluation of pile geotechnical capacity are partial FOS on shaft (F_s) and base (F_b) respectively; and the global FOS (F_g) on total capacity (as described in Section 2.1 above). The lower geotechnical capacity obtained from both methods is adopted as allowable geotechnical capacity, as follows:

For **Driven Piles** (under Compression Load) :

$$Q_{ag} = \frac{Q_{su}}{F_s} + \frac{Q_{bu}}{F_b} = \frac{Q_{su}}{1.5} + \frac{Q_{bu}}{3} \qquad \text{(eq.3)}$$

$$Q_{ag} = \frac{Q_{su} + Q_{bu}}{F_g} = \frac{Q_{su} + Q_{bu}}{2} \qquad \text{(eq.4)}$$

Note: Use the lower of Q_{ag} obtained from eq. 3 and eq. 4 above.

For **Bored Piles** (under Compression Load) :

(i) For bored piles constructed with drilling fluid and without base grouting:

$$Q_{ag} = \frac{Q_{su}}{F_s} + \frac{Q_{bu}}{F_b} = \frac{Q_{su}}{1.5} \text{ (ignore base)} \qquad \text{(eq.5)}$$

$$Q_{ag} = \frac{Q_{su} + Q_{bu}}{F_g} = \frac{Q_{su}}{2} \text{ (ignore base)} \qquad \text{(eq.6)}$$

Therefore, for bored piles eq.6 governs.

(ii) For bored piles constructed in dry holes, or with base grouting, or with fully instrumented preliminary piles loaded to failure and ultimate base capacity verified on site:

$$Q_{ag} = \frac{Q_{su}}{F_s} + \frac{Q_{bu}}{F_b} = \frac{Q_{su}}{1.5} + \frac{Q_{bu}}{3} \qquad \text{(eq.7)}$$

$$Q_{ag} = \frac{Q_{su} + Q_{bu}}{F_g} = \frac{Q_{su} + Q_{bu}}{2} \qquad \text{(eq.8)}$$

Note: Use the lower of Q_{ag} obtained from eq. 7 and eq. 8 above.

In view of the above, it is important that when drafting the Malaysian National Annex for EC7, the partial factors on resistance should be in line with current local practice. Different partial FOS shall be used for shaft and base as the displacement required to mobilize the shaft and base are different as reported in many literatures on piles.

3.2 Comparison of EC7 using Model Factor with Conventional Factor of Safety

Since Malaysian engineers are used to conventional FOS as described in Section 2.1 of this paper, it is important to convert partial factors for actions, soil materials, resistance and also model factors used in EC7 to the conventional FOS which local engineers are familiar with for comparison, despite the fact that the conversion may be indirect with assumption of the ratio of permanent load (e.g. dead load) and variable (e.g. life load) load. Design Approach 1 and Design Approach 2 are referred for comparison.

Tables 5 and 6 summarised the partial factors for actions, soil materials and resistance extracted from EN1997-1:2004 Annex A and UK national Annex to EN1997-1:2004 respectively. UK National Annex (UK-NA) only applies Design Approach 1 (as stated in NA to BS EN 1997-1:2004, page 2). Complying with EN1997-2004, 2.4.1(6), UK-NA also recommends a model factor to be applied to resistances calculated using characteristic values of soil properties. In UK-NA, the value of the model factor should be **1.4**, except that it may be reduced to **1.2** if the resistance is verified by a static load test taken to the calculated, unfactored ultimate resistance (e.g. failure load).

Table 5 Summary of Partial Factors for Actions, Soil Materials and Resistance extracted from EN1997-1:2004 Annex A.

			Design Approach 1							**Design Approach 2**		
			Combination 1			**Combination 2 – piles & anchors**				**Combination 1**		
			A1	**M1**	**R1**	**A2**	**M1 or**	**M2**	**R4**	**A1**	**M1**	**R2**
Actions	**Permanent**	**Unfav**	**1.35**			**1.00**				**1.35**		
		Fav	**1.00**			**1.00**				**1.00**		
	Variable	**Unfav**	**1.50**			**1.30**				**1.50**		
Soil	**tan ϕ'**			**1.00**			**1.00**	**1.25**			**1.00**	
	Effective cohesion			**1.00**			**1.00**	**1.25**			**1.00**	
	Undrained strength			**1.00**			**1.00**	**1.40**			**1.00**	
	Unconfined strength			**1.00**			**1.00**	**1.40**			**1.00**	
	Weight density			**1.00**			**1.00**	**1.00**			**1.00**	
Driven piles	**Base**				**1.00**				**1.30**			**1.10**
	Shaft (compression)				**1.00**				**1.30**			**1.10**
	Total / combined				**1.00**				**1.30**			**1.10**
Bored piles	**Base**				**1.25**				**1.60**			**1.10**
	Shaft (compression)				**1.00**				**1.30**			**1.10**
	Total / combined				**1.15**				**1.50**			**1.10**

A ***model factor*** should be applied to the shaft and base resistance calculated using characteristic values of soil properties by a method complying with EN1997-1, **2.4.1**(6). The value of the ***model factor*** should be **1.4**, except that it may be reduced to **1.2** if the resistance is verified by a static load test taken to the calculated, unfactored ultimate resistance. (extracted from NA to BS EN 1997-1:2004 page 11)

The major differences between Annex A in EC7 and UK-NA are the partial factors used for shaft, base and also total/combined resistance (capacity). UK-NA introduces lower partial factors if there is explicit verification of Serviceability Limit State (SLS) with the following requirements:

a) if serviceability is verified by load tests (preliminary and/or working) carried out on more than 1% of the constructed piles to loads not less than 1.5 times the representative load for which they are designed, OR
b) if settlement is explicitly predicted by a means no less reliable than in (a), OR
c) if settlement at the serviceability limit state is of no concern

Table 6 Summary of Partial Factors for Actions, Soil Materials and Resistance extracted from UK National Annex EN1997-1:2004.

			Design Approach 1								
			Combination 1			Combination 2 – piles and anchors					
						WITHOUT explicit verification of SLS[A)]			WITH explicit verification of SLS[A)]		
			A1	M1	R1	A2	M1	R4	A1	M1	R4
Actions	**Permanent**	**Unfav**	1.35			1.00			1.00		
		Fav	1.00			1.00			1.00		
	Variable	**Unfav**	1.50			1.30			1.30		
Soil	**tan ϕ'**			1.00			1.00			1.00	
	Effective cohesion			1.00			1.00			1.00	
	Undrained strength			1.00			1.00			1.00	
	Unconfined strength			1.00			1.00			1.00	
	Weight density			1.00			1.00			1.00	
Driven piles	**Base**				1.00			1.70			1.50
	Shaft (compression)				1.00			1.50			1.30
	Total / combined				1.00			1.70			1.50
Bored piles	**Base**				1.00			2.00			1.70
	Shaft (compression)				1.00			1.60			1.40
	Total / combined				1.00			2.00			1.70

[A)] The lower partial factor of safety in R4 may be adopted

a) if serviceability is verified by load tests (preliminary and/or working) carried out on more than 1% of the constructed piles to loads not less than 1.5 times the representative load for which they are designed, OR

b) if settlement is explicitly predicted by a means no less reliable than in (a), OR

c) if settlement at the serviceability limit state is of no concern

A ***model factor*** should be applied to the shaft and base resistance calculated using characteristic values of soil properties by a method complying with EN1997-1, **2.4.1**(6). The value of the ***model factor*** should be **1.4**, except that it may be reduced to **1.2** if the resistance is verified by a static load test taken to the calculated, unfactored ultimate resistance. (Extracted from NA to BS EN 1997-1:2004 page 11)

For the easy reference of Malaysian engineers who are familiar with the conventional Factor of Safety (FOS), the Authors have calculated the "indirect" FOS associated with each design approach listed in EC7 (Table 5) and UK-NA (Table 6) for comparison. Table 7 summarises the "indirect" FOS on shaft, base and combined for different design approaches in EC7 and UK-NA. The ratio of permanent load (e.g. dead load) to variable load (e.g. life load, etc.) is taken at 8:2 when calculating the "indirect" FOS. Generally, the "indirect" FOS for EC7 and UK-NA ranges from 1.65 to 2.97 for combined capacity compared to current Malaysia practice of 2.0. The "indirect" FOS for shaft capacity ranges from 1.65 to 2.37 while the "indirect" FOS for base capacity ranges from 1.65 to 2.97.

Table 7 Summary of "Indirect" Factor of Safety (FOS) calculated from EN1997-1:2004 Annex A and UK National Annex EN1997-1:2004.

Methodology / Indirect Factor Of Safety (FOS) values (for Comparison with Conventional Method)		DA1-C1	DA1-C2	DA2-C1	DA1-C2 UK-NA **WITHOUT explicit verification of SLS[A)]**	DA1-C2 UK-NA **WITH explicit verification of SLS[A)]**
Model Factor =1.4						
Driven Pile (Compression)	Base FOS	1.93	1.93	2.13	2.52	2.23
	Shaft FOS	1.93	1.93	2.13	2.23	1.93
	Total/Combined FOS	1.93	1.93	2.13	2.52	2.23
Bored Pile (Compression)	Base FOS	2.42	2.37	2.13	2.97	2.52
	Shaft FOS	1.93	1.93	2.13	2.37	2.08
	Total/Combined FOS	2.22	2.23	2.13	2.97	2.52
Model Factor =1.2						
Driven Pile (Compression)	Base FOS	1.66	1.65	1.82	2.16	1.91
	Shaft FOS	1.66	1.65	1.82	1.91	1.65
	Total/Combined FOS	1.66	1.65	1.82	2.16	1.91
Bored Pile (Compression)	Base FOS	2.07	2.04	1.82	2.54	2.16
	Shaft FOS	1.66	1.65	1.82	2.04	1.78
	Total/Combined FOS	1.90	1.91	1.82	2.54	2.16

Where :
DA1-C1 = EN1997-1:2004 Design Approach 1 – Combination 1
DA1-C2 = EN1997-1:2004 Design Approach 1 – Combination 2 – Piles and Anchors
DA2-C1 = EN1997-1:2004 Design Approach 2 – Combination 1
DA1-C2 UK-NA = UK National Annex to EN1997-1:2004 Design Approach 1 – Combination 2 – Piles and Anchors

Note :
1. The Permanent (Dead Load) and Variable (Life Load) ratio of 80% : 20% is assumed in this table.
2. For DA1-C1 and DA2-C1; the Average FOS on Total Load =1.38;
3. For DA1-C2-Pile and Anchors; the Average FOS on Total Load = 1.06

It is also observed that for driven pile, EC7 Design Approach 1 and Design Approach 2 both adopted the same partial factors for shaft and base, thus yielding the same value of the calculated "indirect" FOS for both shaft and base. For bored pile, the same is observed for Design Approach 2. The Authors are of the opinion that this is against fundamental understanding of soil mechanics especially as the displacement required to mobilize shaft and base varies significantly. This is further confirmed by UK-NA which adopted a higher partial factor for base compared to shaft for Design Approach 1 – Combination 2 – Piles and Anchors.

Both EC7 and UK-NA also assume that the base capacity of all bored pile can be effectively mobilized, which is different from what was published for the base capacity of bored piles in Malaysia (Tan et al. 1998) which showed that for bored piles constructed with drilling fluid, the base capacity is very low and require large displacement to mobilise. Therefore, in Malaysia, it is advisable to ignore the contribution of the base capacity unless the bored pile is constructed in dry hole which can be inspected, or with base grouting, or with fully instrumented preliminary piles loaded to failure and ultimate base capacity verified on site.

EC7 and UK-NA generally allow lower partial factors which yielded lower "indirect" FOS if static load tests on preliminary piles to ultimate resistance are carried out on site to verify the load capacity. This is evident in UK-NA on the reduction of the model factor from 1.4 to 1.2 if there is preliminary pile static load test to unfactored ultimate resistance (e.g. failure load).

3.3 Suggestion on Partial Factors for the Malaysian National Annex of EC7 for Pile Foundations under Compression Load

When suggesting partial factors for the Malaysian National Annex (MY-NA) of EC7, it is important to take into consideration the following :

a) The partial factors should be in line with the current partial factor of safety (FOS) on shaft (F_s) and base (F_b) and the global FOS (F_g) on total capacity that have been extensively accepted and used in Malaysia.
b) There should be a clear distinction between the partial factor of safety for shaft and base which are mobilized at different magnitude of displacement.
c) Requirements for pile testing especially static and dynamic load tests on preliminary piles (sacrificial piles) which are to be loaded to failure and also working piles which are to be loaded to designed test load.
d) The adoption of the same Model Factor as in UK-NA.
e) The adoption of EC7 concept of allowing lower partial factor if more verification tests (e.g. static or dynamic load tests) are carried out at site.
f) Verified suggested partial factors with actual case histories to review the reliability of the suggested values. More case histories are needed before the values of partial factors for Malaysian National Annex can be finalized.
g) Complying to methodology as in EN1997-1, 7.6.2.3(8), where the characteristic values may be obtained by calculating:

$$R_{b;k} = A_b \, q_{b;k} \quad \text{and} \quad R_{s;k} = \sum_i A_{s;i} \cdot q_{s;i;k} \tag{7.9}$$

where $q_{b;k}$ and $q_{s;i;k}$ are characteristic values (in kPa) of base resistance and shaft friction in the various strata, obtained from values of soil/rock parameters. $R_{b;k}$ and $R_{s;k}$ are characteristic base and cumulative shaft capacity (in kN).

NOTE : To apply this procedure, the values of the partial factors for resistance such as base (γ_b), shaft (γ_s) and combined (γ_t) recommended may need to be corrected by a model factor which UK-NA recommended value is 1.4 and 1.2 respectively.

EC7 also has other methodologies as follows:

7.6.2.2 Ultimate compressive resistance from static load tests
7.6.2.3 Ultimate compressive resistance from ground test results (except 7.6.2.3(8))
7.6.2.4 Ultimate compressive resistance from dynamic impact tests
7.6.2.5 Ultimate compressive resistance by applying pile driving formulae

However, these methodologies will not be covered in this paper and will have to be addressed separately in the future.

Table 8 summarises the partial factors for actions, soil materials and resistance suggested for Malaysian National Annex (MY-NA) to EN1997-1:2004. Generally, partial factors for actions and soil materials suggested for Malaysian National Annex (MY-NA) follow the UK National Annex. The only suggested changes are on the partial factors for ***resistance***. The partial factors suggested for resistance will be in line conceptually with the Malaysian's conventional Factor of Safety (FOS) on shaft (F_s) and base (F_b) and the global FOS (F_g) on total capacity. When converting the partial factors to "indirect" FOS similar to Section 3.2, these values can be used indirectly to compare the conventional FOS commonly used in Malaysia.

Irrespective of which design approach is adopted in the design of piles, sufficient and properly planned subsurface investigation (S.I.), including field and laboratory tests, should be carried out to obtain representative subsoil conditions and parameters. Proper full time supervision of S.I. is also important to increase confidence levels in the information obtained. The Board of Engineers Malaysia (BEM) has issued a circular titled "Engineer's Responsibility for Subsurface Investigation" in 2005 which reminded all professional engineers that they are responsible for planning and supervision of the S.I. to be used in their design (in which they act as Submitting Engineer). Failure to do so contravenes the Part IV, Code of Professional Conduct of the Registration of Engineers Regulation (1990) (Amendment 2003) and calls for disciplinary action under the Registration of Engineers Act, Malaysia.

Table 9 summarises the "indirect" FOS on shaft, base and combined for different design approaches in EC7, UK-NA and the suggested Malaysian National Annex (MY-NA). Generally, the "indirect" FOS for the suggested MY-NA ranges from 1.65 to 2.37 for combined capacity compared to current Malaysia practice of 2.0. For "indirect" FOS for shaft capacity ranges from 1.27 to 2.23 while the "indirect" FOS for base capacity ranges from 1.82 to 3.26.

Table 8 Summary of Partial Factors for Actions, Soil Materials and Resistance suggested for the Malaysian National Annex (MY-NA) EN1997-1:2004.

			Design Approach 1								
			Combination 1			**Combination 2 – piles and anchors**					
						WITHOUT explicit verification of SLS[B)]			**WITH explicit verification of SLS[B)]**		
			A1	**M1**	**R1**	**A2**	**M1**	**R4**	**A1**	**M1**	**R4**
Actions	**Permanent**	**Unfav**	**1.35**			**1.00**			**1.00**		
		Fav	**1.00**			**1.00**			**1.00**		
	Variable	**Unfav**	**1.50**			**1.30**			**1.30**		
Soil	**tan ϕ'**			**1.00**			**1.00**			**1.00**	
	Effective cohesion			**1.00**			**1.00**			**1.00**	
	Undrained strength			**1.00**			**1.00**			**1.00**	
	Unconfined strength			**1.00**			**1.00**			**1.00**	
	Weight density			**1.00**			**1.00**			**1.00**	
Driven piles	**Base**				**1.1**			**1.9**			**1.8**
	Shaft (compression)				**1.0**			**1.5**			**1.0**
	Total / combined				**1.05**			**1.6**			**1.3**
Bored piles	**Base**				**1.2***			**2.2***			**1.8***
	Shaft (compression)				**1.0**			**1.5**			**1.1**
	Total / combined				**1.1**			**1.6**			**1.4 (1.3)****

B) The lower partial factor of safety in R4 may be adopted

a) if serviceability is verified by static load tests (preliminary and/or working) carried out on in accordance with the pile testing criteria listed in ***Table 10***$_{(MY\text{-}NA\ suggestion)}$, OR

b) if settlement is explicitly predicted by a means no less reliable than in (a), OR

c) if settlement at the serviceability limit state is of no concern

A ***model factor*** should be applied to the shaft and base resistance calculated using characteristic values of soil properties by a method complying with EN1997-1, **2.4.1**(6). The value of the ***model factor*** should be **1.4**, except that it may be reduced to **1.2** if the resistance is verified by a static load test taken to the calculated, unfactored ultimate resistance. (To follow NA to BS EN 1997-1:2004)

***** For bored pile design, the base resistance should be ignored (not included in calculation) unless for bored pile constructed in dry hole, or with base grouting, or with fully instrumented preliminary pile loaded to failure and ultimate base capacity verified on site.

** (1.3) = Partial factors for Total/Combined capacity of bored pile can be reduced to 1.3 if base is ignored in the calculation of the total/combined capacity.

Irrespective of design approach, proper and sufficient pile load verification tests should be carried out such as static load tests, dynamic load tests and sonic logging (for bored piles) to verify the acceptance of the pile.

Another suggestion for MY-NA which is different from UK-NA is on one of the requirement to satisfy the Serviceability Limit State (SLS). For MY-NA, it is suggested that serviceability is verified by static load tests (preliminary and/or working) carried out as per requirements listed in Table 10 to loads not less than 1.5 times the representative load for which they are designed instead of 1% used in UK-NA. However, the suggested MY-NA requires to carry out dynamic load tests as in Table 10 is not specified in UK-NA. This change is necessary as the pile testing requirements in the UK and Malaysia are different.

In UK-NA, in order to satisfy SLS, the load to be tested is only up to 1.5 times the representative load (e.g. working load*)(* this excludes piles with negative skin friction) compared to approaches commonly adopted in Malaysia which loaded the working pile to 2.0 times working load. In the suggested MY-NA, the same approach as UK-NA is applied by adopting test load the working pile to 1.5 times the working load instead of the old practice of 2.0 times.

Table 9 Summary of "Indirect" Factor of Safety (FOS) calculated from EN1997-1:2004 Annex A, UK National Annex EN1997-1:2004 and suggested Malaysian Annex EN1997-1:2004.

Methodology / Indirect Factor Of Safety (FOS) values (for Comparison with Conventional Method)		DA1-C1	DA1-C2	DA2-C1	DA1-C2 UK-NA **WITHOUT explicit verification of SLS[A)]**	DA1-C2 UK-NA **WITH explicit verification of SLS[A)]**	DA1-C1 MY-NA	DA1-C2 MY-NA **WITHOUT explicit verification of SLS[B)]**	DA1-C2 MY-NA **WITH explicit verification of SLS[B)]**
Model Factor =1.4							**Model Factor =1.4**		
Driven Pile (Compression)	Base FOS	1.93	1.93	2.13	2.52	2.23	**2.13**	**2.82**	**2.67**
	Shaft FOS	1.93	1.93	2.13	2.23	1.93	**1.93**	**2.23**	**1.48**
	Total/Combined FOS	1.93	1.93	2.13	2.52	2.23	**2.03**	**2.37**	**1.93**
Bored Pile (Compression)	Base FOS	2.42	2.37	2.13	2.97	2.52	**2.32***	**3.26***	**2.67***
	Shaft FOS	1.93	1.93	2.13	2.37	2.08	**1.93**	**2.23**	**1.63**
	Total/Combined FOS	2.22	2.23	2.13	2.97	2.52	**2.13**	**2.37**	**2.08 (1.93**)**
Model Factor =1.2							**Model Factor =1.2**		
Driven Pile (Compression)	Base FOS	1.66	1.65	1.82	2.16	1.91	**1.82**	**2.41**	**2.29**
	Shaft FOS	1.66	1.65	1.82	1.91	1.65	**1.66**	**1.91**	**1.27**
	Total/Combined FOS	1.66	1.65	1.82	2.16	1.91	**1.74**	**2.04**	**1.65**
Bored Pile (Compression)	Base FOS	2.07	2.04	1.82	2.54	2.16	**1.99***	**2.80***	**2.29***
	Shaft FOS	1.66	1.65	1.82	2.04	1.78	**1.66**	**1.91**	**1.4**
	Total/Combined FOS	1.90	1.91	1.82	2.54	2.16	**1.82**	**2.04**	**1.78 (1.65**)**

Where :
DA1-C1 = EN1997-1:2004 Design Approach 1 – Combination 1
DA1-C2 = EN1997-1:2004 Design Approach 1 – Combination 2 – Piles and Anchors
DA2-C1 = EN1997-1:2004 Design Approach 2 – Combination 1
DA1-C2 UK-NA = UK National Annex to EN1997-1:2004 Design Approach 1 – Combination 2 – Piles and Anchors

Where :
DA1-C1 MY-NA = Suggested Malaysian National Annex to EN1997-1:2004 Design Approach 1 – Combination 1
DA1-C2 MY-NA = Suggested Malaysian National Annex to EN1997-1:2004 Design Approach 1 – Combination 2 – Piles and Anchors

Note :
1. The Permanent (Dead Load) and Variable (Life Load) ratio of 80% : 20% is assumed in this table.
2. For DA1-C1 and DA2-C1; the Average FOS on Total Load =1.38;
3. For DA1-C2-Pile and Anchors; the Average FOS on Total Load = 1.06

* For bored pile design, the base resistance should be ignored (not included) unless for bored pile constructed in dry holes, or with base grouting, or with fully instrumented preliminary pile loaded to failure and ultimate base capacity verified on site.

In conventional design approaches in Malaysia, generally the pile will have global FOS of 2 which means that the ultimate capacity of the pile is 2.0 times of the working load (WL). When loading the working pile to 2.0 times WL, this means theoretically loading the pile to failure. If the working pile is designed with FOS of 2.0 and when it reaches 2.0 times the working load, the pile top displacement (settlement) would be very large as the pile is supposed to fail if the pile is deemed to fulfil its designed ultimate load. However, loading the working pile to ultimate load (2.0xWL) is illogical as when the pile fails in geotechnical capacity, it can no longer be accepted as it is and has to be downgraded if one wishes to use it as working pile. Many practicing geotechnical engineers in Malaysia adopted a practical approach of not limiting or specifying the allowable or acceptable displacement at 2.0 times working load and this is theoretically correct since the pile is supposed to fail (reach the ultimate limit state) at 2.0 times working load.

In view of this, it would be more reasonable to load test working piles to 1.5 times to prevent failing the working piles. The acceptance criteria of working piles should focus on the acceptable displacement (settlement) at the pile top (more correctly at pile cut-off-level) at 1.0 times working load. Therefore, geotechnical engineer should specify allowable or acceptable pile top settlement at 1.0xWL and also the residual settlement after unloading. In Malaysia, commonly the allowable pile top settlement at 1.0xWL is fixed at not exceeding 12mm and the allowable residual settlement after unloading is fixed at not exceeding 6mm. These values can be a preliminary guide but for long piles or different types of structures, the geotechnical engineer should work with the structural engineer to specify allowable settlement at 1.0xWL and residual settlement after unloading.

To fulfil using model factor of 1.2, a preliminary (sacrificial) pile should be subject to static load test taken to the calculated, unfactored ultimate resistance as follow:

- Load to at least 2.5 times the design load or to failure of the pile to try to obtain ultimate resistance of pile for shaft and base.
- Instrumentation is encouraged to allow proper verification of load-settlement behaviour in shaft and base.
- Without SLT on preliminary piles to achieve unfactored ultimate resistance, then Model Factor of 1.4 should be used.

To fulfil the requirement "***<u>WITH</u>*** explicit verification of SLS" for MY-NA, the testing criteria for piles under compression load should satisfy items (1) and (2) stated below:

1) Static Load Test (SLT) on Working Pile :
   - Load to 1.5 times design load. Acceptable settlement at pile cut-off level should be less than 10% of the pile diameter.[I]
   - Acceptable settlement at pile cut-off-level should not exceed 12mm[II] at 1.0 time representative load.
   - Acceptable residual settlement at pile cut-off-level should not exceed 6mm[II] after full unloading from 1.0 time representative load.
   - To fulfil criteria "with explicit verification of SLS[B)]" (as described in Table 7), the percentage (%) of constructed piles listed in Table 10, should be subjected to SLT (minimum of one (1) number of pile shall be subjected to SLT)

Note:

(I) adopt the "failure" criterion as in EC7 **7.6.1.1** (3) "For piles in compression it is often difficult to define an ultimate limit state from a load settlement plot showing a continuous curvature. In these cases, settlement of the pile top equal to 10% of the pile base diameter can be adopted as the "failure" criterion."
However, for very long piles, elastic shortening will need to be taken into account as the elastic shortening of the long pile itself may reach 10% of the pile diameter and this scenario, the ultimate load shall be defined by the Engineer.

(II) The value indicated as preliminary guide. Geotechnical engineers and Structural engineers shall specify the project specific allowable settlement 1.0xWL and residual settlement to suit the buildings and structures to be supported by the pile.

AND ITEM (2)

2) (A) High Strain Dynamic Load Test (DLT) on Piles:

- To fulfil criteria "with explicit verification of SLS[B)]"(as described in Table 7), minimum percentage (%) of constructed piles listed in Table 10 should be subjected to DLT[(III)]

Note :

(III) DLT can be omitted if it is technically not suitable to carrying out DLT on the pile (e.g. bored pile solely relies on rock socket, etc). Then more SLT shall be carried out.

OR

(B) Statnamic Load Test (sNLT) on Pile :

- To fulfil criteria "with explicit verification of SLS[B)]"(as described in Table 7), minimum percentage (%) of constructed piles listed in Table 10 should be subjected to sNLT[(IV)]

Note :
(IV)

(i) sNLT can be omitted if it is technically not suitable to carrying out sNLT on the pile (e.g. bored pile solely rely on rock socket, etc). Then more SLT shall be carried out.

(ii) Since the reliability of the test results of sNLT lies between SLT and DLT, therefore, higher percentage of tests are needed compared to SLT but lower percentage compared to DLT

In the event where the percentage (%) of SLT has to be increased or reduced due to the type of foundation system selected or the individual project nature, the required % of DLT shall be adjusted accordingly. Table 10 lists the recommended percentage (%) of testing to be carried out on the constructed piles to fulfil the criteria "***WITH*** explicit verification of SLS". The Authors also cross-checked the suggested percentage with 16 project sites that had been successfully completed and randomly selected by the Authors to verified that the recommended percentage is in order.

Table 10 Recommended Percentage (%) of Constructed Piles to be Tested to Fulfil Criteria of "WITH explicit verification of SLS" in suggested MY-NA

Options	**% of Constructed Piles to be Tested to Fulfil Criteria of "*WITH* explicit verification of SLS"**				
	Must Include		**Either**		**Either**
	SLT		**DLT**		**sNLT**
1	> 0.2%	**AND**	> 1.0%	**OR**	≥ 0.5%
2	> 0.1%		> 2.5%		≥ 1.2%
3	> 0.05%		> 5.0%		≥ 2.5%
4 (Especially for bored /barrette pile where its capacity is mainly derived from rock socket friction)	> 0.3%		NIL		NIL
Note: In all cases, the following minimum numbers of SLT shall be carried out: 1. Minimum one (1) number for total piles < 500 numbers. 2. Minimum two (2) numbers for 500 ≤ total piles < 1000 numbers. 3. Minimum three (3) numbers for total piles ≥ 1000 numbers.					

Even for site that meets the criteria of "***WITHOUT*** explicit verification of SLS" for MY-NA, the design engineer still needs to carry out necessary testing of the piles on site despite not being up to the percentage (%) specified in Table 10, to ensure safety.

4.0 Case histories on Driven Pile

Five different driven pile sizes from different sites were analysed based on different methodologies of EC7, UK-NA and suggested MY-NA for both model factors of 1.2 and 1.4. The adopted case histories have included reinforced concrete (RC) square piles of 150mm to spun piles of 600mm in diameter. Among the five case histories presented, both 200mm and 150mm square RC square piles were constructed in sandy SILT material overlaying Kenny Hill and Granite Formation respectively. Meanwhile, the 300mm and 600mm diameter spun piles were driven into SILT material underlain by Kajang formation. As for the fifth case history, 500mm diameter spun piles were installed in a mixture of loosely deposited reclaimed SAND overlying soft CLAY.

4.1 Interpretation of Static Pile Load Test

In order to capture the practicality of EC7, the results of analyses based on different methodologies of EC7, UK-NA and suggested MY-NA are compared with Malaysian conventional design methods and the static pile load test results were interpreted. As most of the selected static pile load tests were not tested to failure (i.e. to the ultimate pile capacity), an ultimate load had to be derived from Chin's method (Chin, 1970) to allow for comparison with EC7 methodologies.

As most of the piles studied in this paper has demonstrated relatively linear load-settlement behavior (even for piles tested up to 3.0 times working load), the ultimate capacity derived through the interpolation method of Chin (Chin, 1970) will lead to larger ultimate capacity. Therefore, the ultimate capacity determined using Chin's method is further downgraded by 30% for all piles studied in this paper, to be on the safe side.

4.2 Comparison of Case Histories for Driven Piles

Based on our analyses, the results are presented in Figures 2 and 3 and the following findings are deduced. For comparison purposes, the ultimate loads interpreted from the static load test are divided by Factor of Safety (FOS) of 1.5, 2.0 and 3.0 to obtain the allowable capacity with different FOS, and the pile head settlement corresponding to the respective allowable working load is also indicated in the figures. For easy comparison, Figures 4 and 5 show the ratio of allowable capacity calculated with various methodologies over the allowable capacity calculated via Malaysian conventional design.

As observed from Figures 2 and 3, methodologies DA1-C1 and DA1-C2 of the EC7 are generally more optimistic compared to UK-NA and the suggested MY-NA. The suggested MY-NA generally fits well with the Malaysian conventional design and the actual static load tests results carried out on site to verify the pile capacity. [It is common for DA1-C1 to yield higher allowable geotechnical capacity compared to DA1-C2 as in DA1-C1, the factors on actions (both Permanent & Variable) are larger. Therefore, the acting load on the working pile is larger in DA1-C1 compared to DA1-C2]

For DA1-C2 of MY-NA (with explicit verification of SLS) which calculated the highest allowable pile capacity, the value still falls within acceptable load and deformation limits, as proven in the SLT results. In summary, load tests results indicated that the suggested MY-NA partial factors are generally acceptable for all five sites and with different driven pile sizes ranging from as small as 150mm to as large as diameter of 600mm.

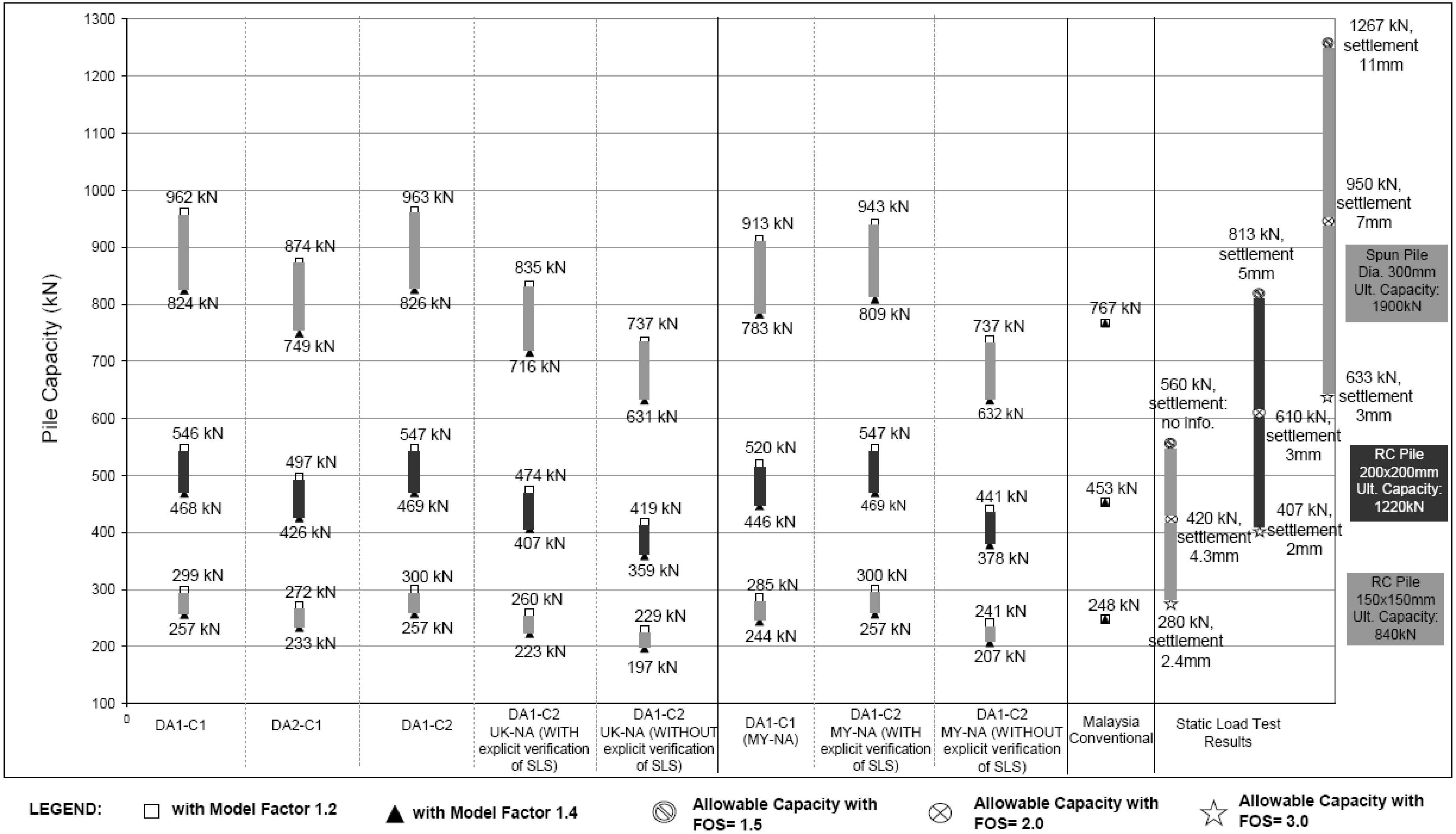

Figure 2 Allowable pile capacity of Driven Piles for various methodologies compared with Malaysian conventional design methods and static pile load test results (150mm to 300mm size piles)

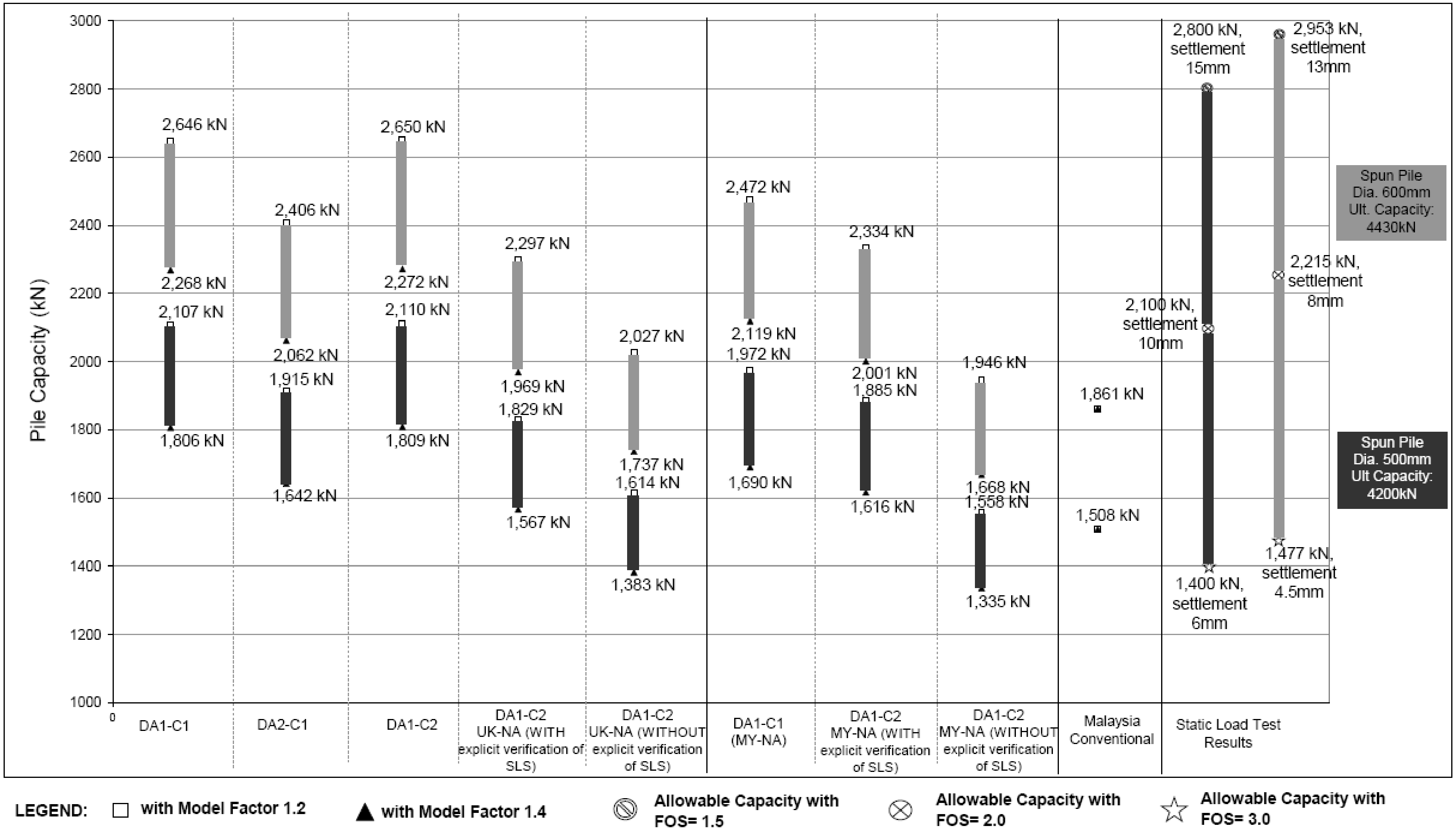

Figure 3 Allowable pile capacity of Driven Piles for various methodologies compared with Malaysian conventional design method and static pile load test results (500mm and 600mm size piles)

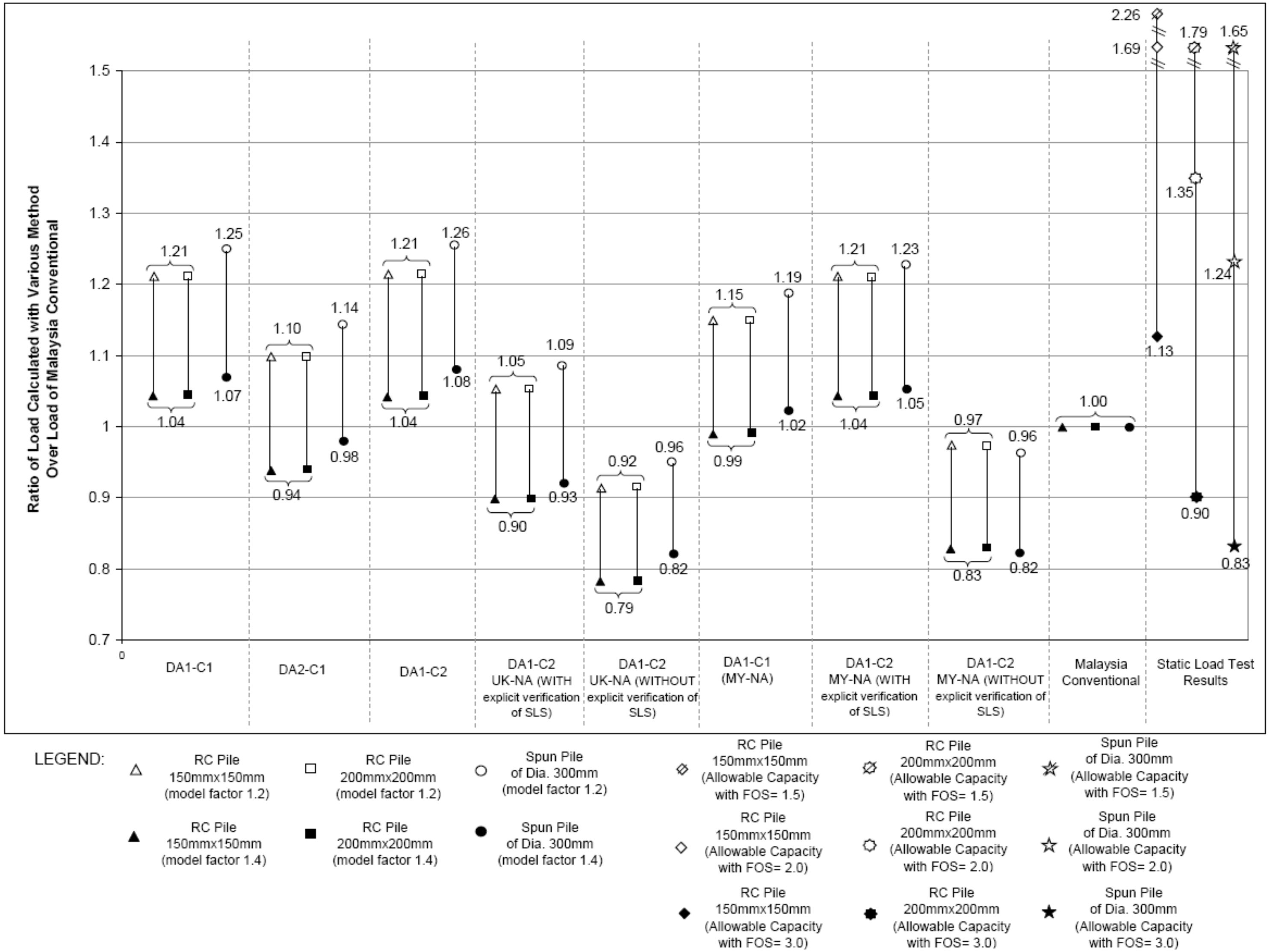

Figure 4 Ratio of Driven Pile allowable capacity for various methodologies over allowable capacity calculated via Malaysian conventional design (150mm to 300mm size piles)

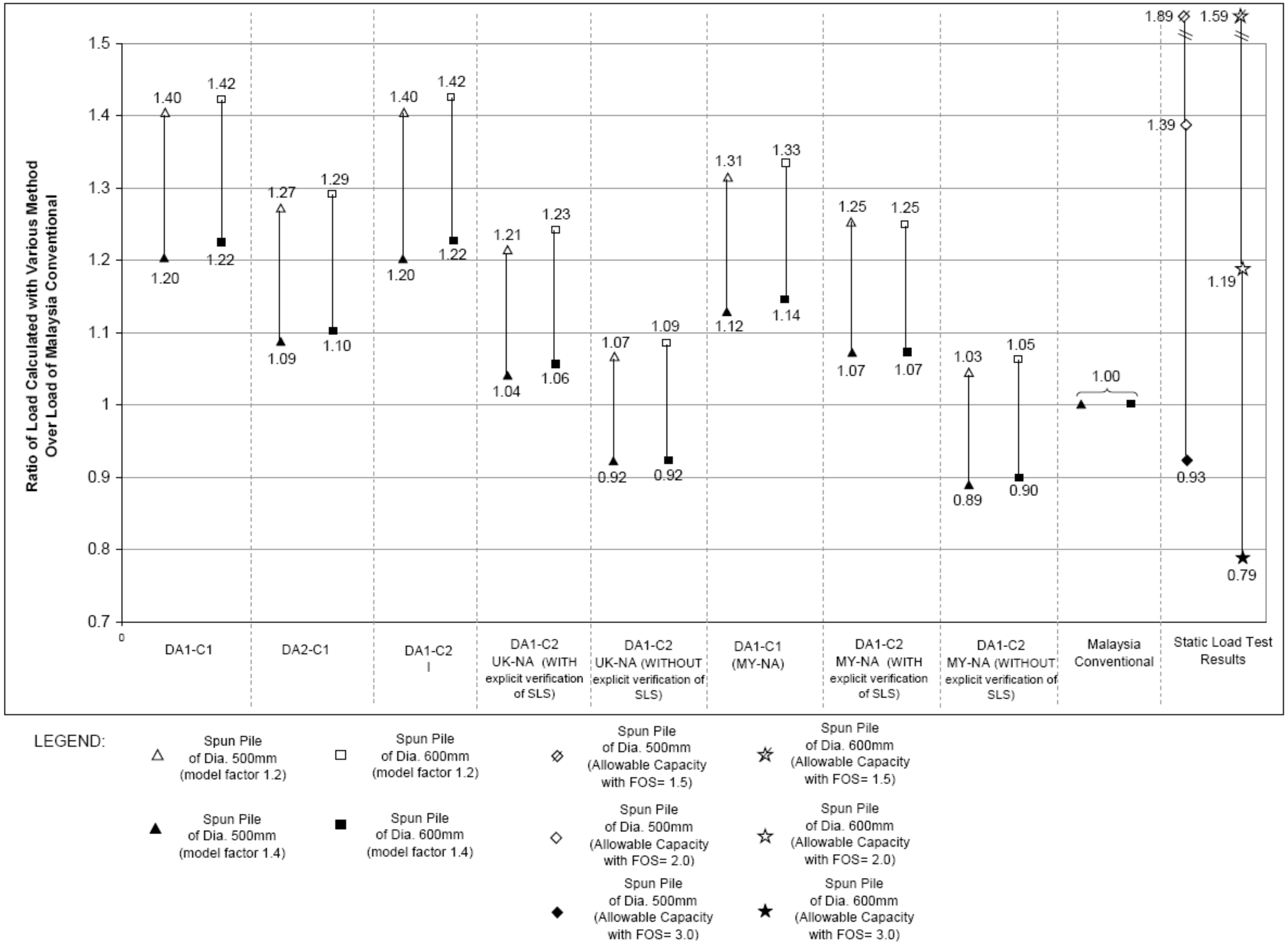

Figure 5 Ratio of Driven Pile allowable capacity for various methodologies over allowable capacity calculated via Malaysian conventional design (500mm and 600mm size piles)

6.0 Conclusion

This paper presents the Malaysian design methodologies for driven pile and bored pile foundation and the way forward in converting to EC7 approach. As the Malaysian practice is based on working state principle with global factor of safety, this paper presents the EC7 methodologies with some suggested values of partial factors for the development of EC7 Malaysian National Annex (MY-NA).

Based on the selected case studies on driven pile foundation for five different pile sizes, the design pile capacity based on both Malaysian Conventional Approach and EC7 Methodologies are compared. The results of analyses are presented in term of design pile capacity based on the proposed model factors and partial factors for suggested Malaysian National Annex (MY-NA) in the application of EC7. Such proposed factors take into consideration the Malaysian experience based on our lesson learnt and construction practicality, by referring to numerous publications by local practitioner. In summary, the paper aims at providing a platform to allow for design adjustment and to ensure smooth transition from current Malaysian practice to EC7.

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