Design parameters for bored piles in a weathered sedimentary formation

Paramètres de projet pour les pieux forés dans les formations sédimentaires altérées

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SUMMARY The Kenny Hill Formation which is a sequence of clastic sedimentary rocks consisting of interbedded shales, mudstones, siltstones and sandstones of the Upper Palaeozoic period extends over a significant part of Kuala Lumpur City and the Klang Valley of Peninsular Malaysia. Design of bored piles is usually by use of a Meyerhof type equation relating pile capacities to S.P.T. values. Bored piles are usually socketed into material with N values ranging from 50 to 200. The results of load tests on several instrumented bored piles were analysed to determine strength and deformation parameters and related to N values in order to examine the validity of the existing design method and to establish future guidelines for design of bored piles in this sedimentary formation.

INTRODUCTION

Residual soils derived from sedimentary rocks are generally complex due to inhomogeneity (layering) and textural variation of the rock. The complexity of the sedimentary residual soils is even greater where the parent rock has been subjected to folding and this poses considerable difficulties in design and construction of bored piles due to significant changes in ground properties over short distances.

Standard Penetration Tests (S.P.T.) are almost always carried out as part of subsurface exploration but the friable nature of the material prevents the recovery of a suitable sample for laboratory strength tests. Chan (1985) reported that the N values at bored pile founding levels generally range from 50 to 200.

The occassional attempts to recover undisturbed samples by using retractable type triple tube core barrels have not been successful due to the presence of gravels. N.M.L.C. triple tube core barrels have been used with bentonite to obtain undisturbed samples but the core recovery ratio is uncertain.

Occassionally pressuremeter tests are carried out by use of the Menard Pressuremeter but the availure instrument has a maximum pressure of 5MPa which is generally less than the pressure limits of the weak rocks. Another reason for the limited use of the pressuremeter is the relatively high costs in the country; the cost of a pressuremeter test is approximately 30 times the cost of carrying a S.P.T. Consequently the design of piles have been by empirical relationships betwen pile capacities and N values adopted from international practice but further developed from local experience.

GEOLOGY

The Kenny Hill formation is a sequence of interbedded sandstones, siltstones and shales/mudstones. The shales usually occur as centimetres to metre bedded strata, while the sandstones and mudstones are thicker and mainly occur as metre bedded strata. The sandstones are mostly fine to medium grained with occasional clay clasts and mudstone lenses, while the mudstones and shales are often phyllitic and show foliation and incipient cleavage. The strata is also strongly jointed and cut by numerous small to large scale faults. The Formation is believed to be of an Upper Palaeozoic age and has given rise to an undulating terrain of low hills and shallow and broad valleys in it's outcrop areas.

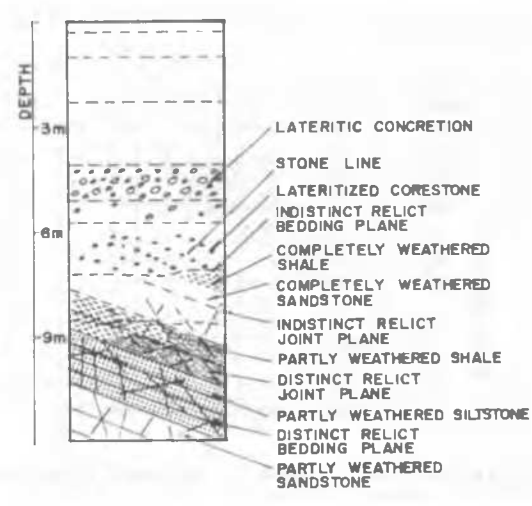

Fig. 1 Schematic diagram of the main features of the morphological horizons of the Kenny Hill Formation from Raj (1983)

Fig. 1 from Raj (1983) is a schematic diagram of the main features of the morphological horizons of the Formation. Raj (1983) reported that the distribution of the different stages of weathering of rock material within the residual soil profile is extremely complex and that different stages of rock weathering material are found at similar levels.

GENERAL PROPERTIES

N Values

Fig. 2 summarizes the N values at a typical site in the Kenny Hill formation.

Generally N values in excess of 50 occur at depths ranging from 10 to 20m. However at locations near granite intrusions, N values of less than 50 may occur up to depths of 30m or more.

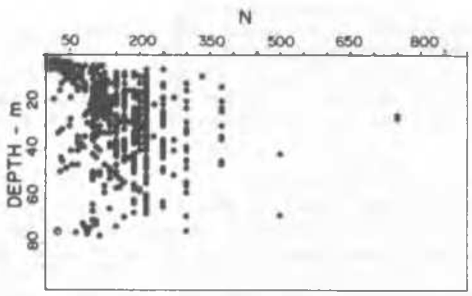

Fig. 2 N values at a typical site.

Pressuremeter Modulus

A correlation of N values with Epm, Pressuremeter Modulus values, for weak rocks of the Kenny Hill Formation is reproduced in Fig. 3 (Toh et al, 1987). Considerable scatter in the results is noted.

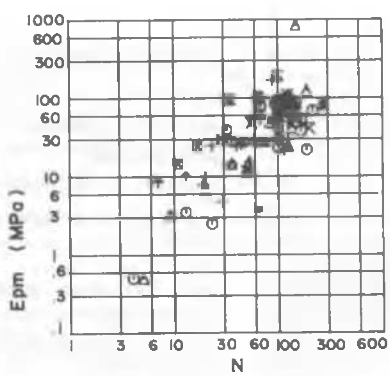

Fig. 3 Pressuremeter Modulus versus N for the Kenny Hill Formation from Toh et al. (1987)

Grain Size Distribution

Fig. 4 illustrates a typical distribution of clays, silts, sands and gravels with depth. The proportions of silt in weathered shales and siltstones range from about 15% to up to about 80%. The proportion of sands and gravels in weathered sandstone can range up to 90%.

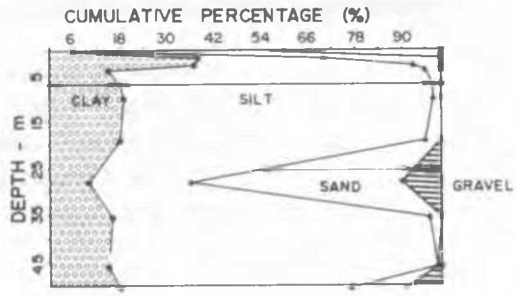

Fig. 4 Grain Size Distribution at a typical site

Plate Load Tests Results

Toh et al. (1987) reported the results of plate load tests using 1m diameter plates on subvertical beds of slightly weathered shales and sandstones. Bi-linear load-settlement curves were observed from the tests on shales with yield stresses occurring at settlements of approximately 10mm or 1% of the plate diameter. The yield stress and Elastic Modulus of the soft shales are 1.3 MPa and 100 MPa respectively. The Elastic Modulus of slightly weathered sandstone was 595 MPa.

Maintained load tests on the shales at press ures corresponding to 65% of yield stress resulted in significant time dependant settlements. The quantum of base load that should be permitted in the design of bored piles in softer shales should therefore be restricted.

SPT-STRENGTH RELATIONSHIPS

Stroud (1974) suggested the correlation: Cu = f_1 N where the f_1 (kPa) ranges from 4 to 6 with decreasing plasticity. Leach and Thompson (1979) found good agreement for mudstones when f_1 was taken to be 5kPa. The use of Stroud's correlation with f_1 of 5 kPa for N values ranging from 50 to 200 gives unconfined compressive strengths for the site material of between 0.5 to 2.0 MPa. Materials with such strengths are classified as Very Weak to Weak Rocks on the scale of strength set out by the Working Party of the Geological Society of London (1970).

PRESENT PILE DESIGN PROCEDURE

The commonly used method for designing bored piles in the Kenny Hill formation is expressed in equation (1a) and (1b) :

$$f_{su} = K_s N_s \qquad (1a)$$

$$f_{bu} = K_b N_b \qquad (1b)$$

Where:

f_{su} is the ultimate shaft resistance;

N_s the average N value along the pile shaft;

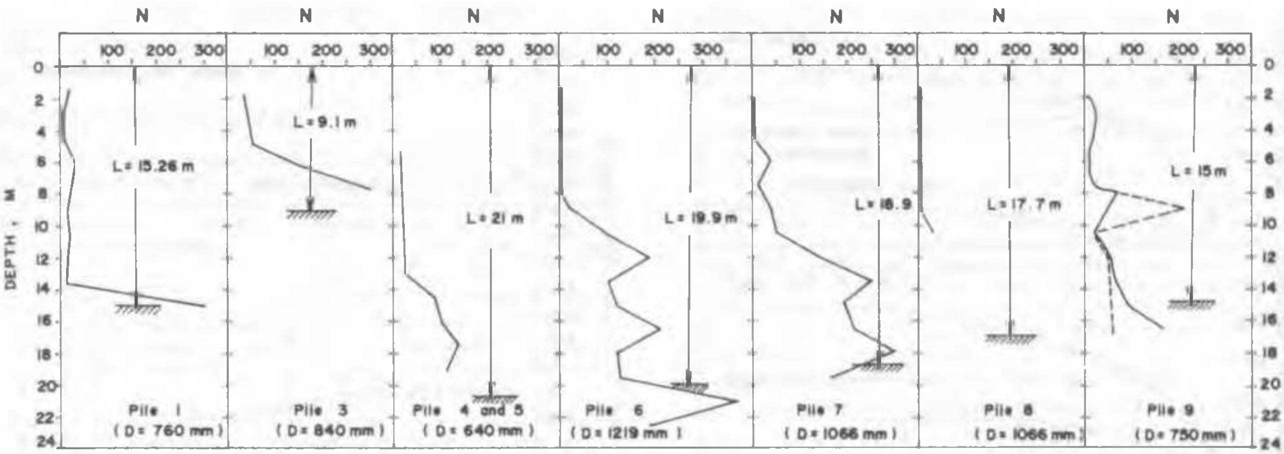

Fig. 5 Profiles of N and Pile Dimensions

K_s the shaft resistance factor;
f_{bu} the ultimate base resistance;
N_b the N value at the pile base;
K_b the base resistance factor.

Values of K_s normally adopted range from 2.5 to 2.7. Values of K_b commonly used range from 0 to 50 depending on conditions encountered during construction and the material at the pile base.

Chiu & Perumalswamy (1987) quoted use of K_s = 2.5 and K_b = 50 but limit the allowable shaft resistance to 80kPa and the allowable base resistance to 3 MPa with Factors of Safety of 2.5.

Assuming that $K_s = 2.5$ and $K_b = 45$ and applying Stroud's correlation with f_1 = 5kPa to equations 1, results in:

$$f_{su} = \alpha C_u \text{ with } \alpha = 0.5 \quad - (2)$$

and

$$f_{bu} = 9C_u \quad - (3)$$

which reverts to the familiar equations for the ultimate carrying capacities of piles in clays.

INSTRUMENTED TEST PILES

Load tests were carried out on 1 instrumented micropile and 8 instrumented bored cast-in-place piles. Strain gauges were located at at least 5 levels along the lengths of the piles within and above the socket length. Rod extensometers were also installed in some piles. Only 2 piles were loaded to failure. The other piles were loaded to the point where it was thought that the shaft resistances along most of the pile lengths were fully mobilized but before full mobilization of base resistances.

The profiles of N values and the lengths and diameters of the piles are given in Fig. 5. The socket length is taken to be that length of pile within material with N values greater than 50. Material with N values less than 50 is classified in this context as soil.

Load Deformation Behaviour

Typically the load-deformation behaviour of the piles (Fig. 6) take the approximate form of bilinear curves with the "yield load" separating the two parts. The yield loads occur at settlements ranging from 3.5mm to 9mm or 0.4% to 2% of the pile diameters (see Table 1) and are observed to be points where the shaft resistance over a significant length of the pile has been fully mobilized. Non-linear behaviour is characterized by a significantly higher proportion of load reaching the base. Settlements at failure load for the 2 piles tested to failure are 17mm and 25mm or 3.3% and 8.4% of the pile diameter respectively.

Pile	Top of Pile Movement At Yield Load		Top of Pile Movement At Failure Load	
	S (mm)	S/D (%)	S (mm)	S/D (%)
1	-	-	-	-
2	4	1.97	17	8.4
3	3.5	0.42	-	-
4	9	1.5	-	-
5	-	-	-	-
6	5	0.41	-	-
7	4	0.38	-	-
8	4	0.38	-	-
9	7	0.93	25	3.3

Table 1 - Movements at Top of Pile

The above settlement diameter ratios are similar to those quoted by Fleming et al. (1985) viz. 0.5% to 2% of pile diameter for mobilisation of shaft capacity and 5% to 10% of pile diameter for mobilisation of base capacity.

Fig. 6 illustrates the load-deformation response of Pile 9 which was loaded to failure. The yield load occurred at a settlement of approximately 7mm coincident with almost full mobilisation of shaft resistance along the full length of the pile. The proportion of load

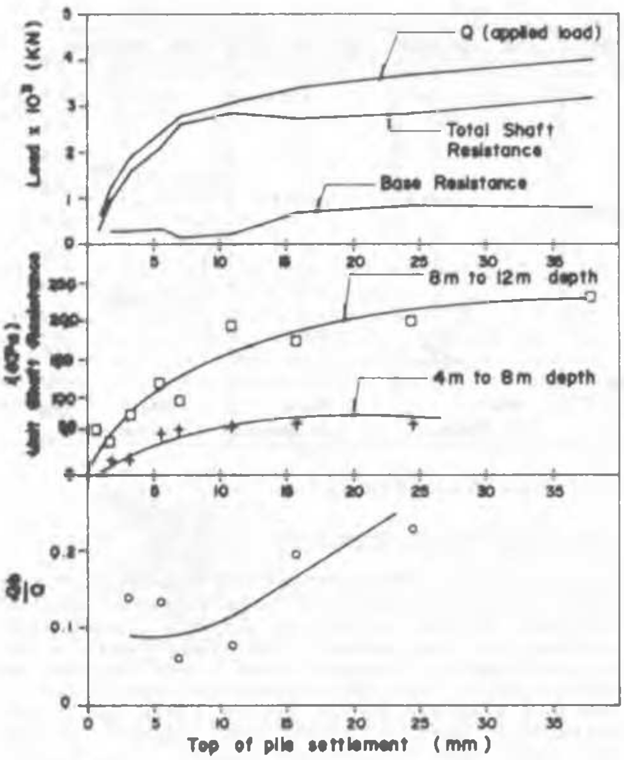

Fig. 6 Applied Load, Base Load, Shaft Resistance versus settlement - Pile 9

reaching the base remains at approximately 10% until past yield where the proportion of base load to applied load increased to a maximum of about 25%. The pile load increased with settlement with no indication of an ultimate load condition for settlements up to 40mm. The load distribution characteristics observed in pile 9 is similar to that reported by Donald et al. (1980) and Williams et al. (1980).

Ultimate Shaft Resistance

Fig. 7 is a plot of K_s (f_{su}/N) against N and is for selecting the ultimate shaft resistance given a value of N. There is considerable scatter in the data for N values of less than 50. However there is a trend of decreasing K_s with increasing N for N values greater than 50. N values greater than 120 would, by use of the average line, result in K_s values less than the values of 2.5 to 2.7 normally used in design. As shown in Fig. 7 the use of a Factor of Safety of 2.5 against ultimate shaft resistance as expressed by the average line would ensure minimal risk of failure.

Use of Stroud's equation with f_1 = 5kPa results in the following direct relationships:

$$K_s = \frac{f_{su}}{N} \; ; \qquad - \; (4)$$

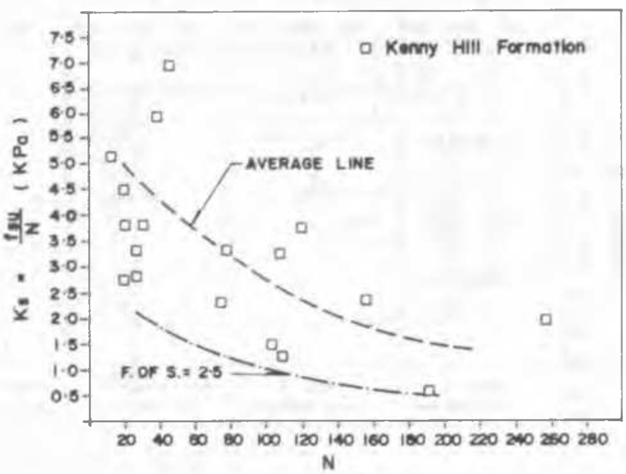

Fig. 7 Shaft Resistance Factor, K_s, versus N

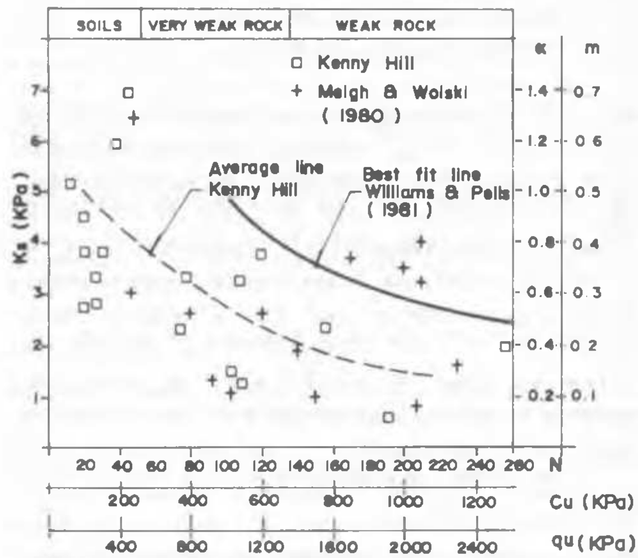

Fig. 8 Comparison of Shaft Resistance Factors

$$\alpha = \frac{f_{su}}{C_u} = \frac{f_{su}}{5N} = \frac{K_s}{5} \; ; \qquad - \; (5)$$

$$m = \frac{f_{su}}{q_u} = \frac{f_{su}}{2C_u} = \frac{f_{su}}{10N} = \frac{K_s}{10} \; ; \qquad - \; (6)$$

which enable comparisons of the results given in Fig. 7 with those by Williams & Pells (1981) and those summarized by Meigh & Wolski (1979). As shown in Fig. 8 the average line by Williams & Pells (1981) lies above those of the Kenny Hill Formation while those summarized by Meigh & Wolski (1979) lie within the same broad band as those for the Kenny Hill Formation for N less than 160. Comparisons are only made for N values up to 260. It should be noted that most of the data by Williams & Pells (1981) are for unconfined compressive strengths (q_u) greater than 2MPa.

Ultimate Base Resistance

Only 2 piles were loaded to failure and therefore comprehensive analyses could not be made for the relationship between N values and base resistances. Table 2 summarizes the 2 test results available.

Pile	Ultimate Base Resistance (kPa)	N_b	Relationship Between Ultimate Base Resistance And N_b
2	6180 kPa	100	$f_{bu} = 60\ N_b$
9	1970 kPa	73	$f_{bu} = 27\ N_b$

Table 2

Elastic Modulus

Back analysis of the pile tests results to match the applied load (before yield load) with the measured top of pile settlement was carried out using the linear elastic finite element method. Analysis of the socket alone and ignoring the overburden soil gave unrealistic results. The piles were subsequently idealized as being embedded in a 2 layer material with the modular ratio of the upper to the lower layer being equal to the ratio of the average N values of the upper to that of the lower layer. The value of Ec (Young's Modulus of the pile) adopted was 29 GPa as obtained from the pile tests.

The back calculated soil/weak rock modulus values (E_s) using the abovementioned model are plotted against the average N values in Fig. 9. The results fall within a narrow band and the relationship may be expressed as:-

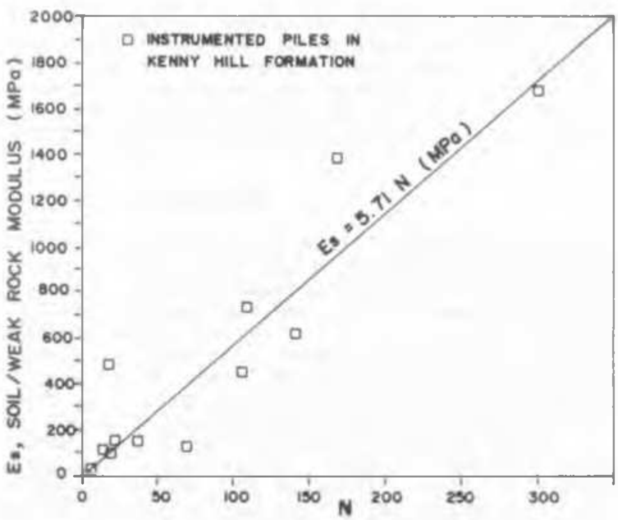

Fig. 9 Soil/Weak Rock Modulus versus N

$$E_s = 5.71\ N\ (MPa) \qquad (7)$$

which is similar to that for free-draining sands and preloaded sands quoted by Stroud (1974). The backcalculated E_s/N ratio corresponds to the upper bound of the Epm/N ratios given in Fig. 3.

Unfortunately no pressuremeter tests were carried out adjacent to the instrumented test piles and therefore no direct comparison may be made between backcalculated and pressuremeter modulus values. However comparison of Fig. 3 with Fig. 9 indicates that the Menard Pressuremeter modulus values are lower than the backcalculated modulus of the weak rocks of the Kenny Hill Formation.

E_s/Cu ratios may be obtained from equation 7 by applying different values of f_1 within the range given by Stroud (1974). The results are summarized in Table 3.

f_1 (kPa) Stroud's Correlation Factor	E_s/Cu
4	1428
5	1142
6	952

Table 3

Distribution of Load Between Shaft And, Base Resistance

The proportion of load reaching the base of each test pile (ratio of the load at the pile base, Q_b to the load at the top of the socket, Q_{tos}) in the linear elastic range is plotted

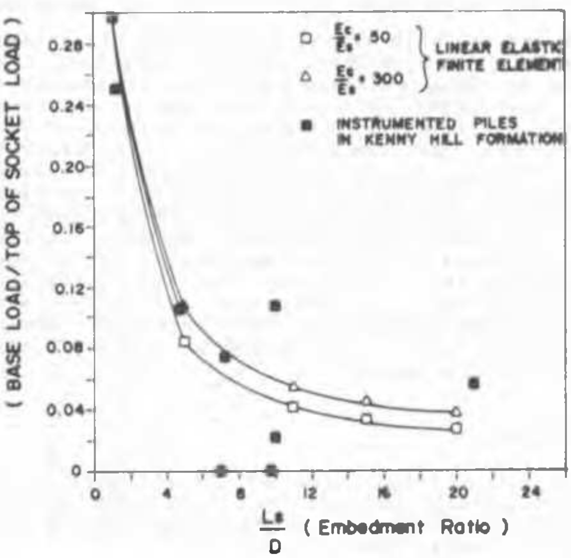

Fig. 10 Proportion of Base Load as a Function of Embedment Ratio

against the embedment ratio (ratio of socket length to pile diameter, L_s/D) in Fig. 10.

Theoretical distributions of shaft and base loads for different Ec/Es ratios from linear elastic finite element analysis with identical base and shaft modulus values are also included in Fig. 10 to demonstrate the applicability of the linear elastic theory for estimating the proportions of base and shaft resistances within the linear part of the load-deformation curve.

CONCLUSIONS

1. The Standard Penetration Test is the most economical and most widely used method of estimating the strength of weak rocks in the country for design of bored piles in the Kenny Hill formation.

2. K_s values of 2.5 to 2.7 usually used in design appear to be reasonable for N values of up to 120. For greater N values, lower K_s values will have to be considered. There is considerable scatter in the results for N values less than 50 and in view of the scatter caution need to be exercised if K_s values greater than 2.7 are adopted. Due to lack of data for N values greater than 200, it is recommended that the design value should not exceed that for N = 200. A Factor of Safety of 2.5 against the average line will minimize the likelihood of failure.

3. Unless there is considerable confidence in overcoming the difficulties of cleaning the pile base, bored piles should be designed to carry loads by shaft resistance only. Creep behaviour of softer shales should be taken into account if the design permits a proportion of the load to be carried by the base. Fig. 10 incorporating curves from linear elastic theories may be used to estimate the relative proportions of base and shaft resistance within the linear part of the load-deformation curve.

4. Fig. 9 relates E_s to N and is obtained from backanalysis of the piles using the linear elastic finite element method modelling the piles as being embedded in a 2 layer continuum viz. overburden soil and weak rock. Analysis of the socket alone ignoring the effects of the overburden soil leads to errors in settlement predictions. It is difficult to produce comprehensive charts for estimating pile settlements which account for variable thicknesses of the overburden soil, different embedment ratios of the socket, different modular ratios of the overburden soil to the weak rock and, different pile diameters. Consequently the finite element method remains the best method for estimating the elastic settlement of piles in the Kenny Hill Formation.

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