Point Pressure versus Length and Diameter of Piles

Pression de Pointe des Pieux à Longueur et Diamètre

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SUMMARY

Piles of more than 1 m diameter have become increasingly used. It was experieced that regulations for conventional smaller piles were useless for them. New rules introduced in the German Codes of Practice are shown, based on a critical appraisal of recent knowledge about the dependence of bearing behaviour on diameter, length and settlement.

1. PRACTICAL REQUIREMENTS

In the development of pile types a caesura occured at the end of the fiftieth. Up to this time piles usually had diameters or breadth not exceeding b = 0.3 m. The admissible load Q_{ad} of such conventional piles was and is determined from an ultimate load Q_u by

$$Q_{ad} = Q_u/(F.S.) \tag{1}$$

with a factor of safety F.S. = 2. F.S. is evidently an empirical value, however, it shall be shown that Q_u is an empirical value too, adjusted to assure that the settlement caused by Q_{ad} should not exceed about 5 mm.

In the last 20 years piles with b > 1 m became increasingly used (see Figs. 1 and 2) and it revealed that the usual definitions for Q_u, developed for smaller conventional piles, did not work when applied to large piles. This was convincingly shown by VESIĆ (1975), who gathered some of the usual definitions for the determination of Q_u for conventional piles (see Table I). For such piles these definitions did not cause deviations of Q_u larger than ± 10 %. But for

Fig. 1 Reinforcement cage of a large bored pile, diameter 150 cm

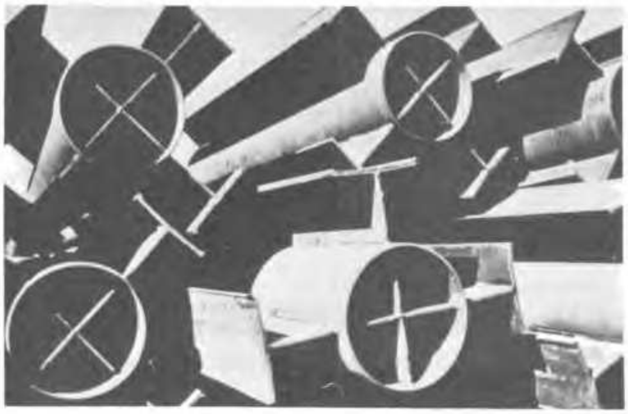

Fig. 2 Enlarged base of steel piles, pipe diameter 60 cm, 1/2 H-profiles of H 60 cm

piles with b > 1 m the results show a large scatter, which is unacceptable. For large bored piles which often are test loaded till Q_u = 10MN, the definitions 5a) and 6a) of Table 1 require corresponding settlements of $s_u \approx 25$ cm. Similar unrealistic requirements are connected with the definitons 5b) and 6b), by which Q_u is defined at settlement increments of 7.5 cm/MN. On the other hand the definitions 1a) and 2) are completely uneconomical as the corresponding settlements are smaller than 1 cm and 0.5 cm resp. under working conditions. This becomes evident considering that large piles have usually spacings of about 5 m and corresponding β-values (as defined by SKEMPTON/MAC DONALD 1956) of 1/500...1/1000. Therefore the development of new regulations in the relevant codes of practice became necessary to better satisfy practical requirements.

2. IMPROVEMENTS IN KNOWLEDGE ON POINT PRESSURE IN SAND AS A BASIS FOR NEW REGULATIONS

TABLE I

Definitions for determination of ultimate load (selected by VESIĈ 1975)

Rules for determination of ultimate load

1) Limiting total settlement

a) absolute 1.0 in (Holland, New York Code)
b) relative 10 % of pile tip diameter (England)

2) Limiting plastic settlement

0.25 in. (AASHO)
0.33 in. (Magnel, 1948)
0.50 in. (Boston Code)

3) Limiting ratio plastic settlement/elastic settlement

1.5 (Christiani and Nielsen)

4) Maximum ratio $\frac{\text{elastic settlement increment}}{\text{plastic settlement increment}}$

(Szèchy, 1961, Ref. 15)

5) Limiting ratio settlement/load

a) total 0.01 in/ton (California, Chicago)
b) incremental 0.03 in/ton - Incremental (Ohio)
0.05 in/ton - Incremental (Raymond Co.)

6) Limiting ratio plastic settlement/load

a) total 0.01 in/ton (New York Code)
b) incremental 0.03 in/ton (Raymond Co.)

7) Maximum ratio $\frac{\text{settlement increment}}{\text{load increment}}$

(Vesiĉ, 1963, Ref. 16)

8) Maximum curvature of log w/ log Q line

(De Beer, 1967, Ref. 17)

9) Van der Veen postulate (1953)

$$w = 8 \ln \left(1 - \frac{Q}{Q_{max}}\right)$$

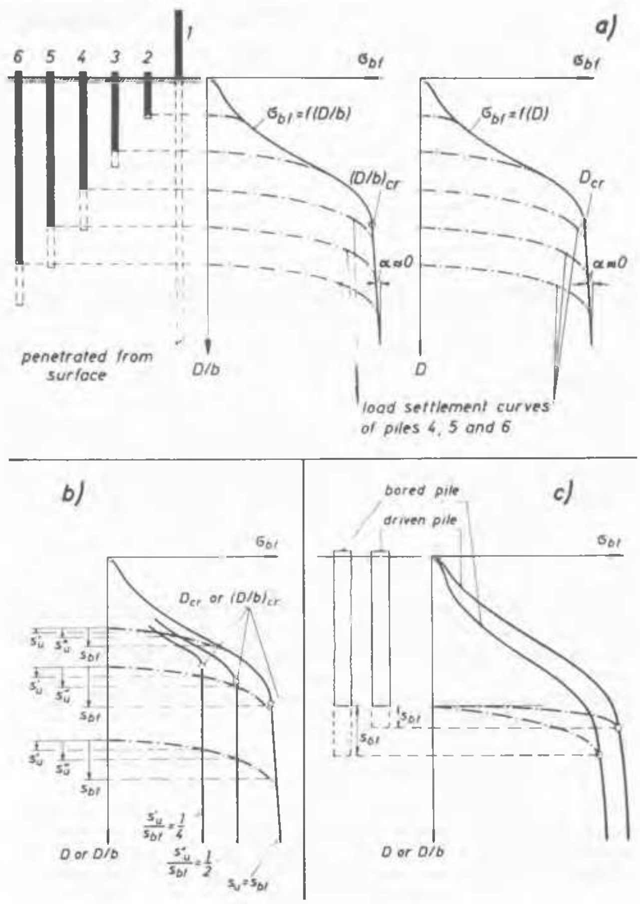

Fig. 3 a) Point pressure at failure σ_{bf} in dependence on depth; definition of critical depth D_{cr} resp. $(D/b)_{cr}$.
b) Decreasing critical depth for $s_u < s_{bf}$.
c) Difference in critical depth for driven and bored piles.

2.1 Two Fundamental Experimental Results

The most important experimental results of the last 20 years are

a) that the point pressure at failure σ_{bf} increases with depth only till a critical value D_{cr} and remains constant beneath it.

b) that σ_{bf} may more or less decrease with increasing diameter or breadth b of a pile when all other conditions remain constant.

Experimental details have become known e.g. by publications of KERISEL (1961) and KERISEL et al. (1965).

On Fig. 3a it is shown, how the existence of a critical depth D_{cr} could be recognized. The dependence of σ_{bf} on depth was gained either by the penetration of pile 1 from ground surface or as the envelope of the load settlement curves of piles 2...6 which were buried, bored or driven and then loaded to failure.

For loose sand the depth dependence of σ_{bf} was not influenced by b. This dependence is schematically shown on Fig. 3a, right side, and can be expressed by

$$\left.\begin{aligned} \sigma_{bf} &= \text{const} \quad \text{for } D = \text{const} \quad &(1a) \\ D_{cr} &= \text{const} \quad &(2a) \end{aligned}\right\} \text{ no dependence on b!}$$

In dense sand, however, the depth dependence of σ_{bf} was influenced by b. For this dependence a unique function $\sigma_{bf} = f(D/b)$ of a normalized depth (D/b) is often assumed like the one on the midpart of Fig. 3a (see e.g. MEYERHOF 1976). It is less well known that this function yields a certain diameter dependence for piles, having the same normalized depth D/b and corresponding different diameters b. This is shown on Fig. 4 by drawing a secant to the normalized load settlement curve $\sigma_b = f(s_b/b)$ with any inclination ß. Then with s_b = settlement of the pile base

$$\sigma_b \cdot b/s_b = \cot ß$$

is valid. Regarding now only those piles having the same settlement s_b = const, it follows

$$\sigma_b \cdot b = \cot ß \cdot s_b = \text{const} = C \qquad (1b)$$

presuming that

$$(D/b) = \text{const} \quad \text{or} \quad (D/b) > (D/b)_{cr} \qquad (2b)$$

(in the latter case it is additionally presumed

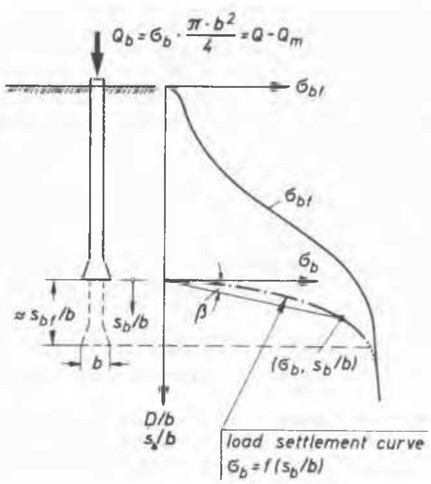

Fig. 4 Schema for Derivation of formula (1b)

that $\alpha \approx 0$ on Fig. 4 which can be done in good approximation).

Two additional remarks on the critical depth may be of interest: For formula (2b) mostly $D_{cr} = 10...20 \cdot b$ is cited as resulting from piles penetrated from the surface. With piles of this sort "complete" failure according to settlements s_{bf} (s. Fig. 3b) is reached. But in practice test loadings are only seldom extented to s_{bf}, often a smaller ultimate settlement value $s_u < s_{bf}$ can be chosen (for details see formulae 4 and the relevant text). In this case smaller D_{cr} values resulted as is schematically shown on Fig. 3b (see e.g. FRANKE/GARBRECHT 1977). - Another remark shall stress that D_{cr} for bored and driven piles is not significantly different, as the s_{bf}-values for both pile types are small compared to D_{cr}, even if great differences in s_{bf} occur for them.

2.2 A Qualitative System Closing the Gap between Formulae (1a), (2a) and (1b), (2b)

Until now no generally accepted theory is available to better quantify the pile bearing problem exceeding formulae (1a), (2a) and (1b), (2b), particularly because of construction influences. To allow for at least a better qualitative judgement it shall instead be tried to establish a qualitative system. This is done by comparing results of calculations and of test loadings. For this comparision the log σ_b - log b - diagram on Fig. 5a is used. It shows that the experimental results are spread between the horizontal line, defined by (1a), and another line, inclined under 45° and expressing (1b) in the form of

$$\log \sigma_b = \log C - \log b = K - \log b \qquad (1c)$$

The following appraisal of this spreading enables us to recognize which parameters determine the position of the experimental results in the diagram.

Already the experiments of KERISEL (1961) had shown that no diameter dependence occured in loose sand. This is expressed by the horizontal line acc. to formula (1a) in the range of small σ_b-values on Fig. 5b, corresponding to the typical experimental line 3c-3c, and approximately

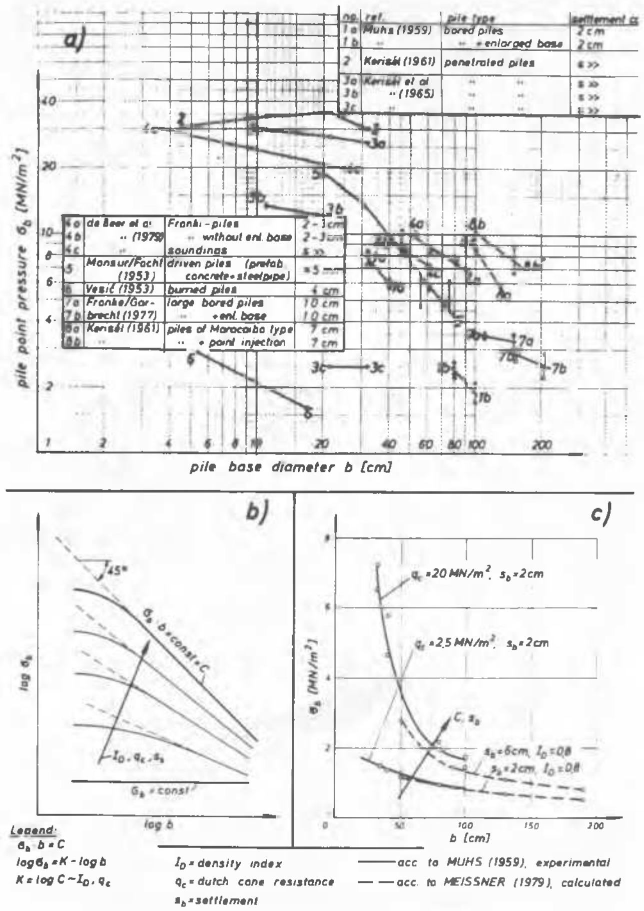

Fig. 5 Diameter dependence of point pressure:

a) measured data
b) qualitative system for an appraisal of Fig.5a
c) influence of settlements and soil strength on the diameter dependence

to the lines 7a-7a, 7b-7b on Fig. 5a. In case of denser sand the hyperbolae $\sigma_b \cdot b = C$ (see Fig. 5c) resp. the lines inclined at 45° on Fig. 5a are representative. Then the constants C resp. K are increasing with the strength of the soil or the density of the sand I_D. This is shown on Fig. 5c, presuming that the curves shown there were hyperbolae. But now allowing for deviations from the hyperbolic form and assuming that a smooth transition from the horizontal line acc. to formula (1a) to the 45° inclined line acc. to formula (1c) does exist due to changing soil strength, this cannot be expressed by K anymore but by I_D or q_c instead (q_c = cone penetration resistance). This dependence is shown on Fig. 5b.

(The available experimental results for piles in stiff fissured clay, published by WHITAKER/COOKE 1966, KERISEL 1967, DVORAK 1976, LEACH et al.1976, JELINEK/KORECK/STOCKER 1977, DE BEER 1979 revealed no diameter dependence similar to the one shown for sand, but a rather irregular spreading which remains in the range of $\sigma_b = 1...2$ MN/m².)

Another phenomenon still to be discussed is the decreasing inclination of some of the experimental curves from the right to the left on Fig. 5a, shwon more systematically on Fig. 5b. The reason is the decreasing strength of sand under increasing stress, e.g. expressed by a decreasing friction angle φ with increasing mean normal stress $\sigma_m = (\sigma_1 + \sigma_2 + \sigma_3)/3$ as published e.g. by DE BEER (1965), VESIC/CLOUGH (1968). The consequence of decreasing soil strength with increasing stress is evidently that a curve inclined at 45° acc. to formula (1b) resp. (1c) for low σ_b (and large b) becomes less inclined for higher σ_b (and smaller b). At least it may become a horizontal like for piles in loose sand acc. to formula (1a). As examples the curves of KERISEL (1961) and DE BEER et al. (1979) at Fig. 5a may be regarded, where 8a-8a, 8b-8b and 4a-4a, 4b-4b are strongly inclined at lower σ_b and where their continuations 2-2 and 4c-4c become flatter.

There is still another effect of the decreasing φ with increasing σ_m on piles, already shown by DE BEER (1963). On Fig. 6, which is taken from his publication, it can be seen that $\sigma_{bf}/(\gamma \cdot D)$ is decreasing with increasing b for constant normalized depth D/b (which would not occur for φ = const, s. FRANKE 1976). From Fig. 6 it must be

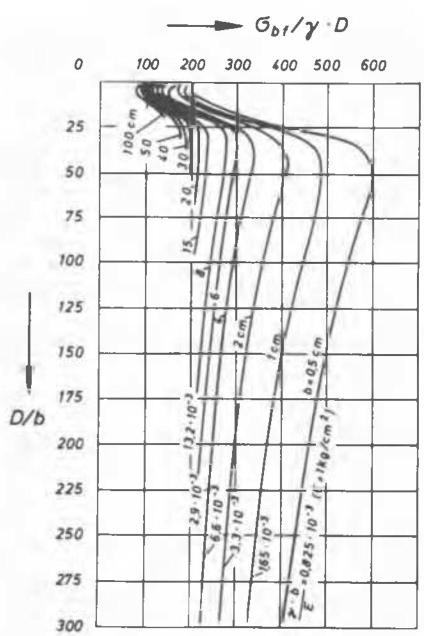

Fig. 6 Depth dependence of point pressure acc. to DE BEER (1963), accounting for soil compressibility E.

concluded that the decrease of σ_b with increasing b according to formula (1b) resp. (1c) is exceeded accounting for $\varphi = f(\sigma_m)$ as DE BEER (1963) has done, and the inclination at 45° on Figs. 5a and 5b may be no upper limitation. (Because σ_b is valid till σ_{bf}, see Fig. 4, this conclusion from Fig. 6 on formula (1b) is allowed for.) However, from the available measurements on Fig. 5a it may be concluded that the 45° inclination cannot be exceeded by notable amounts. (Note: On Fig. 6 besides b the parameter $\gamma.b/E$ is used with E defined as Youngs modulus of the sand grains. It may be helpful to assume E more generally as a modulus of soil compressibility including grain crushing, as the latter may be of more importance beneath piles. The effect of b is caused by E, which may be conceived to be due to the increasing number of sand grains involved with increasing b. The increasing soil compressibility is then caused by the deformation and crushing of the sand grains under increasing σ_m resp. σ_b.)

No clear information from Fig. 5a can be gained about the influence of the settlements. Logically increasing settlement s_b should have the same effect as the increasing soil strength $K \sim I_D \sim q_c$ has. For the comparison of s_b and I_D only FE-calculated curves of MEISSNER (1979) are available, used on Fig. 5c in connection with suitable expermental curves of MUHS (1959) for s_b = const = 2 cm and changing strength.

Last but not least, it should be admitted that only a small part of the available experimental data has been gained for the purpose aimed at here. Therefore they could not be used without reservations resp. the requirements on the accuracy of the data had to be restricted. For example the presumption of equal settlements, the requirement of equal resp. sufficient pile lengths and of equal construction procedure was often not fulfilled with desirable accuracy for the compared piles. In so far Fig. 5a requires checking with additional tests.

2.3 Settlement s_{bf} for Complete Failure

From the differences of the definitions in Table I it can be concluded that formerly no generally accepted definition of failure existed. With the experiments shown on Fig. 3a it now becomes recognizable what "complete" failure is. Moreover, it became known that the settlement s_{bf} at failure (e.g. defined at 95 % of σ_{bf}) is very different in magnitude for different pile types, i.e.,

$$s_{bf} = (0{,}1 \cdot b) \ldots (3 \cdot b) \qquad (3)$$

with the lower limit for displacement piles in dense sand and the upper for bored piles in loose sand. Particularly for bored piles the failure settlements are too large as to be practicable for the derivation of an ultimate load. The conclusion from this is the compulsion to define $Q_u \leq Q_f$ as is shown in the next section.

3. NEW RULES FOR THE PRACTICE

3.1 Definition of an Ultimate Load Q_u when Derived from Test Loadings

The consequence drawn from Table I was that the formerly used definitions of Q_u are not valid for large piles with b > 1 m. On the basis of improved knowledge about the failure state acc. to 2.3, which would enable us to determine a accurate failure load Q_f, it must now be concluded that Q_f is not suitable for the application in formula (1). The reasons are:

a) that the settlement s_{bf} required for complete failure acc. to 2.3 often could not be reached with the usual test loading devices. (For a bored pile of b = 1.5 m in sand of medium density it would be $s_{bf} \approx 0.3 \times 1.5$ m = 0.5 m! To reach the corresponding value of Q_f would require a costly device.)

b) that with safety factors F.S. ≈ 2, which have proved satisfactory, under working conditions admissible settlements s_{ad} would occur, which are by far too large, e.g.

$$s_{ad} \approx s_{bf}/(F.S.)^2 = 0.5\ m/4 = 12\ cm$$

when the load settlement curve is approximated as a parabola $s_b \sim Q^2$.

The consequence is to define $Q_u \leq Q_f$, assuring that in formula (1) usual values of F.S. ≈ 2 can further be used for all sorts of piles, leading to acceptable values of s_{ad} for the resulting Q_{ad}. For this purpose the practically well tried assumption $s_b \sim Q^2$ is used with which a suitable Q_u can be defined in dependence on s_{ad} by

$$Q_u = Q(s_u) \leqq Q_f \quad \text{with} \quad s_u = s_{ad} \cdot (F.S.)^2 \leqq s_{bf} \qquad (4)$$

(In DIN 1054, 5.4.3, ed. Nov.1976, it is defined $s_u = 4 \cdot s_{ad}$.) A distinction between elastic and plastic parts of s_{ad} is not relevant when refering to s_{ad}.)

Naturally it should be tried in practice to reach Q_f but as was shown this is often not possible.

3.2 An Alternative Derivation of the Admissible Load Q_{ad}

For conventional small piles it was formerly presumedin Germany that plastic settlement parts under working conditions are negligible. When calculating statically undetermined systems this means it could be assumed that these have elastic soil reactions only. In case of large bored piles this principle was left as this is justified by technical reasons and an economic need as well:

For large piles the elastic settlement parts are small compared with the plastic ones. The admissible settlements of large bored piles with 1 m < b < 2 m are normally 2 to 4 cm when Q_{ad} is calculated acc. to formula (1), using Q_u acc. to formula (4); and no damages have been experienced in case of rising structures of normal sensitivity to settlements. The explanation is that the pile spacing is usually 3 to 5 m and the ß-values acc. to SKEMPTON/MAC DONALD (1956) then are not exceeding an admissible magnitude of 1/3oo. - However, in cases where the loads differ from pile to pile very much or where the structures are very sensitive, an alternative calculation is required to assure that the settlements are admissible. For this purpose acc. to DIN 4o14, part 2 for large bored piles, the following alternative calculation - besides the one acc. to formula (1) - is required, applying settlement dependent pile loads

$$Q_{ad} = Q(s_{ad}) \qquad (5)$$

Q(s) is to be determined acc. to Fig. 7. The data of this Fig. were gained with 35 test loadings, 4 of which were published by FRANKE (1973) and further 8 by FRANKE/GARBRECHT (1977). (The skin friction τ_m is approximated as diameter indepent having failure settlements of s_{mf} = 2 cm in sand and 1 cm in clay.)

3.3 Extrapolation of Point Pressure resp. Pile Load to Larger Piles Measured with Small Piles

High costs for test loading large piles have caused a tendency to measure the point pressures with small piles and to extrapolate them to larger piles. The question then arises whether a

a) Point-Pressure $\sigma_b(s)$ (Static Cone Penetration Resistance > 10 MN/m^2)

Settlement	Point Pressure
[cm]	[MN/m^2]
Piles without enlarged bases	
1 2 3 15	0.5 0.8 1.1 3.4
Piles with enlarged bases	
1 2 3 15	0.35 0.65 0.90 2.4

b) Skin Friktion τ_m in Sand (τ_m linear till s = 2cm) (τ_m = const for s > 2cm)

Strengh of sand	Static Cone Penetration Resistance	Depth below surface	Skin friction
	MN/m^2	m	MN/m^2
very small	< 5	—	0
small	5 to 10	0 to 2 2 to 5 > 5	0 0.03 0.05
medium	10 to 15	0 to 2 2 to 7.5 > 7.5	0 0.045 0.075
high	> 15	0 to 2 2 to 10 > 10	0 0.06 0.10

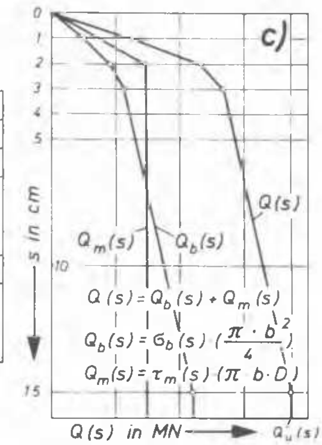

Fig. 7 Extract from DIN 4014, part 2

scale effect does exist. For bored piles in clay and Keuper marl no such effect was found (DVORAK 1976, LEACH et al. 1976, JELINEK/KORECK/STOCKER 1977), also for large bored piles in sand of medium density (FRANKE/GARBRECHT 1977). But from Fig. 5a the consequence must be drawn that no general conclusion is possible. Without approvals in particular cases a scale effect according to formula (1b) should be accounted for, when extrapolating σ_b, as this gives a sufficient limitation to the unsafe side.

In practice it is almost always the pile load Q which must be determined, and it is by far easier to do this without the complicated separation of σ_b and τ_m. As shown in the appendix it is possible to extrapolate the complete pile load Q, if D = const, $(D/b) > (D/b)_{cr}$ and s_b = const. This can be done using the formula

$$\sigma \cdot b = \text{const} \quad \text{with} \quad \sigma = Q/(\pi \cdot b^2/4) \qquad (6)$$

This formula is valid even when $b \neq b_m$, the shaft diameter, if b/b_m = const.

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APPENDIX

$$\frac{Q_1}{Q_2} = \frac{\sigma_1 \cdot (b_1^2 \cdot \pi/4)}{\sigma_2 \cdot (b_2^2 \cdot \pi/4)}$$

$$= \frac{\sigma_{b1} \cdot (b_1^2 \cdot \pi/4) + \tau_{m1} \cdot \pi \cdot b_{m1} \cdot D_1}{\sigma_{b2} \cdot (b_2^2 \cdot \pi/4) + \tau_{m2} \cdot \pi \cdot b_{m2} \cdot D_2}$$

$$\frac{(b_1/b_{m1}) \cdot \sigma_1 \cdot b_1}{(b_2/b_{m2}) \cdot \sigma_2 \cdot b_2}$$

$$= \frac{(b_1/b_{m1}) \cdot \sigma_{b1} \cdot b_1 + \tau_{m1} \cdot D_1 \cdot 4}{(b_2/b_{m2}) \cdot \sigma_{b2} \cdot b_2 + \tau_{m2} \cdot D_2 \cdot 4}$$

$(b_1/b_{m1}) = (b_2/b_{m2})$, $D_1 = D_2 = D$, $\tau_{m1} = \tau_{m2} = \tau_m$

$4 \cdot D \cdot \tau_m = a$

$$\frac{\sigma_1 \cdot b_1}{\sigma_2 \cdot b_2} = \frac{\sigma_{b1} \cdot b_1 + a}{\sigma_{b2} \cdot b_2 + a}$$

As - acc. to formula (1c) - $\sigma_{b1} \cdot b_1 = \sigma_{b2} \cdot b_2 = C$ it follows

$$\frac{\sigma_1 \cdot b_1}{\sigma_2 \cdot b_2} = 1 \quad \text{resp. } \sigma \cdot b = \text{const.}$$

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