The Determination of the Permissible Point-load of Piles by Means of Static Penetration Tests

Détermination de la force portante limite admissible pour des pieux au moyen d'essais de pénétration statique

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Summary

The results of a large number of loading-tests on piles for which static penetration tests were carried out, have been collected, and existing relationships have been investigated by statistical methods. These show the degree of accuracy for a given factor of safety for the determination of the permissible point-load of piles by means of static penetration tests. From this follows the factor of safety which the method requires for its application in practice. In addition, the factors influencing the results are considered and in particular the role which the diameter of the pile and the cone resistance of the penetrometer play. Finally, a formula is presented for the determination of the permissible point-load of piles by means of static penetration tests which is based on a large number of loading-tests.

Sommaire

Pour étudier le rapport existant entre le taux de rupture du sol sous les pointes de pieu et la résistance de pointe de sondages de pénétration statiques, on a rassemblé les résultats d'un grand nombre d'essais de chargement de pieux, pour lesquels des sondages de pénétration statiques avaient été effectués. On a ensuite examiné ces résultats à l'aide de méthodes statistiques. De cette manière il est possible de calculer le degré d'exactitude avec lequel on peut, pour un coefficient de sécurité donné, déterminer la force portante de pointe d'un pieu, à partir de la résistance de pointe trouvée au pénétromètre statique. On en déduit la marge de sécurité à observer.

Introduction

One of the most important questions of foundation technique is that of the permissible load of piles. Whilst for driven piles it is possible to find the bearing capacity approximately from the driving resistance using empirical values, or from a pile driving formula, the reliability of which is proved for local subsoil conditions, it is very difficult to determine the exact failure load for bored piles if a few of them are not subjected to a load test. This is particularly necessary if a piled foundation is carried out in a soil the strength of which is only approximately known. Loading tests are expensive and take up much time, so that they should be reduced to a minimum.

The problem of selecting piles for load tests must be solved in such a manner that the true subsoil conditions are revealed. Unless this is done, a pile foundation may be designed on the basis of the most unfavourable test result. Several theoretical and practical methods are known for determining the bearing capacity of piles. Theoretical methods suffer from the disadvantage that simplifying assumptions must be made to allow for complex stress and strain conditions at the pile point. This frequently produces unreliable results. A model test in the form of a static penetration test can be applied more successfully. The apparatus and the method of carrying out the test has been frequently described (e.g. PLANTEMA, 1948).

There is a relationship between the cone resistance of this penetrometer and the ultimate bearing load of a pile. In Holland and Belgium in particular, where pile foundations must often be used this penetration test was developed almost 30 years ago and used from the beginning to aid the design of pile foundations ((LABORATORIUM VOOR GRONDMECHANIKA DELFT 1936, DE BEER, 1941 HUIZINGA, 1941).

A qualitative determination of the permissible point-load of piles was first proposed by VAN DER VEEN (1950), who concluded from the results of twenty-one load tests that for the permissible point-load of the pile the cone resistance of the static penetrometer has to be divided by a factor of safety of 3 ($F = 3$).

Factor of Safety

This factor of safety is the product of two other factors of safety namely, F_1 and F_2. The value of F_1 was found from the fact that a given ratio of cone resistance of the static penetrometer w, to failure stress of soil under the pile point W, is exceeded only in exceptional cases. For the permissible stress, the failure stress has to be divided by the value of F_2. Van der Veen derived $F_1 = 1{\cdot}75$ from his test results. With this value a factor of safety of 3 is obtained, if the permissible load is taken to 60 per cent of the ultimate load of the pile. After fifteen completing tests, F_1 was later reduced to 1·5, which gives a total factor of safety of 2·5 (VAN DER VEEN and BOERSMA, 1957).

Distribution of the Stress Ratio

VAN DER VEEN's proposal is of great practical importance. However, it is based on an assumption of the possible range of the ratio of come resistance to failure stress of soil at the

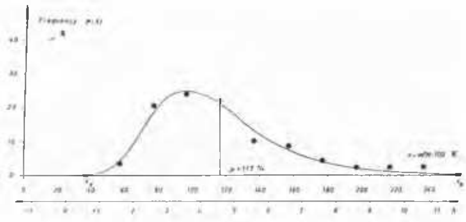

Fig. 1 Frequency distribution for the stress ratio w/W for 88 load tests on piles.
Frequency in per cent an intervall $dx = 20$ per cent.
Distribution as found from all 88 test for
$0 < w < 180$ kg/cm²
$0 < A < 12\,000$ cm²
— Theoretical distribution $\varphi(\lambda) = 0{,}25 \cdot e^{-3 \log 2e \left(\frac{\lambda}{3.5}\right)}$
Mean value $\mu = \int_{x_a}^{x_b} \chi \cdot \varphi(\chi) dx = 117$ per cent
Standard deviation $\sigma = \int_{x_a}^{x_b} (x - \mu)^2 \cdot \varphi(\chi) d\chi = 40$ per cent

Distribution de la fréquence pour la proportion w/W de la pression à la pointe pour 88 essais de chargement de pieux
— Fréquence (pour cent) pour un intervalle $d_x = 20$ pour cent
Distribution trouvée d'après les 88 essais pour
$0 < w < 180$ kg/cm²
$0 < A < 12\,000$ cm²
— Distribution theorique : $\varphi(\lambda) = 0{,}25 \cdot e^{-3 \log 2e \left(\frac{\lambda}{3.5}\right)}$
Valeur moyenne $\mu = \int_{x_a}^{x_b} \chi \cdot \varphi(\chi) d\chi = 117$ pour cent
Divergence moyenne $\sigma = \int_{x_a}^{x_b} (\chi - \mu)^2 \cdot \varphi(\chi) d\chi =$ 40 pour cent

pile point. Therefore, it seems to be useful to investigate the distribution of the stress ratio using a larger number of test results. The table gives eighty-eight results of load tests on piles which can be compared with the results of static penetration tests. The base areas of the piles vary between 109 and 11684 sq.cm. The value of the cone resistance which is found either from the curve of the minimum resistance or better, from the averaged resistance according to van der Veen and Boersma (1957), extends over a range of 25 kg/cm² to 180 kg/cm².

In nearly all tests the soil at the pile point was sand or gravel. The plotting of the frequency of the stress ratio shows a skewed distribution. If this is approximated by a theoretical distribution it is possible to determine, using the methods of the probability theory, with what probability a given stress ratio w/W may or may not be exceeded. Each stress ratio corresponds to a certain probability that the failure stress of the soil under the pile point is not smaller than expected (Fig. 2). If a 95 per cent probability is required, i.e. that for only 1 in 20 piles the determined failure stress is estimated higher than it actually is, one obtains a stress ratio and, hence, a factor of safety $F_1 = 1{\cdot}95$. Using the value chosen by van der Veen and Boersma of 1·5 roughly 17 per cent of all piles are likely to have a failure load smaller than expected (Fig. 2). This, however, is of not so great a consequence as it might seem as only a part of the failure load is permitted for the working load and also 83 per cent are likely to have a bearing capacity that is higher than can be expected. These piles therefore have some reserve of bearing capacity. If the factor of safety is 2·5, only 1 per cent of all piles will be actually loaded with more than the failure load.

On the other hand, the expected ultimate bearing capacity is estimated for 50 per cent of all piles with less than 75 per cent of its actual value.

Scattering of the Stress Ratio

The explanation for the heavy scattering of the stress ratio lies in the fact that it is influenced by several factors. For further investigation of the problem it is necessary to search for at least some of them. The more important are :

(1) The determination of the failure load in a load test : In only a few cases is the failure load obtained from a clearly vertical part of the load-settlement curve.

Therefore, the failure load is mainly defined in a more a less arbitrary manner. Some definitions are based on the shape of the load-settlement curve, others on the absolute settlements or the settlements relative to the diameter of the pile. For this reason, the determination of failure load is not independent of the applied definition.

(2) The diameter of the pile :

The rupture lines existing under the point of the pile will, as a rule only be approximately similar to those under the cone of the penetrometer. Furthermore, both affect different ranges of soil, which leads, to a scattering of the results particularly in soil with thin strata of various strengths. This has to be considered when evaluating the cone resistance, for which a value averaged over the possible range of the rupture lines (Van der Veen and Boersma, 1957) or, less accurately, the curve of the minimum cone resistance, should be taken.

(3) The magnitude of the cone resistance : This is a direct measurement of the strength of the soil which influences the rupture lines.

(4) The type of soil: For equal cone resistance, the strength properties depend to a large extend on the type of soil (Menzenbach, 1959).

(5) The type of pile : For equal subsoil conditions, driven piles usually show a higher bearing capacity than bored piles.

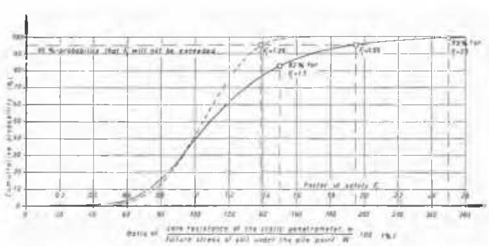

Fig. 2 Comultative probability of the stress ratio
—— for all 80 tests
- - - for 48 test
$A < 2\,000$ cm²
$W < 100$ kg/cm²

Courbes cumulatives de la fréquence pour la proportion de la pression à la pointe
—— pour les 80 essais
- - - pour 48 essais
$A < 2\,000$ cm²
$W < 100$ kg/cm²

Influence of the Base Area of the Pile and the Cone Resistance on the Factor of safety.

It is hardly possible to allow for every factor. As regards the failure load, this is quite impossible, because it is not determined on the basis of standard rules. However, it seems possible with the data available to investigate the influence of the diameter of the pile and the magnitude of the cone resistance. One can easily recognize that they are significant if the stress ratio is plotted against the diameter of the pile for three different ranges of cone resistance (Fig. 3-5). If the high cone resistances ($w > 100$ kg/cm^2) and the large base areas of the piles ($A > 2\,000$ cm^2) are not taken into consideration one obtains from 48 tests a frequency distribution which is almost normal and has a much smaller standard deviation and hence a smaller deviation of the data from the mean (Fig. 6).

The practical significance of this may be seen from Fig. 2. For the same probability of 5 per cent that F_1 is exceeded, the factor of safety for the dotted curve is only 1·39 instead

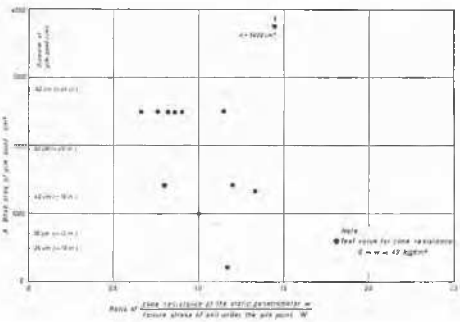

Fig. 3 Stress ratio as a function of the base area of the pile for a range of cone resistance from 0 to 49 kg/cm^2

Rapport des pressions de rupture à la pointe entre 0 et 49 kg/cm^2 en fonction de la section transversale de la pointe de pieu.

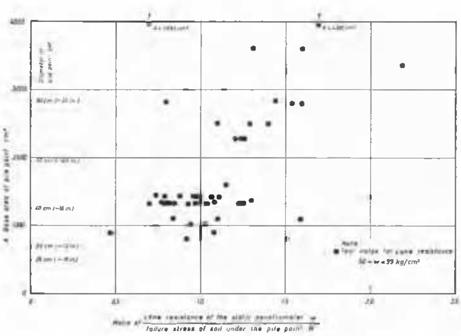

Fig. 4 Stress ratio as a function of the base area of the pile for a range of cone resistances from 50 to 99 kg/cm^2

Rapport des pressions de rupture à la pointe entre 50 et 99 kg/cm^2 en fonction de la section transversale de la pointe de pieu.

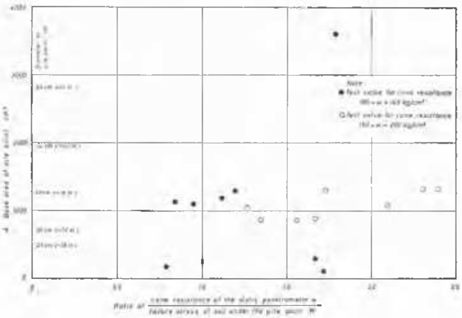

Fig. 5 Stress ratio as a function of the base area of the pile for a range of cone resistances from 100 to 200 kg/cm^2

Rapport des pressions de rupture à la pointe entre 10 et 20 kg/cm^2 en fonction de la section transversale de la pointe de pieu.

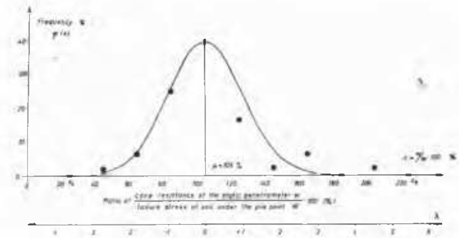

Fig. 6 Frequency distribution for the stress raion w/W for 48 load tests on piles.

Frequency in per cent for an intervall $d_x = 20$ per cent

Distribution as found from 48 tests for :

$0 < W < 100$ kg/cm^2

$0 < A < 2\,000$ cm^2

— Theoretical normal distribution

$$\varphi(\lambda) = \frac{1}{\sqrt{2\pi}.\sigma} . e^{-\frac{\lambda^2}{2}} ; \lambda = \frac{\chi - \mu}{\sigma}$$

Mean value : $\mu = \int_{x_a}^{x_b} \chi \cdot \varphi(\chi) \cdot d_x = 105$ per cent

Standard deviation : $\sigma = \int_{x_a}^{x_b} (\chi - \mu)^2 \varphi(\chi) d_x = 22$ per cent

Distribution de la fréquence du rapport des pressions de rupture w/W de la pression à la pointe pour 48 essais de chargement

— Fréquence (pour cent) pour un intervalle $d_x = 20$ pour cent

Distribution trouvée dans 48 essais pour

$0 < W < 100$ kg/cm^2

$0 < A < 2\,000$ cm^2

— Distribution normale

$$\varphi(\lambda) = \frac{1}{\sqrt{2\pi}.\sigma} . e^{-\frac{\lambda^2}{2}} ; \lambda = \frac{\chi - \mu}{\sigma}$$

Valeur moyenne : $\mu = \int_{x_a}^{x_b} \chi \cdot \varphi(\chi) . d_x = 105$ pour cent

Divergence moyenne $\sigma = \int_{x_a}^{x_b} (\chi - \mu)^2 \varphi(\chi) d_x = 22$ pour cent of 1·95 for all 88 tests. For a closer investigation of the influence of the diameter of the pile and the magnitude of cone resistance the author suggests starting from the consideration that, for very small diameters of piles which are approximately the same as the diameter of the penetrometer, F_1 is equal to 1 and that, secondly, for $w = 0$, F_1 is also equal to 1 as in that case W will be zero. For any given cone resistance w (kg/cm²) and base area of the pile A (cm²) the equation for the factor of safety can be written in the general form

$$F_1 = 1 + a \cdot w^{b} \cdot A \quad \ldots\ldots (1)$$

which has proved to be suitable from preliminary investigations. In equation (1), a and b are constants which are calculated by the method of the least squares (Σv^2_i) by putting

$$\frac{\partial(\Sigma v^2_i)}{\partial a} = \frac{\partial(\Sigma v^2_i)}{\partial b} = 0 \quad \ldots\ldots (2)$$

One obtains for a standard deviation of $s = \pm\ 0{\cdot}3$

$a = 5{\cdot}10^{-7}$

$b = 1{\cdot}3$

The dependance of F_1 on A and w is shown in Fig. 7.

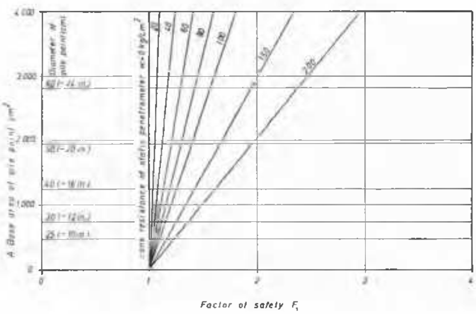

Fig. 7 Dependance of the factor of safety F_1 of the base area of the pile A and the cone resistance w

$F_1 = 1 + 5{\cdot}10^{-7} w^{1,3} . A$

Standard deviation : $s = \pm\ 0{,}3$

Rapport entre le coefficient de sécurité F_1 la section transversale de la pointe de pieu A et la résistance w à la pointe.

$F_1 = 1 + 5{\cdot}10^{-7} .\ w^{1,3} . A$

Divergence moyenne : $s = \pm\ 0{,}3$

Hence, one can write for the determination of the permissible point-load of a pile from the result of a static penetration test

$$P\text{permissible} = \frac{1}{F_1} = \frac{1}{F_2} \cdot \frac{w \cdot A}{1\,000} \cdot \frac{w \cdot A}{1\,000\ F_2(1 + a \cdot w^{b} . A)} \quad \ldots\ldots (3)$$

Where :

Ppermissible = permissible point load of the pile with a base area A (cm²) in tons.

F_2 = Pfailure/Ppermissible.

For a constant cone resistance, the permissible point-load decreases with the diameter of the pile, as the failure load is often fixed on the basis of a given settlement which is reached for piles of larger diameters at smaller point stresses. For equal base areas, the factor of safety increases with cone resistance. This is probably partly due to the fact that in soils of high strength a clear failure could not be produced in all cases. Hence, the factor of safety for is more likely to be too great than too small. The value $F_1 = 1{\cdot}5$ proposed by van der Veen and Boersma is valid for a range covering the most ¡frequently used piles of medium diameter and cone resistance.

Conclusion

These investigations have proved that for the determination of the permissible point-load of piles from the result of a static penetration test, the cone resistance and the base area of the pile must be taken into account.

The factor of safety is based on the results of 88 load tests. The standard deviation is 0·3.

From a greater amount of data improved accuracy can be expected because the influence of the type of soil and the type of pile can then be further investigated.

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Author and reference	Type of soil	Depth	Static penetration test		Pile		(5)/(7) × 100	Notes
			cone resistance w'	averaged cone resistance w^*	base area of pile point	failure stress W		
——	—	m	kg/cm²	kg/cm²	cm²	kg/cm²	per cent	———
1	2	3	4	5	6	7	8	9
LOHMANN	loamy coarse sand	16·0	111	76	1 017	74	103	Load tests and penetration tests Amstelstation point resistances of static penetrometer averaged from 4 tests
De Ingenieur,		16·0	111	76	2 289	63	121	
1938, p. 117	— —	16·5	116	79	1 017	84	94	
	loamy fine sand	15·6	93	82	2 826	57	144	
	l.c.S	16·2	125	82	2 289	66	124	
	l.c.S	16·4	126	83	2 289	66	126	
	l.f.S	15·1	60	57	2 826	71	80	
BOONSTRA,	S	14·5	195	150	1 325	65	231	
De Ingenieur,	S	10·7	130	70	1 325	72	97	
1940, p. 33	S	14·5	115	95	1 325	76	125	
	S	16·6	110	50	1 325	61	82	
	S	19·4	125	75	1 325	79	9	
HUIZINGA,	clay f.S	12·0	18	16	1 325	12	133	Load tests and penetration tests Zwijndrecht (Rotinoff-Caisson ⌀ 122 cm)
De Ingenieur,	clay f.S	15·6	63	54	1 325	64	85	
1940, p. 55	clay f.S	16·0	63	52	1 325	50	104	
	clay f.S	16.1	64	53	11 684	76	70	
	f.S and gravell	17·3	140	80	1 325	65	123	
	f.S and gravell	19·7	84	84	1 325	120	70	
		16·8	100	105	~ 1 100	110	95	Load tests Ysselstein
		18·0	92	92	~ 1 100	110	84	
		16·0	80	70	~ 1 370	54	130	Load tests Amstelstation II
		16·2	80	78	~ 950	78	100	
		23·8	134	134	~ 1 130	160	84	Load tests Wadvinxveen
		11·1	40	40	~ 1 000	40	100	Load tests Jutfaas I
		11·1	40	40	~ 1 000	52	77	
		11·5	50	50	~ 1 000	68	74	
		25 3	160	150	~ 1 050	118	127	Load tests Berg'sche Hoek
		16·0	130	80	~ 2 800	52	154	Load tests Amstelstation IV
		16·0	170	80	~ 2 800	50	160	
		16·2	84	84	~ 900	78	108	Load tests Sliedrecht
		20·4	70	70	~ 900	148	47	
		15·6	90	82	~ 890	82	100	Load tests Woerden
		18·0	210	180	~ 890	108	167	
		11·0	130	64	~ 1 100	58	110	Load tests Diefdyk
		12·2	74	70	~ 1 100	44	159	
		15·0	190	160	~ 1 100	76	210	
		20·7	116	110	~ 1 200	98	112	Load tests Alblasserdam
Plantema *Proc. Rott.*	f.S	13·4	60	60	1 422	30	200	Continuous loading of on pile at different depths
1948, vol. IV, p. 112	and	14·4	42	36	1 422	30	120	
	g.S	15·4	45	40	1 422	50	80	
		16·4	68	62	1 422	56	111	
		17·4	72	72	1 422	72	100	
		18·4	82	78	1 422	73	107	
		19·4	90	80	1 422	83	96	
		20·4	91	82	1 422	93	88	
		21·4	80	78	1 422	80	98	
		23·4	50	50	1 422	63	79	

Author and reference	Type of soil	Depth	Static penetration test: cone resistance w'	Static penetration test: averaged cone resistance w*	Pile: base area of pile point	Pile: failure stress	(5)/(7) × 100	Notes
—	—	m	kg/cm²	kg/cm²	cm²	kg/cm²	per cent	—
1	2	3	4	5	6	7	8	9
Van der Veen *De Ingenieur*, 1950, p. 67			70 52 140 85 52 85	57 55 70 85 55 57	1 340 800 1 340 1 340 3 000 3 000	57 60 64 110 60 50	100 92 109 77 92 114	Tests of BOONSTRA at the Bridge over the Old Maas at Dordrecht
			125 95 125 125 28 240	125 80 100 100 28 100	176 805 286 259 207 109	159 53 60 100 24 58	79 151 167 100 117 172	Tests of Afdeling Utiliteitsbouw van de Publike Werken for Schiphol air-port (steel piles)
	c.S + G S f.S f.S S + G	14·5 10·7 14·5 16·6 19.4	195 130 115 110 125	156 75 95 50 75	1 325 1 325 1 325 1 325 1 325	65 72 76 61 79	240 104 125 82 95	Tests of BOONSTRA at the Hendriks Ido Ambacht (Rijkswaterstraat) concrete piles
			100 130 105	75 80 100	3 600 3 600 3 600	47 61 56	160 131 179	Tests at the Electric Zentral Hemweg
			90	72	805	72	100	Navigationbuilding Schiphol
Mierlo A. Koppejan " Bouw ", Jan. 1952	S	18·8	90	90	4 300	53	170	
Van der Veen and Boersma *Proc. London*, 1957, vol. II, p. 72	S S S S S S S S S S S S S S	13·1 13·2 12·6 14·2 12·6 13·1 24·3 12·6 14·0 14·0 12·1 12·0 16·2 12·8		36** 41 25 60 37 44 75 49 72 70 46 53 110 51	2 500 2 500 3 000 2 500 2 500 2 500 2 500 2 500 1 600 3 364 2 500 2 500 1 296 1 444	44 54 52 43 56 49 58 57·2 62·5 32 40 48 92 69	82 76 48 140 66 90 129 86 115 219 115 110 120 74	Tests in Amsterdam : Precast Slotermeer Precast Slotermeer Franci Slotermeer Precast Geuzenveld Precast Geuzenveld Precast Geuzenveld Vibro Geuzenveld Precats Geuzenveld Precast Kattenslot Precast Nemdvo Precast Slotervaart Precast Slotervaart Precast Slotervaart Precast Slotervaart
Muhs 1959, *Bauverlag*, Wiesbaden	medium-c.S mS-c.S mS-c.S mS-c.S m.S	5·8 5·8 6·2 6·3 6·7	178 178 180 180 23	170 170 180 170 23	1 370 1 310 865 860 5 020	85 98 133 127 159	200 173 135 156 145	Failure stress = Maximum reached stress

* Averaged cone resitance from curve of minimum resistances.

** Averaged cone resistance according to VAN DER VEEN and BOERSMA (1957) ;
averaged cone resistance over a depth of
1 diameter of pile under pile point
3·75 diameter of pile above pile point

The static penetration tests were carried out with the Dutch penetrometer ;
base area of the point 10 cm²
angle of cone 60°