Variability among Piles of the Same Foundation

Variabilité entre Pieux de la Même Fondation

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SYNOPSIS. The load-settlement relationships of supposedly identical piles, belonging to the same foundation, show actually an appreciable variability due to small unavoidable constructional differences and to the variability of the subsoil from point to point. Experimental evidence of such a variability, coming from proof loading tests on 8 large piled foundations in 5 different sites, is presented and discussed. The influence of the variability of piles behavior on the design of foundation structures is analyzed, referring to two simple schemes: a column supported by a group of 3 piles and a beam supported by a row of 25 piles. In the cases considered, the influence has been found to be rather significant.

INTRODUCTION

The load-settlement relationships of supposedly identical piles belonging to the same foundation show actually an appreciable variability. Differences in the behavior of the piles can arise from the inherent variability existing within piles due to small unavoidable constructional differences, and also from the variability of the soil from point to point.

Published data on this problem, that has an unknown bearing on the design of foundations, are very scarse (Whitaker, 1976 p.113). The aim of this paper is to present some experimental evidence about the variability of behavior of piles belonging to 8 different large piled foundations and to discuss its influence in the design of piles cap, with reference to simple foundation schemes.

SUBSOIL PROPERTIES

The 8 foundations considered are located in five different sites (A to E) in Southern Italy and Sicily; at all sites the subsoil consists essentially of sand deposits, with different percentages of fine gravel and/or silt. A simplified sketch of subsoil conditions is reported in fig. 1.
The mechanical properties of the soils have been evaluated on the basis of static or dynamic cone penetrometer soundings. A detailed evaluation of subsoil properties at site A

(*) The Authors have given an equal contribution to the investigation reported in the present paper.

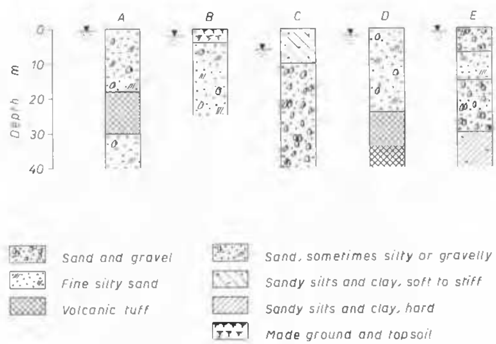

Fig. 1. Simplified sketch of the subsoil condition at the five sites considered.

is reported in fig. 2. In order to smooth out local irregularities, the penetration resistances is represented by its mean value q_1 over vertical lengths Δ_1 = 4 m, that is considered a minimum significant length affecting the pile behavior. A number of profiles of q_1 versus depth are available at the site; in fig. 2 the mean values $\bar{q}_1$ of q_1 for an increasing number of profiles belonging to zones of increasing area are reported. The corresponding coefficients of variation v_{q1} (v_{q1} = standard deviation $\tilde{q}_1$/mean value $\bar{q}_1$) are plotted in the same figure.

It is clearly seen that soil properties and their scatter depend on depth below ground surface but are practically unaffected by

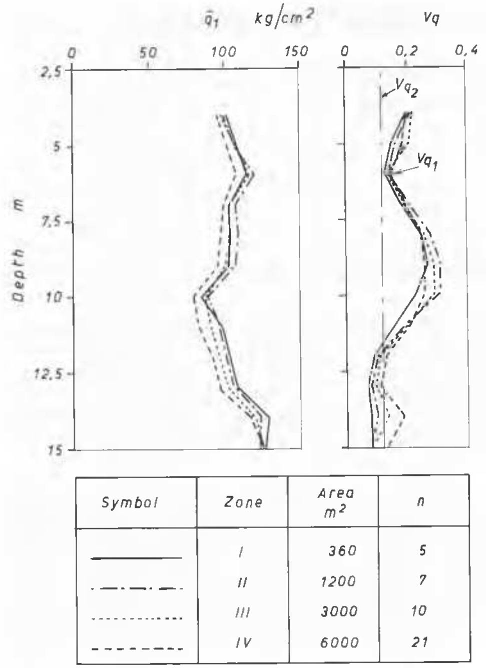

Fig. 2. Variability of subsoil properties at site A. q_1 is the mean value of cone penetration resi= stance over lengths of 4 m; $\bar{q}_1$ is the mean value of q_1 over the profiles belonging to a zone of the site. Zones of increasing dimensi= ons in plan are considered, each inclu= ding the smaller previous one.

increasing the dimensions in plan of the in= vestigated area. In the same fig. 2 is also reported the coefficient of variation v_{q2} of the cone resistance averaged, for each pro= file, over the entire thickness Δ_2 = 15 m of the deposit. It appears that the value of v_{q2} is lower than v_{q1}. The same trends have been obtained for other deposits.

In conclusion, available evidence seems to indicate that, in cohesionless deposits and within the dimensions under consideration:

(i) the mechanical properties of the soil, averaged over lengths relevant to the behavior of a single pile, are varia= ble from a vertical to another; neve= rtheless, their mean value and their scatter do not change increasing the investigated surface;

(ii) the variability of soil properties de= creases when they are averaged over a larger length, that is passing from a length relevant to the behavior of a single pile to a length relevant to the behavior of a pile group.

To characterize the five sites, for each pro= file the penetration resistances have been averaged over the length L of pile shaft (q_s) and below pile point (q_p). The value of q_p refers to a thickness of 0,5·L that, accor= ding to elastic theory (Dente, 1974), is re= sponsible for the major part of the settle= ment of a single pile.

In Table I the mean values and the variabil= ity of q_s and q_p for the five sites are re= ported. Average penetration resistances range between 80 and 150 kg/cm² for static and between 10 and 40 blows/ft for dynamic cone; the relative density of the soils is accord= ingly between "medium" and "dense".

PILES

Six out of the eight pile foundations consi= dered are made up by cast-in-situ driven piles with expanded base of the Franki type; the remaining two, by piles bored in bento= nitic mud. Relevant data are listed in Table II. A number of piles belonging to each fou= ndation have been subjected to proof loading tests, kept as a rule to 1,5 times the inte= nded service load P_s. A few piles have been tested under larger loads, but failure has never been attained. Altogether, 94 loading tests are considered.

The load-settlement response of the piles is variable; a first glance at the scatter of the results may be obtained from the load-settlement curves reported in fig. 3. It may be seen that the scatter is rather signifi= cant, and such that account should be taken of it in the construction specifications and in the design of the foundations.

To smooth out experimental results, each load settlement curve has been interpolated by means of an hyperbola (Chin, 1970) with equ= ation:

$$\frac{w}{P} = a + b\,w \qquad (1)$$

where w = settlement at load P; a, b = coef= ficients determined by fitting experimental results by least squares.

With reference to the specifications and to the design of a flexible foundation, the analysis has been focused on the settlement

TABLE I. Subsoil properties at the five sites considered

Site	Penetration tests		Pile length	Average penetration resistance					
				over shaft length L			between depths L and 1,5 L		
	Type	number	L (m)	$\bar{q}_s$	$\tilde{q}_s$	v_{qs}	$\bar{q}_p$	$\tilde{q}_p$	v_{qp}
A	S	29	8	99	16	0,17	80	19	0,24
			12	90	15	0,17	116	17	0,14
			18	99	16	0,16	(*)	(*)	(*)
B	S	5	9	82	12	0,15	125	3	0,03
C	D	6	15	8,5	2,4	0,28	15,5	5,2	0,33
D	S	8	15	152	19	0,13	107	20	0,I9
E	D	16	20	23,1	4,9	0,21	42,9	14,6	0,34

N.B. The cone penetration resistance is expressed in kg/cm^2 for static and in blows/ft for dynamic tests. S = static; D = dynamic cone penetrometer.

(*) Piles are point bearing over a tuff layer. E moduli of the tuff measured in uniaxial compression on 14 core samples are: $\bar{E}$ = 16.5 CC kg/cm^2; E = 2.878 kg/cm^2; v_E = C,17

TABLE II. Pile foundations

Site	Foundation	Piles		
		Type	Length L (m)	Nominal dia= meter D (m)
A	1	Driven	8	0,40
	2	"	12	0,40
	3	"	18	0,52
B	4	"	7,3	0,36
	5	"	7	0,46
C	6	"	15	0,50
D	7	Bored	14,5	0,45
E	8	"	20	0,80

TABLE III. Load-settlement data

Foun= dation	P_s (ton)	Settlement w (mm)						Load	
		P = P_s			P = 1,5 P_s			v_P	
		$\bar{w}$	$\tilde{w}$	v_w	$\bar{w}$	$\tilde{w}$	v_w	at $w=\bar{w}_{P_s}$	at $w=\bar{w}_{1,5Ps}$
1	60	1,89	0,78	0,41	4,36	2,31	0,53	0,23	0,25
2	60	2,19	0,83	0,38	5,61	2,14	0,38	0,19	0,12
3	120	3,35	1,02	0,30	11,68	8,14	0,70	0,16	0,19
4	40	2,14	0,23	0,11	3,55	0,49	0,14	0,11	0,12
5	60	1,75	0,33	0,19	2,92	0,46	0,16	0,15	0,14
6	80	1,43	0,51	0,36	2,76	1,07	0,39	0,22	0,20
7	60	1,10	0,21	0,19	2,40	0,68	0,28	0,11	0,12
8	110	0,92	0,20	0,21	2,03	0,69	0,34	0,13	0,13

at load P_s and 1,5 P_s. Their values have been calculated for each test by means of eq. (1); mean values $\bar{w}$, standard deviations w and coe= fficients of variation v_w are listed in the Table III.

At service load, the mean values of settle= ment are in the range 0,9 to 3,3 mm. Back-calculating the values of E modulus of the subsoil by means of the elastic theory (Poulos, 1972), these rather small settle=

ments correspond to E = 300 ÷ 1000 kg/cm^2, that are in the range of the expected values for medium to dense sands. The coefficient of variation v_w is rather large, ranging from 0,1 to 0,4; the variability increases further at P = 1,5 P_S.

With reference to the design of a rigid foundation, the significant parameter is the load P needed to produce in each pile a settlement equal to the mean settlement under loads P_S and 1,5 P_S. The values of these loads have been calculated by means of eq.(1); their coefficients of variation are listed in

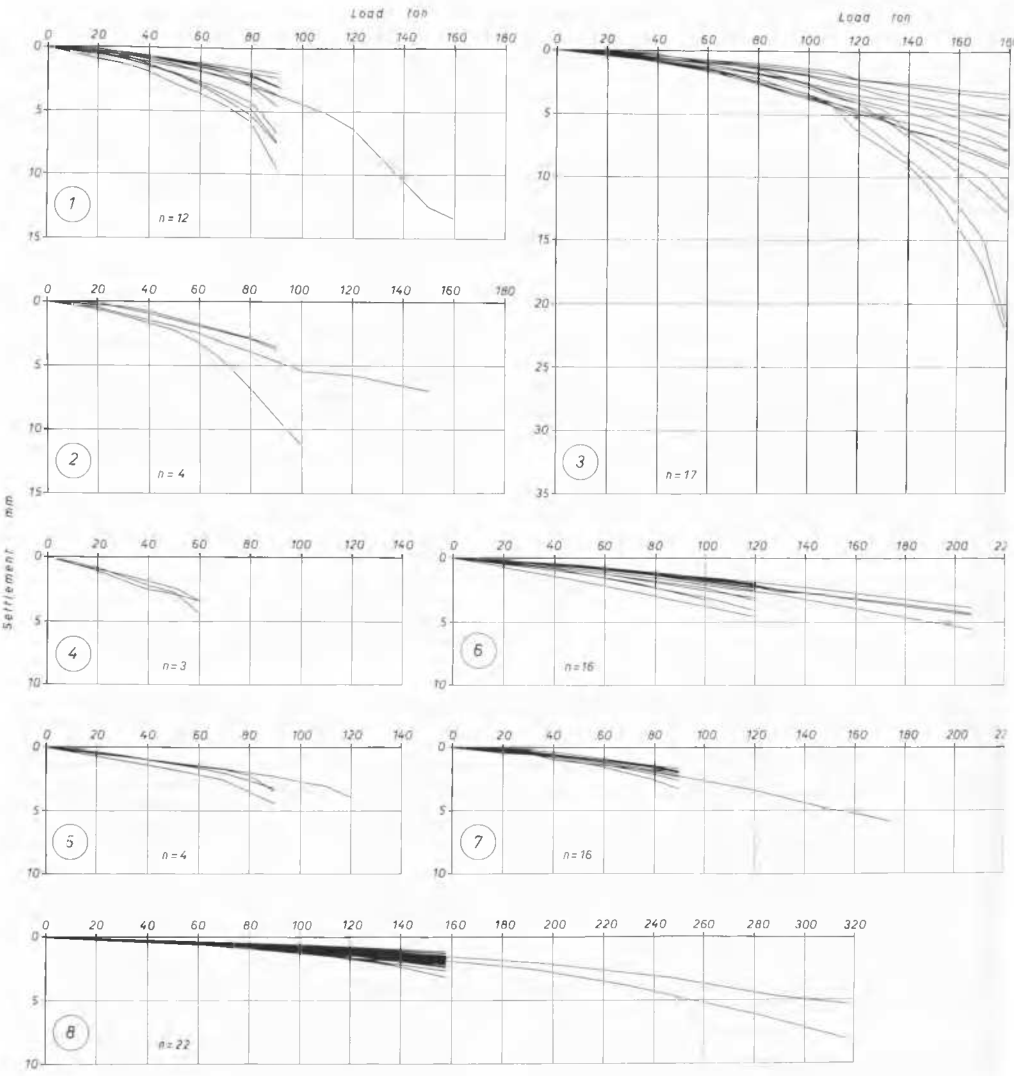

Fig. 3. Experimental load-settlement curves

table III. The variability of P is smaller than that of w, being v_P in the range 0,10 to 0,25 for both settlements.

DISCUSSION OF PILES BEHAVIOR VARIABILITY

The variability of load-settlement behavior of supposedly identical piles of a same foundation, outlined in the preceding paragraph, may be ascribed to some or all of the following factors:

1. misfeasances during pile construction; e. g., discontinuities in the concrete;
2. variability of the geometry (length, diameter) of the pile;
3. variability of the properties of the concrete;
4. variability of the effect of pile construction on the properties of the surrounding soil;
5. variability of the properties of the soil in the influence zone of each single pile.

The first four factors are connected with piling operations; however, the regularity of the experimental results and a careful inspection of the construction records at the five sites lead us to the conclusion that, for the cases considered, factor 1 may be excluded and factors 2 and 3 are of minor importance.

To evaluate the relative importance of the factors connected with piling operations and of the variability of the soil, the following analysis may be performed. On the basis of elastic theory, the settlement w of a pile under load P may be expressed as:

$$w = \frac{P}{ED} I \qquad (2)$$

where E is the Young modulus of the soil, and I a dimensionless influence coefficient depending on Poisson ratio of the soil, on the thickness of the deformable layer and on length to diameter ratio L/D of the pile. Eq. (2) may be written:

$$P = \frac{ED}{I} w \qquad (3)$$

For a cohesionless soil, the E modulus may be correlated to static cone resistance as follows (Mitchell, Gardner, 1975):

$$E = \alpha\, q \qquad (4)$$

where α depends on the nature of the soil (*).

(*) In the cases where dynamic penetration resistances are available, eq. (4) may still be employed making use of existing empirical correlations between the two tests (Mitchell, Garner, loc. cit.).

Within a pile group I (eq. 3) and α (eq. 4) may be assumed to be constant. Since the settlement of a single pile is due almost exclusively to the deformation of the soil between depths L and 1,5 L, the values of q_p (Table I) may be used. Combining eqs. (3) and (4) one gets:

$$P = K\, q_p\, w \qquad (5)$$

where $K = \alpha D/I$ is a constant.

If the values of q_p at the location of each tested pile were known, the influence of soil variability could be ascertained in a deterministic way. Since such data are not available, use may be made of the proportionality between the load P needed to produce the settlement w and the value of q_p. If the variability of load-settlement behavior were due only to factor 5, the coefficient of variation v_P should be equal to v_{qp}. This claim is substantiated by the data shown in fig. 2 demonstrating that, for each site, the scatter of soil properties does not depend on the particular location investigated.

In fig. 4 the coefficients of variation of P at $w = \bar{w}_{Ps}$ and $w = \bar{w}_{1,5Ps}$ (Table III) are plotted against the corresponding values of v_{qp} (Table I). It may be seen that some of the data plot on, or very close to, the 45° line through the origin; for these piles groups it could be argued that the variability of behavior originates essentially in the variability of soil properties.

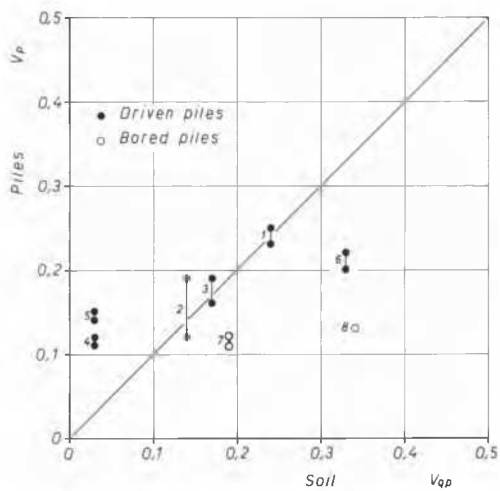

Fig. 4. Comparison between the variability of piles behavior and that of soil properties.

For groups 4 and 5, where v_{qp} has the smallest values, v_p is found to be larger than v_{qp}. In these cases the construction of the piles seems to have increased soil variability.

On the contrary, for groups 6 and 8, where v_{qp} has the largest values, v_p is found to be lower than v_{qp}. In these cases piles construction seems to have decreased soil variability.

INFLUENCE ON THE DESIGN OF FOUNDATIONS

The influence of piles behavior variability on the design of foundations is affected by many factors. As an example, it is analyzed in this paragraph with reference to two very simple schemes.

Let us consider first a group of 3 piles, subjected to an axial load Q (fig. 5).

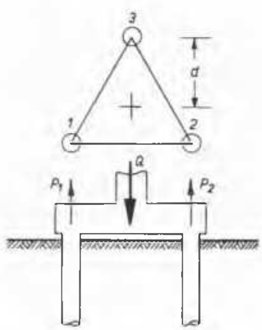

Fig. 5. Group of 3 piles, supporting a column subjected to an axial load Q.

If the structure is such that the column can only settle, without rotation, the settlement of the 3 piles will be the same; they will react with different forces, P_1, P_2 and P_3 giving rise to an eccentricity of the resultant and hence to a bending moment M in the column. It may be shown that:

$$M = d\ (P_1^2 + P_2^2 + P_3^2 - P_1P_2 - P_2P_3 - P_1P_3)^{1/2}$$

and, being $P_3 = Q - (P_1 + P_2)$:

$$M = 3\ d\ (P_1^2 + P_2^2 + P_1P_2 + Q\ (Q/3 - P_1 - P_2))^{1/2} = F\ (P_1,\ P_2) \qquad (6)$$

In this case, for simmetry, the behavior of each pile is unaffected by its position within the group; P_1 and P_2 may be therefore considered as two independent random variables, both having mean value equal to Q/3 and the same value of the standard deviation $\tilde{P}$. By the method of the second moment, we can evaluate the mean value and the standard deviation of the function (6) as follows:

$$\bar{M} = F\ (\bar{P}_1,\ \bar{P}_2) \qquad (7)$$

$$\tilde{M}^2 = \tilde{P}_1^2\ \left(\frac{\partial M}{\partial P_1}\right)^2 + \tilde{P}_2^2\ \left(\frac{\partial M}{\partial P_2}\right)^2 = 2\ \tilde{P}^2\ \left(\frac{\partial M}{\partial P}\right)^2 \qquad (8)$$

where all partial derivatives are calculated in the point $P_1 = \bar{P}_1$; $P_2 = \bar{P}_2$. Eqs. (6), (7) and (8) give:

$$\bar{M} = 0$$

$$\tilde{M} = 2,45\ d\ \tilde{P} \qquad (9)$$

Eq. (9) allows an evaluation of bending moments to be expected in the columns as a consequence of the variability of piles behavior. Being $\tilde{P}$ of the order of 0,1 ÷ 0,2 $\bar{P}$ (Table III), it may be seen that the occurrence of significant bending moments is rather probable.

As a second case, let us consider a symmetrical foundation beam resting on a row of 25 piles and subjected to 4 equal concentrated forces of 375 ton each, as shown in fig. 6. The analysis of the influence of the variability of piles behavior has been performed in this case by simulation, assuming that the compliances w_i/P_i of the 25 piles vary according to a normal distribution with mean value and standard deviation equal to the values of foundation 1 at service load ($\bar{w}$ = 1,89 mm; $\bar{P}$ = 60 ton; v_p = 0,23; see Table III).

By means of the random number generator of the system UNIVAC 1106, 100 random distribution of the 25 compliances of the piles below the beam have been generated, and for each of them four beams of different stiffness (EI = 7; 56; 450; 3600x10^4 tm^2) have been analyzed. It has been assumed that the beam itself is not in contact with the soil.

The reciprocal influence between the piles may be simulated by different models, the simplest one being a Winkler-type model in which each pile is assumed to settle only under the load directly acting upon it. More realistic models are obtained by introducing the influence coefficients $w_{i,j}$, i.e. the settlement of the pile i due to the load acting upon the pile j. Such influence coefficients could be calculated, for instance, by elastic theory (Dente, 1974).

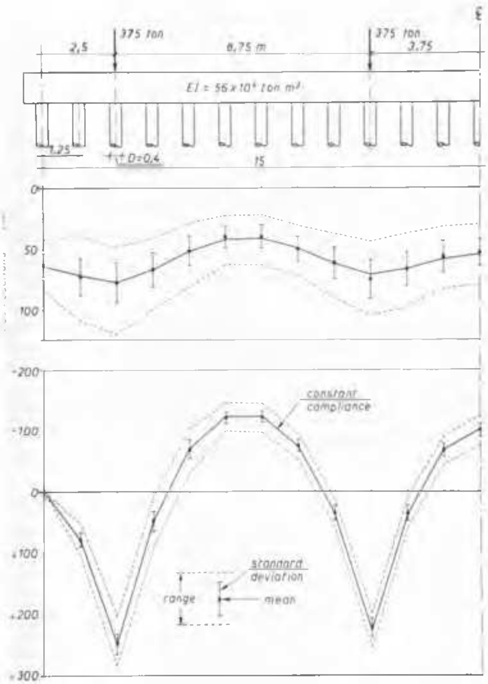

Fig. 6. Foundation beam resting on a row of 25 piles and acted upon by 4 concentrated forces.

In the present case, some experimental resu= lts on the reciprocal influence between the piles of the same foundation n.1 were avail= able, and are reported in fig. 7. Based on these results, as a first approximation it has been assumed:

$$\frac{w_{i,i\pm1}}{P_{i\pm1}} = \frac{0,17\ \text{mm}}{60\ \text{ton}} \ ; \qquad \frac{w_{i,i\pm2}}{P_{i\pm2}} = 0 \qquad (10)$$

In other words, each pile is affected only by the loads acting upon the two immediately adjacent piles; the influence of the other piles, though not zero, is small enough to be neglected in the present case. Such in= fluence, however, could be easily accounted for if needed.

Fig. 6 shows the range, mean value and sta= ndard deviation of piles reactions and ben= ding moments of the 100 distribution analy= zed for $EI = 56x10^4\ tm^2$. On the same figure have been superimposed, by full line, the values obtained in the usual design hypothe= sis that the piles compliance is constant.

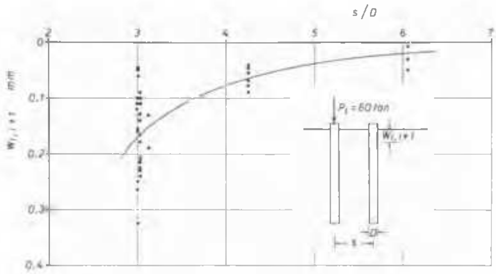

Fig. 7. Foundation n. 1. Settlement induced in the ne= arby piles by the load acting on the test pile.

The influence of the beam stiffness is depi= cted in fig. 8.

The piles below the beam react with different forces due to the combined influence of two factors: the variability of their response and the stiffness of the beam (for instance, in a relatively flexible beam the piles be= low the loads react more than the piles at mid span). The influence of the latter fac= tor is depicted in fig. 6 by the full line, referring to the case of constant piles com= pliance. In the case of fig. 6, the influe= nce of the two factors may be seen to be of the same order.

With increasing beam stiffness (fig. 8), as a consequence of the assumed model (eq. 10) the differences of piles reactions due to their location below the beam tend obviously to vanish, while the random variability is practically unaffected. Even for a very stiff beam, the reactions of the piles may be sig= nificantly different.

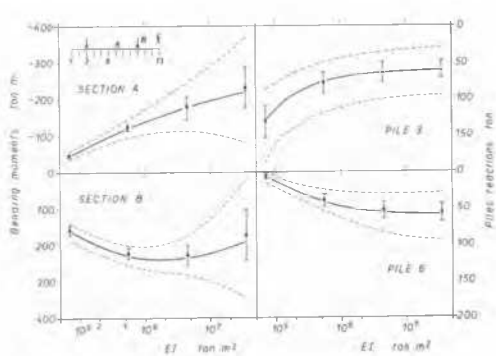

Fig. 8. Influence of the beam stiffness on the bending moments and on the reactions of the piles.

The variability of the bending moments inc= reases markedly with increasing beam stiff= ness, and is very significant for the highest values of EI.

It may be concluded that an increase of the beam stiffness, in an attempt to make piles reactions uniform, is not very effective above certain values of EI. Due to the ran= dom variability of piles behavior, the reac= tions remain affected by a significant vari= ability and furthermore both the values of bending moments and their variability incre= ases markedly above certain values of EI.

CONCLUDING REMARKS

Some experimental evidence on the random variability of behavior of piles in cohesi= onless soils has been presented; available data are far from being exhaustive, but are believed to represent a first significant contribution to the study of this problem.

The factors influencing said variability have been discussed, with particular refe= rence to the variability of the subsoil pro= perties, that is characterized by means of penetration test results.

The influence on the design of foundation structures has been found to be rather sig= nificant in two simple schemes, that have been analyzed by different techniques and referring to a particular set of data.

These conclusions, of course, apply only to the cases examined; further theoretical and experimental research is needed before the significance of piles behavior variability in more general conditions can be evaluated.

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