4th Intern. Conference on the Aplication of Stress Wave Theory to Piles
Application of Stress-Wave Theory to Piles, F. B. J. Barends (ed.) © 1992 Balkema, Rotterdam. ISBN 90 5410 082 6
Netherlands - 21-24 Sept. 1992

155 163.2

Dynamic loading test curves

N. Aoki
Foundation Engineering IME-RJ, São Paulo, Brazil

V. F. B. de Mello
São Paulo, Brazil

ABSTRACT: The results of a dynamic loading test during redriving for set are presented showing that when using the Smith model the damping and quake are not constant soil behaviour parameters as generally postulated, but are significantly dependent on energy level. In routine pile driving, at each project some dynamic load tests are being systematically performed with successively increased heights of hammer drop at the final set, giving good correlation with static load tests (Aoki, 1989). At the microstrain levels currently imposed in piling projects such dynamically determined behaviours on gradually varying elastic and plastic components, as well as of shaft and point damping factors, seem logical. Thus the generalized practice of employing single, constant, blow-and-set data is cleary insufficient and wrong, and preference points toward more developed models than Smith's. CAPWAP monitoring by Pile Driving Analyser PDA are furnished, supporting the above rationale of progressive variations with total pile head displacements per blow.

1 INTRODUCTION

In the practice of pile driving it is routine to apply a constant fall of the hammer weight. The same procedure is applied when measuring the final penetration "set" and also when conducting a PDA (Pile Driving Analyser) driving monitoring. The ultimate dynamic resistance of pile is estimated by applying the historic dynamic formulae, or the wave equation (Smith, 1960), employing the values of the set or of the accelerations and deformations measured at the pile head. The ultimate static resistance is presumed derivable from the determined ultimate dynamic resistance which has been religiously taken as that derived from ONE blow of constant energy.

Fjelkner and Broms (1972) merit mention as having investigated different heights of hammer fall regarding influence of shaft friction on a so-called Amplitude Damping, really a down-the shaft attenuation factor on maximum stress amplitude. The claim that their equation's composite damping coefficient proved approximately independent of height of fall, leading to Amplitude Damping approximately proportional to the particle velocity, calls for closer scrutiny within the tabulated data. The point is that the presentday definitions of damping have no link with that pioneering effort.

Whitaker and Bullen (1981) postulated an obvious corollary that if there is a single and definite ultimate resistance as implicit in calculations by dynamic formulae, based on whatever analytic failure criterion be applicable, then such a value should remain constant independently of changing the height of fall.

Fujita and Kusakabe (1988) raised the question of how to select a specific significant set of data from various data obtained from a series of blows during a monitoring of driving set. Evaluated values of bearing capacities were recognized to vary depending on which blow was used. They reflect that there is experimental evidence that the (presumed) failure mode of soils around the base of the pile changes with the magnitude of the impact energy reaching the pile toe, and the deformation patterns around the toe and down the shaft differ from those pertaining to static penetration. They conclude that this evidence raises a question as to whether the driven pile bearing capacity (and specifically the static one, that constitutes the desired goal) can really be evaluated when the magnitude of deformation around the toe, and/or the pile-soil interface displacements are SMALL.

The load-displacement relationships for the soil in Smith's rheological model assume that quake (maximum elastic displacement at each section down the pile) occur concomitantly with the ultimate soil resistance (static) at that section. The dynamic behaviour is introduced as characterized by the Smith damping constant Js. Both values are taken as parameters dependent only on the soil type and condition.

Gobble et al. (1981, 1985) describe the Case Method that is a closed form solution based on a few simplifying assumptions simple enough to be evaluated "in real time", i.e. between hammer blows, using the PDA. The dynamic soil resistance is computed from a soil damping factor Jc. The CAPWAPC Method combines the wave equation pile and soil model with the Case Method measurements. This procedure permits deriving the resistance distribution along the shaft and at the toe, as well as the Smith and Case damping and quake values. The total static resistance value proved to be in good agreement with the measured static ultimate resistance as evaluated by the so-called Davisson's failure criterion (see Fellenius, 1980). It should be remarked that in the case referred, the small pile-soil displacements were small, far from those typically needed to mobilize the full soil resistance for the single blow of the hammer.

The present paper submits a job case in which CAPWAPC analyses were run on a centrifuged reinforced concrete pile of 60 cm external diameter and 10 cm wall thickness, driven to satisfactory near-toe bearing in dense sandy materials, using a free-fall hammer of 51 kN weight. The final redriving was conducted under the procedure denominated Dynamic Loading Test (Aoki,1989) with progressively increasing heights of hammer fall.

The CAPWAPC analyses run on each successive blow lead to the conclusion that the Smith and Case toe damping, as well as the toe quake, are definitely energy dependent values, and not constant as hitherto postulated.

The derived dynamic load-displacement curve shows that there is a low-energy initial portion exhibiting elastic behaviour, wherein the total displacement is equal to the elastic rebound (plastic penetration equal to zero). Subsequently at higher energy levels the pile starts to penetrate into the soil with growing permanent plastic deformations and an almost constant elastic rebound. Such a trend was also observed by Lapshin (1989).

2. SUBSOIL PROFILE

The subsoil profile is summarized in Fig.1. It comprises about 11 m of sediments overlying a granito-gnaisse saprolite that was investigated down to a depth of about 22 m. Down to about 11 m, the soft upper sediments, principally silty clays, are described with detailed geotechnical behaviour parameters by Cunha and Lacerda (1991).

The first underlying stratum, 11 to 15 m, is interpreted as residual, with disperse angular gravel-size grains and mica. Below 15 m the dense clayey silt involved in the nominal toe pressure bulb is definitely a saprolitic horizon of gneiss.

3 DYNAMIC LOADING TEST RESULTS

The test procedure has been established with progressively increasing driving energies in order to be essentially analogous to the conventional static load test

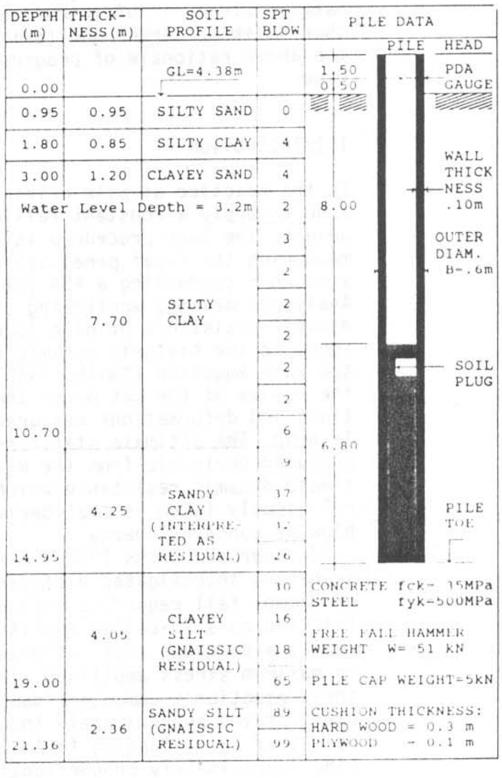

Fig. 1 Soil profile and pile data

routines. Thereupon, it must be recognized that if there are any cumulative effects of successive micro-penetrations, that should overflow sequentially into affecting subsequent ones, the separate successive CAPWAPC analyses have disregarded them as of secondary interest. Future developments may consider the interest of investigating the interference of this factor, possibly by intermittently varying impact energy levels in the chronological sequence.

All data are presented in graphical form for visual impact, and are accompanied by tabulated results for convenience.

The data on variations of the applied energies with increasing height of drop of the hammer are summarized in Fig.2 (Table 1). It is of some side interest to note that there is no noticeable discrepancy between the trends shown by calculations by the basic idealized dynamic FORMULA as compared with ENTHRU, the latter having given systematically somewhat higher results. This result seems explainable by the fact that, consistent with present-day routines, both in the rheological model and the basic dynamic formula computations the radiation damping losses into surrounding soil are neglected, but should intervene in different degrees.

Fig. 3 (Table 2) summarizes the results of the CAPWAPC analyses of the variation of mobilized resistances vs. the increasing successive heights of fall. The results are striking in themselves, as well as in their resemblance to typical results from static load tests with down-the-shaft stresses monitored. The shaft resistance increases almost linearly up to an intermediate value of hammer fall, and, thereafter, for additionally increased fall heights remains perfectly constant. Meanwhile the toe resistance continues to increase steadily, which tallies well with longsince established radical differences (cf. London LARGE BORED PILES Symposium, ICE, 1966) between strains mobilizing shaft friction as compared with base resistance and ultimate failure (about 10% D).

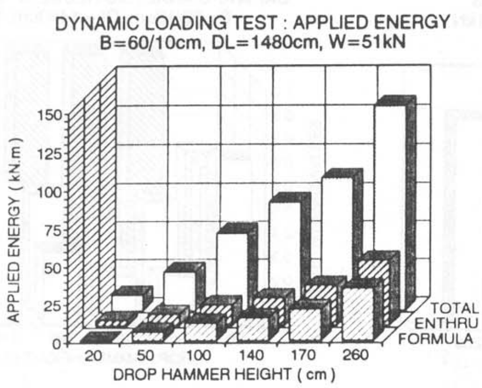

Fig. 2 Drop hammer height x energy

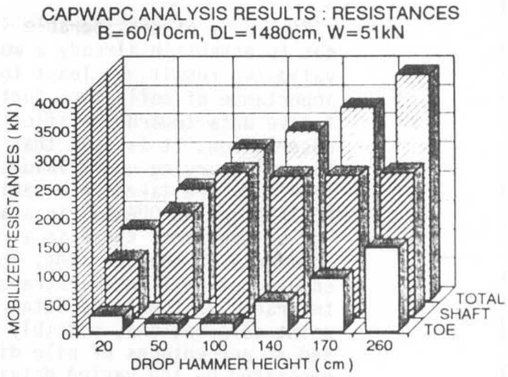

Fig.3 Drop hammer height x resistances

From Fig. 4 (Table 3) the successive distributions of local friction values derived from the CAPWAPC analyses merit a comment: they appear to be strikingly rational in comparison with monitored data from static load tests. Suffice it to compare visually the distribution diagrams for the drop heights of 0.2, 0.5, 1.0, and 2.6 m, to observe the progressive growth of lateral friction as increasing shear strains are generated.

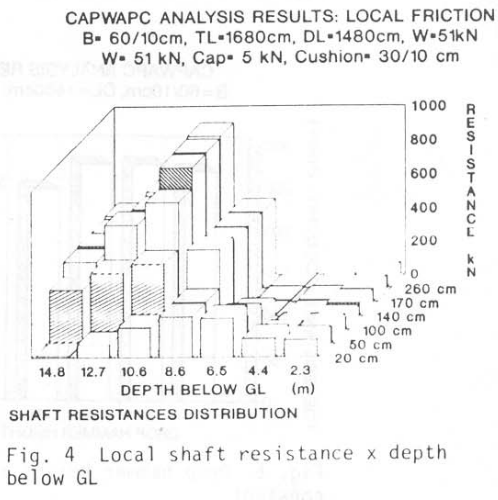

Fig. 4 Local shaft resistance x depth below GL

Fig. 6 is of considerable interest, if not to establish already a worthwhile and validated result, at least to suggest the importance of collecting further representative data towards ratification of the observation. It is seen that if the significantly varying quake values of Fig. 5 (Table 2) are taken in a ratio of (TOE RESISTANCE)/(TOE QUAKE), in analogy to a toe spring constant, there is a remarkable constancy of the quotient. This observation should not be lightly extrapolated beyond the range of the field data regarding point deformations, possibly to be expressed as percentages of pile diameters, generated by the varied driving energies. It is suggestive, however, for future improvements to be introduced in the rheological models' simplified parameters. The point to be emphasized is that since the rheological models have assumed that any plastic deformation, no matter how small, would already represent a rigid-plastic failure, postulated constant, the aberration becomes flagrant when the sets are of but few mms. for modern large-diameter piles, and really a microstrain elastic condition is at stake.

Figs. 7 (Table 2) and 8 (Table 2) further emphasize the fact that computed damping factors vary greatly with the variation of driving energies, a fact discrepant with the Smith model formulation that adopts them as constant. In Fig. 7 there is an appearance of approximate constancy in the Smith shaft damping values, possibly associated with similarities that accompany very low levels of dynamic straining. Concomitantly there is a significant systematic decrease of toe damping values with

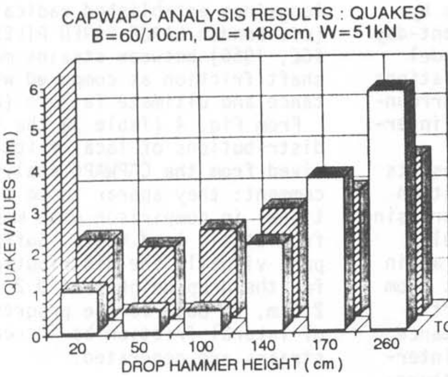

Fig. 5 Drop hammer height x quakes

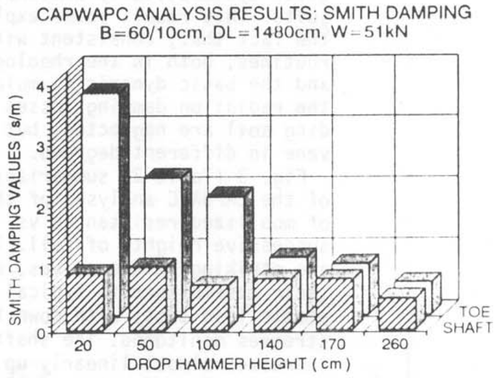

Fig. 7 Drop hammer height x Smith damping factor

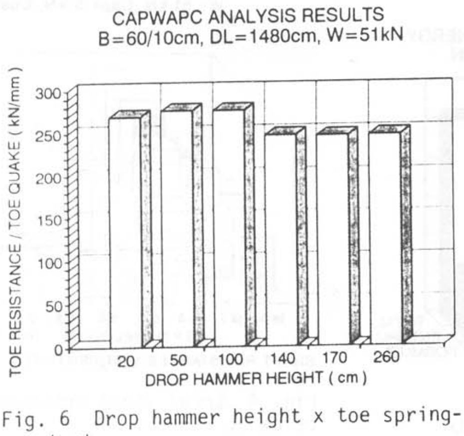

Fig. 6 Drop hammer height x toe spring-constant

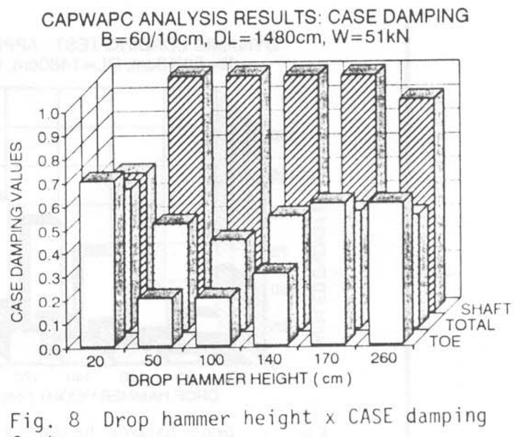

Fig. 8 Drop hammer height x CASE damping factor

increased drop height, for this soil profile and driven pile. In comparison, Fig.8 furnishes the same panorama of non-constancy for the Case Method damping factors, shaft, toe, and total. All three factors are irrevocably concluded to be dependent on energy levels. The trends appear, however more erratic, and might even seem incongruent at places (for instance, the unjustifiably high toe damping factor at the lowest energy level, a result possibly affected by decreased precisions of data-recording).

4 DYNAMIC LOADING TEST RESULTS

Beside the interest and significance attributed to the above observations, the fact is that the primary object of the dynamic loading test accompanied by the CAPWAP computations is to furnish the interpreted load-displacement data analogous to those of static load tests. The computed data are presented in Fig. 9 (Table 1), dimensionless coordinates having been preferred in order to exhibit more flagrantly the geomechanically logical principles exposed. One should even emphasize the accrued interest, compared with routine static load testing, that in such dynamic analyses there is a separation (rheologically-mathematically idealized) between the strictly elastic and the plastic (irrecovered) deformations. The strain levels at which such separation is to be considered allowable-questionable-unacceptable is a question of engineering decision.

In Fig. 9 the TOTAL loads are plotted without separation into shaft and toe contributions. The symbols used are the routine ones of the dynamic analyses, reproduced below for convenience. It is of primary interest to note that the project's design allowable load for the pile was 1800 kN.

RE = static structural failure load of pile section
RMX = max. mobilized total resistance for the blow
DMX = max. corresponding downward displacement at the PDA gage level (cf. Fig. 1)
K,S = respectively, elastic and plastic part of the displacement

One first emphasis is that with the total gage level (head) displacement of 2,7%B (B = pile diameter), at hypothetical "failure loading" (dynamic) the restriction to 1.6 cm of pile head settlement constitutes a stringent criterion, geotechnically unrealistic, limited to microstrain levels absolutely incapable of developing conventional postulated base bearing capacity plastification zones.

Moreover, up to the load of 3100 kN, roughly 50% of RE but 170% cf the design allowable load one obtains indications of a purely elastic (dynamic) behaviour, with total displacement equal to rebound, and therefore zero set. It may be a coincidence that at this point of changing idealized elasto-plastic behaviour, the displacement was about 1.0 cm, which would approximate the roughly 1 cm generally associated with mobilization of directshear lateral friction, independent of pile diameter. In this specific case, because of the roughly 60% of upper soils geotechnically incapable of perceptible contribution to static lateral friction, the comparison dynamic-to-static resistance modelling would require reevaluation for compatibilizing rheologies and computational modelling.

5 CONCLUSIONS

The routine use of monitored-computed pile capacity results from a SINGLE blow-and-set derives from a wrong concept, increasingly wrong as pile diameters have increased, and furnishes correspondingly questionable results. After execution of a representative dynamic loading test, the constant single blow-set condition, of generalized practice, may be used for execution-inspection aiming at uniformity, intervening factors assumed to be invariant.

At the levels of microstrains at play under current routines dynamic loading test

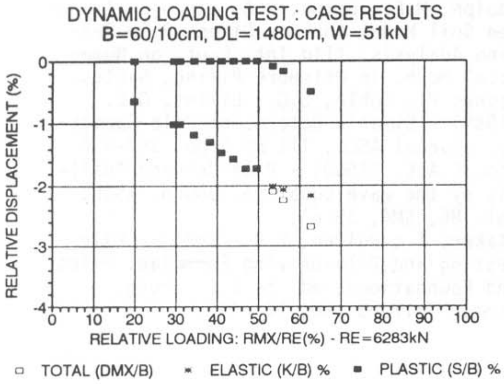

Fig. 9 Non-dimensional load-displacement curve

Table 1. Case Field Results : Jc=0.45

Fall (cm) H	Displac. (mm) TOT. DMX	EL. K	PL. S	Energy (kN.m) TOT. W.H	ENTH. EMX	FORM *	Tot. RES. RMX
20	3.9	3.9	0.0	10	3.2	2.5	1280
50	6.1	6.1	0.0	26	7.3	5.9	1940
100	8.8	8.8	0.0	51	13.6	11.4	2585
140	10.3	10.3	0.0	71	18.4	15.2	2950
170	12.5	12.0	0.5	87	26.5	21.9	3375
260	16.0	13.0	3.0	133	42.6	37.4	3940

*FORMULA= RMX(DMX+S)/2 c=3200m/s

Table 2. CAPWAP Analysis Results

ITEM	H = Fall Height (m) 20	50	100	140	170	260
	Mobilized resistances (kN)					
TOE	268	139	140	517	910	1430
SHAFT	972	1801	2480	2413	2420	2470
TOTAL	1240	1940	2620	2930	3330	3900
	Case damping Jc					
TOE	0.75	0.24	0.21	0.35	0.58	0.58
SHAFT	0.65	1.40	1.38	1.53	1.43	0.91
TOTAL	0.60	0.45	0.38	0.48	0.50	0.48
	Smith damping Js (s/m)					
TOE	3.58	2.22	1.93	0.86	0.82	0.52
SHAFT	0.86	1.00	0.71	0.81	0.76	0.47
	Quakes (mm)					
TOE	1.00	0.48	0.50	2.10	3.70	5.80
SHAFT	1.90	1.67	2.15	2.64	3.00	3.77

Table 3. Local Shaft Resistance (kN)

Depth (m)	H = Fall Height (m) 20	50	100	140	170	260
2.3	106	96	63	73	73	74
4.4	114	57	17	9	9	10
6.5	237	162	109	9	9	10
8.6	247	249	481	455	456	456
10.6	180	484	765	888	891	906
12.7	78	429	633	557	559	577
14.8	10	324	412	422	423	437

results analysed by PDA seem to become analogous to those of static load tests, and even furnish indications on threshold of microplastification. Incidentally, from many analogous case-analyses one notes that this type of dimensionless graph as a function of RE fosters clearly distinguishing predictability of tendency to pile-soil or pile-structure failure (RE).

Stringent design-construction codes and practices on driven piling should be promptly subjected to intelligent revision with recognition of geotechnical-rheological modelling and computationally exposed results on-the-spot. The simple idealized hypothesis of the Smith model should be relinquished in favour of such better modelling as those of Holeyman (1985) and Randolph and Simon (1986), and radiation damping losses should begin to be incorporated in all dynamic formulae.

6 ACKNOWLEDGEMENTS

A special acknowledgement is rendered to PDI Engenharia, of Copacabana, Rio de Janeiro, Brasil, both for this specific case's analyses and for their continued advances in the procedures involved.

7 REFERENCES

Aoki, N. (1989) - A New Dynamic Load Test Concept. Proc.Disc. Ses.14, XII ICSMFE, Tech. on Pile Drivability, Vol.1, 1-4.

Aoki, N. (1989) - Interv. in the Disc. Ses 15 on Static and Dynamic Tests on Piles, XII ICSMFE (in publication).

Cunha, R.P. da; Lacerda, W.A. (1991) - Anal. of Sanit. Embank. Failure on Rio de Jan. Soft Clay Depos. Can.Geot.J. 28, 92-102.

Fellenius, B.T. (1980) - The Analysis of Results from Routine Pile Load Tests. Ground Engg., Sept., 19-31.

Fjelkner, G.; Broms, B. (1972) Damping of Stress Waves in Piles during Driving. Swed. Geot. Inst., Repr. and Prel.Rep.50.

Fujita, K.; Kusakabe, O. (1988) - Evaluation of Static Bearing Capacity. IIId Intern. Conf. on Applic. Stress-wave Theory to Piles, Ottawa, 525-534.

Goble, G.G.; Rausche, F.; Likins, G.E. (1981) - The Anal. of Pile Driving - SOA. Proc. Inter. Sem. on the Applic. Stress-Wave Theory on Piles, Stockholm 130-161.

Holeyman, A. (1985) - Dynamic non Linear Skin Friction of Piles. Proc. Symp.Penetrability, Drivability of Piles. San Francisco, XI ICSMFE.

Lapshin, F.K. (1989) - The Definition of Bearing Pile Capacity with Regard for Elastic Deformations, Proc. Disc. Sess. 14, XII ICSMFE, Tech. Com. on Pile Drivability, Vol. 2, 18-19.

Randolph, M.F.; Dimon, H.A. (1986) - Improved Soil Model for One Dimen. Pile Driving Analysis. IIId Int. Conf. on Numerical Meth. in Offshore Piling, Nantes.

Rausche, F.; Goble, G.G.; Likins, G.E. (1985) - Dymanic Determ. of Pile Capacity. Journal ASCE, 111 nº 3 pp. 367-383.

Smith, F.A.L. (1960) - Pile Driving Analysis by the wave Equation. Journ. ASCE, Vol. 86, SM4, 35-61.

Whitaker, T.; Bullen, F.R. (1981) - Pile Testing and Pile-Driving Formulae. Piles and Foundations, ed. by F.E. Young. Thomas Telford Ltd. 121-134.

(163.1)
155.1 novo

(1) standardization of pile-soil interaction models; of course only those models should be selected that may be expected to describe the physical phenomena as realistic as possible;

(2) standardization of methods to establish the parameters of the pile-soil interaction model for a given measurement;

(3) establishment of the field of application for the various methods and quantification of accuracies on the basis of systematic test series; this means that one should find the mean and standard deviaton of the discrepancies between model and reality; reality is in this respect a standardized static load test;

(4) based on the information found under (3) one can derive design values for the load bearing capacity which can be used within the modern load and resistance factor design approach.

Of course this is a research program for years. And what is more, it is a program that no company, institute or university is able to do alone. To carry out the above program is possible only if there is a world wide coordination on standardisation of models and comparison between models and reality.

REVIEW OF INDIVIDUAL PAPERS

L. Aaltonen, O. Tirkkonen, R. Wikstrom: A new design method of (rammed) piles.

A new field measurement system is shown, by which the bearing capacity of a mini-pile is used, to predict the ultimate bearing capacity of full-scale piles. The system is rather complete with penetration test, soil parameters, theoretical bearing capacity, simulation of driving, and analysis of the bearing capacity. Standard methods as CASE and CAPWAP are used to interprete the measurements.

Remarks: The paper is rather suggestive, actually more commercial than scientific. Scale effects are not properly studied. The interpretation of the graphs is rather incomplete. It is mentioned in the paper that in Pori some piles were broken. Is this assessed only theoretically?

N. Aoki and V. de Mello: Dynamic loading test on concrete pile in resudial soil.

The paper discusses the limitations of SMITH, CASE and CAPWAP approaches. The interpretations of measurements show several deficiencies in the models, especially the dependancy of parameters on the energy level of driving. Proposals for improvements of the models are given, such as the introduction of the ratio toe resistance over toe quake.

The paper urges the necessity of better models for the pile-soil interaction and for more advanced models.

Remarks: The literature research is good! Can the dependency of the quake and damping on energy level be translated in soil behavior more specifically? How important are the quake-value and J-value for the global behaviour? What is the effect of the plug, see fig. 1? The paper confirms the results of paper 86. A shortcoming of the paper is that the recommended improved models have not been used in the paper itself.

M. Bustamante, L. Gianeselli, C. Schreiner: Comparitive study on the load bearing capacity of driven H-piles in a layered marl.

The paper relates the results of a series of full scale static and dynamic pile load tests, performed on steel H piles, driven into very hard marl with a particular structure. The dynamic tests were interpreted using CASE and CAPWAP methods. One pile was instrumented for the static test. The aim of the tests was the comparison of the appropriateness of different methods for predicting the bearing capacity of the piles and for determining the distribution of forces along the shaft. The comparison shows that CAPWAP gives a good prediction of the point resistance however a large overestimation of the shaft friction.

Remarks: How does the chosen value for J affect the results? The dynamic estimation of the bearing capacity has been determined after the static test, which makes the result of less value.

L. Chen, M. Fan, R. Zhao: Pile integrity analysis from lateral mechanical admittances.

A visco-elasticcally supported Euler-Bernoulli beam model has been used to investigate the use of lateral vibrations of piles for integrity testing, additional to axial testing. Some experimental results are shown.

Remarks: The type and the way of loading in the tests is not specified. The approach is only valid in the low-frequency band because of the Euler-Bernoulli model used. A Timoshenko model would have advantages especially for the Mode II vibrations. The lateral soil reactions have a large impact on the results, especially the proposition of constant stiffness and damping per length. Is steady-state theory used for a transient loading?

Application of Stress-Wave Theory to Piles, F.B.J. Barends (ed.) © 1992 Balkema, Rotterdam. ISBN 90 5410 082 6

Dynamic loading test curves

155.2

N. Aoki
Foundation Engineering IME-RJ, São Paulo, Brazil

V.F.B. de Mello
São Paulo, Brazil

1 REMARKS OF THE GENERAL REPORT ON THE AUTHORS' PAPER

The literature research is good! Can the dependency of the quake and damping on energy level be translated in soil behavior more specifically? How important are the quake-value and J-value for the global behaviour? What is the effect of the plug, see figure 1? The paper confirms the result of Danziger a.o. The recommended improved models have not been used in the paper itself.

2 AUTHORS' RESPONSES TO THE COMMENTS OF THE GENERAL REPORT

The proposed special procedure of dynamic loading test, wherein the pile is striken with increasingly hammer all height, has showed that the dynamic pile-soil load transfer is done in a continuous way: starting with the side friction mobilization, when the displacements are almost all elastic and, thereafter, by the point resistance mobilization, when the pile starts to penetrate in the soil.

The rupture load is fully mobilized when the applied energy level has induced large strain levels either in the pile material itself or in the soil under the point of the pile.

It is not possible to forecast the rupture load, in the engineering sense, from data obtained in a single stroke of the hammer. In this case the monitoring allow us to know the values of the applied energy, displacement, velocity, force and the mobilized soil resistance, for this blow.

In this context the model of Smith is a load transfer model and not a rheologic model, which would require a stress-strain rather than a load-displacement relationship.

In the Smith load transfer model the rupture load is defined as any load that generates any small plastic strain in the soil under the pile point (set, permanent penetration of the point of the pile in the soil, bigger than zero).

This confusion originates the wrong idea that quake is a soil parameter rather than a component of the displacement vector. So being the point quake is the elastic component of the pile base displacement and varies with the applied energy level.

The proposed dynamic loading procedure shows the limitations of Smith model and the need for the development of a more suitable load transfer model. The displacement of the pile base must be separated not only in elastic and permanent components but also in the components due to the loading imposed by the friction and by the point of the pile, taking into account that soil is a continua media.