General report/Discussion session 15: Static and dynamic testing of piles

Rapport de spécialistes/Séance de discussion 15: Essais statiques et dynamiques des pieux

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There are 20 papers in this Discussion Session, which may be classified into the following groups:-

1. PREDICTIONS OF STATIC LOAD TEST RESULTS AND RESIDUAL STRESSES.
2. STATIC VERSUS DYNAMIC RESISTANCE.
   Papers No 15.1, 15.2, 15.5, 15.6, 15.7, 15.8, 15.10, 15.11, 15.12, 15.13, 15.14, 15.16, 15.17, 15.18, 15.20 (15 papers). Since several papers discuss both the themes, they will be reviewed together.
3. RESPONSE OF PILES AND PILE GROUPS UNDER STATIC AND DYNAMIC LATERAL LOADING
   Papers No 15.3, 15.4, 15.9, 15.15, 15.19, (5 papers)

See Table at the end for details of soils piles and test procedure etc.

1. PREDICTIONS OF STATIC LOAD TEST RESULTS AND RESIDUAL STRESSES.
2. STATIC VERSUS DYNAMIC RESISTANCE.

Akhtar and Kibria (Paper No. 15.1) reported comparisons of predictions and measured ultimate capacities and settlements on 10 Rcc bored piles from 5 project sites. Several methods of predictions as well as load test intrepretation were used. It has been shown that (1) prediction of pile capacity by Terzaghi method yields the most acceptable results, (2) Butler and Morton criterion gives better estimates of settlement.

The variations in the predicted and measured capacities and settlements have been briefly described.

Aoki and Alonso (Paper No 15.2) re-analysed the driving data on 19 concrete piles on which static load tests had also been performed, by 1) CASE and CAPWAP methods and Chellis (1962) rebound method modified by Uto (1957). On the basis of this comparison, it was estimated that 1) Uto's and Chellis methods show a good agreement with the results of static load tests, 2) CASE and CAPWAP methods give more conservative results.

Briaud, Tucker, and Ng (15.5) present partial results of a research program on piles in sand sponsored by the FHWA. The subsoil essentially consists of a medium dense to loose sand (SP), hydraulically filled, with the water table at a 2.4 m depth. The piles consist of closed-end steel tubes, with a diameter of 0.273 m and wall thickness of 9.3 mm, that were driven to a depth of 9.15 m. Axial load test was performed on a single pile and on a rigidly connected group of five piles, installed on a centered square pattern with a minimum pile spacing ratio of 3. The piles were instrumented with strain gages, top and toe load cells, and toe telltales. The single pile and the pile group were both load tested using 30-minute-long load increments and maintaining the maximum load for at least 6 hours. Results consist of the residual load distribution of the piles after driving; the load-settlement curve of the single pile, pile group, and of each pile in the group; the load versus depth profile for each pile; and the load transfer curves. The authors conclude the following, among other things:

- residual loads were more significant for the single pile than for the pile group
- group efficiency factors were estimated to be 1.83 for the friction, 0.67 for the point, and 0.99 for the total resistance
- when residual loads are taken into account, the critical depth below which friction would no longer increase becomes less apparent.

The authors finally provide their judgment regarding the relative performance of the following prediction methods: CPT and SPT based methods for single pile capacity, TTI and WEAP86 program for driving features, and the case and capwap methods for pile bearing capacity from driving measurements.

The paper provides a wealth of information on a well-documented case. The full-scale measured residual load profiles are impressive. The paper confirms the factors that encourage large residual loads and the circumstances under which it becomes worthwhile to take residual loads into account. One may add to the mentioned factors the ratio of shaft resistance to base resistance. It should prove interesting to further define the "re-strike" procedure: which blow is interpreted, the very first blow after rest, the last one of the series of blows, or a representative blow after a given number of blows?

Bustamante, Frank, and Gianeselli (15.6) present a first synthesis of the LCPC Method to predict the load bearing curve of a single pile using the results of Menard-type pressuremeter tests. The backbone mobilization curves for the shaft (t-z curves) and for the base (q-z curves) resistances adopted for the study were initially developed by Frank and Zhao (1982) for bored piles in fine-grained soils. The stiffness coefficients of the backbone curves depend on the pressuremeter modulus, and the pile radius.

The present study involved 33 piles of various types at 18 sites of variable geotechnical conditions. The piles were statically tested as to induce a settlement at the pile tip of at least 10% of the base diameter. The piles were instrumented with retractable extensometers to determine the mobilization of the shaft and base resistances. The ultimate shaft and base resistances, as measured from the pile load tests, were used to scale the backbone mobilization curves. The authors then provide the ratios of the calculated settlements to the measured settlements at the admissible load for the 33 piles.

Except for closed-end tubes driven in sand and for bored piles in clayey marl, the presented ratios are generally close to unity, with extremes of 0.43 and 2.14. Based on these comparisons, the authors conclude that the presented method is satisfactory and operational, thanks to the direct measurements of the pressuremeter tests.

One may wonder what is actually demonstrated in the paper because the amplitudes of the mobilization curves have been scaled using the ultimate resistances as measured from the static load tests. It is the writer's opinion that provided the pile section modulus and the ultimate values of the local shaft and base resistances are correct, any reasonable set of backbone mobilization curves should generally yield a reasonable prediction of the pile settlement at the admissible load. It may be interesting to know how the measured shaft and base ultimate resistances are accommodated to account for residual stresses in the simulation proposed in the paper. Residual stresses may explain the relatively small settlements measured by the authors for the closed-end steel tubes driven in sand.

The paper provides an elementary step in the overall French methodology to predict load bearing curves for piles based on the PMT tests. Comparison of predictions on the sole basis of PMT tests should provide a more challenging demonstration.

Darrag and Lovell (15.7) suggest a simplified procedure to predict the residual load of piles driven in cohesionless soils. After stressing the significance of residual stresses in the interpretation of static load tests, the authors review the existing procedures which in their opinion, are either too simplistic or involve incorrect assumptions. Based on the conclusion that the wave equation method is capable of incorporating all important factors that contribute to residual loads, the authors have run parametric analyses using the computer code CUWEAP (Hery, 1983). The code assesses the relative importance of the driving system, total pile capacity and its distribution, pile dimensions and material, and pile-soil interface stiffness. The parametric analyses are used to characterize a typical residual pile load distribution in terms of a set number of adimensional parameters. Through the analysis of the results of more than 250 computer runs, the authors then suggest using a set of charts for either concrete or steel piles to determine the residual mobilization ratio of the base resistance of piles. The charts have been established for piles that derive 40 percent of their resistance from friction and an equation is suggested to account for different friction resistance ratios. Additional charts are provided to take into account the interface stiffness. A formula derived from the numerous numerical simulations is proposed to assess the residual load distribution along the pile. The authors show that predictions according to the suggested method compare well with published results.

The paper provides a rational and simple description of the factors affecting the residual load of driven piles. The authors confirm that residual load increases when the relative shaft resistance increases and when the flexibility of the pile increases. As far as the interface stiffness is concerned, it seems that the parametric analyses were run for a homogeneous profile. It would certainly help the reader to know how the pile-soil interface stiffness is converted into spring constants at the base and along the shaft. This might also help in clarifying why a pile modulus of 9,630 kN/m is presented in Figure 2b of the paper. It would be interesting to know the number computer runs on which Figures 2a and 2b were based to better appreciate the generalization of the results in terms of non-dimensional parameters. A very interesting conclusion that the authors derive is the quasi-insignificant influence of the driving system on the residual loads: Would a simulation of the static loading and unloading of the pile produce similar results? The promising procedure would gain further usefulness if specifically extended to end-bearing piles.

Drescher et al (Paper No 15.8) report static and impulse load tests (t = 2-4 s) on 4 cast in place concrete piles, 17 m long installed on 5V:1H slope and at the ground spacing of 1.4 m. The pile cap was 2.6 m square and 1.5 m thick. The pile foundations will support both vertical loads and horizontal loads from accelerating and braking trains running on this pile supported bridge.

The subsoil consists of soft to medium stiff clays, with soil strength and modulus increasing with depth. A compacted gravel cushion was installed near the pile tip to reduce plastic deformations in the soil.

In static test, an allowable group load of 1800 kN at a settlement of 8.1 mm was observed. The vertical pile stiffness was 2.22 MN/cm. Results of 3 impulse load tests gave vertical pile stiffness of about 8 MN/cm, which is about 4 times the static value.

Without given data of an alternative system, it is concluded that using the modified cast-in-place piles, static and dynamic bearing capacity and stiffness requirements can be met in this case.

The authors missed opportunity to make a forceful presentation and hope they can supplement the missing information on pile sizes and gravel cushion during the discussions.

No information on horizontal load tests is included.

Heritier (15.10) provides a succinct summary of the CEBTP Method of predicting the load-settlement curve of piles from dynamic loading tests. The testing procedure involves dynamic loading of the pile head with hammer blows of increasing energy and recording the force, the acceleration, and the displacement at the pile head. The dynamic resistance corresponding to a particular blow is obtained by the difference between the upward moving force wave of the pile, if it were free, and the upward moving force of the embedded pile. The succession of blows of increasing energy is used to establish an empirical relationship between the static and the velocity dependent dynamic resistance. The provided example of the load bearing curve behavior predicted by the CEBTP Method agrees very well with the curve obtained from the static load test.

The two-page-long paper does not provide the rationale of the author's method. It should be noted that the testing procedure involving blows of increasing energy was used in 1987 for the prediction exercise carried out within the framework of the Belgian Symposium on Pile Dynamic Testing. It would have been interesting to see what the velocity dependent function correlating the static and dynamic resistance looks like. One may also wonder how the effects of a larger displacement can be dissociated from those due to the increased velocity. Also, are the results dependent on the sequence of blows used for testing?

The proposed method offers the definite advantage of establishing a specific correlation between static and dynamic resistance of the tested pile, and as such, one would expect it to yield more reliable results than methods based on more general correlations.

Jaime et al (Paper No 15.11) present data and analysis of static tests on 4 concrete piles in Mexico City clay. Failure of buildings on friction pile in 1985 Mexico earthquake prompted this study. The soils at this site are 1) 5 m very hard crust and between 5 m and 15.5 m, there are three clay layers with mean water contents of 375, 300 and 325% for the top, middle and lower layers respectively. Between 12 ad 13 m depth, a stiff layer of sandy clay occurs.

A 15 cm diameter hole was made between elevation 5 m - 15 m and then 4- precast piles 30 cm square and 15 m long were installed in a grid of 6 m x 6 m. Mean undrained strength (Su) of this clay from UU triaxial tests was 34 kPa. For tip capacity, Su was estimated as 68 kPa. A hole 50 cm diameter and 5 m deep was made in the stiff top crust and a pipe was installed around the pile so that effective pile penetration was in clay only.

In addition to load and pullout, quick pile penetration tests were performed with loading time of 30 - 40 S.

One slow and two quick penetration tests were performed 5 months after installation. In slow test, the load displacement relationship was almost linear up to 350 kN load. The peak load was 540 kN at a settlement of 22 mm.

In quick tests, P-δ relationship is linear up to 400 kN and the maximum load was estimated as 750 kN. In pull out test, the P-δ curve was non linear from the very beginning and the maximum load was 460 kN.

A comparison of the P-δ curves, showed that their slopes in quick tests is about 1.6 times stiffer than that in slow test and the maximum load in quick test is about 1.5 times that in the slow test. The slope of the pull out P-δ curves is much smaller than that of the penetration (slow) test. These results are of the same order as on strength and modulus values of clays in quick loading (Prakash 1981).

The authors have corrected the P-δcurves in penetration and pull out due to shortening of the pile by making simplified assumptions and have concluded that mode of friction mobilization in clay in penetration and pull out is different. This is not in fact true in all field data reported and analysed else where (Prakash and Sharma 1990).

α-factor for this pile-clay system has been estimated as 1.2. After accounting for weight of the pile in pull out and penetration and α- of 1.2, it was concluded that "peak friction capacities determined in penetration and pull out tests are similar".

The authors did not measure the skin friction and point bearing loads independently. Therefore, their quantitative conclusions are subject to certain degree of uncertainty.

Jardine and Bond (Paper No 15.12) report tests on eight 100 mm diameter 5-7 m long steel closed ended pipe piles in London clay. This clay is precompressed to a pressure of 1 MPa giving minimum overconsolidation ratio (OCR) of approximately 30 at 2 m depth and approximately 14 at 6 m depth. Water table is about 1 m below the ground surface.

The data on 3 piles instrumented with axial load cells, and pore pressure probe at different locations along the pile are presented. The piles were jacked hydraulically in situ The maximum velocity of penetration was 600 mm per min. The top 2 m length of pile was cased with a pipe of larger diameter to prevent its contact with the soil.

The load increments were applied at a fixed rate until creep-yield point was reached, after which a constant rate of displacement was maintained. The three piles were tested 63, 79 and 79 days after installation.

During installation on a pile (CP2) a negative pore pressure up to 0.5 atm was observed near the central section from 2.5 m to 5.0 m from the ground level. The maximum positive pore pressure was measured up to 450 kPa below 5.3 m depth. Negative pore pressures were acting along most of the pile (CP2) after installation, although theories predict positive pore pressures. The pore pressure varied with time as expected.

Total radial and shear stresses were also monitored during installation. Both these stresses were found to vary rapidly with depth.

Pile (CP1) was jacked at a rate of 95 mm/min while CP2 at a rate of 425 mm/min. The observed pore pressures and total radial stress do not appear to be affected by the rate of penetration. In CPI, with slower rate of jacking, the average value of shaft shear stress (τ_{rz}) toward the end of jacking was approximately 40% smaller than those in faster test (CP2). The values of interface friction angle ϕ' (= $\tan^{-1} \frac{\tau_{rz}}{\sigma'_r}$) were interpreted varying from 9° - 14° in pile CP1. This value compares well with the Laboratory ring shear tests. The corresponding range of ϕ' values for pile CP2 was 14° - 17°, which also compare with the rapid laboratory tests. The rate of penetration, seems to control the soil fabric behavior close to the shaft. Therefore, it may affect the peak friction developed in the subsequent monotonic tests. Since the load tests are carried out at a relatively low rate of penetration, lower interface friction angles (ϕ') may develop than those observed during installation.

The pore water pressures equalized to hydrostatic condition after about 24 hours at most locations. However, negative pore pressures still remained near the top of the pile. Total radial stress also equalized to a about 95% of the constant value at a particular depth in the same period.

Limited data of pile load test has been interpreted in terms of 'α' and 'β' coefficients. The long term shaft capacity was smaller than the maximum capacity developed during installation. More importantly, the jacking rate is the most important single parameter in controlling long term capacity. A pile installed at 500 mm/min. appears to have 60% more shaft resistance as compared to the one jacketed at 20 mm/min.

The authors have presented very interesting and unique data which may not be substantiated by available theories (e.g. negative pore pressure development during jacking) which most certainly reflects on the validity of the theories. The information on shaft capacities is in accordance with the strength of clays under slow and quick load tests (Prakash 1981).

Jarominiak (15.13) presents an ingenious and original method to improve and check the performance of bored piles which do not meet their projected bearing capacity. The method was applied to piles bored into dense sands with some gravel layers below a river bed and cased in a 1.8 m diameter steel tube. The question posed during construction was whether 11.6 m of embedment was sufficient to provide the expected 2.2 MN design bearing capacity. Rather than performing a conventional compression test, which was precluded for practical reasons, the author proposed to drive a 0.510 m-diameter tube pile inside and below the casing sunk for the bored pile and use it as reaction to test the pull-out resistance of the outer casing.

A single test allowed assessment of both the uplift shaft resistance of the casing and the downward base capacity of the driven element. After the test indicated that the shaft pull-out resistance was only on the order of 1 MN, the internal tube pile was driven to greater depth and later connected to the casing to provide a consistently reinforced base to the bored pile.

Jarominiak's paper demonstrates that innovative solutions are found under difficult construction circumstances. The author is to be commended for devising a system which provides both improvement and control of the quality of a foundation product. In the particular set-up adopted, one should be concerned with the potential internal reaction that may develop between the tube pile and the casing, via the casing plug. It would be worthwhile to know how that potential reaction was assessed or accommodated, as it could lead to an overestimate of both the casing and base resistances.

Kruizinga (15.14) presents comparisons between pile ultimate shaft and base resistances calculated on the basis of Menard-type pressuremeter tests and those measured during static load tests. A single closed end steel tube ([phi] =0.355 m, e=12.7 mm) was driven to different depths and load tested generally at least 21 days after the driving had been interrupted. The steel tube was instrumented with strain gages at 0.4, 1.0, and 2.0 m from the pile tip. Five load tests were performed with pile embedments of 11, 16.8, 18.8, 20.8, and 22.8 m. The load was applied in increments using three reaction battered piles, with five unloading cycles at the end of each load increment. Results are presented in terms of total ultimate capacity, base ultimate capacity, total shaft ultimate capacity, and shaft ultimate capacity for the four lower meters of shaft. Load-settlement curves are also presented for the five load tests. The pressuremeter tests were performed using the retro-jet system in the upper 16 m of generally soft clays and using a bentonite-stabilized, hand augered, pre-drilled hole in the deeper dense sand layers.

Three types of pressuremeter rules were used by the author to calculate the components of the ultimate bearing capacity: Menard (1975), Baguelin et al. (1978), and Bustamante et al. (1981). From the comparison of the calculated resistances to the measured ones, Kruizinga concludes that:

- end bearing capacities are correctly predicted using Bustamante's method, while they are generally overestimated using Menard's method
- shaft capacities are generally overestimated by the three types of pressuremeter rules applied.

The author suggests that a reduction factor of 0.8 on Bustamante's method would provide a more accurate calculation of the shaft resistance and points out still a discrepancy for the deepest level test.

Kruizinga's paper indicates that the pressuremeter test may be gaining some acceptance among Dutch engineers: it is refreshing to see that pressuremeter rules may provide useful results in the motherland of the CPT test. The paper, which is rich in specific information that may be further interpreted by other practitioners and researchers, stimulates the following issues for discussion:

- What would have been the prediction using the CPT test results?
- To what extent can the high friction mobilized on the shaft segment closest to the pile tip be dissociated from a high base resistance?
- To what separation between shaft and base resistances does the interpretation of the unloading cycles of the static load test lead? (See Van Weele, 1957.)
- How can the low friction measured in the deepest level test be explained?

Milovic and Stevanovic (15.16) show the results of load tests on four bored piles with diameter varying between 0.9 and 1.5 m and lengths varying between 15 and 22 m. The bored piles were installed in 10 to 12 m of medium to stiff clays and deeper medium dense sands, with occasional gravels. The authors derive the ultimate bearing capacity from the load-settlement curve using three different criteria: Van der Veen (1957), Mazurkiewicz (...) and a bi-linear interpretation of the settlement-load ratio as a function of settlement. They compare the ultimate bearing loads resulting from the load tests with CPT-based design methods due to Mohan et al. (1963), Bustamante and Gianeselli (1982), and Meyerhof (1978).

From these comparisons, the authors conclude that a reasonable estimate of the bearing capacity of piles may be obtained from the Bustamante and Gianeselli (1982) interpretations of the CPT tests; the method of Mohan et al. as well as the bi-linear interpretation of the load test results is thought to produce too high results. The soil moduli back-calculated from their interpretation of the load-settlement curves indicate that the ratio of the soil modulus to the cone resistance varies between 10 and 19, which is felt to be higher than that usually used.

It is difficult in the writer's opinion to appreciate the validity of the comparisons presented in the paper because of the following reasons: clays have been identified where the average core resistance is about 18 MPa (180 tsf); the water levels are not presented; and the method of boring the piles is not described.

It would be interesting to see a measure of the lateral friction (local or total) on the cone resistance diagram to better appreciate what friction could be expected along the shaft of the bored piles. It would also be worthwhile to understand at what stress (or strain) level was the soil moduli back-calculated. Finally, it is the writer's opinion that the back-calculated soil modulus may strongly depend on the modulus assumed for the reinforced concrete. In that respect, it would be interesting to know the assumptions made by the authors and their assessment of the interval of confidence of such back-calculations.

Niyama et al (Paper No 15.17) describe results of load transfer along shaft and tip on a prestressed concrete pipe pile by two methods 1) CAPWAP method during pile driving in the last blow and 2) Static load test after pile driving. The pipe pile was 80 cm OD with 15 cm wall thickness and 35m long driven through marine sediments to a very dense sandy silt ($N \simeq 45$) with 20.7 m length embedded in soil. The soil was slightly erratic in character up to about 26 m depth. The pile was driven open ended and was instrumented with

4 sets of strain gages and 6 sets of tell-tales, at 3-levels, with strain transducers and accelerometers at the pile top.

The maximum load was 4.64 MN in the static test and the failure load was estimated as 5.5 MN according to Van der Veen's method.

A comparison of the side friction showed that up to about 10 m depth, the skin friction in both static and dynamic tests is almost equal. However, below this depth, skin friction in static test was greater than that in dynamic test. At the toe, the skin friction was estimated as 46% of the total load by CAPWAP method and 10% from static computations. The maximum total resistance in dynamic and static tests was 4.7 and 4.55 MN respectively. The skin friction in corresponding tests was estimated as 2.15 and 3.94 MN respectively (initial) and 1.85 and 3.64 MN (Moidied) respectively. The discrepancy in shaft resistance values in static and dynamic test may be due to regain of shaft resistance with time. In the writers opinion there is no reason why the 2- shaft resistance values be equal or similar.

Noren et al (Paper No 15.18) report results of static and dynamic pile tests in a mudstone or claystone with lenses of cemented sand and silt, with water content of 20-30%. The pile should develop a working load of 750 kN in vertical compression and horizontal resistance with a design seismic coefficient of 0.3 selected for the site.

In 320 piles, driving was stopped when the bearing capacity according to Case-method was 3-times the working load. Also CAPWAP analysis was performed on 15% of the piles.

In this paper, data for 2-piles are reported; Pile 1, 650 mm diameter in normal mustone and pile 2, 550 mm diameter in a more firm rock. In pile 1, the base loads computed by 3-methods ie 1) considering rock as a cohesive soil, 2) CAPWAP method and 3) statistical method, were 3050, 2100 and 920 kN, while in pile 2, the corresponding values are 4560, 3314 ad 1370 kN. It was concluded that the CASE and CAPWAP methods give good values of the mobilized pile bearing capacity.

Apparently, the authors seem to have good data on their piles, which could not be included in this paper probably due to space limitations.

Selby, et al (Paper 15.20) report on an extensive pile testing program undertaken by the Ministry of Transportation of Ontario (MTO) from 1977 to 1981. A total of 26 piles of various types and lengths were driven and load tested at five different sites. The paper briefly describes the specific site conditions and compares the ultimate capacities predicted by:

- o Static Formulae (Meyerhof, 1956 and Tomlinson, 1957)
- o Pile Driving Formulae (Hiley, Janbu, and Gates)
- o Wave equation (Case and Capwap methods).

Based on the results obtained from the reported tests, the authors observe that the prediction methods considered are erratic; predictions of the ultimate bearing capacities vary from the measured ones by the following ranges:

- o Static formulae: -50 to +178%
- o Pile driving formulae: -55 to +206%
- o Wave equation: -57 to +78%.

In spite of recognizing the higher reliability of the predictions resulting from the wave equation interpretation of the measurements taken at the end of driving, the authors conclude that the method has not yet demonstrated a sufficient accuracy to supersede the current MTO approach. The current MTO approach consists of extrapolating load test results from an extensive local data bank (more than 200 load tests accumulated since the mid-1950s).

While the reporting of a large number of case histories can be used to indicate certain statistical trends, the reader might be more specifically convinced if the following information would be provided:

- o Criterion used to deduce the ultimate capacity from the static load test results
- o Results of predictions obtained with the current MTO procedure
- o Assumed or measured parameters necessary to apply the pile driving formulae
- o Driving equipment and set at refusal
- o Type of blow for the wave equation measurements: last blow of continuous driving or re-strike.

The paper points out the strength provided by an extensive local data bank and the need for further clarification of the change of pile bearing capacity with time.

3. RESPONSE OF PILES AND PILE GROUPS UNDER STATIC AND DYNAMIC LATERAL LOADS

Baguelin, Frank and Jezequel (15.3) compare the results provided by different design methods to the results of a long-term lateral load test on a free head single pile. The pile consisted of a steel square profile (B=0.284 m) driven into 4 m of low plasticity soft clay and 2.5 m of medium dense silty sand. The lateral load was applied for 54 days 1 m above the mud line, right at the water level. Measurements included the lateral displacement and rotation of the pile head and multiple strain gauges at various depths. The ultimate and allowable lateral capacity as well as maximum bending moments are presented using the following methods:

- o Brinch-Hansen (1961)
- o Broms (1965)
- o Menard (1962)
- o API (1987).

The lateral load bearing curves have been calculated using both the p-y curves approach, according to either Menard (1962-1968) or API (1987), and the elastic theory (Poulos and Davis, 1980). The authors conclude that:

- o The applied design methods define an allowable lateral load in the range of 60 to 90 kN
- o Both p-y methods give satisfactory results
- o The elastic theory method allows one to back-figure the soil equivalent Young's modulus at E=40Cu.

The calculations provided indicate that the allowable lateral load could be overpredicted to some extent by Broms' method and to a larger extent by Brinch-Hansen's method. However, it is not clear from reading the paper what the ultimate lateral capacity of the pile is because the load test results are provided only up to 60 kN, which according to the authors, corresponds to about the allowable lateral capacity.

The maximum bending moment predicted by Menard's ultimate capacity method in Table III of the paper is 147 kNm for the predicted allowable lateral load of 56 kN. However, Figure 3 presents a maximum moment calculated by Menard's displacement method of about

117kNm for the same lateral load of 56 kN. Would the discrepancy between the moments indicate that the maximum bending moment should be obtained by dividing the maximum bending moment under the ultimate load by a factor higher than the factor of safety used to determine the allowable load? It is the writer's opinion that the elastic theory method could have been used in a predictive mode with the relation E=EM/[EQN "[alpha]"] (see Menard, 1965), which would have led to the 1,000 kPa value back-calculated from the load test.

The paper does correctly point out that different methods should be applied with their specific factors of safety. The back-calculated values of the soil modulus for different load levels clearly indicate the non-linear behavior of piles under lateral loads, even at small strains.

Bonaz, et al (Paper 15.4) compare experimental results with their numerical simulation of pile group behavior under harmonic lateral loading. The numerical simulation uses sub-structuring techniques, combining integral equations for the visco-elastic modeling of the soil domain with finite elements for the pile domain.

The lateral tests reported were performed at the Plancoet site (see Paper 15.3) on two H-piles driven into soft saturated clay (B=0.27 m, l=6.5 m). The piles had a center to center spacing of 0.8 m and were rigidly connected at their heads. The dynamic characteristics of the clay were determined from resonant column tests: G=10 to 30 MPa, and D=5% for $\gamma = 10^{-4}$. The frequency of the harmonic load was varied between 1 and 60Hz and for each frequency, three levels of displacement were selected between 10 and 40m. The piles were instrumented with accelerometers down to a depth of 4 meters.

The experimental verification of the numerical simulation was carried out with respect to amplitudes of displacement, displacement profiles, and phase shifts. Based on their comparisons, the authors conclude that the calculated displacements are generally in good agreement with the measured ones but that the calculated phase shifts are much lower than those measured. They hope that this discrepancy will be resolved through a finer soil investigation.

It would have been useful for the authors to present their comparison between the strain rate enforced by the loading test and those obtained in the laboratory tests. The measured displacement profiles indicate that the head of the piles was allowed to rotate, in spite of a stiff connection of widely spaced flexible piles. It would be worthwhile to understand the deformation mechanism of the pile group and determine how the connection between the two piles was modeled in the computer code as no calculated displacement was provided above ground level.

Hassini and Woods (Paper No 15.9) report tests on 2 and 4- pile groups at different spacings. The soil was fairly uniform fine to medium, poorly graded sand with uniformity coefficient Cu of 2.9, and effective size of 0.13 mm. The minimum and maximum void ratios were 0.57 and 0.76 respectively. Steel pipe piles were 6.0 cm in outside diameter and 5.1 inside diameter. These were embedded 1.98 m in the sand. In order to ensure that the pile cap was rigid, the ratio of bending stiffness of cap (Bc) to that of pile (4EI/lc) was kept greater 55.

The piles were installed by first excavating a large pit in a bin filled with sand (6.70 m diameter and 2.13 m deep) and then replacing the soil around the piles by vibratory compaction in 13 cm lifts. An average unit weight of soil was 16.95 kN/m^2. The caps were built after the installation of the piles. Heavy steel plates were rigidly connected to the cap to provide for inertia and maintain resonant frequencies within the operating frequency of the vibrator used.

The shear wave velocities were determined by cross hole and resonant column tests, and varied from 137 m/sec at 30.5 cm depth and 244 m m/sec 1.83 m depth. The damping values varied from 3% to 1.3% at the same depths.

The tests of 2- pile group was performed in 2-stages. In the first stage, an electromagnet vibrator was used. The horizontal force levels used were 22 N to 156 N. The maximum amplitude was 0.6 mm ie 1/100 diameter of the pile. In the second stage, a mechanical vibrator was used and at the maximum force of 401 N, the pile group amplitude was 1.5 mm (1/40 pile diameter).

In low strain, the system was analyzed as a SDOF system.

Also, the natural frequency of the system decreases with increasing force level. Therefore, even in low strain tests the material shows non-linearity.

Damping increases with vibration amplitudes. In both 2- and 4- pile groups, the damping coefficients increase significantly with spacing for spacing ratios of less than 8, although the damping factor (ratio of damping to critical damping) varied only from 6.1 - 6.8% in 2- pile groups and 5.36 - 6.21% in 4- pile groups.

Effect of spacing of piles on stiffness was studied in terms of Relative change (RC = $\frac{K_i - K_{i-2}}{K_{i-2}}$ i = 4, 6, 8, & 10) for both vertical and horizontal vibrations. It was found that at a) spacing of 14 diameter and beyond, small pile groups show no interaction. At small vibrations, the RC values for vertical vibrations were generally smaller than those for horizontal vibrations.

The group stiffness have been shown to be frequency dependent in a companion paper of Novak. Also, on the basis of piles instrumented with strain gauges and tested in a centrifuge by Finn and Gohl (1987), it was found that the interaction effects do not extend to beyond 6-diameters. Prakash and Sharma (1990 have shown that these effects extend to at best 8- diameters. Therefore, both these questions need further study and deliberation in this session.

The authors do not mention if any vibration absorbing material was used on the boundary of the test bin. Also, the results of high strain tests are not described.

Guedes de Meto and Ferreira (Paper No 15.15) report test on an 0.8 m diameter 42 m long castin-situ reinforced concrete pile driven through layered alluvial deposit. Horizontal displacements and rotations of the pile at several depths were monitored with an inclinometer embedded in a pipe inside the concrete pile. Also, the pile was tested in 2-directional loading with a maximum load of 200 kN.

It has been shown that 1) up to a 60 kN load, the soil pile behavior is elastic and 2) beam on elastic solution matches the measured displacements along the depth with k =αE, where E is Young's Modulus measured independently (see Figure 1) and α is a multiplying factor, determined as 1.1 for this soil-pile system. It was concluded that pile behavior is practically without bending for a depth of about 13 m.

A correlation between 'K' and 'E' depends upon 1) pile size 2) type on soil and 3) probably method of pile installation, and 4) other factors not identified. It is good to see such a correlation for this site.

Novak and Jones (Paper No 15.19) report tests on full sized pile group under horizontal loads and interpret group action in terms of 1) pile interaction factors and 2) group stiffness. For small displacements assuming principle of superposition, expressions have been derived for vertical and horizontal pile group stiffness in terms of pile interaction factors in each mode.

Rotation response of a 11.7 m long compressor foundation supported on 10 composite pile showed that 1) individual resonance regions occur at very different frequencies, which is obvious and 2) depth of embedment of the foundation affects the response about vertical axis the most. The figure is instructive but a lot of desired information has not been included.

For an off-shore tower, the response of pile group has been described in terms of Group Efficiency Ratio (GER) for 1) group stiffness and 2) group damping considering dynamic interaction factors. Both GER for stiffness as well as damping have been shown to be frequency dependent.

They carried out 4 static lateral load tests on a group of 6- closely spaced close ended pipe piles, 101 mm diameter, 3.05 m long and wall thickness 6.35 mm. The soil was silty fine sand with gravel seam. Three individual pile tests and one group test with a rigid pile cap were conducted to large displacements. Hysterias loops for 1- cycle of unloading and reloading were also determined.

A plot of lateral load (P), deflection (y), and static interaction factor 'α' has been arbitrarily divided into 3 regions, 1) elastic 2) transition and 3) yield.

A plot of interaction factors for several ratios of spacing (s) to diameter (d) and incidence angles (β) at different deflections shows that the interaction factors decrease with increasing deflections. However for equal s/d ratios, and β angles from 0^o - 180^o, the interaction factors decreased considerably.

For non-linear prediction of pile-behavior, a weak zone around the upper one-third of the pile is assumed. This can be incorporated in PILAY/DYNA programs used. The above assumption is arbitrary. The extent of soil disturbance depends upon the pile size, spacing in groups and the soil type. Therefore, more fundamental research on the change in soil properties around the pile is needed.

It will be interesting to follow their work and see how it will be used by a practicing engineer!

The following questions have been identified by the General Reporters for discussion in this session:-

1. Effect of Rate of Loading on Pile Resistance.
2. Residual Loads on Driven Piles.
3. Static and Dynamic Lateral Loading and Pile Group effects.

ACKNOWLEDGEMENTS

"Any misunderstanding or misinterpretation of the papers reviewed for this Session are the responsibility of the Reporters, and to those Authors whose papers may be misrepresented, apologies are offered. Comments regarding the papers have been expressed from the perspective of stimulating lively discussions during the Session. The General Reporters wish to express their appreciation to Mrs. Charlena Ousley, Mrs. L. Kidd, and Mrs. S. Gwinn for their typing and editing assistance."

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Paper No.	Authors	Title	Pile Types (section dimension/length)	Soil/Geologic Conditions	Test Set-up/ Procedure/Criteria	Item Predicted, Prediction Method	Soil Investigation Results Provided	Main Theme for Discussion
15.1	J. Akhtar and S. Kibria	Prediction and performance of Bored Cast-in-Situ RCC Piles in Load Tests in Pakistan	RCC Bored Piles 10.0 m to 17.5 m long and 0.46 m to 0.56 in diameter at 5 sites	1. Medium dense sandy silt 2. Medium dense to dense sand at base, soft to stiff silty clay along shaft 3. Hard siltstone at base, weathered shale along shaft 4. Highly weathered soft to hard shale at base, medium dense to dense sand along shaft 5. Medium dense to dense silty sand	Pile Load Test Interpreted by Criteria Proposed by: 1. Brinch Hansen 2. Swedish Piling Commission 3. AAHSTO 4. Intersection 5. Butler and Morton	Ultimate Load Capacity. Methods used for Prediction: 1. Meyerhof 2. Skempton 3. Hansen and Terzaghi	None	Prediction of Static Load Test Results
15.2	N. Aoki and U. Alonso (Brazil)	Correlation between different evaluation procedures of static and dynamic load tests and rebound	19 hollow reinforced piles 20 cm-50 cm and 9.8 m-30.9 m	Not described	Not described	Limit loads	None	- Prediction of Static Load Test Results - Loading rate effects
15.3	F. Baguelm, R. Frank, and J.F. Jezequel (France)	Interpretation d'essais de chargement lateral d'un pieu Isole	1 driven steel square profile (284x284 mm/7.5 m)	0-1 m water 1-5 m: soft clay of low plasticity 5-7.5 m: medium dense silty sand	- long-term lateral loading 1 m above ground level - measurement of y and y' at pile head - strain gages at various depths	Ultimate lateral capacity using Brinch-Hansen, Broms, Menard & API (87). Lateral load-displacement curve using p-y curves and elastic theory	Laboratory tests on undisturbed samples, PMT tests	Lateral loading of piles
15.4	R. Bonaz, P. Bozetto, A. Pecker and J.P. Tourct (France)	Validation experimental d'un Code de calcul dynamique de Fondations sur pieux	1 group of 2 H-driven steel piles (270 mm/7.5 m)	See paper 15.1	- Fixed head harmonic lateral loading (1 to 60Hz) - displacements of 10 to 40 μm - accelerometers at various depths	Displacement profiles and phase shifts, using substructuring numerical technique combining FE and integral equations	Resonant column tests	Lateral loading of piles
15.5	J.L. Briaud, L.M. Tucker, and E. Ng (U.S.A.)	Axially loaded 5 pile group and single pile in sand	Closed end driven steel pipe piles (273x9.3 mm/9.15 m)	- 1.4 to 12.2 m: hydraulic fill consisting of medium dense to loose clean sand (SP) - 12.2 to 14.3 m: Same as above interbedded with medium stiff to stiff silty clay (CH) - 14.3 m and below: Bedrock - Water table at 2.4 m depth	- 30 min maintained load increments, last load maintained at least 6 hours - Pile instrumented with strain gages, top and toe load cells, toe tell-tales - Soil instrumented with extensometers and piezometers	- Pile driving features using TTI and WEAP86 programs - ultimate bearing capacity using case and Capwap methods	CPT tests, SPT tests, PMT tests, shear wave velocity shear modulus, direct shear tests, grain size, γ_d, w.	- Residual loads of driven piles - loading rate effects
15.6	M. Bustamante, R. Frank, and L. Gianeselli (France)	Prevision de la Courbe de Chargement des Fondations Profondes Isolees	33 piles: - Driven concrete (400 to 545 mm/10.2-12 m) - Driven steel (273 to 514 mm/8-28 m) - Bored (550 to 1,120 mm/6-30.3 m)	18 sites: - clay - sand - chalk - marls	- static loading up to a tip settlement of at least 10% of the base diameter - recoverable extensometers	Load bearing curve, using t-z and q-z mobilization curves with ultimate values determined from pile load test	Laboratory tests on undisturbed samples, CPT tests and Menard PMT tests	- Residual loads of driven piles. - Prediction of static load test results
15.7	A. Darrag and C. Lovell (U.S.A.)	A simplified procedure for predicting residual stresses for piles	Intended for driven piles	Applicable to cohesionless soils	No specifics	Residual loads from driving, using proposed charts and equations	No specifics	Residual loads of driven piles
15.8	J. Drescher H. Meyer O. Neubaner (FRG)	Large scale impulse loading tests on a group of cast-in-place piles on cohesive soil	4 cast-in-place pile group 1.4 m x 1.6 m spacing pile cap 2.6 x 2.6 m x 1.5 m thick	Soft to stiff 4-6 m thick clay layers	a) Jacks-2 No. 4 MN capacity and 1, 15 MN capacity used to test piles. Settlement and tilt monitored with dial gauges and electronic transducers. b) Impulse tests details not given	a) Static load 1800 KN @ 8.1 mm settlement b) Static failure load 4000 KN @53.4 mm settlement c) Static stiffness 2 202 KN/cm d) Stiffness in impulse test 8000 KN/cm (app)	Static cone penetration resistance increases marginally with depth, except in gravel sand mixture close to the surface. Soil modulus increases significantly with depth	a) Prediction of static load test results b) Loading rate effects
15.9	S. Hassini and R.D. Woods (Egypt - USA)	Dynamic experiments with model pile foundations	Steel pipe piles 6 cm O.D. and 5.1 cm I.D. and 1.98 m long. Rigid pile cap.	Sand in a bin 6.7 m dia. and 2.13 m deep	Horizontal excitation - Low strain with electromagnetic vibrator - High strain with mechanical oscillator having 2 eccentric masses	- Natural frequency in horizontal vibrations - damping as a function of spacing in pile groups - Relative change in group stiffness	Sand	Response of piles and pile groups under dynamic lateral loads
15.10	B. Hentier (France)	Capacite Portante par chargement dynamique-la methode C.E.B.T.P.	Reader referred to other publications	- sand - Flanders clay	free fall hammer with increasing drop height	Load settlement curve, using wave equation and static-dynamic resistance correlation	Reader referred to other publications	Loading rate effects
15.11	Jaime P. Alberto, Miguel P. Romo, Jose A. Ponce, Adrian M. Mitre (Mexico)	Static tests on friction piles in Mexico City	Four precast concrete piles 30 cm x 30 cm x 15 m long - top 5 m of pile cased with a sleeve and - a hole 15 cm diameter, 10 m deep prebored before installing the pile	Typical Mexico City Lake bed deposit a) three layers between 5 m and 15.5 m depth in which pile capacity is determined b) Water content of 3 layers of this clay are 375,300 and 325% respectively c) A stiffer stratum of sandy clay between 12-13 m	- 600-700 kN load reached in 30-40 s by jacking piles - displacement measured with dual gauges - Slow and quick penetration and pull out tests performed	a) Penetration resistance b) Pullout resistance c) Point bearing d) Skin resistance	Mean Su = 34 kPa, Su at tip = 68 kPa.	Prediction of static load test results

Paper No.	Authors	Title	Pile Types (section dimension/length)	Soil/Geologic Conditions	Test Set-up/ Procedure/ Criteria	Item Predicted, Prediction Method	Soil Investigation Results Provided	Main Theme for Discussion
15.12	R.J. Jardine, A.J. Bond (U.K.)	Behavior of displacement piles in a heavily overconsolidated clay	three steel pipe piles 100 mm dia. and 5.7 m long	No details	Piles installed by jacking at variable rates of penetration 95 mm/min to 500 m/min	a) Pore pressures during installation b) Total radial stress c) Shear stresses	heavily overconsolidated long clay a) OCR~30 at 2 m depth b) OCR~14 at 6 m depth	a) loading rate effects b) prediction of static load test results
15.13	A. Jarominiak (Poland)	Test of bored pile without ballast and anchor	Bored piles cased in a steel tube below river bed (1,500x12 mm/11.6 m)	Dense medium sand, with some gravel layers below river bed	- Use of a bottom driven internal tube pile as reaction to pull out the casing - measurement of casing pull-out and internal pile settlement	Not specifically addressed	Not specifically addressed	Interpretation of static load tests
15.14	J. Kruizinga (Netherlands)	Bearing capacity of a test pile compared with predictions from pressuremeter rules	1 closed end steel tube (355x12.7 mm/12.7 m) Driven with diesel D-30	0-16 m: NC soft sandy clays and silts with bands of peat and a sand interbed 16-25 m: medium dense fine sand, NC from Pleistocene age	- 5 static load tests on same pile driven at different depths - maintained loads followed by 5 unloading cycles for each load increment - strain gages at 0.1, 2.0, and 4.0 m from pile tip.	Ultimate shaft and base capacity using 3 types of pressuremeter rules	CPT, PMT, continuous boring with identification tests, pore pressure measurements	Prediction of static load test results
15.15	F.A. Guedes de Melo M.I. Ferreira (Portugal)	Horizontal load test on free head pile	Cast in place 0.8 m dia. and 42 m long, reinforced with 6 bars 20 mm dia. and 6 bars 15 mm dia.	Alluvial valley excavated into rock formation of schist and filled with recent deposit of mud, sandy mud and silty sandy mud up to 70 m depth	- Reaction from another pile - Top deflection - Bending moment with depth with the help of electric resistance gauges - Rotation at pile top and with depth	Horizontal displacement and rotations along the pile	A plot of E with depth and SPT values given	Lateral loading of piles
15.16	D. Milovic, and S. Stevanovic (Yugoslavia)	Deformation modulus determined by pile load test	4 bored piles (900 mm/16 m, 1,200 mm/18 m, 1,200 mm/22 m & 1,500 mm/15m)	- 10-12 m of medium to stiff clay - deeper medium dense sand, with gravel	- static vertical load test - 3 criteria to assess the ultimate bearing capacity: Van der Veen, Mazurkiewic and bilinear approximation	- ultimate bearing capacity from CPT test using Mohan et al. (1963), and Bustamante and Gianeselli (1982) - Soil modulus back calculated with elastic solutions	CPT tests	Interpretation of static load tests
15.17	S., Niyama N. Azevedo C.M. Polla M.A. Dechichi (Brazil)	Load transfer in dynamically and statically tested pile	0.8 m diameter prestressed concrete piles	Marine sediments to very dense silt (residual soil)	Pile driven by an air-stream BSP 24 B hammer with a rated energy of 120 KN (12t ram weight)	Load transferred along the shaft in static and dynamic test condition (i.e. long term loading and during driving).	SPT results with depth	Loading rate effects
15.18	C. Noren, C.J. Gravare C. Ahlen (Sweden)	Dynamic measurements of pile performance in soft mudstone	320 piles, variable sizes, not given	Mudstone claystone, with lenses of cemented sand and silt	Piles driven using a 5-7 t hydraulic Banut hammer. Driving stopped when the bearing capacity according to case method was 3 times the working load	Ultimate pile capacity using - static formulas - Capwap - statistical methods	a) core drilling through mudstone or claystone b) cone samples classified with respect to hardness and recuperation c) a relationship between unconfined compression strength and elastic modulus given	- Prediction of static load results and - Loading rate effects
15.19	M. Novak and M. Janes (Canada)	Dynamic and static response of pile groups	a) 10 composite piles supporting a machine foundation b) 10 large piles supporting an off-shore platform c) Static test on 6 steel piles, 3.0 m long, 101 mm dia. and 6.3 mm wall thickness	None	None	a) (i) Rotation response of compressor foundation, (ii) Group efficiency ratio (GER) for stiffness and damping b) Power spectrum of offshore tower response c) Interaction factors for lateral deflection	a) None b) None c) Layered non-cohesive soil stratum consisting of fine sand with gravel seam	Response of piles and pile groups under dynamic lateral loads
15.20	K.G. Selby, M.S. Devata, P. Payer, and D. Dundas (Canada)	Ultimate capacities determined by load test and predicted by the Pile Analyzer	26 driven piles: - Timber (size 36/ 3.5-13.5 m) - Concrete (305x30.5 mm/ 14.6-34.8 m) - Steel HP (310x79 &110/ 14.8-45.3 m) - Steel tube (324x6.3 mm/ 14.7-32.7 m)	5 sites: Thick lacustrine and glacial deposits, shallow ground water table	ASTM D1143-74 Pile driving	Ultimate pile capacity, using: - Static formulae - Pile driving formulae - Wave equation (case, capwap)	Soil description, SPT values w_p-w-w_l, grain size fractions, ground water levels	Loading rate effects