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Static and dynamic lateral response of a 15 pile group

Réponse laterale statique et dynamique d'un groupe de 15 pieux

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ABSTRACT
To improve our understanding of the lateral resistance of pile groups, we have performed a series of static and dynamic lateral load tests on a full-scale pile group. The pile group consisted of 0.324 m-diameter steel pipe piles driven closed-ended into cohesive soil with an undrained shear strength of about 45 kPa. The test piles were driven in a 3x5 arrangement at a normalized spacing of 3.92 pile diameters in the direction of loading. A single pile test was also performed for comparison purposes. Lateral resistance was found to be a function of row position with piles in the first row carrying about the same average load as a single pile. The average load decreased for the second and third row piles but then remained about the same for subsequent rows. P-multipliers were determined for each row and were found to decrease as deflection increased. After 15 cycles of loading, the peak static resistance decreased by about 17%. Following static loading, dynamic loads were applied using a statnamic device. The dynamic load-displacement curves were stiffer and had larger hysteretic loops than the static curves. Analyses using an equivalent single degree of freedom model indicate that the damping ratio was between 30 and 40%.

RÉSUMÉ
Pour amèliorer notre compréhension de la résistance laterale des groupes de pieux, nous avons fait une série de tests statiques et dynamiques charge latérale sur un groupe de pieux. Le groupe de pieux a consisté de tuyaux en acier de 0,324 metres en diametre, enfoncés, bouts fermés, dans du sol cohésif avec une résistance au cisaillement non-drainé de 45 kPa. Les pieux de test étaient enfoncés dans une disposition 3 X 5 avec une espacement normalizé de 3,92 diamétres de pieux dans la direction du chargement. Un test à un seul pieu a été aussi fait pour faire une comparaison. La résistance laterale a été constaté d'être en fonctionne de la position des pieux avec ceux du premier rang portant approximativement la même charge qu'un seul pieux. Le changement moyen a diminué pour les deuxième et troisième rangs mais est resté invariable pour les rangs suivants. P-multipliers ont été déterminés pour chaque rang et ils ont diminués comme la déflection a augmenté. Après 15 cycles de chargement, la résistance statique de pic a diminué par approximativement 17 pourcent. Suivant le chargement statiques les charge dynamiques ont été mis en place en employent un appareil statnamic. Les courbes de déplacement du charge dynamic étaient plus raides et avaient de plus grands boucles d'hystérésis que les courbes statiques. Les analyses employant une modèle de single degré de liberté indique que le rapport d'amortissement était entre 30 et 40 pourcent.

1 INTRODUCTION

The lateral load resistance of pile group foundations is often a significant factor in the design of structures subjected to dynamic loads such as earthquakes and ship impacts. Although fairly reliable methods have been developed for predicting the lateral resistance of single piles, there is very little information to guide engineers in the design of closely spaced pile groups. Reduction factors (p-multipliers) to account for group interaction have been back-calculated based on several full-scale group load tests; however, essentially all of these tests have been performed at three pile diameter spacing and little information is available to define p-multipliers as a function of spacing.

Previous dynamic lateral load tests conducted on another full-scale pile group showed that damping resistance could produce significant increases in lateral resistance (Weaver et al, 1998). However, these tests only involved one cycle of loading and gaps were not generally present while the tests were conducted. Therefore, testing in this study was also designed to ascertain if damping would still be significant when gaps in the soil were present due to cyclic loading prior to the dynamic loading. Dynamic loads were applied using a statnamic load sled after application of 15 load cycles under static conditions.

2 GEOTECHNICAL CONDITIONS

A comprehensive geotechnical investigation was carried out to define the soil profile and properties at the test site. This investigation consisted of conventional sampling and laboratory testing as well as in-situ testing. Conventional sampling included 76.2 mm (3 inch) diameter thin-walled Shelby tube undisturbed samples and disturbed soil samples from a hand-auger. In-situ tests included standard penetration testing (SPT), cone penetrometer testing (CPT), dilatometer testing (DMT), pressuremeter testing (PMT), vane shear testing (VST), and downhole shear wave velocity measurements. Laboratory testing performed on the field samples determined particle size distribution, Atterberg limits, soil classification, shear strength and consolidation characteristics. Additional geotechnical data is provided elsewhere (Rollins et al, 1998).

Since laterally loaded piles typically receive most of their resistance from the soil in the upper 5 to 10 pile diameters, the shallow surface layers are of greatest interest for this study. The soil profile near the surface consists of layers of silt and clay underlain by a sand layer. The water table was located about 0.3 m below the natural ground surface during the testing.

As shown in Fig. 1, the cohesive surface soils extend from the ground surface to a depth of 3.05 m and consist of low-plasticity silts and clays classifying as ML, CL-ML or CL according to the Unified Soil Classification system (USCS). Hydrometer analyses indicate that a majority (50 to 75%) of the cohesive soil near the ground surface (1.7 to 4.5 m) consists of silt-size particles with a clay-size content generally between 10 and 25%. The undrained shear strength is typically between 25 and 50 kPa. The consolidation testing indicates that the soils are overconsolidated to a depth of about 10 m. The measured

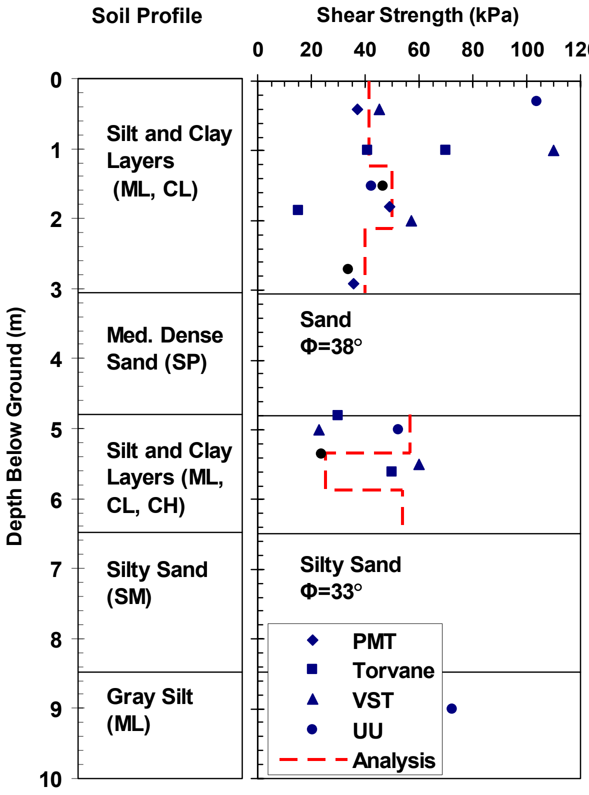

Figure 1. Soil profile and strength properties.

shear wave velocity was 120 m/sec from 0 to 1.5 m below the natural ground surface and 150 m/sec from 1.5 to 4.5 m depth.

The underlying cohesionless soil layer extends from a depth of 3.05 to 4.8 m and consists of poorly graded medium-grained sands and silty sands classifying as SP or SM according to the USCS. SPT blow counts and CPT tip resistances indicate that the sand is dense to very dense with corresponding relative densities (D_r) of 65 to 85%.

3 PILE PROPERTIES, TEST LAYOUT, AND INSTRUMENTATION

The test piles consisted of 324 mm-diameter steel pipe with a wall thickness of 9.5 mm. The piles were driven closed-ended in a 3x5 arrangement at a normalized spacing of 3.92 pile diameters in the direction of loading. The test piles conformed to ASTM A252 Grade 3 specifications and had a yield strength of 400 MN/m^2. The moment of inertia of the pile alone was 1.16 x 10^8 mm^4. To protect strain gauges on instrumented piles, angle irons were welded to each side along the length of the pile. This action increased the moment of inertia to 1.43 x 10^8 mm^4. The piles were driven to a depth of approximately 11.6 m, leaving 2.1 m above the ground surface. A single test pile having the same properties was also driven and tested for comparison purposes.

The layout for the pile group test is shown in Fig. 2 Load was applied to the pile group by reacting against the two 1.19 m-diameter drilled shafts as shown in Fig. 2 A 1.34-MN hydraulic jack was placed south of each drilled shaft to apply the load to a W 760 x 284 beam. Eight threaded (Dywidag) bars then transferred the load from this beam to an identical beam that was bolted to the pile group load frame. The load frame for the pile group tests was essentially rigid in comparison to the stiffness of the piles so that each pile was constrained to have essentially the same pile head displacement.

Each pile was attached to the frame with a pinned connection to provide a free-head boundary condition. The tie-rod connecting the pile to the frame was instrumented with strain gauges so that the load carried by each pile could be measured during the test. Displacement of each pile was measured with LVDTs attached to an independent reference frame. Bending moment versus depth was obtained using strain gauge pairs located at 15 depths along the length of the pile. Downhole accelerometers were magnetically attached to the row 1 center pile at 8 depths along the length of the pile. The acceleration time histories were integrated to provide velocity and displacement time histories along the length of the pile.

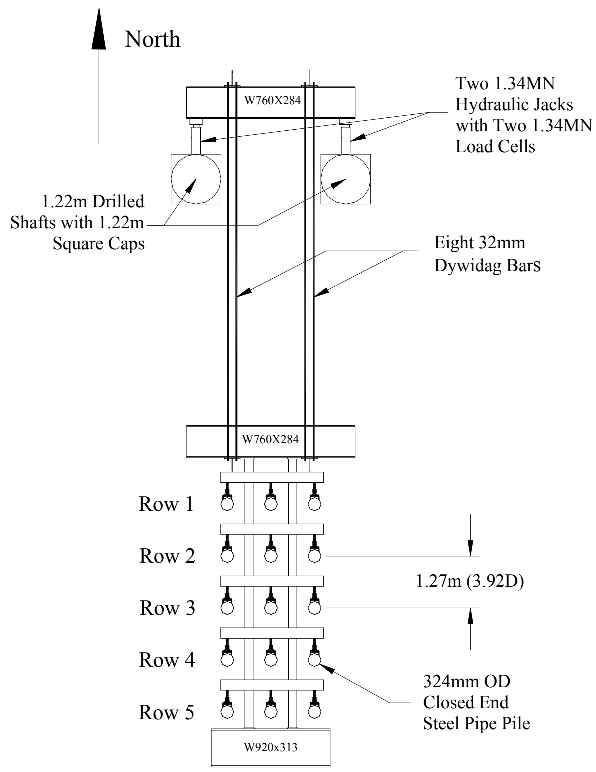

Figure 2. Layout for pile group load test.

4 STATIC LOAD TESTS

4.1 *Test Procedure*

The static load test was performed incrementally using a deflection controlled approach. At each increment 15 cycles were applied to approximately the same deflection level to evaluate the decrease in lateral resistance that would occur due to repeated loading and the formation of gaps during a large earthquake. A Magnitude 7.0 earthquake would be expected to produce about 15 cycles of loading.

4.2 *Test Results*

A plot of the total peak load vs. deflection curves for the 1st and 15th cycles of load are provided in Fig. 3. The peak load typically decreased by about 17% from the initial cycle to the last cycle. Most of this strength loss occurred within a few cycles and stabilized after about 10 cycles.

In contrast to expectations based on elastic theory, there was no consistent trend of load distribution within a given row. However, the lateral resistance was a function of row position within the group as has been observed in other full-scale tests in clay (Rollins et al, 1998). A plot of the average load vs. deflection curves for piles in each row in the group is presented in Fig. 4. Piles in the first row carried the largest average load and the load-deflection relationship was similar to that for the single pile. However, for a given deflection, the lateral resistance

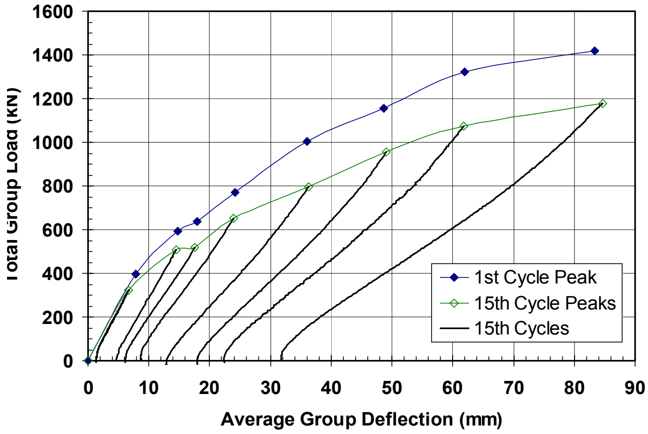

Figure 3. Peak total load vs. average group deflection for the 1st and 15th cycles.

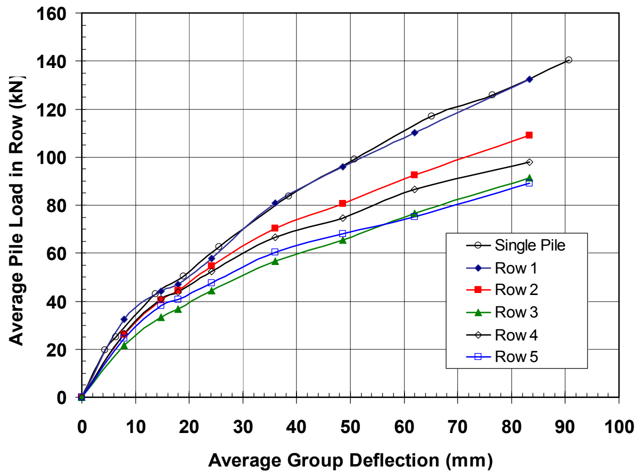

Figure 4. Average load vs. deflection curves for each row

decreased successively from the second to the third row piles, but then remained roughly constant for the remaining rows although the fourth row exhibited somewhat higher resistance. The increase in lateral resistance for the fourth row is attributed to random variations in strength properties around the group. These results suggest that the lateral resistance for the third and higher rows in a pile group can be treated as constant for design purposes.

For a given load, the maximum bending moment for the front row piles was about the same as for the single pile. However, for trailing row piles the maximum moment was as much as 30% higher.

5 DYNAMIC LOAD TESTS

The dynamic force was applied using the statnamic loading sled which pushed the pile group in the same direction as for the static load tests shown in Fig. 1. Prior to dynamic loading, the Dywidag bars were disconnected from the frame. The statnamic system could produce a lateral force of up to 3500 kN (787 kips) in 0.15 to 0.25 seconds. Generally, the statnamic load was applied after the application of 15 static cycles so that it would represent a 16th cycle.

A comparison between the 15th static group load-deflection curve and the subsequent dynamic group load-deflection curve is provided for several deflection increments in Fig. 4. Despite the presence of gaps, the dynamic resistance was typically 30 to 60% higher than the peak static resistance. In addition, the area inside the load-deflection curve for the dynamic test is significantly larger than that for the static test indicating that damping in providing a significant contribution to the total measured resistance.

Group load reduction effects were not as significant during the dynamic loading as they were during the static loading, but they were still evident with the first row carrying the greatest load.

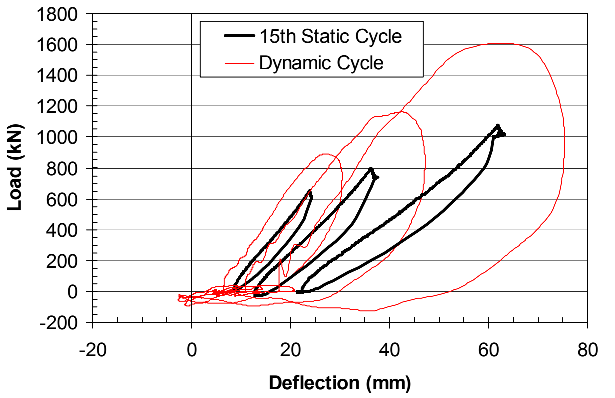

Figure 5. Comparison of 15th static load-deflection curve with dynamic (16th) load-deflection curve.

6 ANALYSIS OF TEST RESULTS

6.1 *Static Test*

The lateral response of the single pile and the pile group was analyzed using the computer programs LPILE (Reese and Wang, 1994) and GROUP (Reese et al, 1996). These programs treat the pile as a beam and the soil as non-linear springs using p-y curves. The p value is the horizontal force per length along the pile and the y value is the horizontal displacement. For the single pile test analysis, standard p-y curve shapes for soft clay and sand were used without modification. The undrained shear strength profile used in the analysis is also shown in Fig. 1 and is within the range of measured strength values. The strength profile was varied in order to improve the match between the measured load vs. deflection and load vs. moment curves.

The soil properties were then held constant during the analysis of the pile group. To account for group interaction, the p values for a given row were reduced using a constant multiplier (f_m) until a match was obtained between the measured and computed response as suggested by Brown et al (1987). Based on this analysis, the f_m values for deflections from 0 to 38 mm were determined to be 1.0, 0.87, 0.64, 0.81, and 0.70 for Rows 1 to 5, respectively. However, for higher deflections it was necessary to decrease the f_m values to match the measured results. The back-calculated *p*-multipliers for deflections from 38 to 89 mm were determined to be 1.0, 0.81, 0.59, 0.71, and 0.59 for Rows 1 to 5, respectively. Excellent agreement was obtained between the measured and computed load–deflection curves with these adjustment factors.

6.2 *Dynamic Tests*

Analyses were also performed to separate out the components of lateral resistance developed during the statnamic testing. These components include static "spring" resistance, damping resistance, and inertial resistance. These analyses were performed by treating the pile group as an equivalent single degree-of-freedom system, which is admittedly a simplification of a complex reality. The response of the group was evaluated using the fundamental equation of force equilibrium

$$F_i = ma_i + cv_i + kx_i \quad (1)$$

where:

F_i = Total force applied to the group of piles
m = mass of the group and soil in some case
a_i = acceleration of the group at the load point
c = Total damping coefficient
v_i =Velocity of the pile group at the load point
k = Total static stiffness constant
x_i = Average deflection of the pile group

The mass of the pile group was initially assumed to be the mass of the piles within the active length (l_a) as defined by Gazetas and Dobry (1984). However, to match the measured response, it was necessary to add the mass of the soil between the piles after the gaps had been closed. Eq. (1) could be used directly to back-calculate the damping coefficient with other known parameters. However, this approach was very sensitive to electrical noise and slight instrumental errors in the acceleration time histories which were not of high quality. In an effort to smooth the measured acceleration curves, the acceleration, a_i at each time step was computed using the equation

$$a_i = \frac{F_i - cv_{i-1} - kx_{i-1}}{m} \quad (2)$$

where v_{i-1} and x_{i-1} are the velocity and displacement for the previous time step.

Acceleration, velocity and displacement were zero for the first time step. The static secant stiffness, k, in Eq. (2) could be estimated using the static load-deflection curve for the pile group; however, the initial value of c had to be estimated and then refined through subsequent trials. Once acceleration was computed, the pile head velocity and deflection were computed using the equations,

$$v_i = a_i t + v_{i-1} \quad (3)$$

$$x_i = v_i t + x_{i-1} \quad (4)$$

The appropriate value for the damping coefficient was then determined by minimizing the sum of the differences between the computed and measured deflection time histories. This optimization was accomplished using tools within the spreadsheet program excel. Back-calculated damping ratios were typically between 30 and 40%. A typical plot of the measured deflection time history in comparison with that computed using Eq. (5) is provided in Fig. 6. The agreement is generally quite good although some divergence is evident in the rebound portion of the curves.

The measured load-deflection curve for the same statnamic test is provided in Fig. 7 along with the measured static load-deflection curve for the 15th cycle. The calculated static load versus deflection curve is also shown in Fig. 7 and it lies nearly on top of the measured static load versus deflection curve. Therefore, the assumption of a linear load versus deflection curve for this test is appropriate.

7 CONCLUSIONS

Based on the results of the test program the following conclusions can be made:

1. Fifteen cycles of loading at a given deflection typically produced a 17% degradation in lateral load resistance at the peak load. Greater reductions occurred at smaller deflections due to the presence of gaps around the piles.

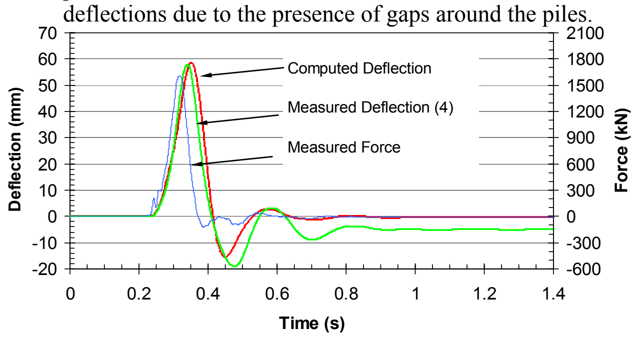

Figure 6. Measured and computed load vs deflection curves time.

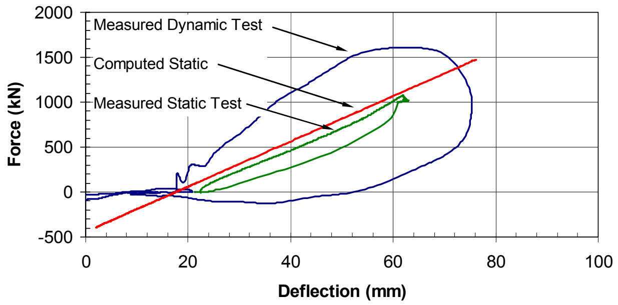

Figure 7. Comparison of measured static load-deflection curve and curve computed using statnamic load-deflection curve

2. Group interaction effects significantly reduced the lateral resistance of the piles in the group even at 3.92 D spacing for a given deflection relative to a single isolated pile.
3. Reduction in lateral resistance was a function of row within the group rather than location within a row. The front row piles carried the largest average load which was similar to the single pile. The resistance decreased successively from the second to the third row, but then remained roughly similar for subsequent rows although the fourth row was somewhat higher.
4. P-multipliers could adequately account for group effects.
5. The load-deflection curves for the dynamic load tests were significantly stiffer than the 15th static load-deflection curves and enclosed larger loops suggesting significant damping even with the presence of gaps around the piles.
6. Simplified single-degree-of-freedom analyses suggest that the increased dynamic resistance was primarily due to damping and that group damping ratios were 30 to 40%.

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