# VIBRATIONAL TESTING OF A FULL-SCALE PILE GROUP IN SOFT CLAY

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## SUMMARY

A 3 x 3 pile group and pile cap were constructed in a soft clay site located near the Salt Lake City International Airport. This pile group was tested under vibrational lateral loads at several times during the overall testing program for this structure. The horizontal loading was accomplished using an eccentric mass shaking machine as well as an impact loading using a twelve pound hammer. The results obtained from this study, are compared to the results obtained from other work [Peterson and Rollins, 1996, Weaver *et al*., 1999] performed on this structure.

The purpose of the work was to determine the dynamic response of the pile group as a function of excitation frequency. The dynamic response includes the natural frequencies or stiffness, and damping. Substantial changes in these parameters were observed due to changes in the physical state of the structure. Also observed were the substantial changes in dynamic characteristics as a function of the amount of strain induced in the system. This has significance in the extrapolation of small strain test data to the use for large strain problems, and also is significant because of the substantial changes in a system before and after an earthquake or other major event.

The outcome is a series of plots showing the variation in stiffness of the foundation system as a function of frequency. Also determined is the damping of the system in the two orthogonal horizontal directions. Of particular interest is the change in these quantities as other testing is performed on the structure that alters the condition of the piles.

This paper concludes that the dynamic response of a pile group is very much a function of the frequency of the excitation, and therefore must be included in the structural dynamic model if accurate predictions of behavior are desired. Also, it was determined that for soft clays the effect of large displacement testing, an earthquake, or other major loading may result in large changes in the dynamic response of the foundation system.

## INTRODUCTION

The lateral load capacity of pile foundations under dynamic loading is important for structures subject to earthquakes, wind, wave action, and ship impacts. Although fairly reliable methods have been developed for predicting the lateral capacity of single piles under static loads, there is very little information to guide engineers in the design of closely spaced pile groups under dynamic loads. Assessment of dynamic behavior of piles and groups of piles necessitates the development of alternatives to the standard static load test methodologies. A test method which more accurately approximates a typical loading could be beneficial in predicting the true behavior of pile groups.

In this paper the results from forced vibration testing on a pile group are presented. Both sinusoidal forcing as well as impact forcing were used. The objectives of this study were to determine the dynamic characteristics of a pile group and to determine how these characteristics depend on, and vary with, frequency of excitation. Also of

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interest is the effect that different condition states have on the response of a pile cap. The results of this work will be compared with the results of other testing on the specimen structure which was conducted by others.

## TEST STRUCTURE AND TEST SITE

The test site was located 300 meters north of the new control tower of the Salt Lake City International Airport in the state of Utah.

Nine piles were driven to 9.1 m depth with nominal spacing of three pile-diameters centered in 3 x 3 pattern to form a pile group. The piles were driven in October 1995. The steel circular piles had a nominal outer diameter of 305 mm with a 9.5 mm wall thickness. The elastic modulus of the steel was 200 GPa and the minimum yield stress was 331 MPa. The piles were filled with pea-gravel concrete with 18.6 MPa compressive strength. The steel jacket of each pile was terminated at the bottom of the pile cap.

The pile cap was reinforced concrete with dimensions of 2.74 m x 2.74 m x 1.22 m. Steel reinforcing included 32 mm bars at 178 mm centers at the top and bottom of the pile cap. Eight 25 mm bars, 2.3 m long, were embedded 1.22 m into each pile to connect the piles to the pile cap. Circular ties with 13 mm bars at 300 mm centers were used to tie the vertical reinforcing bars. The pile cap was placed on 150 mm of compacted granular fill that was placed on the undisturbed native soil. For both the vibration tests performed in this study, there was no backfill on any side of the pile cap. Therefore, the resistance to lateral loads was a result of the piles as well as load transfer at the bottom of the pile cap. Figure 1 shows the test structure and dimensions.

The soil profile near the surface consisted of a cohesive layer of low-plasticity silts and clays (3 m thick) underlain by a sand layer (1.5 m thick). The water table was located near the natural ground surface during the testing. The undrained shear strength was typically between 25 and 50 kPa, although some layers had strength above 100 kPa. The cohesive soil had a shear wave velocity ranging from 120 to 150 m/s. The underlying cohesionless soil layer consisted of poorly graded medium-grained sands and silty sands. SPT blowcount and CPT tip resistance values indicated that the sand was dense to very dense with corresponding relative densities of 65 to 85%. The shear wave velocity was 150 m/s.

The backfill, consisting of sandy gravel (75 mm) was compacted to approximately 95% of the modified proctor maximum density using a vibratory drum-roller along with hand-operated compactors immediately adjacent to the pile cap. SPT blow counts were between 35 and 45. The friction angle was determined to be 42 degree. A complete description of both the soil and the structure can be found in Peterson and Rollins [1996].

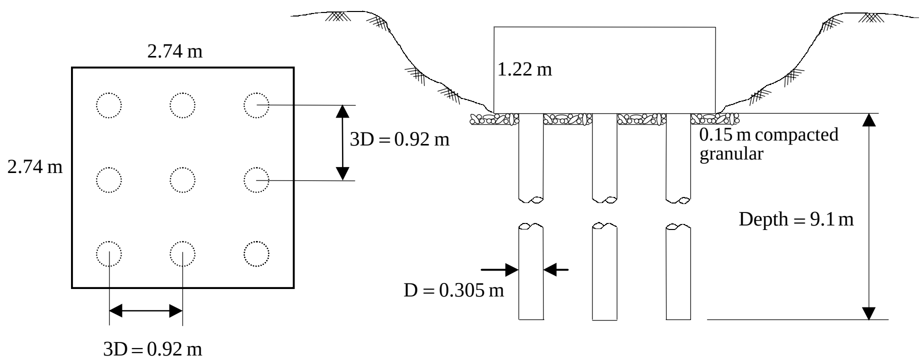


**Figure 1. Dimension of the pile group**

## TESTING

The vibrational testing presented here represents only one part of the overall testing that was performed on this particular structure. In order to properly interprete the results from the vibrational testing, Table 1 lists the complete set of tests that were performed at this particular site and some of the important information regarding the tests.

**Table 1. Chronological listing of lateral load tests**

| Test | Pile Cap Existing? | Backfill against Cap? | Direction of Forcing | Date | Approx. Max Displacement (mm) | Research Team |
|---|---|---|---|---|---|---|
| Static Loading | piles only | no | E | 6/10/96 | 60 | BYU |
| Statnamic I a | piles only | no | W | 7/25/96 | n/a | BYU |
| Statnamic I b | piles only | no | W | 7/25/96 | 16 | BYU |
| Statnamic I c | piles only | no | W | 7/25/96 | 27 | BYU |
| Sinusoidal Vibration I | yes | no | N-S,E-W | 8/2/96 | 0.07 | USU |
| Impact Vibration | yes | no | N-S,E-W | 8/2/96 | <0.05 | USU |
| Statnamic II a | yes | no | N | 8/6/96 | 20 | BYU |
| Statnamic II b | yes | no | N | 8/6/96 | 29 | BYU |
| Statnamic II c | yes | yes | N | 8/6/96 | 24 | BYU |
| Sinusoidal Vibration II | yes | no | N-S,E-W | 8/4/97 | 0.3 | USU |

The static load was applied in the East direction with no pile cap present. The loads were applied using hydraulic rams. Both load and deflection were monitored. For details of this testing see Peterson and Rollins [1996].

The statnamic I tests were performed by applying a lateral force to the pile group by launching a reaction mass away from the pile group. This was accomplished by igniting a solid fuel propellant inside a cylinder (piston). The ignition of the propellant caused a rapid expansion of high-pressure gas that caused a sudden lateral force to the pile group which propels the reaction mass in the opposite direction. The three tests (a, b, c, in table 1) were performed with increasing load, with the loading of the piles in the West direction. The load rise time was typically about 0.13 seconds and load duration was about 0.30 sec.

The first set of sinusoidal and impact testing was performed after the free-head statnamic tests, with the pile cap in place, previous to the second set of statnamic tests.

The Statnamic II tests were conducted with the pile cap in place. Again three tests were performed with two (a, b from table 1) occurring with no backfill, and the third (c from table 1) with a compacted granular fill to develop passive resistance against lateral motion. These tests were performed in the North direction.

The second set of sinusoidal vibration testing was performed after the statnamic II tests had occurred.

**Sinusoidal Vibration I**

An eccentric mass shaking machine, Vibration Generator System Model VG-1, provided the forcing. Excitation frequency of the machine ranges from approximately 0.5 Hz to 9.7 Hz.

Response of the pile cap was recorded using Kinemetrics ranger velocity transducers mounted on the pile cap. The data acquisition system utilized was an IOTECH Daqbook 216, 16 bit analog to digital card connected to a notebook computer for data manipulation and storage. The data was collected at a sampling rate of 100 Hz.

Steady state excitation was provided from 0.60 Hz to 2.30 Hz at increments of 0.10 Hz. Then from 2.30 to 2.90 Hz at an increment of 0.20 Hz. Due to technical problems the shaker was unable to provide excitation above 2.9 Hz. The data retrieved from this testing is plotted in Figures 2 and 3 for the North-South and the East-West directions, respectively. As can be seen, the resonant frequencies of the system were not reached in this testing, but some data regarding low strain stiffness of the system was recorded.

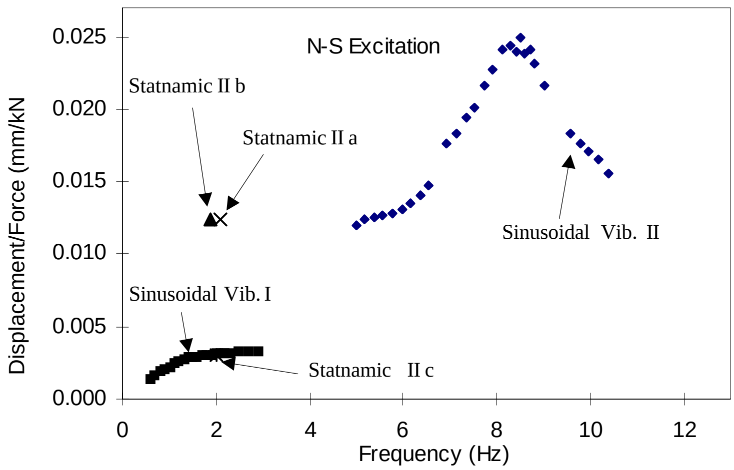


**Figure 2. Displacement per unit force from sinusoidal vibration and statnamic II**

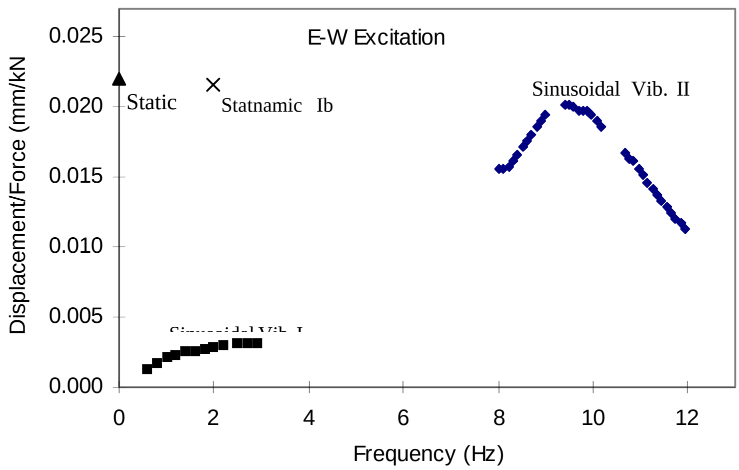


**Figure 3. Displacement per unit force from sinusoidal vibration, static, and statnamic I**

**Impact Vibration**

A 12 pound sledge hammer was used to induce the vibration in the pile group. The pile cap was hammered manually at the middle of the pile cap in the South and in the West directions.

The data was acquired as in the sinusoidal testing as described in the previous section. An example of the vibrational response as a result of the impact forcing is given in Figure 4. From many of these time domain records in both the North-South and the East-West directions, the damped natural frequency and equivalent viscous damping ratio were determined.

It was found that the first natural frequency in the E-W direction was 9.5 Hz with 14% damping. In the N-S direction the first natural frequency was determined to be 17.5 Hz with 15% damping. These values are summarized in Table 2.

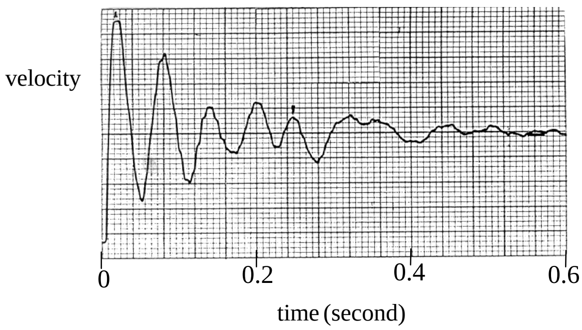


**Figure 4. Velocity recorded on strip chart for the N-S impact vibration test**

**Sinusoidal Vibration II**

The sinusoidal forcing was supplied for this second vibration testing by a new AFB Engineered Test Systems Model 4600A eccentric mass shaking machine.

Data was collected at a sampling rate of 100 Hz with a portable data acquisition system, Kinemetrics VS-3000, that utilizes an IOTECH Daqbook 216, 16 bit external analog to digital card. This system was then fed to a desktop personal computer for storage using Daisylab data acquisition software. Data processing was accomplished after field tests using MATLAB digital signal processing software.

Vibration tests were conducted in both the North-South and East-West directions. Six accelerometers were placed on the pile cap and two were placed in the free field approximately 9 meters from the pile cap. The placement of the accelerometers and eccentric mass shaker are shown in Figure 5.

Steady state excitation was performed for frequencies from 5.00 Hz to 8.00 Hz at increments of 0.20 Hz. Then from 8.00 to 10.00 Hz at increments of 0.10 Hz, and from 10.00 to 20.00 Hz at frequency increments of 0.50 Hz. This testing was completed for both the N-S as well as the E-W directions. Figure 2 presents the results of the N-S testing, indicating the fundamental frequency at 8.5 Hz and the damping ratio calculated by the half-power band width method to be 17% of critical damping. Figure 3 presents the results of the E-W testing, indicating the fundamental frequency at 9.5 Hz and the damping ratio calculated by the half-power band width method to be 19% of critical damping.

To confirm the damping ratio for this testing, Figure 6 shows a plot of the phase lag between the sinusoidal forcing and the displacement response of the pile cap. As can be seen, the phase shift is 180 degrees through the resonance and the best fit curve for damping ratio is approximately 20% of critical damping.

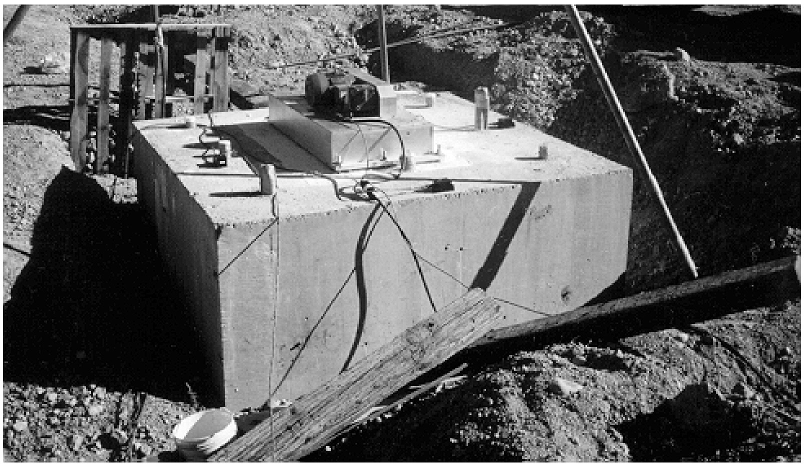

**Figure 5. Pile cap and eccentric mass shaker**

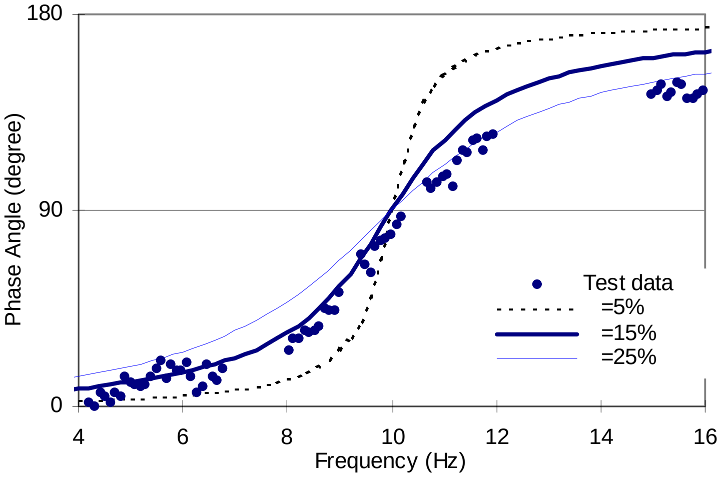


**Figure 6. Phase angle versus frequency E-W excitation sinusoidal vibration II**

**Table 2. Comparison of natural frequency and damping ratio at each test**

| Test | Direction | First Mode | |
|---|---|---|---|
| | | Nat. Freq. | Damping |
| Impact Vibration | N-S | 17.5 Hz | 15% |
| | E-W | 9.5 Hz | 14% |
| Sinusoidal Vibration II | N-S | 8.5 Hz | 17% |
| | E-W | 9.5 Hz | 19% |

## RESULTS

The first vibrational testing was performed after significant testing had already been performed in the East and West directions, but the piles were relatively undisturbed at that point in the N-S direction.

The significant difference between the natural frequency of the pile cap in the N-S and the E-W directions is almost certainly due to the extreme loadings of the piles, both static and statnamic, in the East and West directions prior to the vibrational testing. The fundamental frequency in the E-W direction was determined to be 9.5 Hz whereas, in the N-S direction which was relatively undisturbed the fundamental frequency was determined to be 17.5 Hz.

Figure 2 contains the results from the statnamic test at approximately 2 Hz for comparison with the results from the vibrational tests. The statnamic tests were plotted at 2 Hz because the rise time for these tests was on the order of 0.125 seconds which corresponds to a quarter cycle of a 2 Hz sinusoid forcing. As can be seen, test (a) and test (b) result in significantly higher displacements than those of the vibration testing. This is due to the softening of the system at larger strains. Test (c) is very comparable with the vibrational testing deflections presumably because of the added stiffness of the granular backfill.

Figure 3 also shows the larger displacements of both the static and the statnamic tests in the E-W direction due to the softening of the system under large strains and the fact that in the E-W direction these tests were performed on the piles in the free-head state.

It is also significant to note that the high fundamental frequency of the system in the N-S direction between the first vibration tests (17.5 Hz) was reduced significantly (8.5 Hz) because of the large loadings inflicted in the N-S direction during the time between the two tests.

## CONCLUSIONS

There was a substantial change in natural frequency of the system between the natural state and after it had been loaded using static or statnamic methods. This indicates the substantial effect that major loading can have on a foundation system even during a single event.

The small strain stiffness of the system (vibrational testing) was considerably higher than the stiffness of the same system loaded under large strain conditions. Again this is important for consideration in the design of earthquake resistant structures in which considerable lateral resistance is provided by the foundation system.

The damping ratio of this structure was very high, in the range of 14 to 19 % of critical damping.

## REFERENCES

Peterson, K. T. and Rollins, K. M. (1996). *Static and Dynamic Lateral Load Testing of a Full-Scale Pile Group in Clay*. Civil Engineering Department Research Report #CEG. 96-02, Brigham Young University, Provo, Utah. 222 pp.

Weaver, T. J., Rollins, K. M., and Peterson, K. T. (1999). Lateral Statnamic Load Testing and Analysis of A Pile Group (to be published).