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Full-Scale Vertical and Horizontal Dynamic Testing of a Double Helix Screw Pile

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ABSTRACT
Helical screw piles can provide immense construction and performance advantages over the conventional concrete and steel piles. To further exploit their potential advantageous performance, there is a need for thorough and systematic characterization of their performance. For applications that involve dynamic loading, it is necessary to qualify and quantify their dynamic characteristics. This paper presents the first full-scale dynamic field testing program executed on large capacity helical screw piles. The complete test setup and test procedures are described to provide the basis for future dynamic load testing of screw piles. A series of quadratic type harmonic loading tests was conducted on a 324 mm diameter steel pile installed 9.0 m into a clayey deposit. The pile was fitted with two helices each of diameter 610 mm. A Lazan type mechanical oscillator was used to generate frequency sweep loadings from frequencies below to frequencies above the expected resonant frequency of the pile-soil-cap system at five force intensities for each loading direction. The acceleration at the level of the centre of gravity of the pile cap-oscillator system was recorded. Conventional soil boring and testing methods were used to determine the subsurface soil profile and static soil properties. Dynamic soil properties in the test area pile were determined using the seismic cone penetration technique. The initial analysis of the testing results reveals basic dynamic characteristics of the pile-soil system and gives some insights into the interface conditions. In addition, the results of the current project can be employed to evaluate the adequacy of the existing theoretical approaches in order to predict the dynamic response of screw piles. The data presented is therefore considered valuable to other researchers and engineers considering the dynamic performance of helical screw piles.

RÉSUMÉ
Motoréducteurs à vis des piles peut fournir d'immenses avantages en matière de performances et de la construction sur le béton conventionnel et de pieux en acier. Pour exploiter leur potentiel de performance avantageux, il est nécessaire pour la caractérisation approfondie et systématique de leurs performances. Pour les applications qui impliquent le chargement dynamique, il est nécessaire de qualifier et de quantifier leurs performances dynamiques. Ce document présente la première grande échelle verticale et horizontale dynamique programme d'essais sur le terrain exécutées sur une grande capacité à vis hélicoïdales pile. Le test de configuration et les procédures d'essai sont décrites à fournir une base pour de futurs essais dynamiques effectués sur des piles à vis. Une série de type quadratique chargement harmonique tests a été réalisée sur un diamètre de 324 mm en acier 9.0 m pile installée dans un dépôt argileux. La pile a été muni de deux hélices de chaque diamètre 610 mm. Un oscillateur mécanique Lazan type a été utilisé pour générer des charges de la fréquence de balayage des fréquences inférieures aux fréquences au-dessus de la fréquence de résonance de la pile-sol-système de plafonnement à cinq intensités vigueur pour chaque direction de chargement. L'accélération au niveau du centre de gravité de la pile système de cap-oscillateur a été enregistrée. Sol conventionnel ennuyeux et méthodes d'essai ont été adoptées pour déterminer le sous-sol et statique, les propriétés du sol. Dynamique des propriétés du sol à côté de la pile testés ont été déterminés en utilisant la technique de pénétration au cône sismique. L'analyse initiale de la base des résultats des tests révèlent les caractéristiques dynamiques de la pile et le sol et donne un aperçu de l'interface conditions. En outre, les résultats du projet en cours peut être utilisé pour évaluer la pertinence de l'existence d'approches théoriques afin de prédire la réponse dynamique de la vis piles. Les données présentées est donc considéré comme très précieux pour d'autres chercheurs et d'ingénieurs envisagent la performance dynamique des piles à vis hélicoïdales.

1 INTRODUCTION

Since the nineteen forties and helical screw anchors and piles have been used on a large scale as a foundation system for several engineering applications. The popularity of helical piles stems from their ease of installation, superior performance in certain soil profiles and other salient features compared to the conventional piling systems. Helical screw piles are a cost effective alternative, can be easily installed using minimal equipment, and can be removed and reused. Furthermore, they allow immediate loading upon installation. Likewise, in the case of high ground water level, helical piles save dewatering and/or pumping of the construction site (Bobbitt and Clemence, 1987). Helical screw piles are light in weight and don’t require very heavy drilling rigs, thus they are more convenient to be installed in weak soil conditions and with little soil disturbance.

Helical screw piles are mostly designed to sustain static loading, especially the uplift loading conditions. Such applications included transmission towers, pipelines, residential and industrial buildings, and supporting retaining structures (Zhang 1999; Carville and Walton 1995; Adams and Klym 1972). Recently, helical piles are considered for seismic applications. El Naggar and Abdelghany (2007a,b) carried out an extensive full-scale field testing program to investigate the performance of plain and composite helical screw piles under cyclic loading.

Pile foundations are commonly employed in industrial applications that generate dynamic loads such as power plants, petroleum platforms, oil refineries, offshore structures, wind turbines, compressor stations, and machine foundations. Many field tests have been carried out on single and group conventional piles installed in different soil conditions (Boominathan and Ayothiraman 2006; El-Marsafawi and Novak 1992; Blaney and O'Neill 1986; Muster and O'Neill 1986; El Sharnouby and Novak 1984; Novak and Grigg 1976)

The dynamic performance of helical screw piles is of significant interest because they can offer an efficient alternative to conventional piling systems in many applications where the foundation would be subjected to dynamic loads such as machine foundations, wind turbines power plants. This interest initiated the need to understand their dynamic performance and evaluate their dynamic characteristics (stiffness and damping) through series of in-situ tests. The current research represents the first full-scale dynamic loading program of helical screw piles. The research includes two stages: the field testing of helical screw piles with different geometrical configurations and the developing of analytical solutions and design charts to estimate the dynamic parameters.

2 OBJECTIVES

This paper presents the detailed description of the test setup, experiment procedure, and test results of the dynamic vertical and horizontal performance of a full-scale isolated soil-double helix screw pile-mass system. The system was tested under both steady-state and free vibration excitation.

3 SITE AND GEOTECHNICAL CONDITIONS

The test site is located about 11.0 km to the north of the town of Ponoka, Alberta. The subsurface investigation was carried out by the seismic cone penetration testing (SCPT) and a mechanically augered test hole to a depth of 15.0 m. The site soil consisted of an upper layer of organic silt to silty sand of 1.0 m thick underlain by 2.1 m of medium stiff to stiff clay to silty clay soil, 1.5 m of layers of silty clay, and clayey silt, 0.5 m of clayey silt to silt, 1.0 m of medium silty sand to sand, and 7.7 m of homogeneous layer of very stiff to hard clay till with occasional lenses of silty clay mixture overlying 1.2 m of silty sand, silt, and clayey silt. The ground water level was established at 1.2 m below the ground surface. Figure 1 illustrates the locations of the test pile, SCPT soundings, and augered hole.

In-situ density: The in-situ density was determined using the soil classification charts provided by Robertson *et al*. (1986) in combination with the cone tip resistance and friction ratio corrected for the effect of pore pressure. In addition, the moisture content was measured by a conventional laboratory water content test carried out on the extracted soil samples from the borehole (Table 1).

Shear wave velocity: The small-strain shear wave (S-wave) and compression wave (P-wave) velocities were determined using the economical downhole measuring technique incorporated with the seismic cone. The source consisted of a steel beam pressed against the ground by the weight of the CPT truck and a hammer with an electronic trigger for both S and P waves generation. The S-wave was generated by hitting the beam ends horizontally with the hammer in the direction of the long axis, while P-wave was generated by hitting one beam end vertically using the same hammer. The seismic cone penetrometer was pushed into the ground and penetration was stopped at 1.0 m intervals. During the pause in penetration, the S and P waves were generated consequently at the ground surface and time required for the wave to reach the triaxial accelerometer in the cone penetrometer was recorded. The distribution of the shear wave velocity with depth is shown in Figure 2. Based on the field velocity measurements of the S and P waves, the values of Poisson's ratio were estimated to vary between 0.4 and 0.47. Such high Poisson's values were considered to have insignificant influence on the dynamic response of the soil-pile system based on Dobry *et al*. (1982).

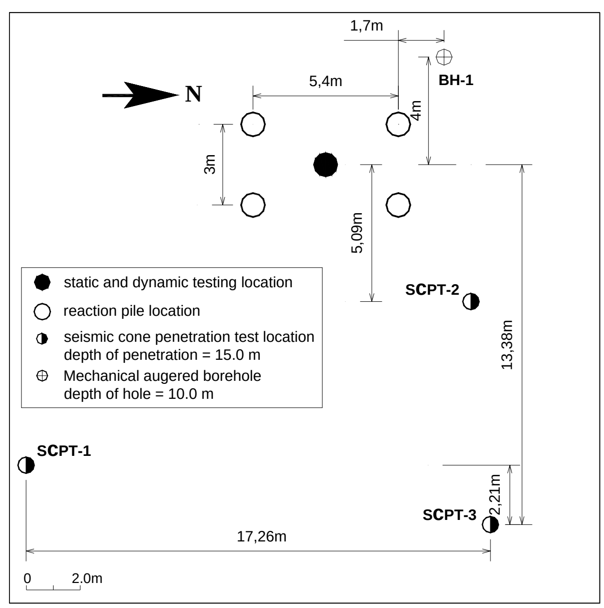

Figure 1. Location of piles, seismic cone penetration tests, and borehole

Table 1. Soil properties

Depth (m)	Bulk unit weight (kN/m^3)	Moisture content (%)
0-1.0	18.2	16.1
1.0-3.1	17.7	18.3
3.1-4.6	17.7	17.8
4.6-5.1	18.0	17.3
5.1-6.1	18.8	29.0
6.1-13.8	17.5	21.0
13.8-15	18.0	-

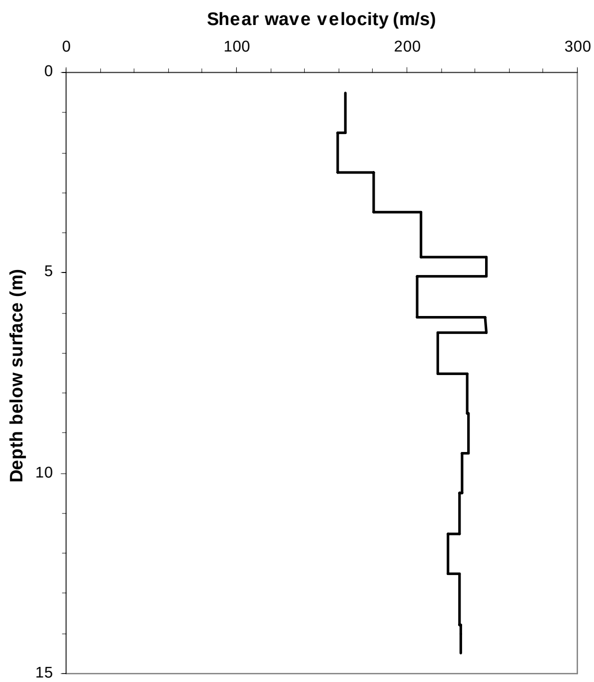

Figure 2. Shear wave velocities distribution

4 EXPRIMENTAL SETUP

4.1 Test Pile

The test pile was a 9.0 m long double-helix screw pile with outer steel pipe shaft diameter of 0.324 m, helix diameter of 0.61 m, and inter-helix spacing of 0.91 m. The leading edge on the helical plate was rounded back and sharpened to facilitate the installation of the pile with minimal soil disturbance, while the pile tip was cut at 45^o to aid in targeting the pile during installation. The pile was closed-ended with a flush closure steel plate. Table 2 and Figure 3 provide all required geometrical and material properties of the test pile. The pile was installed into the soil by applying a clockwise turning moment (torque) to the pile shaft using a hydraulic torque head, while sustaining a constant rate of penetration of one helix plate pitch (6.0 in, 152.4 mm) per revolution.

Table 2. Properties of the test pile

Property	Value
Pile type	Steel pipe pile with double helix
Outer diameter	0.324 m
Inner diameter	0.305 m
Moment of inertia	1.164×10^{-4} m^4
Area	9.4102×10^{-3} m^2
Length	9.0 m
Helix plate diameter	0.61 m
Helix plate thickness	0.019 m
Young's modulus	210 GPa
Poisson's ratio	0.3
Damping ratio	0.01
Unit weight	78.46 kN/m^3

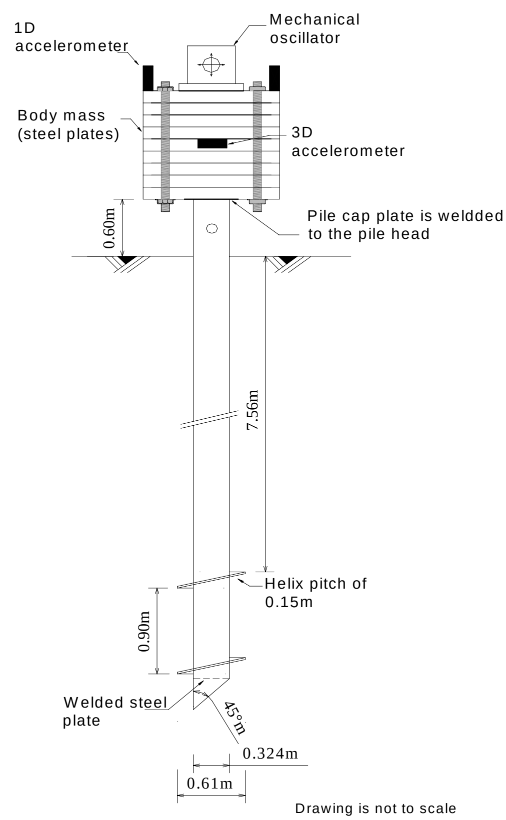

Figure 3. Pile-mass system

An axial compressive force was applied to the shaft to prevent the pile rotation in place without penetrating the soil and to maintain the specified penetration rate which minimizes any soil disturbance. The unsupported pile length protruded 0.6 m with a test body mass attached at the top of the pile as shown in Figure 3.

4.2 Test Body

To ensure that the resonance frequencies were well defined and within the frequency range of the exciting machine, a steel test body was added on top of the pile cap. This also allowed simulating the effects of a superstructure on the response of the pile-soil system. The pile cap was a machined steel plate, 700 × 460 mm rectangular and 38.0 mm thick, and its weight was approximately 88 kg. The head of the pile was machined after completing its installation in order to keep a clean levelled edge to facilitate placing the cap perpendicular to the centreline of the pile. A circle equal to the diameter of the pile was drawn at the centre of the pile cap plate. The plate was then slipped over the pile head and its position adjusted until the drawn circle coincided exactly with the pile circumference in order to assure that both the plate and pile shared the same vertical axis. The plate was levelled in both directions and tack welded to the pile until a fillet weld could be completed around the circumference of the pile (Figure 3).

When the cap plate was securely affixed to the pile head, the steel test body plates were stacked on its top. The test body comprised of 59 machined circular steel plates each with diameter of 724 mm, thickness of 254 mm, and mass of 79 kg. The steel plates had machined contact surfaces to prevent slippage between the plates. The lower and the upper test body plates were centered and welded to the pile cap plate and the Lazan oscillator base plate, respectively. The entire stack of the test body plates was rigidly fastened together with four threaded steel rods to force the steel-plate mass to act as a rigid body (Figure 4). Threaded holes were drilled in few of the test body plates before testing in order to mount the accelerometers. One of the selected plates is the upper test body plate and the other was located as close to the elevation of the centre of mass of the entire system (oscillator-plates system) based on the number of plates used for each specific vibration test. The properties of the test bodies, oscillator, and pile cap are given in Table 3.

Table 3. Properties of test body, oscillator, and pile cap

Properties	Vibration direction: Vertical	Vibration direction: Horizontal
No. of plates	59	59
Mass of cap-test body-oscillator (kg)	4849.5	4849.5
Height of centre of gravity (CG), Z_c, (m) [1]	0.7911	0.7931
Height of excitation above centre of gravity (CG), Z_e, (m)	0.8606	0.9381
Mass moment of inertia about y ($kg.m^2$) [2]	1152.5	1166.9

[1] Height of CG, Z_c, is measured from the bottom surface of the pile cap plate, which is located at 0.6m above ground surface.
[2] y-axis lay horizontally through the CG and is parallel to the flexible shaft direction.

4.3 Excitation Mechanism

The vertical and horizontal dynamic excitations were produced by means of Lazan mechanical oscillator, Model MO2460. The dynamic excitation was quadratic and was characterised by harmonic forces proportional to the square of the driving frequency. The oscillator comprised of two counteracting shafts each carried a set of eccentric masses to generate the harmonic excitation. The magnitude of the excitation force was varied by altering the degree of eccentricity of the rotating unbalanced masses via an external knob at the end of one shaft assembly. The oscillator was driven by a 7.5 HP 220 V three phase motor capable of generating sinusoidal force of 23.5 kN peak-to-peak. The speed of the motor was controlled by a variable frequency AC speed drive, yielding stable operating speeds between 4 and 60 Hz. A well-balanced, flexible drive shaft was utilized to connect the oscillator to the motor through its end couplings (Figure 4).

The oscillator, of mass 51.5 kg, was placed on top of its base plate and was welded to the upper steel plate of the test body mass. Four holding rods were used to connect the holding channel frame to the oscillator base plate in order to keep the oscillator stable under vibration. The dynamic excitation was applied in the vertical and horizontal directions. For horizontal vibration, the oscillator was placed vertically on its base plate and in reverse for vertical vibration (Figure 4).

4.4 Instrumentation

To measure the dynamic response of the pile, it was necessary to determine the displacement amplitude of the test body mass at each frequency accurately. The vibration measuring apparatuses consisted of two uniaxial piezoelectric accelerometers Model 333B50, one triaxial accelerometer Model 356B18, frequency measurement unit (tachometer) Model KM2235B to measure excitation frequency, National Instruments data acquisition system Model SCXI-1520, and half-Poisson's bridge strain gauge circuits configured to indicate axial strain along the entire length of the pile.

The accelerometers were located on the test body in such way that two uniaxial accelerometers were mounted at equidistant positions from the foundation centre on the axis of symmetry. The triaxial accelerometer was mounted on one side of the test body, at the height of the centre of gravity. This location was assigned since the horizontal vibration is referred to the centre of gravity, and, therefore, would be no effect of rotation on measurements. For vertical vibration, the displacement responses derived from the three accelerometers were averaged to eliminate the rocking mode component. For horizontal vibration, the displacement was derived from acceleration measurements of the triaxial accelerometer, while the other two accelerometers provided measurements for the rocking component (Figures 3 to 5). To monitor the strain distribution and force distribution along the pile, the half-bridge strain gauge circuits were affixed on the inner and outer pile surfaces at specified locations. Each level of gauges encompassed four half bridges allocated equidistantly from each other.

Figure 4. Dynamic testing; (a) horizontal vibration; (b) vertical vibration

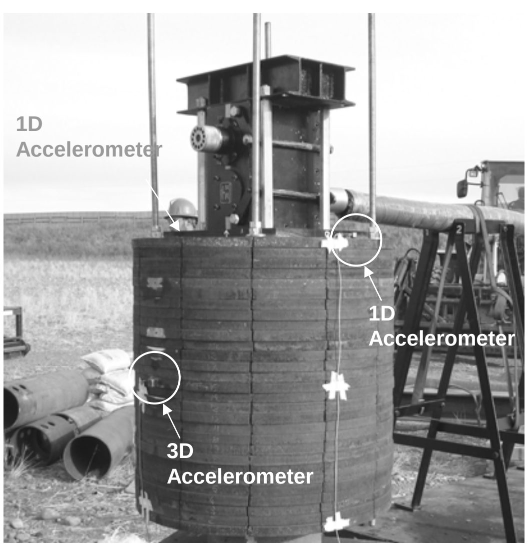

Figure 5. Location of accelerometers

5 TEST PROCEDURE

Steady-state vibration: The Lazan oscillator was connected to drive motor through the flexible shaft to complete the necessary mechanical connections. The harmonic excitation was measured after a suitable size of the test body mass was determined. Approximately 4661 kg of steel plates were added to the pile cap for dynamic testing in order to keep the resonant frequencies within the frequency range of the oscillator. For the vertical and horizontal excitation the test body consisted of 59 plates test body (4661 kg) and the pile was tested under 5 excitation intensities.

Initially, the Lazan oscillator was operated at low-force level (low excitation intensity) in order to keep the vibration amplitudes small enough to avoid any pile-soil separation and strong nonlinearity. The oscillator was driven to cover a frequency spectrum from about 4 to 60 Hz. The steady-state acceleration time history was recorded after reaching equilibrium by maintaining the frequency long enough at each specified frequency. After recording the acceleration along the frequency range of the oscillator for the lowest adopted excitation intensity, the eccentricity of the rotating masses was increased and the test was repeated. The peak-to-peak generated force levels varied between about 0.091 and 20.72 kN, which was deemed sufficient to bracket the vibration response of most full scale foundations.

Plucking test: At the end of the steady-state vibration tests, additional data were collected by applying a plucking (free vibration) test. It was conducted on the same soil-pile-test body configurations. The test body steel mass was subjected to a slight horizontal displacement using a robe that was horizontally connecting the body mass to a picking truck. The horizontal excitation load was applied suddenly by releasing the robe, while the acceleration response was recorded.

6 ANALYSIS OF TEST RESULTS

6.1 Plucking Test

Figure 6 shows the trace of the horizontal free vibration test. The damping ratio D was obtained from the acceleration time history using the logarithmic decrement method, as given by the following equation:

$$D = \frac{1}{2\pi} \ln\left(\frac{\ddot{U}_x}{\ddot{U}_{x+1}}\right) \qquad [1]$$

where $\ddot{U}_x$ and $\ddot{U}_{x+1}$ are the acceleration peaks in two successive cycles. This resulted in a damping ratio of approximately between 4.0 and 5.1 %.

6.2 Dynamic Response Curves

The response curves measured under harmonic vertical and horizontal excitation are plotted in Figures 7 and 8 in the form of single-amplitude displacements. The true displacement amplitude U_z and U_x for the vertical and horizontal vibrations, respectively, were computed through three steps: base line correction, filtering, and double integration. First, the acceleration time history was baseline corrected using cubic polynomial type correction in order to force the acceleration records to oscillate about zero. The corrected time history was filtered using the Butterworth bandpass filter to suppress all noise frequencies in the acceleration record. A Fast Fourier Transform (FFT) analysis was applied on the filtered record to obtain the dominant response frequency to corroborate its closeness to the excitation frequency. Then, a double integration process was employed using Simpson's rule to compute the displacement time history.

The excitation intensities are given in kg.m as $m_e.e$ in which m_e and e are the oscillator eccentric mass and the eccentricity of the rotating mass, respectively. The magnitude of the dynamic force is related to the eccentricity of the oscillator as follows:

$$P_d = m_e.e.\omega^2 \sin\omega t \qquad [2]$$

where ω is the circular frequency. The vertical response for the adopted 5 excitation intensities, shown in Figure 7, is quite linear with clear resonance peaks. The resonant frequency is linear for the low excitation intensities ($m_e.e$ = 0.091 and 0.12 kg.m) as the resonant frequency is ranged between 37 and 38 Hz. However, the resonant frequency decreased slightly from 37 to 35 Hz as the excitation intensity increased from 0.12 to 0.21 kg.m, respectively. It is, therefore, indicating a slight nonlinear response of the tested soil-pile system under vertical vibration. Figure 8 presents the response for horizontal vibration. The resonant frequency is practically the same for all eccentricities and is ranged between 3.43 and 3.67 Hz. At higher frequencies, the horizontal response amplitudes started to increase showing another peak, which is ascribed to the influence of the rocking mode of vibration.

Table 4 provides the resonant frequencies and the damping ratios determined from the frequency response curves using the half-power bandwidth method. It is obvious that computed damping ratios are larger than the value obtained from the free vibration test by approximately an average factor of 2.0.

The dimensionless response amplitudes (magnification factor, R_d) for the vertical and horizontal responses are determined by:

$$R_d = \frac{m}{m_e.e}.U_z \qquad [3]$$

where m is the mass of the oscillator-body mass-pile cap system. For horizontal vibration the true horizontal amplitude U_x replaces U_z. Figures 9 and 10 plot the dimensionless vertical and horizontal amplitudes versus frequency. These dimensionless response curves measured with different excitation intensities demonstrate nonlinear behaviour in case of vertical vibration. For horizontal vibration the curves corroborate the slight nonlinearity near the resonant frequency that turns to significant nonlinear behaviour at higher frequencies with the influence of the rocking component.

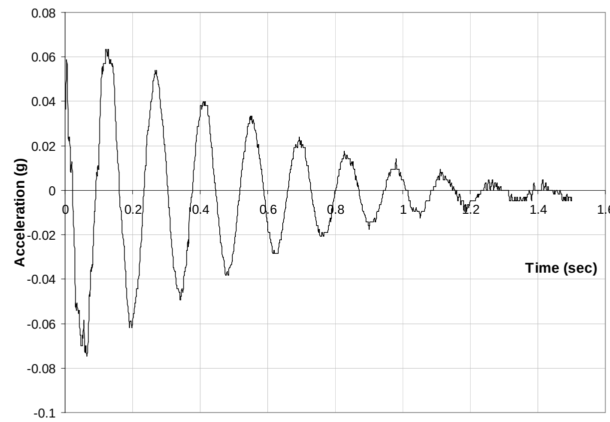

Figure 6. Typical free vibration record

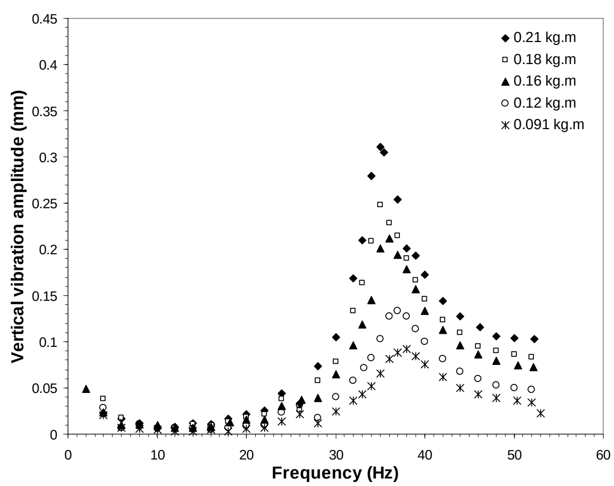

Figure 7. Vertical response curves

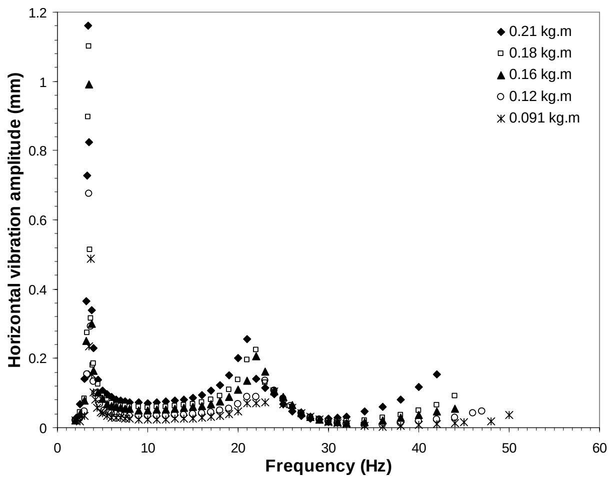

Figure 8. Horizontal response curves

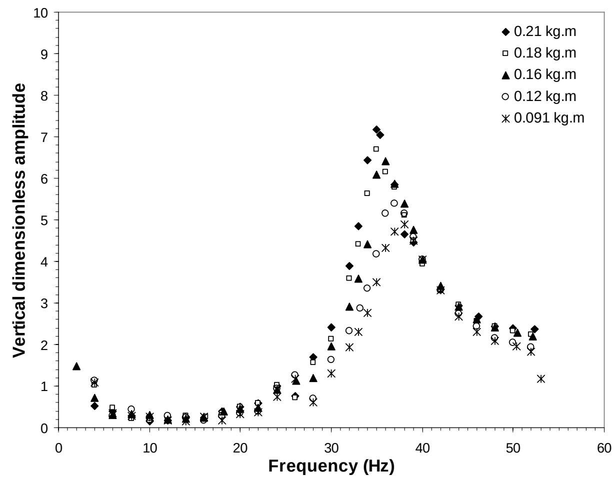

Figure 9. Dimensionless vertical response curves

Table 4. Resonant frequencies and damping ratios from response curves

Excitation Intensity (kg.m)	Resonant frequency (Hz)	Damping ratio (%)
	Vertical vibration	
0.091	38	7.5
0.12	37	7.29
0.16	36	7.36
0.18	35	7.57
0.21	35	6.85
	Horizontal vibration	
0.091	3.67	2.72
0.12	3.5	2.85
0.16	3.5	2.85
0.18	3.45	2.89
0.21	3.43	2.91

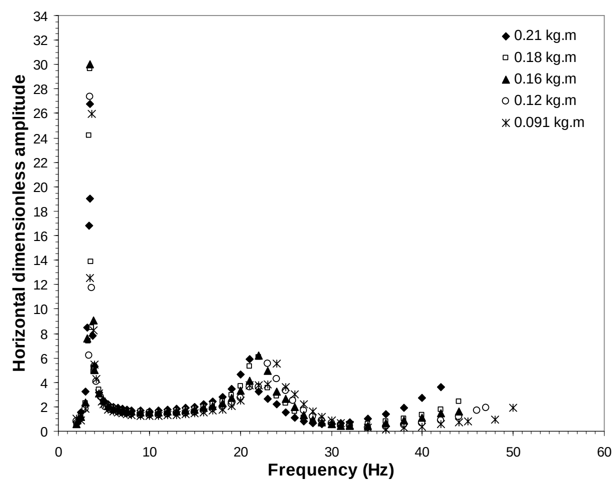

Figure 10. Dimensionless horizontal response curves

7 CONCLUSIONS

Dynamic testing of a 9.0 m double-helix screw pile under vertical and horizontal vibrations was carried out in field in clayey soil profile. Complete response curves were measured under different excitation intensities. The resonant frequencies, resonant amplitudes, and damping ratios were obtained. The dimensional and dimensionless response curves indicated a slight nonlinear response for resonance vertical and horizontal response amplitudes. An indication for significant nonlinearity was observed under large excitation frequencies.

Additional testing results and theoretical analyses will be provided in future papers.

8 ACKNOWLEDGEMENTS

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