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Performance of laterally loaded H-piles in sand

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ABSTRACT

Results from experimental testing of four approximately one-third scale laterally loaded H-piles, subjected to monotonic and cyclic loading are presented. The test setups were designed to prevent torsion in the pile during testing and to eliminate the self-weight of the hydraulic actuator that could otherwise induce moment on the model piles. The tests were conducted in compacted medium dense sand and all the piles were extensively instrumented. Test results indicate that the lateral force–displacement responses under cyclic loading exhibited slight pinching behavior due to the gap that opened at the top of the soil-pile interface. Numerical simulations show that *p*–*y* curves based on the American Petroleum Institute (API) recommendations and that proposed by Reese et al. can reasonably predict the lateral response of the piles though slightly underestimate the ultimate capacities. The general pile behavior such as force–displacement response and moment distributions along the pile depth show slight sensitivity to the subgrade reaction modulus at large displacements.

1. Introduction

Since the installation of offshore platforms in the 1950s, the effects of pile-soil interaction emerged as an important consideration in the design of structure subjected to high wind, wave action and extreme conditions such as ship impact and earthquakes. Some of the earliest representative tests of laterally loaded piles were conducted by Matlock [1] and Cox et al. [2] on clay and sand, respectively. Georgiadis et al. [3] carried out a series of centrifuge model tests on single laterally loaded stainless steel pipe piles of medium length in sand. Based on these tests, a *p*–*y* relationship was proposed. Yan and Byrne [4] presented a model study of three piles made of aluminum tubing subjected to monotonic laterally loading at the pile head. A new parabolic-shaped *p*–*y* curve for sand was proposed, and the effect of varying soil stress levels, pile diameters, loading eccentricity, and constraint conditions of pile head on the *p*–*y* curves was discussed. To investigate the seismic performance of extended pile-shafts in bridge structures, Chai and Hutchinson [5] conducted four full-scale single reinforced concrete piles with 406 mm diameter under combined axial compression and reversed cyclic lateraling loading considering three different parameters: confining steel ratio, aboveground height and relative density of sand. Kim et al. [6] performed 18 model tests of steel pipe piles subjected to monotonic lateral loading, and the recommendations for *p*–*y* curves of sand were proposed considering different pile-head restraints and different pile installation methods.

McVay et al. [7] conducted centrifuge tests on 1/45 scale single open-ended pipe piles and found that the predicted lateral force versus deflection curves using the *p*–*y* curve developed by Reese et al. [8] matched well with the measured pile response curves. Dyson and Randolph [9] performed centrifuge tests on aluminum alloy model pile and found that the fictional angle calculated from the API method [10] can underestimate the lateral load for a given displacement and can overestimate the bending moment for a given load.

In current practice, steel H-piles, with relatively small cross sectional area causing little soil displacement and minimal heave of surrounding ground have been widely used because of their ease of handling, driving and extension or reduction in length. In particular, in the United States, thousands of bridges stand on integral abutments supported by H piles [11]. However data on the response of H-piles to lateral loads is non-existent in the literature. Consequently, in this study four approximately one-third scale single steel H-piles in sand subjected to lateral loading was investigated. The response of the piles was also simulated numerically using the *p*–*y* curve proposed in API [10] as well as by Reese et al. [8] and compared to experimental results.

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2. Experimental program

2.1. Model steel H-piles

The steel H-piles used in the existing Russian River Bridge located in the San Francisco bay area were selected as the prototype piles from over 100 candidate bridges based on a series of research studies conducted by Xiao et al. [12,13]. The bridge is a three-lane single-column bent bridge, which has five single-column bents and two abutments at the ends, and was built in 1982. The H-piles from the second column bent were chosen as the prototype for the model piles. The selected column bent consists of a 5×5 pile group with length of 15.2 m. The sectional dimensions of the prototype piles are as follows: depth of 350.5 mm, width of 373.4 mm and thickness of 15.6 mm for both flange and web. The piles are embedded into a layered soil with an upper layer of depth of 3 m made up of loose, dark gray, fine to coarse sand and pebble gravel, middle layer also with depth of 3 m composed of dense, dark gray, silty, fine to coarse sand and pebble gravel sitting on gray, weathered sandstone, and the third layer with depth of 6 m having gray, extremely weathered and sheared shale with some gray sandstone, finally reaching very stable sandstone base. In order to extend the previous research of Xiao et al. [13,14] by incorporating sub-surface conditions, the model H-piles considered in this study utilized the same sectional dimensions as those adopted by Xiao et al. [14]: depth of 100 mm, width of 100 mm, thickness of 8 mm and 6 mm for flange and web, respectively, as indicated in Fig. 1. The soil condition in the prototype bridge foundation was not specifically simulated, instead was simplified with a single layer of compacted sand. The total length of the model H-pile is 3.6 m which includes the 0.8 m of pile left above the ground surface. Fleming et al. [15] proposed the equation $L_{cr} = 4\sqrt[5]{EI/N_h}$ to calculate the critical length of piles, where, E=elastic modulus of the pile; I=sectional moment of inertia; N_h=the rate of increase of modulus of horizontal subgrade reaction (which is equal to K_h/z where K_h is the modulus of horizontal subgrade reaction). For the piles used in this study $N_h = 14736$ kN/m^3 [16], and the critical length of the pile was calculated as 2.2 m and 1.8 m for piles loaded in the strong and the weak directions, respectively. Both these values were less than the embedment depth of 2.8 m, therefore, the model piles can be classified as long (or flexible) piles.

The yield strength, ultimate tensile strength and elastic modulus of the steel comprising the flange and web of the steel H-piles are 290.5 MPa, 440.4 MPa, 192 GPa and 306.7 MPa, 440.6 MPa, 194 GPa, respectively, based on tensile tests of samples taken from specimens. The following notations are used to designate the specimens: HP-MS, HP-MW, HP-CS and HP-CW wherein 'M' and 'C' refer to monotonic and cyclic loading, respectively; and 'S' and 'W' refer to bending about the strong and weak axis, respectively.

2.2. Instrumentation

The instrumentation details for the testing piles are also shown in Fig. 1. Electrical resistance strain gages with a gage length of 3 mm and a maximum range of 15,000$\mu\varepsilon$ were used to measure the flexural behavior of all H-piles at close intervals along the length. As shown in Fig. 1(c) and (d), the interval between each strain gage was 100 mm for a depth of 800 mm below the ground surface, then varied to 200 mm, 300 mm and 500 mm, respectively for the lower depths. To prevent the strain gages and earth pressure cells (EPC) from being damaged during pile driving, the four test piles were pre-installed before the placement of the sand. Further, all electrical wires were placed on lateral surfaces of the specimen to avoid any interference in the loading direction of the piles. All the electrical resistance strain gages were covered with a protection coating.

Earth pressure cells (EPC) with diameter of 30 mm and depth of 13 mm shown in Fig. 1(b) were embedded in the soil at the locations indicated in the figure to measure change in soil resistance. The EPCs were located on the middle of the web at intervals of 200 mm, 200 mm and 300 mm, respectively, below the ground line for HP-CW, while they were placed on the mid-line of the flange at intervals of 300 mm, 300 mm and 400 mm, respectively for HP-CS, as shown in Fig. 1(c) and (d). The only difference in the EPC locations between monotonic and cyclic tests was that they were placed on both sides of the specimen whereas they were installed only on the face of the piles under monotonic testing. A 10-ton hydraulic actuator fixed to the reaction beam was used to apply the horizontal load at a position 170 mm above the ground line. The imposed displacements at the loading point and also at a location 730 mm above the ground line were recorded using displacement transducers.

2.3. Construction process and soil properties

The model pile tests were conducted in a large soil pit with plan dimensions of 6 m by 4 m and 3 m in depth at the MOE Key Laboratory of Building Safety and Energy Efficiency, at Hunan University. The particle size distribution of the sand used in the tests is indicated in Fig. 2. The coefficient of uniformity was about 6.23, and the coefficient of curvature was approximately 1.03, thus, the test soil can be classified as clean, well-graded sand based on the Unified Soil Classification system [17].

The model steel H-piles were first fixed vertically on temporary shoring. The sand was then placed by vibration compaction (each layer was compacted six times) in layers of approximately 20 mm. The density of the sand in each layer was controlled by measuring the weight and water content of the sand before and the volume of each layer after compaction. During the sand placement, care was taken to ensure that the piles remained vertical. The average dry and wet density over the total depth of the soil pit was 15.7 kN/m^3 and 16.3 kN/m^3, respectively. The test pit with the piles in place during the process of sand placement is shown in Fig. 3. The layer of compacted sand below the pile tips was 200 mm thick. It should be recognized that the soil condition with the compaction procedure used in this study may be different from the true soil conditions that result from in-site pile driving methods. The tests also do not consider interaction between soil and piles as would be expected in a pile group.

The in-situ properties of the sand after placement were assessed by cone penetration tests (CPT) and shear wave velocity measurements. Five points shown in Fig. 1(a) were chosen for CPT and the results are indicated in Fig. 4. Based on results from CPT, the average tip resistance in the zone of interest was estimated as 4.5 MPa. In this study, the zone of interest equals to a depth of 1.25 m below the ground line, or 12.5 times the pile section width. Based on the method suggested by Liao and Whitman [18], the average tip resistance was multiplied by the coefficient, C_N, to consider the effect of overburden pressure. The procedure for calculating the coefficient is given by $C_N = (P_a/\sigma'_{vc})^{0.5}$, here, P_a is the reference pressure (0.1 MPa), σ'_{vc} is the effective overburden pressure, and the maximum value for C_N is 2.0. In the zone of interest, the averaged tip resistance was about 4.5 MPa, therefore the normalized tip resistance was calculated as 9 MPa. The relative density of the in-situ sand was estimated to be 51%, and the angle of internal friction, $\varnothing$, was about 38° by using the assumed normalized tip resistance, based on the data presented in Meyerhof's work [19]. In addition, the modified method proposed by Idriss and Boulanger [20] was adopted to validate the estimated values. In the modified method, $C_N = (P_a/\sigma'_{vc})^m$, where $m = 0.784 - 0.521D_r$, and