PERFORMANCE OF THREE DIFFERENT PILE FOUNDATIONS IN URAYASU CITY DURING 2011 TOHOKU EARTHQUAKE

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Abstract

Performance of three different types of pile foundation (single, 2x2 and large group pile foundations) then under construction thus without superstructures during the 2011 Tohoku earthquake was examined on the basis of field investigation including inclinometer and camera survey together with pseudo static pushover analysis. It was shown that: (1) all single and one 2x2 group pile foundations were vitally damaged at the bottom of a liquefied layer and displaced horizontally by up to 60 cm, whereas the other large group pile foundation survived without any damage; (2) the largest bending curvature occurred near the bottom of the liquefied layer in all the damaged pile foundations, confirming strong effects of ground displacement of the liquefied sand; and (3) the difference in pile moment capacity and pile head constraint might have differentiated the performance of the three different pile foundations during the earthquake.

Keywords: 2011 Tohoku earthquake, case history, pile foundation, soil liquefaction, ground displacement

1. Introduction

The Tohoku earthquake (M=9.0) of March 11, 2011, triggered liquefaction-induced damage to various structures in the reclaimed land area of Kanto region. Not only numerous buildings with spread foundations but also a few pile-supported buildings settled and/or tilted without significant damage to their superstructures [1, 2]. In the Urayasu Sports Park, three different structures (Stand Building, Camera Tower, and Illumination Towers) supported on pile foundations were then under construction thus without superstructures during the 2011 event. While Stand Building survived without any damage, Camera Tower and Illumination Towers experienced apparent permanent horizontal displacement. This leads to the question, what would have caused the difference in damage between the three structures built in the same park? To answer this question seems important, first, to identify the major factors differentiating the observed pile performance and, second, to provide a good benchmarking case history for calibrating seismic design procedures for pile foundations.

The objective of this paper is to describe the performance of the three pile foundations, based on detailed field investigation including borehole camera and inclinometer survey. Pseudo-static pushover analysis using p-y curves is then performed for the pile foundations to identify the major factors differentiating the difference in pile performance.

2. Characteristics of Structures Investigated

Fig. 1 shows a map showing the location of Urayasu Sports Park, located in land reclaimed in 1970s [3], in which three types of structure with pile foundation were under construction during the quake. Also shown in the figure is the nearest strong motion station (K-NET Urayasu) situated on Holocene deposit.

Fig. 2 shows the location of the target structures supported on piles for this study, including Stand Building, Camera Tower, and eight Illumination Towers, as well as plans of Stand Building and Camera Tower. These structures were then under construction thus without superstructures during the 2011 event. Hence, only

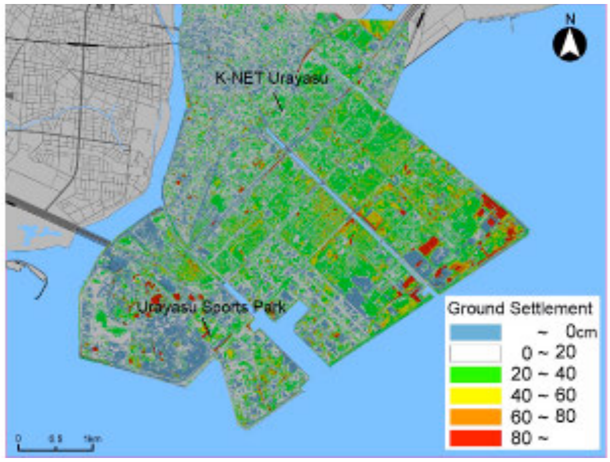

Fig. 1 - Map showing location of Urayasu Sports Park

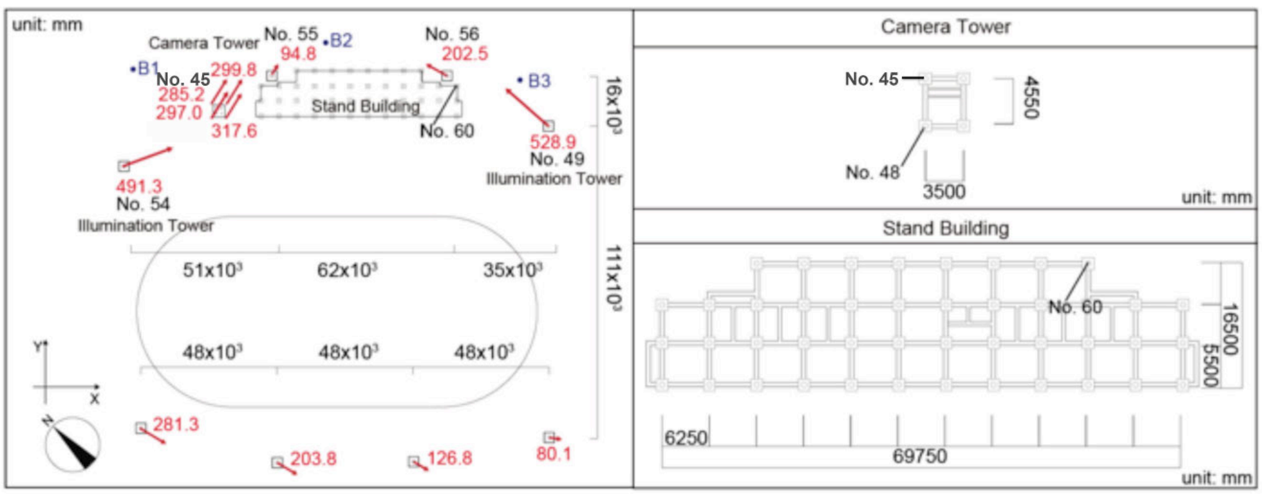

Fig. 2 - Layout and foundation plans of structures in Urayasu Sports Park and permanent horizontal displacement of piles [2, 4]

the pile caps, grade beams, slabs and some columns of the first floor were carried on piles in Stand Building and the pile caps in other structures.

Fig. 3 shows three boring logs obtained before the quake at B1-B3 (Fig. 2) in the site [2, 4]. The groundwater table was located at a depth of 1.4 m. The surface layer to a depth of about 10 m is a reclaimed fill consisting of loose sandy soils, which is underlain by layers of thick silty soil and dense sandy gravel at a depth of about 40 m.

All the piles used for the three structures were follower piles with a diameter of 600 mm, each of which consisted of four different precast piles along the length of about 48 m, as shown in Fig. 3. Stand Building was supported on 44 piles 49 m long, each of which consisted of an upper steel reinforced concrete (SC) pile 5 m long, one Prestressed High-strength Concrete (PHC Type B) piles 14 m long and two PHC Type A piles 15 m long each. Camera Tower was founded on 4 piles 48 m long, each comprising an upper PHC Type C pile 7 m long, and three PHC Type A piles 13 m long each. Each Illumination Tower was supported on one follower pile, comprising an upper SC pile, one PHC Type C pile 9m long and two PHC Type A piles 15 m long each. Type A, B and C piles had an effective prestress of 4, 8 and 10 N/mm^2, respectively.

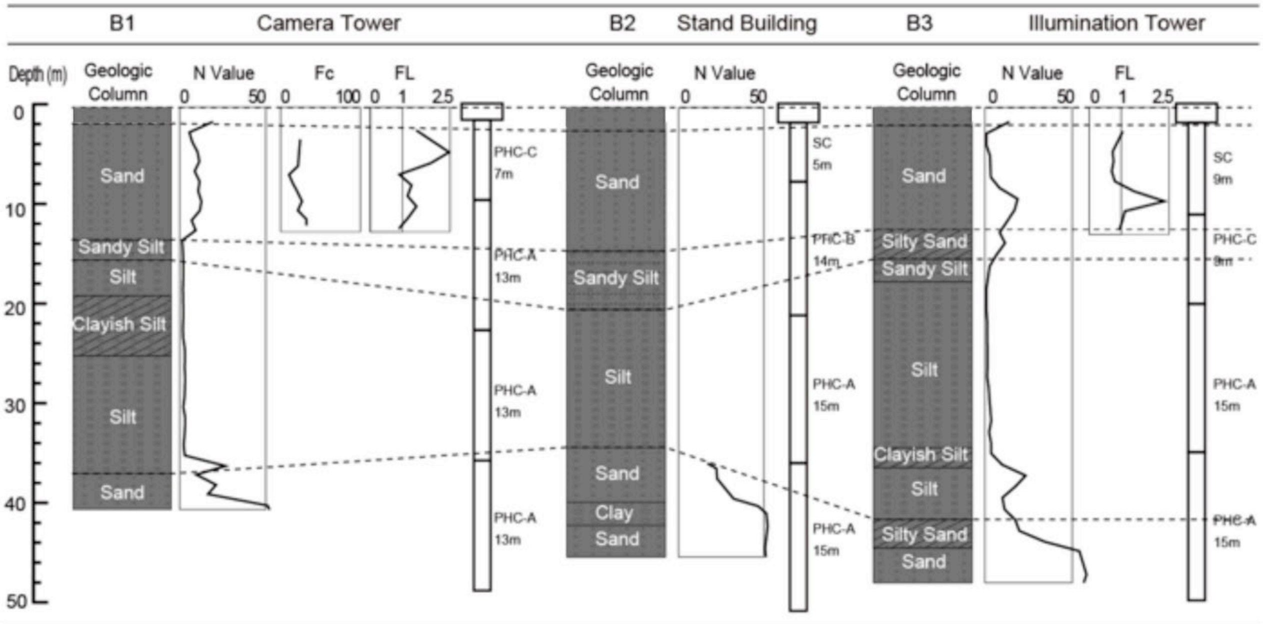

Fig. 3 - Boring logs and pile types used for three structures in Urayasu Sports Park [2, 4]

Fig. 4 shows the relationship between bending moment and curvature for the four precast piles without any vertical load, in which M_u is the bending moment at concrete crashing at extreme compression fiber of PHC and SC piles; M_y is the bending moment at yielding of tension bars of PHC piles or at yielding of steel at extreme tension fiber of SC piles; and M_c is the bending moment at concrete cracking at extreme tension fiber of PHC piles.

The pile caps of Stand Building and Illumination Towers were 1500 mm wide, 1500 mm long, and 2000 mm high, while those of the Camera Tower was 1500 mm wide, 1500 mm long, and 1300 mm high. The grade beams of Stand Building were 400 mm wide and 1500 mm high and those of Camera Tower were 450 mm wide and 1200 mm high.

3. Pile Damage Detected by Field Survey

Photo 1 shows a pile cap of Illumination Tower founded on Pile No. 54 that appeared to tilt slightly rightward (southeastward), suggesting permanent horizontal movement of the pile. Thus, a field survey was made to

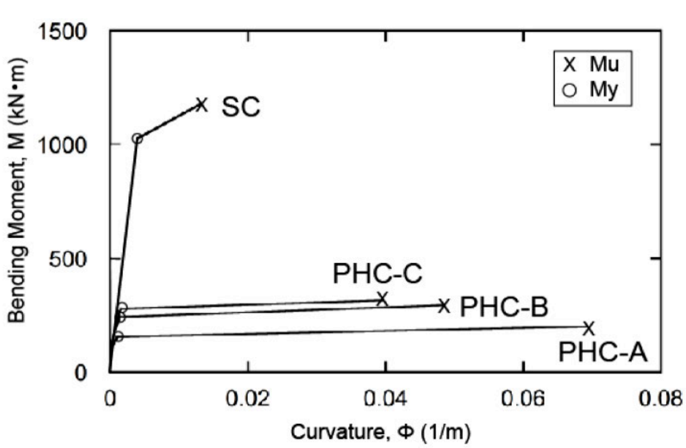

Fig. 4 - Moment-curveture relations for piles investigated

Photo 1 - Damage to Illumination tower No. 54 [2] determine horizontal displacements of each structure, assuming that the undamaged Stand Building remained as was before the quake. The result of the field survey was shown in Fig. 2 with red arrows.

Camera Tower was displaced horizontally by about 30 cm, while Illumination Towers moved up to 52 cm. It is interesting to note that the displacement of Illumination towers tends to be larger with approaching to and to occur toward Stand Building. Pile integrity test made for three piles revealed that, while Pile No. 60 of Stand building had no damage, Pile Nos. 54 and 56 of Illumination towers experienced damage at some depth in the ground.

Borehole camera observation and inclinometer survey were made by Urayasu city for Pile No. 49 of one Illumination Tower [3]. Similar investigation was subsequently made to Pile No. 45 of Camera Tower and Pile 54 of another Illumination Tower. The camera used for this observation had a high sensitivity, enabling to provide a 360-degree view of the inside wall of the pile with depth continuously. The inclinometer used was similar to one used in the previous study [5] but newly developed after the 2011 event. It can inscribe the interior wall of any precast hollow cylindrical pile, providing its slope angles in the two orthogonal directions simultaneously with depth. The measurements were made to a depth of about 20 m for Illumination Towers and a depth of about 15 m for Camera Tower.

Assuming the measured slope angle in one direction is θ, the classic beam theory leads to the following equations:

$$\theta = \frac{dy}{dz} \tag{1}$$

$$\frac{1}{\rho} = -\frac{d\theta}{dz} \tag{2}$$

in which y is the horizontal pile displacement, z is the depth, and ρ is the radius of curvature. Hence, integration of slope angle with depth with Equation (1) yields the distribution of horizontal pile displacement with depth and differentiation of slope angle with depth with Equation (2) the distribution of pile curvature ϕ ($=1/\rho$) with depth.

Figures 5-7 show the distribution of slope angle, horizontal displacement, and curvature with depth for the three piles invested. The X and Y directions in the figures correspond to those shown in Figure 2 and the depth was measured from the bottom of each pile cap. The horizontal pile displacement occurred shallower than 10 m at which some damage was detected with borehole camera observation. The detected pile curvature at the 10 m depth takes the peak value in all three piles, being consistent with the damage portion of the piles and claiming the effectiveness of inclinometer survey. The peak curvature in piles at Illumination Towers occurs immediately below the welded connection between the upper SC and lower PHC piles.

Black lines in Fig. 8 show the two-dimensional horizontal permanent displacement vectors with respect to those at a depth of either 15 or 20 m, which were detected for the three piles from the results of inclinometer survey shown in Figures 5-7. The horizontal displacement of the pile head relative to a depth of 15 to 20 m is 80-528 mm, with its direction varying from pile to pile. Also shown in the figure in broken red arrows are the vector displacements of pile head from the field survey as shown in Fig. 2, which are consistent with those obtained by inclinometer survey. The fairly good agreement in pile displacement between different tests appeared to warrant the reliability of the test results shown in Figs. 2 and 5-8.

The pile displacement at Camera Tower was less than about a half of those observed at the Illumination towers. This was probably attributed to the difference in rotational constraint at the pile head between the two structures; i.e., fixed at Camera Tower and free at Illumination Towers. This is evident by the fact that, unlike at Illumination Towers, the slope angle of pile at Camera Tower decreased with approaching to pile head as shown in Fig. 6.

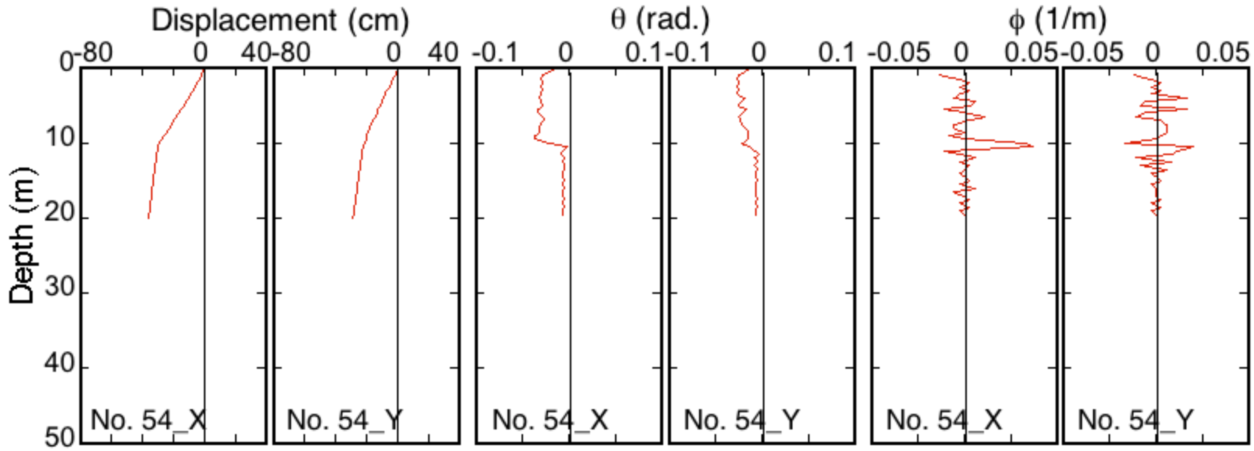

Fig. 5 - Distribution of slope angle, horizontal displacement and curveture with depth for Pile No. 54

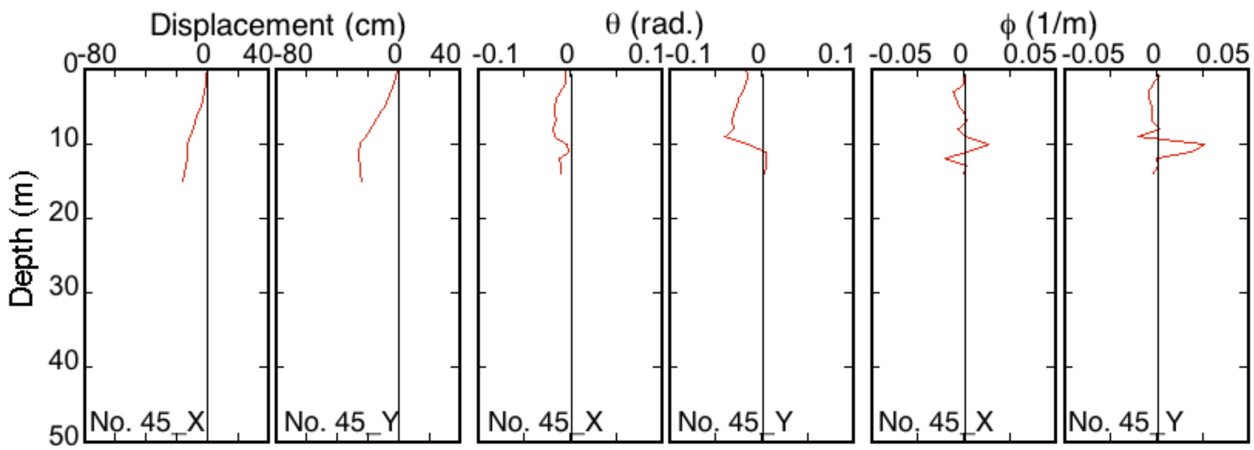

Fig. 6 - Distribution of slope angle, horizontal displacement and curveture with depth for Pile No.45

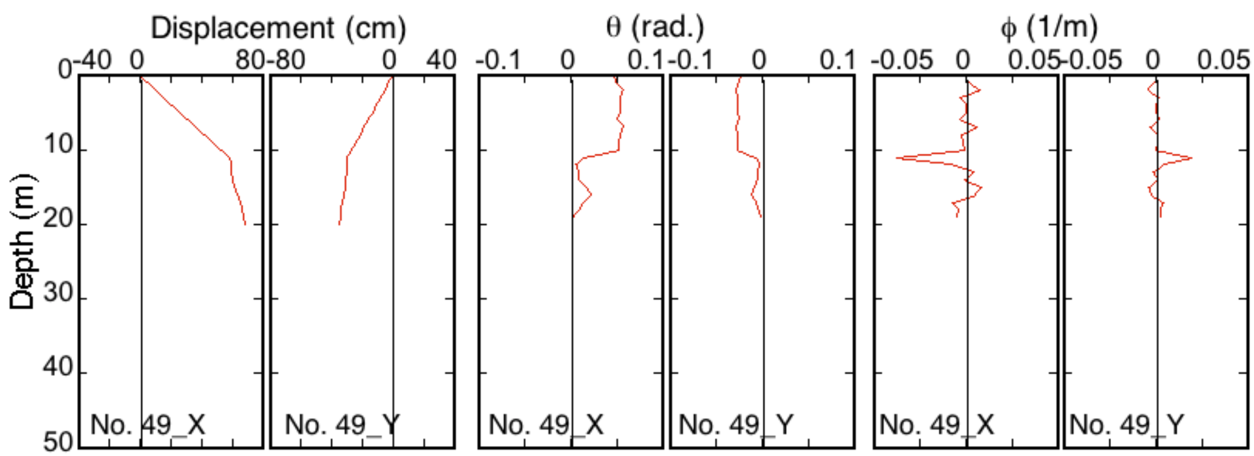

Fig. 7 - Distribution of slope angle, horizontal displacement and curveture with depth for Pile No. 49

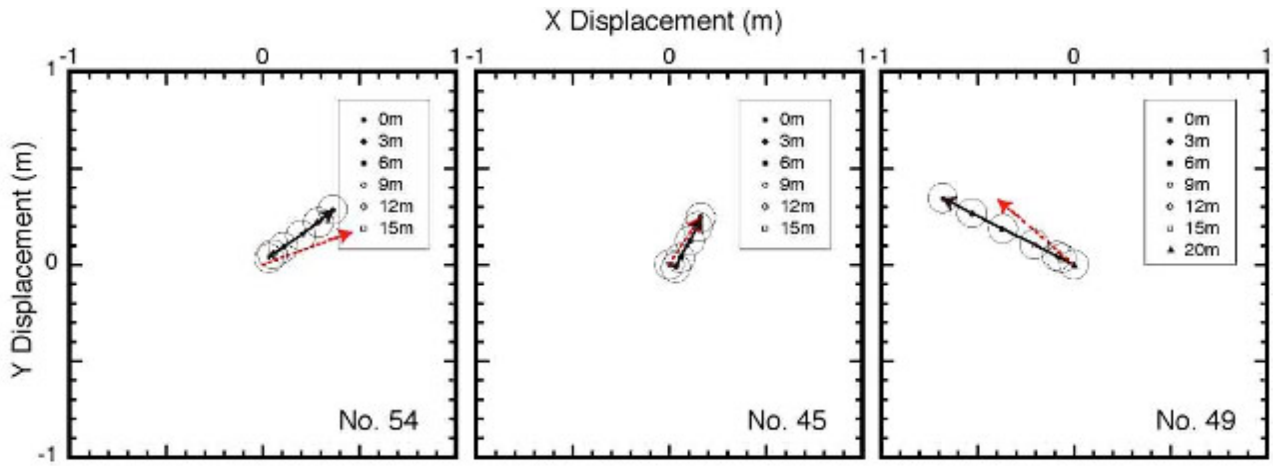

Fig. 8 - Horizontal permanent displacement vectors detected of three piles detected from inclinometer survey (black line) compared with conventional surveying (red line)

4. Pile Performance estimated from Pseudo-Static Pushover Analysis

To investing factors affecting the difference in damage between the three pile foundations, pseudo static pushover analysis was performed to estimate stresses likely developed in piles, which were then compared with the observed pile performance.

In advance of the pushover analysis, dynamic response analyses were performed to estimate ground response that might have developed at the site during the 2011 event based on the following two steps: (1) back-calculation of the outcrop bedrock motions from the ground surface motions observed at K-NET Urayasu using equivalent linear analysis and (2) estimation of site response at Urayasu Sports Park using the back-calculated bedrock motions using effective stress analysis.

Fig. 9 shows the geologic and geophysical logs at K-NET Urayasu and Urayasu Sports Park. The very dense sand layers with V_s grater than 400 m/s that occur at depths about 40 m were considered as the common bedrock for the dynamic analyses. The details of a similar analysis made for various sites in Urayasu has been described elsewhere [4, 6]. The estimated peak ground acceleration (PGA) and maximum ground surface displacement with respect to the bottom of the liquefiable layer were 1.57 m/s^2 and 0.11 m in the X direction and 1.45 m/s^2 and 0.08 m in the Y direction.

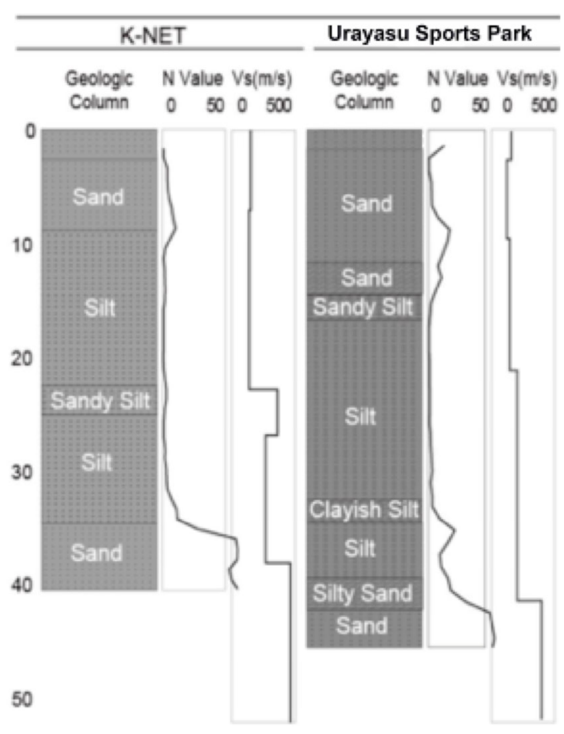

Fig. 9 - Boring and shear wave velocity logs at K-NET Urayasu and Urayasu Sports park

Beam-on-Winkler-springs method

Seismic design of foundations may be made using either dynamic response analysis or pseudo-static pushover analyses. This paper uses the later based on Beam-on-Winkler-springs method that has been incorporated with horizontal forces induced by both structural inertia and ground displacement. The detailed method including how to determine the earth pressure acting on the embedded part of the foundation and subgrade reaction has been described elsewhere [4, 7].

Fig. 10 schematically illustrates soil-pile-structure modes in the unsafe direction of the three structures. In the analytical mode, the inertial force is applied to the gravitational center of the foundation and the cyclic ground displacement to the pile through p-y springs and they were stepwise increased in the same direction simultaneously from zero to the maximum values estimated in the effective stress analysis. The change in axial pile force supporting foundation without a superstructure was very small and thus neglected.

Fig. 11 shows the distribution of bending moment with depth computed from the pushover analysis for the three foundations. Also shown in broken lines are M_c (the bending moment at concrete cracking at extreme tension fiber of PHC piles) or M_y (the bending moment at yielding of steel at extreme tension fiber of SC piles) and M_u (the bending moment at concrete crashing at extreme compression fiber of PHC and SC piles). The peak bending moments occur at both pile head and at depth of 10 m at Stand building and Camera tower with fixed rotational pile head, while it occurs only at depth of 10 m at Illumination tower with free rotational pile head. In addition, those at Camera tower and Illumination tower reach to M_u at depth of 10 m, suggesting extensive damage to piles. In contrast, it is less than M_c or M_y throughout the depth at Stand building, suggesting no damage. These estimates are fairly well consistent with the field observation including camera and inclinometer surveys. It is noteworthy that the design bending capacity of any pile was reduced significantly at the connection between the upper first and second sections, at a depth shallower than 10 m where the pile damage was detected, i.e., the bottom of the liquefied layer. This is likely because soil liquefaction, which was the major factor causing the pile damage, was not taken into consideration in the design of the pile foundations.

The difference in computed result between Stand building and Camera tower indicates that the difference

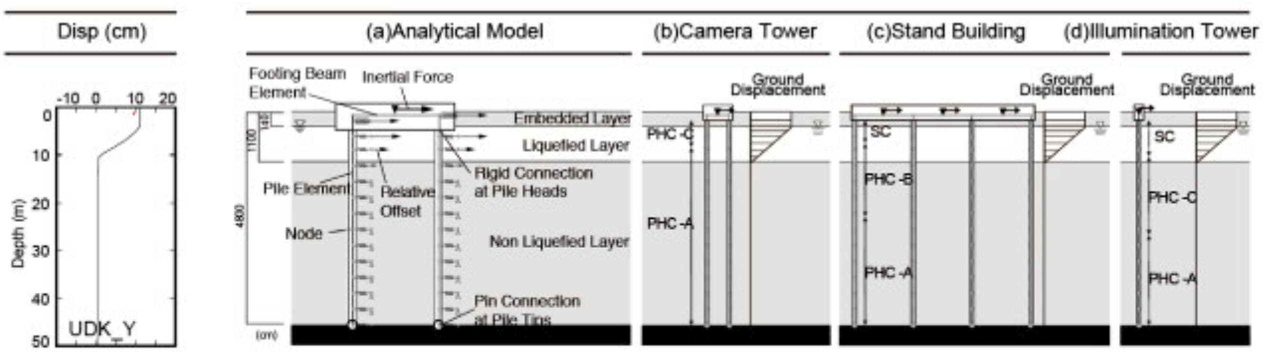

Fig. 10 - Soil-pile-foundation models used for pushover analysis

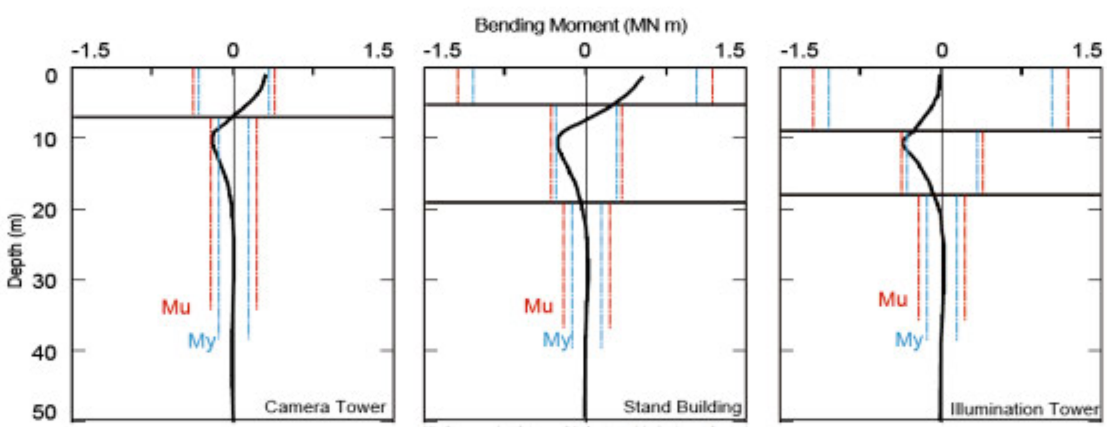

Fig. 11 - Distribution of computed bending moment with depth in unsafe direction for three pile foundations

in pile capacity at the bottom of the liquefied layer and/or the pile group effects against liquefaction-induced ground displacement might have differentiated the damage level between the two. In addition, the difference in computed result between Stand building and Illumination tower suggests strong effects of rotational constraint conditions at the pile head, i.e., fixed for the former and free for the latter.

5. Conclusions

Performance of three different pile foundations then under construction thus without superstructures during the 2011 Tohoku earthquake was examined on the basis of field investigation including inclinometer and camera survey together with pseudo static pushover analysis. The field investigation and pushover analysis lead to the following conclusions:

(1) A single and a 2x2 group pile foundations were vitally damaged at the bottom of the liquefied layer, and displaced horizontally by about 10 to 60 cm, whereas the other large group pile foundation did not suffer any damage.

(2) The largest bending curvature in the two damaged pile foundations occurred at a depth of about 10 m, suggesting the strong effects of ground displacement of the liquefied sand.

(3) The difference in pile moment capacity and pile head connection might have differentiated the performance of the three pile foundations subjected to liquefaction-induced ground displacement.

6. Acknowledgements

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7. References

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