BEHAVIOR OF PHC PILE FITTED WITH ASEISMIC SPLICE UNDER THE CYCLIC HORIZONTAL LOADING AND VARIABLE VERTICAL LOADS

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SUMMARY

The basic idea for this investigative in-situ loading test is derived from the post-earthquake investigation of the damage to piles during the 1978 Miyagiken-oki earthquake. In a building with most of the column footings having 4 piles, it was observed that one of the four piles in some of the footings remained undamaged while the remaining three had completely failed, apparently in bending shear mode [Sugimura & Ohka, 1981]. On closer examination, it was noted that the undamaged piles had been accidentally overdriven during installation, requiring a short pile segment to be added, thus resulting in a welded splice near the top. It appeared that the increase in the otherwise low ductility of the prestressed precast high strength concrete (PHC) pile body due to the existence of welded joint prevented the spliced pile from failure.

A special type of mechanical splice for PHC piles was developed [Miyasaka et al., 1996] as an alternative to the welded splice, while at the same time providing adequate ductility. To investigate the effectiveness of the mechanical splice in relation to the ordinary pile without splice, a full-scale test was designed and implemented at a typical ground condition where such piles are utilized in Japan. Cyclic horizontal loading test under varying vertical loads was conducted on a set of two PHC piles, one ordinary and the other fitted with the aseismic splice, with a footing (foundation slab) at the top. Replicating the essence of the observation during the Miyagiken-oki earthquake, the spliced pile was found to show higher ductility, resulting in the failure of the ordinary pile. The results of the test are discussed in detail and in relation to the seismic resistance of PHC piles.

INTRODUCTION

Prestressed precast high strength concrete (PHC) piles are commonly used to support building structures in Japan. However, seismic resistance of PHC piles has been a matter of concern following the typical failure patterns observed during the 1978 Miyagiken-oki earthquake [Sugimura, 1981, Sugimura & Ohka, 1981, Sugimura et al., 1984]. Of particular concern is the need to ensure adequate ductility, which is found to be emphasized quite often [Sugimura & Hirade, 1987, Kokusho et al., 1989 etc.]. Earthquake damage to pile foundation is also covered in a more recent report on safety evaluation of the apparently undamaged buildings in the Kobe area, undertaken assess the suitability for continued occupation following the Hyogoken Nambu earthquake of 17 January 1995. The instances of damage to piles identified from the inspection is reported to increase with the passage of time following the earthquake, and the instances of damage to PHC piles is reportedto be particularly prominent [AIJ-Ed., 1998]. The damage at relatively deeper part of the pile due to liquefaction and lateral spreading is reported to be a unique feature of the nature of damage observed in Kobe area. However, most of the cases of damage to PHC piles close to the top appear to have initiated in the bending shear mode resulting in the crushing of the weakened section ultimately, indicating the nature of failure observed in the Miyagiken-oki earthquake.

The nature of the failure of PHC piles during the Miyagiken-oki earhtquake as well as in the Hyogoken-Nambu earthquake, clearly indicate the urgency to consider ways of increasing the ductility of PHC piles if they are to find continued use in practice. The need to provide adequate ductility in PHC piles, particularly close to the top, has already been emphasized in the revised Japanese specification for highway bridges [JRA, 1997].

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Investigations on a building with most of the column footings having 4 piles, affected by the 1978 Miyagiken-oki earthquake, is reported by Sugimura and Ohka [1981]. It was observed that one of the four piles in some of the footings remained undamaged while the remaining three had completely failed, apparently in bending shear mode. Closer examination revealed that, in each case the undamaged pile had been accidentally overdriven during installation, requiring a short pile segment to be added, resulting in a welded splice near the top. The sketch in Figure 1 clearly shows the distinct difference in behavior of the pile with a welded splice (P2) compared to the remaining three plies without the splice (P1, P3 & P4). The sketch shows the condition of all the four piles in the footing, based on the excavation and direct inspection carried out after the earthquake. The distinct difference in behavior can also be seen in the photographs of the top portion of piles P1 and P2 given in Figure 1. It appears that the existence of the welded joint enhanced the otherwise low ductility of the pile body, thus preventing the spliced pile from failure.

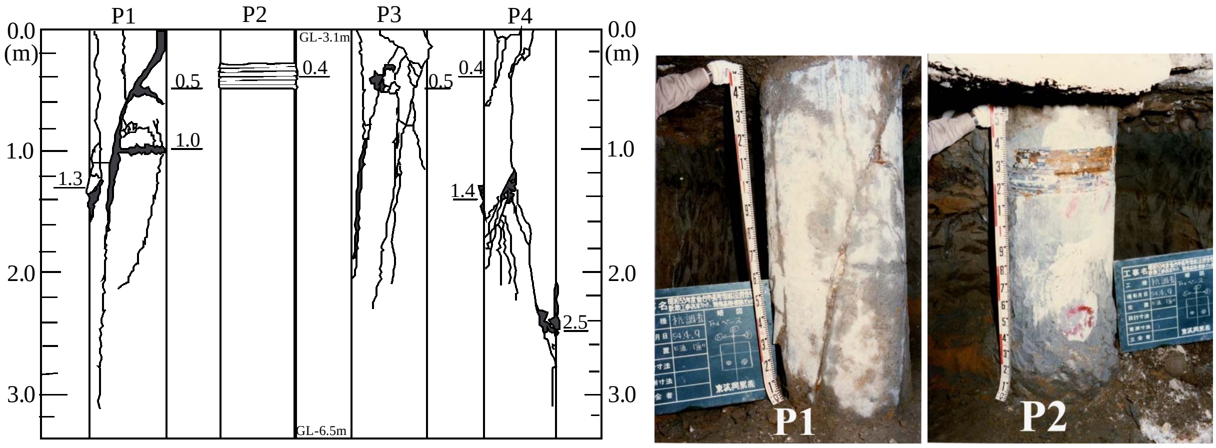


Figure 1:Difference in behavior between PHC piles with (P2) and without welded splice near the top

Based on the hint derived from the damage observation during the Miyagiken-oki earthquake, a setup for the full scale cyclic horizontal loading test on a pair of bored PHC piles, with and without the splice, was designed and successfully implemented [Sugimura et al., 1998]. Considering the long time needed for welding at the site, during which the construction equipment tend to remain idle, the mechanical pile splice is regarded as effective in increasing the construction efficiency of the spliced piles. In addition, mechanical pile splice can be designed to allow adequate joint rotation, thus making up for the deficiency in the ductility of PHC piles [Miyasaka et al., 1996]. Consequently, mechanical pile splice, rather than the welded pile splice, was utilized in the experimental investigation. Considering the adequate ductility of the mechanical splice [Miyasaka et al., 1996] utilized in the test, it is referred to as the aseismic splice in this paper, and the terms joint and splice are used interchangeably. The allowance for the joint rotation provided in the aseismic pile effectively increases the ductility of the PHC pile, in a manner different from the usual approach of increasing the lateral reinforcement for effective confinement of concrete. The full scale in-situ loading test program had the duel purpose of investigating the observation made during Miyagiken-oki earthquake and evaluating the effectiveness of the aseismic pile splice to provide enhanced and sustained ductility to PHC piles under realistic loading conditions expected during severe earthquake shaking.

The cyclic horizontal loading test undertaken and described here is rather unique in that the arrangement was devised for the application of adequate vertical load on the piles, making it possible to apply different combinations of vertical and horizontal loads. The vertical load on piles due to the superstructure, and its expected variation due to rocking motion during earthquake shaking, is not found to be accounted for in the horizontal loading test on piles and pile groups undertaken in practice. The loading test setup consists of two bored PHC piles installed at a site with soil condition generally typical for such piles in Japanese practice.

THE TEST SETUP, GROUND CONDITION AND THE MEASUREMENT SYSTEM

The experimental setup for the load application is shown in Figure 2 and the soil profile of the test site together with the layout of the set of test piles is given in Figure 3. The test piles consist of ϕ400mm PHC piles of type B (thickness t=65mm, concrete strength F_c=85MPa, Young's modulus of concrete E=40,000MPa and with 12 ϕ9.2mm prestress strands). The two piles, one with aseismic splice (designated as pile A) and the other ordinary pile without the splice (designated as pile O), are connected to a rigid base slab (3.2m long, 1.2m deep and 1.0m wide) by embedding a length of piles equal to the diameter (0.4m). Thus the system tested consists of a single span structural unit as shown in Figure 2. The center to center spacing between the piles is 2.2m (5.5 times the pile diameter). Attempt was made to place the splice as close to the base slab as possible. However, due to steel band detailing the center of the splice was actually at 25cm below the bottom of the base slab. For a simple and direct comparison of the performance of the two piles, it was decided not to locate the splice below ground. The center of the splice was at a level of GL+0.35m as shown in Figure 2.

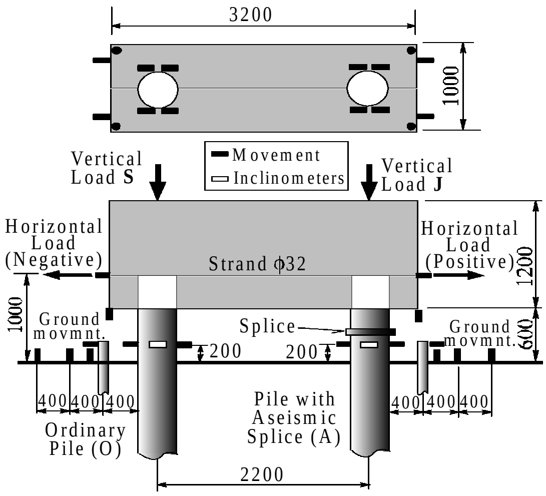


Figure 2: The outline of the loading test arrangement

The horizontal load was applied through the ϕ32mm strand embedded in the base slab at the level of GL+1.0m as shown in Figure 2. The load was applied by pulling the strand by hydraulic jacks suitably arranged on both sides of the base slab. Two hydraulic jacks located centrally above the two piles, designated as S and J in Figure 2, were used for application of the vertical load. The vertical and the horizontal jacks on both sides of the base slab were manually operated independently, thus making the load application system completely asynchronous.

Based on the results of site exploration and the soil tests, the ground condition of the test site consists of a fill material to a depth of about 1.7m, underlain by a mainly clayey soil layer to a depth of about GL-10.m. As shown in Figure 3, the standard penetration test (SPT) N-value in the fairly soft clayey layer ranges from 2 to 4 and the undrained compressive strength q_u ranges from 40 to 90 kPa. The coefficient of horizontal subgrade reaction k_h in the clayey material, estimated from the lateral loading test (LLT), was found to be about 100kPa. It can also be noted in Figure 3 that the soil condition below about GL-10m consists of alternate bedding of silty and sandy material with a large variation in N-value ranging from less than 10 to more than 40. The soil close to the pile toe region is of sandy type with N-value of about 40.

Arrangement was made to measure the movement, inclination and strain at different locations and in different directions together with the measurement of applied cyclic horizontal load (positive and negative) and vertical load (S above pile O and J above pile A). The location of the movement sensors, inclinometers and strain gages utilized to measure and evaluate the behavior of the system being tested are also indicated in Figures 2 and 3.

TESTING PROGRAM AND THE LOADING SEQUENCE

As mentioned above, a unique feature of the testing program consists of an attempt to represent the loading condition in piles during earthquakes by employing different combination of horizontal and vertical loading. The actual procedure followed is illustrated schematically in Figure 4. Different series of cyclic horizontal loading under different combination of vertical loads are depicted in Figure 4 together with the maximum values in each series. The preliminary series consists of applying the design vertical load (long term allowable bearing capacity) on both the piles. That is, the cyclic horizontal load was applied under equal vertical loads on each pile. The long term allowable vertical load R_a (also referred to as the design capacity) of the bored PHC pile for the ground condition shown in Figure 3 was estimated to be about 600kN (60t) from Equation 1 generally recommended for design practice in Japanese. The total vertical load capacity of 1200kN (120t) for the two piles forms the basis for the scheme of vertical load application depicted in Figure 4.

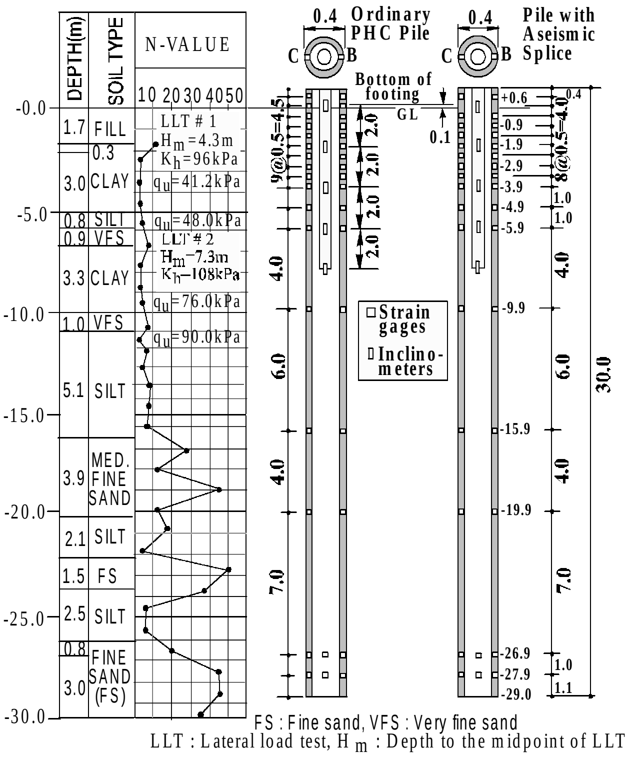


Figure 3: Profile of site condition and the test piles

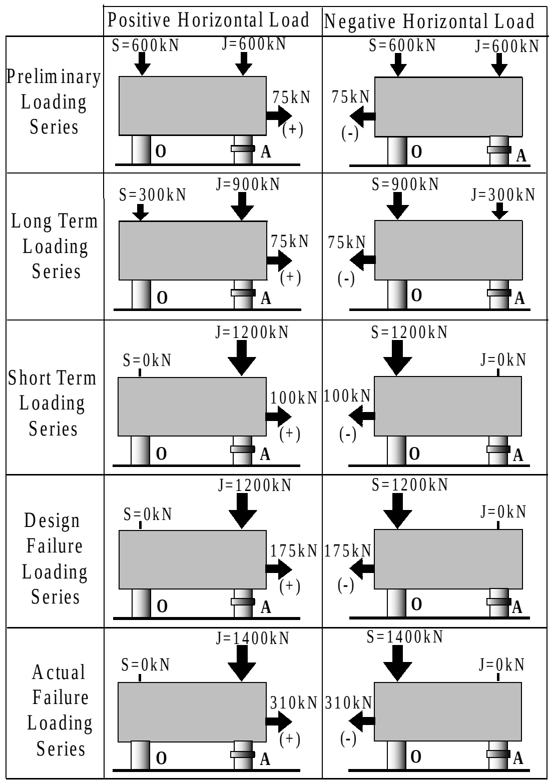
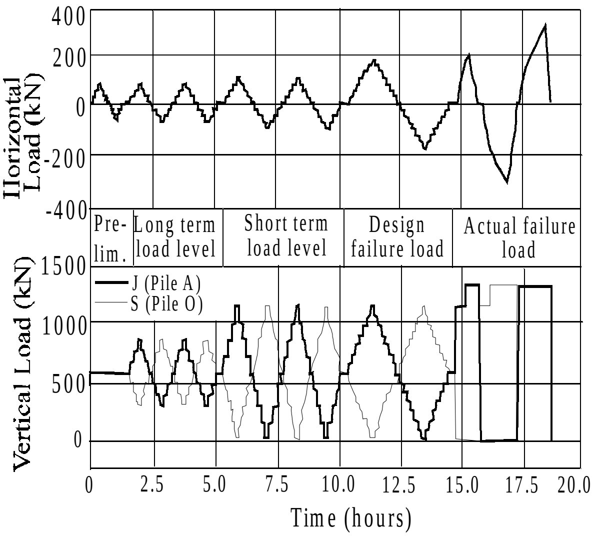


Figure 4: Loading scheme & the maximum loads

$$\boldsymbol{R_a} = \frac{1}{3}\left\{25\overline{\boldsymbol{N}}\boldsymbol{A_p} + \left(\frac{1}{5}\overline{\boldsymbol{N}}_s\boldsymbol{L_s} + \frac{1}{2}\overline{\boldsymbol{q}}_u\boldsymbol{L_c}\right)\boldsymbol{\psi}\right\} \tag{1}$$

In Equation 1, $\overline{\boldsymbol{N}}$ is average N-value within the range 1*B* below and 4*B* above the pile toe (*B* is pile diameter), $\overline{\boldsymbol{N}}_s$ & $\boldsymbol{L_s}$ are the average N-value & thickness of sandy soil layer, $\overline{\boldsymbol{q}}_u$ & $\boldsymbol{L_c}$ are the average value of undrained compressive strength and thickness of clayey layer, and $\boldsymbol{A_p}$ & $\boldsymbol{\psi}$ are the cross-sectional area & perimeter of the pile of diameter *B*. The maximum horizontal load of 75kN in the preliminary series corresponds to long term bending strength of the pile material. It is the horizontal load required to produce a moment corresponding to the long term allowable bending strength of the pile material, and was obtained from the theory of beam on elastic foundation, following the Japanese design practice [Sugimura, 1988]. One cycle of horizontal load with a maximum of 75kN was applied during the preliminary series, while two cycles of the same were applied during the long term loading series, when the vertical load on individual piles was also varied, while keeping the total to 1200kN. As shown Figure 4, the vertical load was raised to J=900kN and S=300kN when loaded in the positive horizontal direction. It was reversed to S=900kN and J=300kN when loaded in the negative direction. In principle, the virgin horizontal load was always applied in the positive direction (Figures 2 and 4) so as to avoid severity on the ordinary pile.

In the next series, considered to represent the short term loading condition as per the Japanese design practice, the maximum horizontal load was estimated to be 100kN based on the short term allowable bending strength of the pile material in a manner similar to that of the long term case. Again two cycles of load was applied. However, the maximum vertical load was raised to be twice the design capacity (J=1200kN) above pile A while keeping S=0kN, when being loaded in the positive horizontal direction. The vertical loads were reversed to S=1200kN and J=0kN in case of negative horizontal direction. This loading situation may be considered quite severe because the pile foundation is designed to maintain some compression throughout under the action of short term allowable bending moment. Consequently, the vertical loading schedule during the single cycle of the design failure series, consisting of the maximum horizontal load of 175kN corresponding to the design failure moment of the pile material, was kept same as that of the short term series. The short term bending strength of the pile material being exceeded, the testing condition in the design failure series is an attempt to depict the limiting design situation. As the system of hydraulic jacks for load application are not synchronized, the sequence of loading involved first setting the vertical load and then applying the horizontal load manually at all the steps. The actual scheme of the horizontal and vertical loads is given in Figure 5.



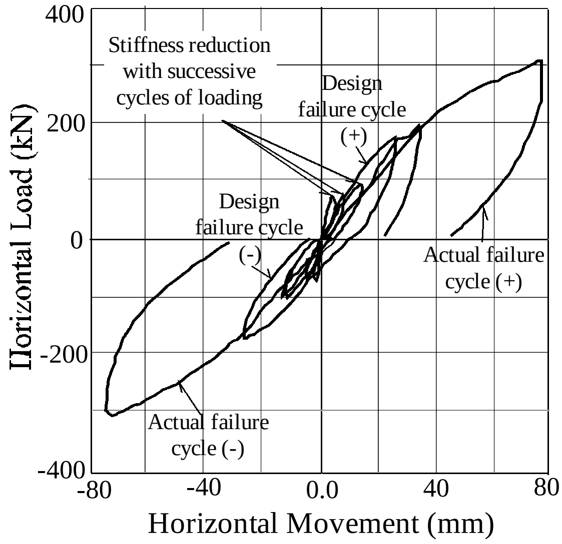


Figure 5: Steps and time schedule of loading scheme **Figure 6: Horizontal movement of the footing**

The final series involved application of load to actually fail the system tested. Attempt was made to evaluate the failure load through trial and error. First of all, maximum horizontal load of 175kN, similar to that of design failure series, was applied in positive direction while keeping J=1200kN and S=0kN right from the beginning. After that, the vertical load was increased to J=1400kN, which is still below the ultimate capacity, and the horizontal load was increased to 200kN. Actually, at this stage it would have been desirable to apply a pull out load on the tension side, but this was not possible with the loading system available. Consequently, the best that could be done was to keep S=0kN. Once the horizontal load was cycled back to zero, vertical load was set to be J=0kN and S=1400kN and the horizontal load was increased to 200kN in a manner similar to that in the positive direction. At this stage, the nonlinearity in pile O began to become conspicuous with the increase in horizontal movement. In order to fail the system being tested, the horizontal load was increased further beyond the 200kN without unloading. When the load reached 310kN level in the negative direction, the nonlinearity in pile O became very large, and it was decided to unload in view of the abnormalities that began to appear in some strain gage readings as well. The load was again applied in the positive direction with J=1400kN and S=0kN and was increased to 310kN level. At this stage, the top of pile O had substantially cracked indicating imminent failure (collapse) and the loading test was concluded.

OBSERVATIONS AND DISCUSSIONS ON TEST RESULTS

Horizontal Movement of the Footing

The cyclic horizontal load movement relation of the footing is shown in Figure 6. During the preliminary series, when equal vertical load (S=J=600kN) was applied, both the piles settled uniformly by about 3mm. Next, when the horizontal load was initially applied in the positive direction, the horizontal movement was 4.4mm, while it was only 2.5mm when the load was reversed next in the negative direction, indicating a clear imbalance in the movement. However, the movement was close to the linear range and there was no appreciable tilting in the footing. As noted above, the long term loading series differs from the preliminary series only in that the vertical load is varied, for example J=900kN and S=300kN when loaded in positive direction. Because of the unequal vertical load, the horizontal movement increased by about twofold to 8.5mm in the positive direction and to 5.2mm in the negative direction. The inclination of the footing was found to be appreciable at this stage.

In short term loading series, although the horizontal movement became comparatively large (positive direction 14.3mm, negative direction 12.8mm), practically linear behavior seemed to be still maintained. However, the lateral stiffness of the system declined with successive cycles. The inclination of the footing was slightly larger when loaded in the negative direction, and it followed to a smaller value in the positive direction. The movement of the footing became clearly nonlinear during the design failure series of loading. Also the horizontal movement in the negative direction became slightly larger (26.6mm) than that in the positive direction (25.4mm). This reversal in horizontal movement may be attributed to the lopsided inclination of the footing in the negative direction during the long term loading series, which became even worse in the design failure stage. During the actual failure series, the horizontal movement was already quite large under the large vertical load difference of 1200kN and 0kN alone, even before the horizontal load was applied. When the maximum horizontal load of 310kN was reached, the horizontal movement increased to 71.4mm in the negative direction and 76.3mm in the positive direction. However, the inclination of the footing increased rapidly during negative horizontal loading and did not recover when the horizontal load was reversed to the positive direction. Consequently, the inclination in the positive direction at the maximum horizontal load of 310kN was even smaller than that at the maximum horizontal load in the short term series, resulting in the overall residual inclination in the negative direction.

The Horizontal Movement and the Inclination of Piles

Figure 7 shows the horizontal movement of the pile near the top (at GL+0.2m). Although the movement is slightly larger in the positive horizontal load direction, towards which the virgin load was initially applied, there is practically no difference in the load movement behavior between the two piles up to the long term series. As may be noted in Figure 7, the difference between piles A and O tends to be noticeable from short term series. The movement in pile A is slightly larger than that in the pile O when loaded in the positive direction, while reverse is true when loaded in the negative direction. As it is, the two piles are connected to a rigid footing at the top, and it is no surprise that the piles A and O show similar horizontal load movement behavior close to the top.

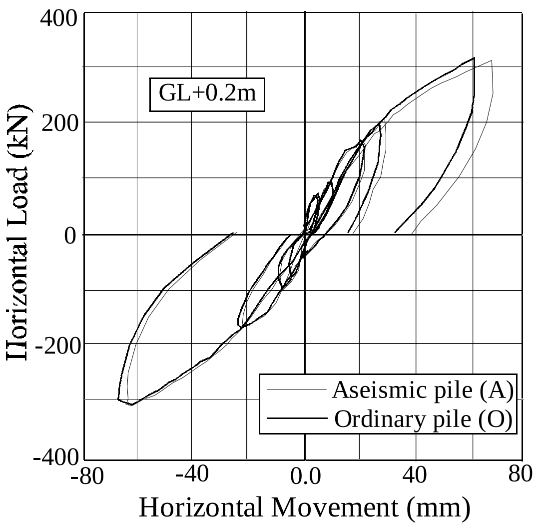


Figure 7: Horizontal movement of piles close to top

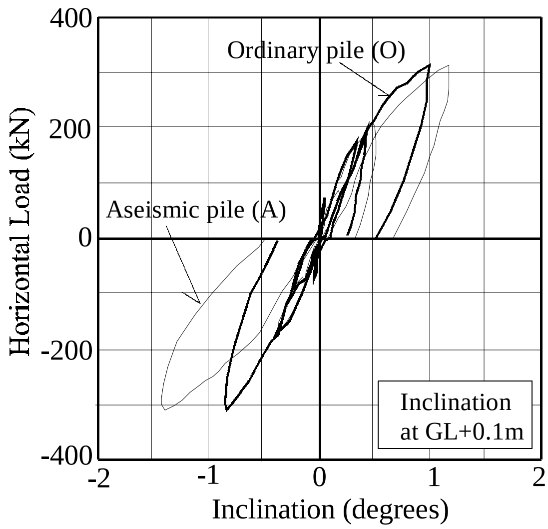


Figure 8: Inclination of piles close to the top

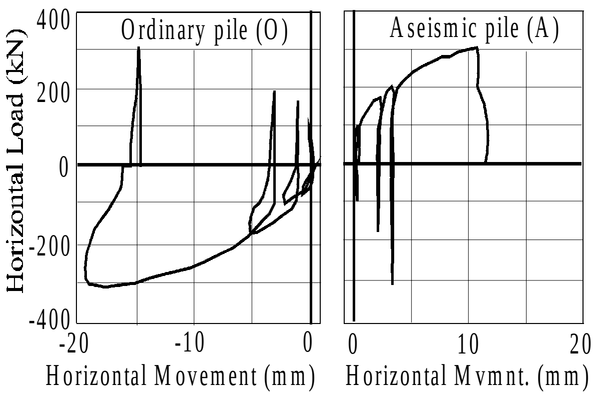
Figure 8 shows the inclination of the piles close to the top (GL+0.1m), where it is seen that the inclination of pile A is larger than that of the pile O irrespective of the loading level and the direction of horizontal load. It may also be noted in Figure 8 that the inclination in the negative direction in pile O shows the tendency to peak during the actual failure series, while it is still increasing in pile A in either direction. The peaking of inclination in pile O when loaded in the negative direction may be attributed to the increased restraint to rotation due to the very large vertical load. Owing to the presence of aseismic splice, application of similar vertical load does not seem to result in any appreciable increase in restraint against rotation in case of pile A.

The Movement of Ground Surface Adjacent to the Piles

Figure 9 shows the horizontal movement of the ground adjacent to pile O and pile A under the action of the cyclic horizontal loading. The horizontal movement adjacent to pile A occurs only when loaded in the positive direction and that on the side of pile O occurs only when loaded in the negative direction. Figure 9 also shows that most of the horizontal ground movement occurred after the short term loading series, and the movement on the side of pile O is much larger than that on the side of the pile A, particularly at the failure loading cycle. The later is most likely due to the increased restraint against rotation in case of pile O as mentioned above. In this sense, the horizontal movement of adjacent ground in Figure 9 seems to relate with the inclination of the pile top shown in Figure 8. As noted above, the inclination of pile A is larger than that of pile O, whichever way the load is applied, indicating easing of the restraint against rotation by the splice in pile A. Vertical movement of adjacent ground, due to swelling of the ground as it is pushed horizontally, was also observed. Although conspicuous close to the piles, the vertical movement due to swelling was found to decline rapidly with distance.

Condition of the System at Conclusion of the Test

The pile and footing system and the surrounding ground was closely inspected after conclusion of the loading test to failure as described above. The piles were cut off at a depth of about 90cm below the bottom face of the footing and turned over for inspection of the crack patterns. The observations made from the inspection are depicted in Figure 10. Also, the aseismic splice in pile A was unlocked and carefully inspected to confirm that there was no undue distress to it due to the loading. It was observed that the body of pile A, as well as its aseismic splice, was free from any signs of damage. This is reminiscent of the case of PHC pile with welded splice in a building affected by the 1978 Miyagiken-oki earthquake discussed above and shown in Figure 1. As can be observed in Figure 10, the failure of the system was solely contributed by pile O, which itself had developed two prominent diagonal cracks starting from the ground surface on the inside and inclined outward. These cracks may be regarded as resulting from the shear force in the pile body protruding above the ground surface due to the applied horizontal load, and are similar in nature to those observed during bending shear tests in the laboratory [Sugimura and Hirade, 1987 and Sugimura et al., 1984].



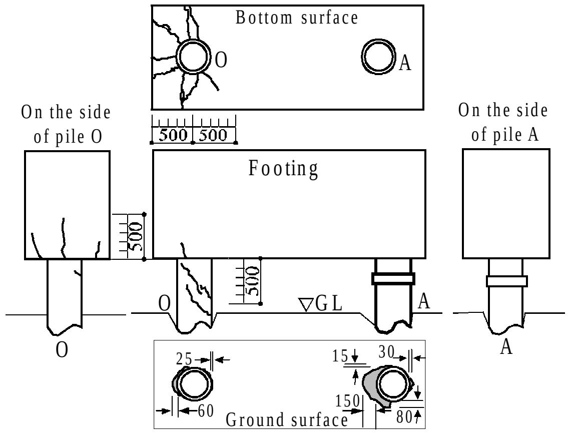


Figure 9: Horizontal ground movement adjacent to piles **Figure 10: Failure condition after the test**

In addition to the cracking in pile O, the footing had cracked quite extensively on the side of pile O, while it was totally intact on the side of pile A, indicating a very distinct difference in behavior. Three cracks on the face of footing on the side of pile O were clearly visible as depicted in Figure 10. The cracks observed on the bottom surface of the footing is sketched at the top of Figure 10, where the cracks seemed to have developed radially outward from pile O and penetrating across the depth to appear on the side, front and the back. The crack appearing on the front of the footing can also be seen in Figure 10 right above pile O.

The gap behind the pile created when it is loaded horizontally is also sketched in Figure 10. The gap with the ground behind pile A was quite wide when loaded in the positive direction, the direction of the virgin load. It appears that most of the gap around the pile was created during initial phase of the loading when the horizontal movement of the footing was behaving in a linear manner under cyclic horizontal loading as mentioned above. It was noted in connection with Figure 6 that the horizontal load movement behavior of the footing was linear while the stiffness declined with successive cycles up to the short term loading series. It is inferred that the gap with the ground behind the piles contributed to this reduction in stiffness. Figure 9 shows most of the horizontal ground movement to have occurred after short term series. Which means, most of the gap with the ground occurs before substantial movement of ground. That is, the gap between ground and pile is more reflective of the direction of initial loading cycles, whereas the extent of actual ground movement tends to reflect the restraint against rotation. Larger gap with the ground in Figure 10 but smaller ground movement in Figure 9 in case of pile A may be as a result of this phenomenon. Accurate confirmation was not possible, but most likely the narrower gap in pile O ran much deeper compared to the wider gap with ground in the pile A seen in Figure 10.

CONCLUSIONS

The full scale in-situ cyclic horizontal loading test clearly illustrate the observation made in a building foundation damaged by the 1978 Miyagiken-oki earthquake, where some of the PHC piles with welded splice near the top remained intact while others had completely failed. The field loading test and investigation undertaken in this study constitutes the first attempt at applying the cyclic horizontal load in combination with the variation in vertical load on piles to depict rocking behavior during earthquake shaking. The effectiveness of this approach is demonstrated and verified.

Increase of the vertical load on piles has the effect of increasing the restraint against rotation near the top of piles. The effect is clearly observed in the inclination measured near the top of ordinary PHC pile. However, increase in restraint against rotation is minor if any in case of PHC pile fitted with the aseismic splice, resulting in a sustained ductile behavior. Larger restraint against rotation of the ordinary PHC pile also contributes to larger horizontal movement of the ground adjacent to it as clearly observed from the loading test program.

The failure of the system of two bored PHC piles connected to a rigid footing under the action of cyclic horizontal loading in combination with variable vertical loads is solely contributed by the failure of the ordinary PHC pile, other PHC pile with aseismic splice remaining completely intact. The ordinary PHC pile developed diagonal cracks similar to those observed in the bending shear test in the laboratory and to those observed in PHC pile damaged by earthquake. The footing also failed on the side of the ordinary PHC pile while other side remained completely sound.

Large gaps with the ground behind the piles were found to develop due to horizontal loading. Such gaps seem to contribute to the decline in the lateral stiffness of the footing in successive cycles of loading, even though the horizontal load movement behavior was practically linear up to the short term loading series. It was also noted that the gap with the ground behind the piles is more reflective of the direction of initial loading cycles, where as the extent of actual ground movement is more reflective of the restraint against rotation of the pile body.

The aseismic splice serves as an effective means of enhancing the otherwise low ductility of PHC piles, and the enhanced ductility is sustained even under severe loading conditions. This is achieved through easing of the restraint against rotation by the splice, thus increasing the overall lateral movement performance of the PHC pile.

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