SEISMIC PERFORMANCE OF PILE-SUPPORTED SLAB WHRAF DECK

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SUMMARY

This study conducted seismic testing on a full-scale precast concrete pile to slab connection considering realistic loading and boundary conditions. This test program introduced several seismic loading protocols determined from nonlinear dynamic analyses of a typical container port structure in U.S. subjected to seismic ground motions with various seismic risk levels. In addition, conventional reversed cyclic loading tests were also conducted on a pile-deck slab connection damaged by seismic loading protocols before and after retrofit. The specimen showed excellent seismic performances in terms of strengths and ductility. It was confirmed that the lateral drift response of the PC pile-wharf connection can be mainly dominated by a rocking mechanism, and also spalling in the pile and deck near the joint region appeared as a main cause of stiffness degradation of the connection. As a quick and fast retrofitting method, section enlargement technique was introduced to damaged pile-slab connection, and the retrofitted pile-slab connection with seismic damage showed higher stiffness and improved capacity, but its deformation capacity and ductility were inevitably limited due to previously accumulated damage in the precast pile, where no sectional strengthening was applied.

Keywords: *precast, wharf slab, pile, seismic performance, strengthening.*

INTRODUCTION

Container port is a critical infrastructure component to the world economy. They are utilized as the beginning and ending points for the exporting and importing of goods, energy and etc., and container ports employ millions of workers and support local and global economic growth. However, as presented in Foltz et al. (2022), many container ports in the United States are unfortunately located in high seismicity regions, and thus they are vulnerable to potentially devastating physical and economic damage. This serves to emphasize the importance for container ports not only to avoid collapse, but also to maintain operational performance after a large seismic event. As shown in Fig. 1, the structural system most commonly used in container ports in the United States is composed of vertical precast prestressed concrete (PC) piles with moment-resisting connections to the cast-in-place reinforced concrete (RC) wharf slab. To better understand the capacity, ductility, and damage evolution of the common types of pile-wharf connections and to explore possible alternative connection details, previous researchers have conducted several experimental campaigns (Joen 1988; Silva et al. 1997; Sritharan and Priestley 1998; Harries and Petrou 2001; Xiao 2003; Roeder et al. 2005; Krier 2006; Jellin 2008; Stringer 2010; Lehman et al. 2013). In particular, comprehensive experimental efforts have been made by the University of Washington group for the last decade (Roeder et al. 2005; Jellin 2008; Stringer 2010; Lehman et al. 2013). The investigations included the evaluations of the effects of bearing pads, debonded dowel bars at the pile-deck interface, and a foam wrap around the embedded pile length on the structural behavior of T-headed dowel bar connections in the precast pile-wharf deck system. It was observed that all the details provided favorable results. According to the existing studies and experiences, it can be confirmed that the seismic detail using the headed dowel bars from the vertical pile anchored into the RC wharf deck can be a simple, practical, and excellent design alternative to achieve the sufficient seismic performances of the PC pile-wharf connection. Therefore, this study adopted the T-headed dowel bar connection, and the sophisticated tests on a full-scale PC pile-wharf connection specimen with more realistic loading and boundary conditions were conducted by utilizing the multi-axial full-scale sub-structured testing and simulations (MUST-SIM, Elnashai et al. 2004) facility at the Newmark Structural Engineering Laboratory (NSEL) of the University of Illinois at Urbana-Champaign (UIUC). In addition, as quick and simple retrofitting method for PC pile-wharf connection with T-headed dowel bar was also presented and applied to the specimen damaged by severe earthquake loading, and its strengthening effect was also explored by testing.

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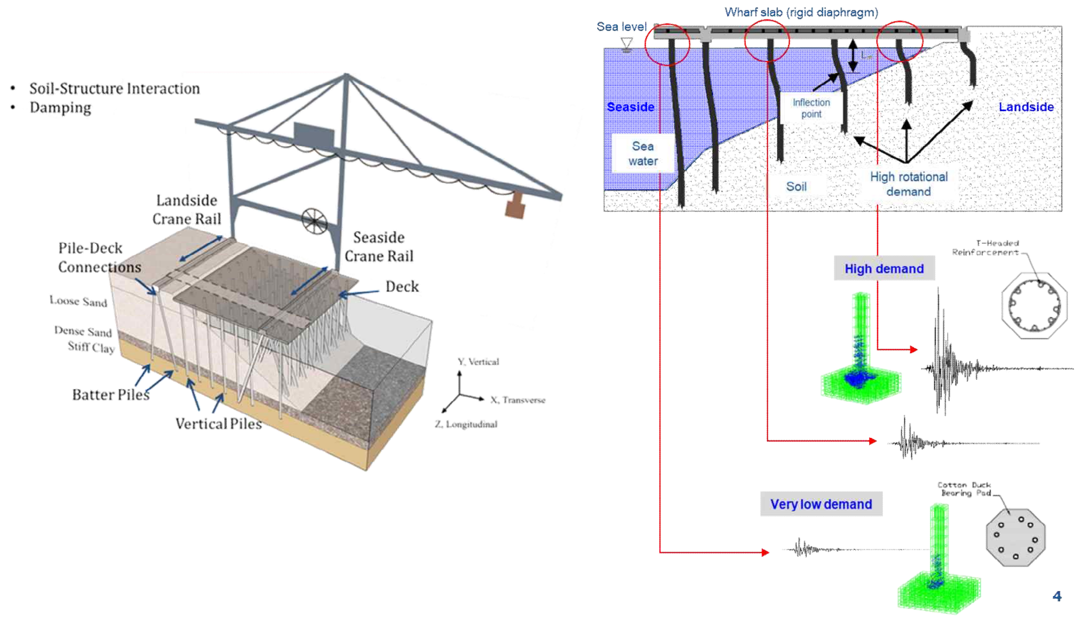

Fig. 1 – Typical container port system

EXPERIMENTAL PROGRAM

The experimental plan is composed of three phases: small-scale testing of a 1/5-scale rubber specimen, seismic loading tests on a full-scale pile-wharf connection for the three seismic hazard levels (i.e., three target return periods: 2%, 10%, and 50% probability of exceedance in 50 years), and typical cyclic loading tests on the damaged full-scale pile-wharf connection before and after retrofitting. The objective of the small-scale testing on rubber specimen was to validate the key aspects of test control parameters on the proposed complex loading protocols as a pilot testing before full-scale experiments, to which the detailed explanations can be found in authors' previous studies (Foltz et al. 2022). The large-scale seismic tests were aimed to simulate the realistic boundary conditions that a pile may experience during a seismic event. In the existing studies, simple testing set-up (e.g., cyclic loading testing as presented in ACI T1.1-01, 2001) was usually adopted without considerations on actual boundary and loading conditions of the precast pile interaction with soil. Unlike the existing studies, this investigation adopted a loading and boundary condition box (LBCB), as shown in Fig. 2, which is a hallmark of the MUST-SIM facility at UIUC. The LBCB utilizes six actuators in an arrangement enabling complete six degree-of-freedom control at the end of the pile, and it allows for precise control of both forces and displacements in each degree of freedom, thereby permitting the application of equivalent loads and displacements, such as an axial force, lateral displacements, and rotation profiles of the PC pile during an actual earthquake event, as estimated from a realistic computational model of a full-scale port structure. The reversed cyclic loading tests on the damaged pile-wharf connection before and after retrofitting were also conducted to examine to overall capacity of the connection and to evaluate the effectiveness of the repair technique for restoring structural integrity after an earthquake event.

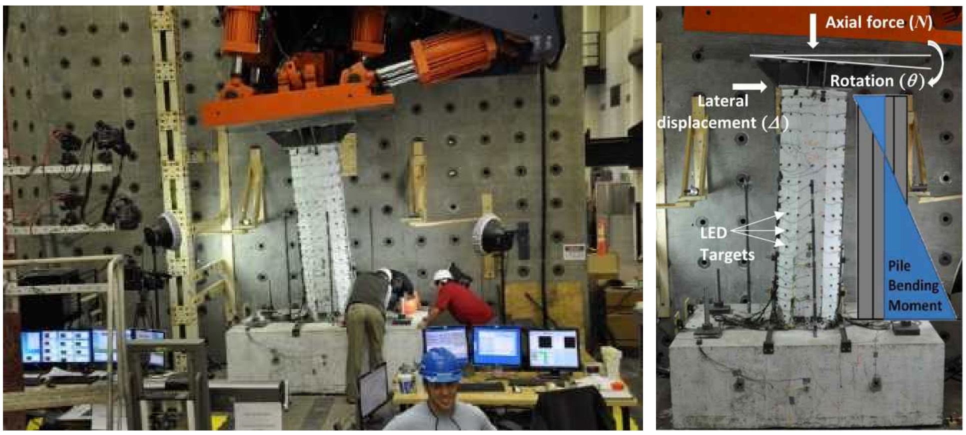

Fig. 2 – Test set-up

A full-scale precast prestressed (PC) pile-wharf connection specimen was fabricated in this study, which consisted of a 24 in. (610 mm) octagonal-shaped precast prestressed pile connected to a cast-in-place (CIP) reinforced concrete wharf-deck slab. Fig. 3 shows dimensional details of the test specimen. All the material properties were presented in Table 1, and the average compressive strength of concrete used in the PC pile was measured at 9.6 ksi (66 MPa), which was somewhat greater than the design strength. The PC pile was reinforced with twenty-two 0.5 in. (12.5 mm) diameter, 270 ksi (1860 MPa) low-relaxation strands, with each strand prestressed to 31 kips (138 kN), which was 75 % of the tensile strength of the prestressing strand. The W11 (0.374 in. / 9.5 mm diameter) smooth wire was used as spiral reinforcements. Spiral pitch varied from 1 in. (25 mm) at the end of the pile to 3 in. (75 mm) along the middle of the pile. As shown in Fig. 3, the PC pile was embedded 2 in. (50 mm) into the cast-in-place RC wharf deck as done in the current design and construction practices, and it shows further details of the test specimen fabrication process. A moment connection was finally achieved by grouting the embedded dowel bars into the corrugated ducts prefabricated in the PC pile, which means the base moment capacity of the specimen is fully dependent on those headed bars. The embedded longitudinal steel bars consisted of eight #10 T-headed bars (Grade 60 in ASTM A706). The T-headed dowel bars were grouted 62 in. (1575 mm) into the precast PC pile, and they were embedded 22 in. (560 mm) into the deck to secure enough anchorage performance and development length. The wharf deck dimensions were 96 in. (2440 mm), 66 in. (1676 mm), and 30 in. (762 mm) in width, length, and height, respectively. The wharf deck slab was reinforced with #4 (D13), #7 (D22), #8 (D25) and #9 (D29) reinforcing bars, and the mechanical properties of those reinforcements were also presented in Table 1. The reinforcement layout and deck depth were representative of those used in the current field practices. The other deck dimensions were chosen as large as possible, given the physical constraints of the laboratory. In typical container port structures, since the PC pile spacing often varies from roughly 16 ft. (4876.8 mm) to 22 ft. (6705.6 mm), the deck dimensions used for the specimen represented approximately one-third of the tributary deck area in the loading direction and one-fourth of the tributary deck area in the orthogonal direction.

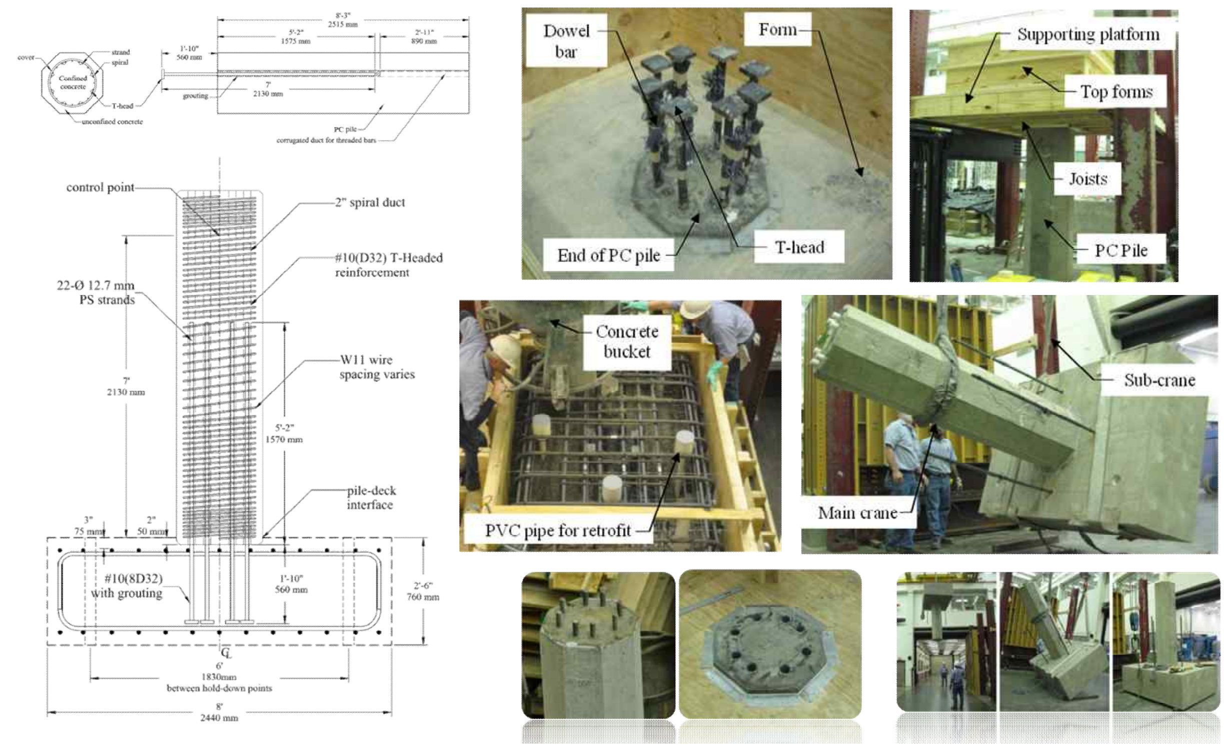

Fig. 3 – Dimensional details and fabrication process of specimen

Table 1 – Mechanical properties of materials

<table>
<tr><td colspan="4">f_c' for Deck, ksi (MPa)</td><td colspan="4">f_c' for Pile, ksi (MPa)</td><td colspan="4">Grout, ksi (MPa)</td></tr>
<tr><td colspan="2">Design</td><td colspan="2">Actual</td><td colspan="2">Design</td><td colspan="2">Actual*</td><td colspan="2">Headed bar**</td><td colspan="2">Threaded rod†</td></tr>
<tr><td colspan="2">5 (34.5)</td><td colspan="2">7.4 (51.3)</td><td colspan="2">8 (55.2)</td><td colspan="2">9.6 (65.9)</td><td colspan="2">7.8 (53.9)</td><td colspan="2">7.6 (52.5)</td></tr>
<tr><td colspan="12">Reinforcement used in PC pile</td></tr>
<tr><td colspan="4">f_y for Grade 60, ksi (MPa)**</td><td colspan="6">Smooth spiral, ksi (MPa) ††</td><td colspan="2">Strand‡</td></tr>
<tr><td colspan="2">Design</td><td colspan="2">Actual</td><td colspan="2">Area, in^2 (mm^2)</td><td colspan="2">f_y, ksi (MPa)</td><td colspan="2">f_u , ksi (MPa)</td><td colspan="2">f_{pu}, ksi (MPa)</td></tr>
<tr><td colspan="2">60 (420)</td><td colspan="2">68.1 (469.7)</td><td colspan="2">0.1093 (70)</td><td colspan="2">65 (448)</td><td colspan="2">121 (834)</td><td colspan="2">270 (1860)</td></tr>
<tr><td colspan="8">Reinforcement used in wharf deck, f_y, ksi (MPa)§</td><td colspan="4">Rod, f_y, ksi (MPa)§§</td></tr>
<tr><td colspan="2">#9</td><td colspan="2">#8</td><td colspan="2">#7</td><td colspan="2">#4</td><td colspan="4">#9 (Threaded)</td></tr>
<tr><td colspan="2">89.3 (616.1)</td><td colspan="2">83.1 (573.1)</td><td colspan="2">75.0 (517.2)</td><td colspan="2">70.2 (484.1)</td><td colspan="4">60 (413)</td></tr>
</table>

* The strength measured at 28 days after casting, ** #10 and grout used for headed reinforcing bar, † Grout for threaded rods to connect pile specimen end to LBCB, †† Measured values
‡ Specified strength, § Measured values, §§ Specified yield strength for steel rod used in retrofitting
Notes: 1 mm = 0.039 in.; 1 MPa = 0.145 ksi

EXPERIMENTAL RESULTS

The Imperial Valley record (LA44) was selected for the lowest seismic hazard level (50% probability of exceedance in 50 years); Northridge (LA18) was selected for the 10% in 50 year return period; and Kobe (LA2) was selected for the highest seismic hazard level (2% in 50 years). Using Response-2000 (R2K), it was estimated that model displacements from the Imperial Valley record would cause cracking, but no yielding in T-headed dowel bars; the Northridge record would cause yielding of the dowel bars; and the Kobe earthquake would bring the specimen near to its failure state. Nonlinear time history analysis results with the ground motions identified a nearly linear relationship between pile lateral displacement and rotation at 84 in. from the pile-to-wharf joint, which was selected as a control point about which all rotations, displacements, and forces were applied during testing (after being discretized into appropriately small steps).

During Imperial Valley testing, no pronounced damage was observed (just limited cracking, as expected). It was still valuable for characterizing elastic stiffness of the pile-wharf connection (to a maximum lateral displacement of 0.13 in.) and understanding the deformation mechanism of the specimen. Over 70% of the total rotation at the pile-wharf joint was attributable to rigid-body rocking behavior. The maximum lateral displacement during application of the Northridge earthquake record was 1.69 in., which roughly corresponds to 2% drift (see Fig. 4). During this largest displacement excursion, the extreme dowel bars experienced first yielding, initial spalling of the pile face was observed, and flexural cracks opened at several discrete locations along the height of the pile. Crack opening was also seen at the pile-wharf interface, flexural cracks in the wharf deck were visible, and the amount of total joint rotation attributed to rigid-body rocking behavior increased to nearly 80%. This satisfies the life safety performance level according to ASCE 41-17, where 2% and 1% transient and permanent drift ratios and minor spalling and cracks are allowed. During Kobe testing, the maximum lateral displacement was 3.54 in. (above 4% drift, per Fig. 4). This was accompanied by further opening of existing cracks, along with additional pile and wharf deck spalling; about 85% of total joint rotation was by now from rigid-body rocking. Fig. 2 also shows the normal strain contour and deformed shape of the test specimen at peak displacement of the Kobe record. Based on observed damage and permanent drift levels, seismic performance of the test specimen is satisfactory for the collapse prevention level per ASCE 41-17.

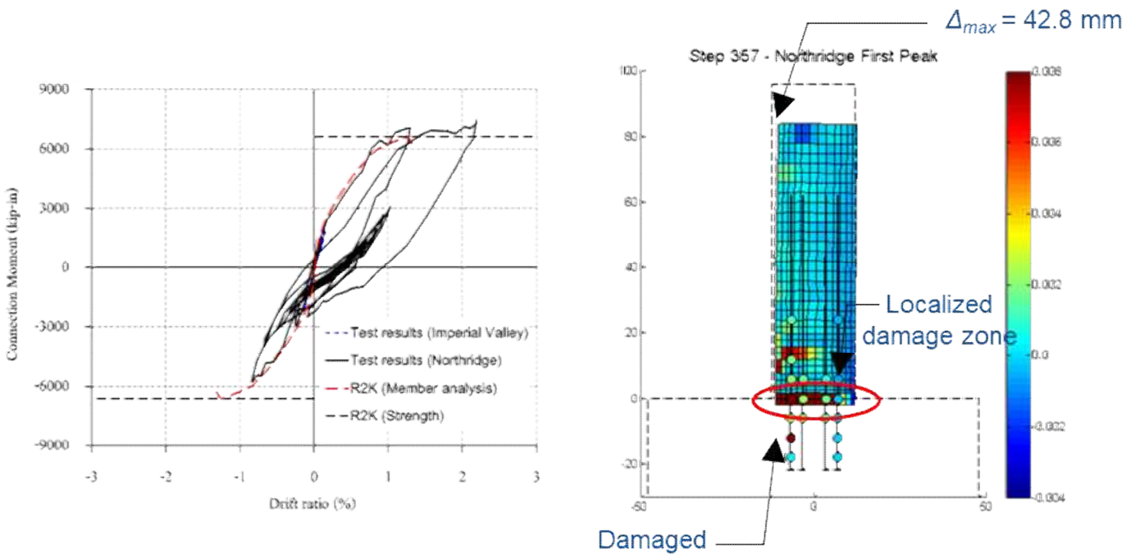

Fig. 4 – Test results of earthquake records

The damaged concrete cover of the plastic hinge region in the pile-wharf connection was removed after the earthquake record and additional cyclic load testing, until the pile hoop reinforcement and longitudinal prestressing strands were exposed. Two diameters (33 in. and 46 in.) of #4 circular hoop reinforcement were positioned, with a pitch of 2 in., surrounding the existing damaged pile, and cast-in-place concrete was placed into a form in order to create a 35 in. high by 53 in. diameter circular concrete section, as shown in Fig. 5, which also contained four #9 (pre-installed) longitudinal reinforcing bars around the PC pile. Compressive strength of concrete used for strengthening was 6000 psi. Cyclic loading was applied to the strengthened specimen starting from 0.5% drift ratio, with a 1% drift increment; measured cyclic loading behavior of the strengthened specimen is also shown in Fig. 3. No stiffness degradation in the cyclic response was observed up to 1% drift, and the capacity was increased by nearly 50% (or more) in the two loading directions. However, an unexpected failure occurred due to severe damage at an existing pile flexural crack, away from the retrofit, that developed before retrofitting. The flexural strength of the retrofitted section is estimated at 16,000 kip·in. (from R2K), which affirms that the specimen did not fail in the retrofitted section due to bending. At the ultimate condition, macro cracks appeared in the existing pile section with no strengthening, in part because section enlargement was only introduced up to 35 in. from the interface between the pile and wharf deck. This was however mainly due to accumulated damage from the previous severe seismic and cyclic loading tests on the test specimen. Nevertheless, the section enlargement method applied in this study showed ample strengthening effects, and it could be used for a damaged pile-wharf deck structure as a cost-effective and rapid strengthening approach. In addition, the observations reveal that proper repairing may also be necessary for other regions outside the plastic hinge to ensure adequate deformation capacity without premature failure.

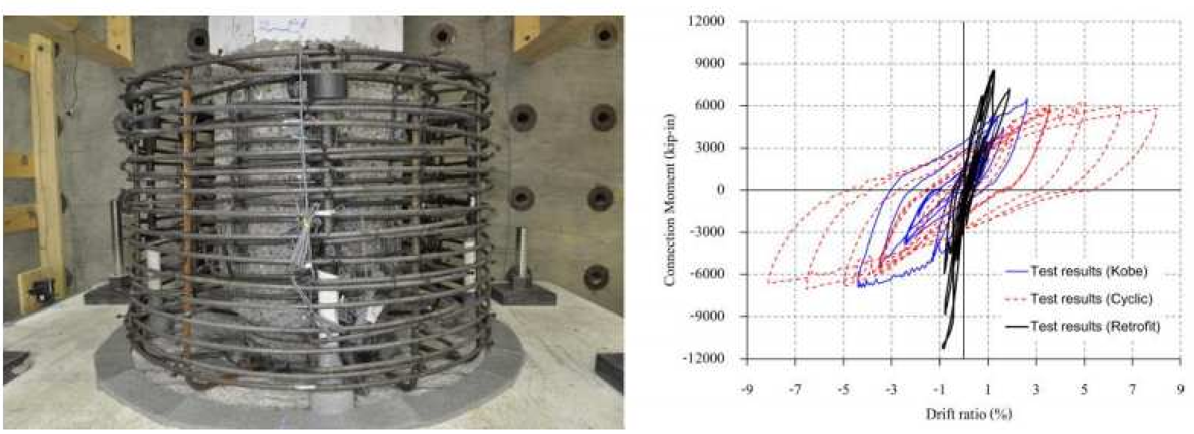

Fig. 5 – Test results of test specimen after retrofitting

CONCLUSIONS

Full-scale pile-wharf connection tests with realistic boundary conditions were conducted in this study, and the realistic loading protocols were developed from the results of a nonlinear port system model. In addition, the conventional reversed cyclic loading tests were also conducted on a pile-wharf connection damaged by seismic loading before and after retrofit. On this basis, following conclusions can be drawn:

1. The primary deformation mechanism of the precast prestressed pile-wharf deck connection is the rigid-body rocking rotation at the pile-wharf connection and not through flexural action of the pile.
2. At lower loads, conventional flexural rotation of the pile accounts for about 30% of the total rotation of the pile, with 70% being attributed to the rocking mechanism, and the rigid-body rocking rotation accounts for as much as 90% of the total rotation after significant damage occurs in the pile-wharf connection.
3. Pile spalling and deck spalling lead to a decrease in the capacity of the pile-wharf connection; however, if the confined concrete core remains intact, the pile-wharf connection can accommodate large lateral deformations with only a modest reduction of capacity.
4. The measuring system has proven to be very useful to obtain reliable data once the traditional instrumentation begins to fail during the test, as well as for the visualization of the overall state of the specimen throughout testing, rather than just localized readings.
5. The section enlargement method was introduced to strengthen the damaged pile-wharf deck structure, and it was clearly confirmed from the reversed cyclic test that the stiffness and strength of the damaged specimen can be significantly enhanced, however, careful repairing should be done to un-retrofitted region in existing damaged pile to prevent premature failure.

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