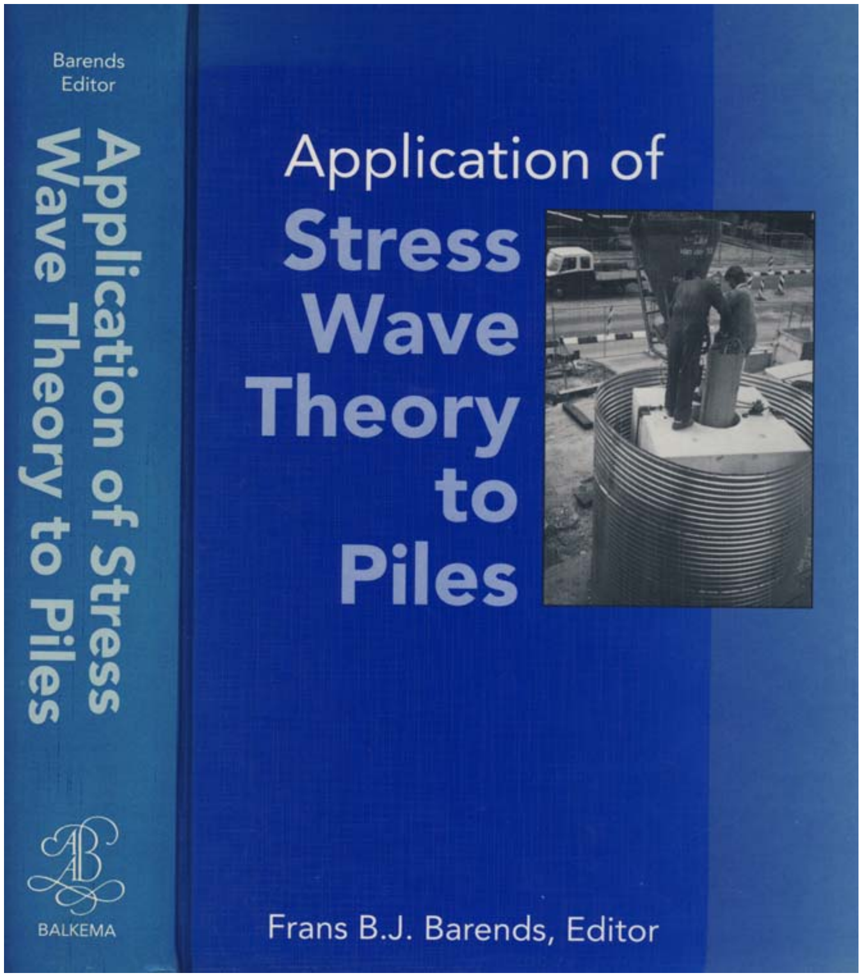

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A case history of prestressed concrete pile splice problems

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ABSTRACT: Many problems were encountered with the mechanical wedge-type splices used for 380 mm square precast prestressed concrete piles installed at an industrial plant site. Many of the wedges were breaking during installation of the splices and after pile driving resumed following splicing. Dynamic monitoring of the piles and uplift loading tests confirmed that the problems were associated with cracked or damaged wedges. More ductile, annealed wedges were subsequently used and, while not completely satisfactory, they were found to perform better than the original more brittle wedges.

1 INTRODUCTION

Long precast prestressed concrete piles were installed to support heavy, settlement-sensitive structures at an industrial plant in Port Alberni, British Columbia, Canada (Fig. 1). The piles were driven through soft compressible silts and clays to toe bearing in a dense sand and gravel deposit or on bedrock. Each pile consisted of two or three segments joined with mechanical wedge-type splices. Shortly after construction began, problems arose during pile installation, and these problems were, later, identified to be related to the wedges, or clamps, used to connect the splice plates.

This paper describes the problems encountered with the splice during pile installation and summarizes the dynamic monitoring and static tension loading tests conducted to identify the source of the problems and to aid in the resolution of the splice problems.

2 SOIL CONDITIONS

The piles were installed at two sites, Site A and Site B, within the existing plant site. Fig. 2 shows the soil profiles at the two sites. The distinctly different soil conditions at the two sites, which are only 400 m apart, are related to the complex sedimentary history of the valley where the plant is located.

The site is located in an old fjord scoured by glacial ice during the Pleistocene Ice Age. The sediments underlying the site consist of remnant glacial deposits and marine sediments deposited after the retreat of the glaciers. The depositional history of the area during post-glacial times has been dominated by successive periods of erosion and sedimentation resulting from changes in sea level. These processes have concentrated the glacial and marine sediments into two distinct areas: shallow

Fig. 1 Site location, British Columbia, Canada

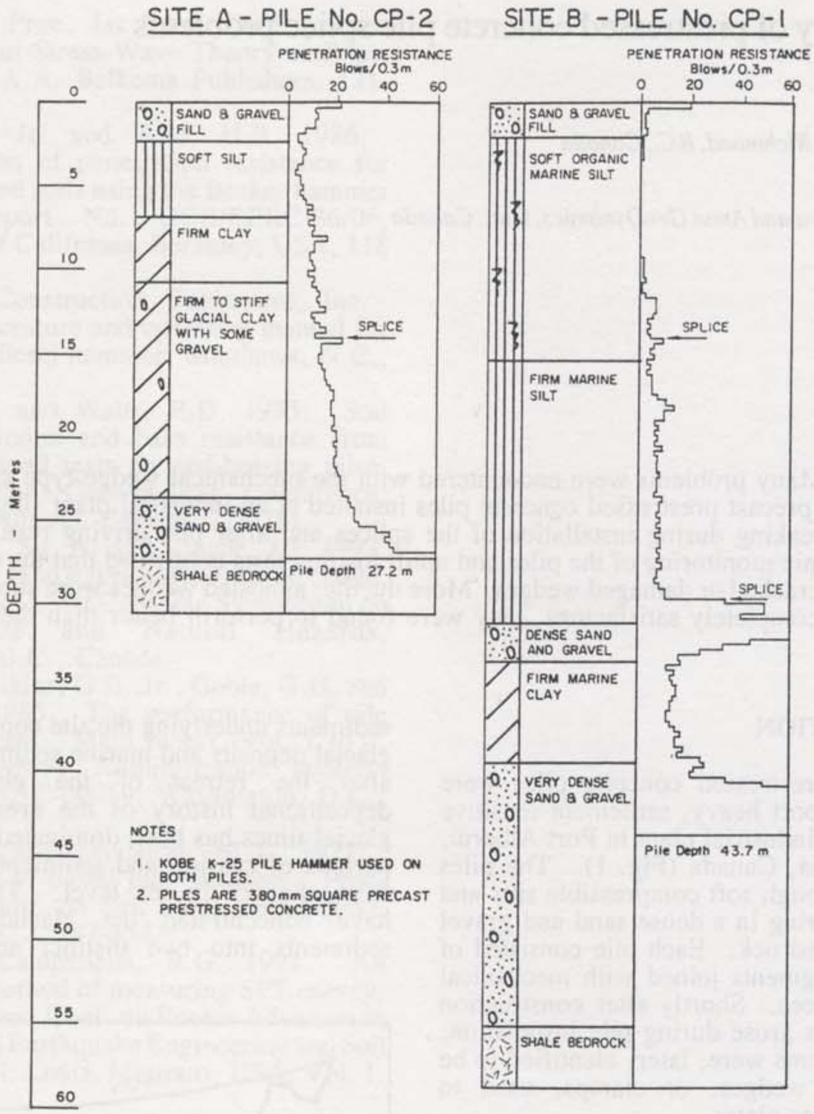


Fig. 2 Typical soil profiles and pile penetration records

deposits of soft marine silts overlying glacial clays and dense outwash materials consisting of sand and gravel characterize the surficial sediments at the portion of the plant site where Site A is located. In contrast, deep deposits of soft marine sediments interbedded with sand and gravel underlie much of the area where Site B is located. A relatively steep erosional discontinuity separates the glacial (Site A) and marine (Site B) sediments.

3 PILE AND SPLICE SPECIFICATIONS

The 380 mm square precast concrete piles were designed for a maximum allowable load of 1330 kN per pile. Some piles were also designed to resist tension loads of up to 360 kN per pile. The concrete had a minimum unconfined compressive strength of 48 MPa at the time of driving. Each pile had ten 13 mm diameter strands and was prestressed to about 7.5 MPa. The required pile lengths varied from 27 m to 60 m, and pile segment lengths ranged from 10.4 m to 23.8 m. Except for the head and toe of the pile, each pile segment end was attached with a 355 mm mechanical splice plate.

There are many splices available for precast prestressed concrete piles (Bruce and Hebert, 1974). Mechanical-type connectors or splices are gaining wide acceptance because they are rapid in execution, economical, and can be

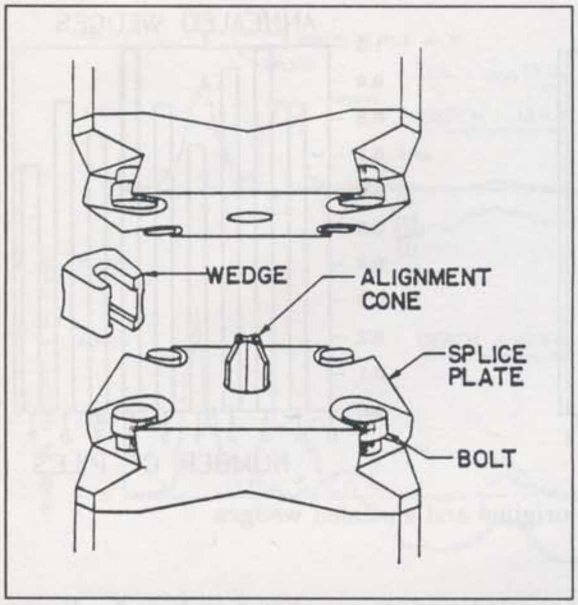


Fig. 3 Major components of the mechanical wedge-type splice

designed to develop the full structural capacity of the pile section. The mechanical wedge-type splice used consists of two steel splice plates, each attached to the end of a pile segment by four reinforcing dowels cast into the pile. After the lower pile segment is driven to about a metre above ground surface, the upper pile segment is placed on top of the bottom segment. With the splice plates in contact, four corner wedges, or clamps, are driven with a sledge hammer over the aligning bolts (one set at each corner) and locked in place with locking pins to complete the splice. The major components of the splice are shown in Fig. 3.

Venuti (1980) conducted an extensive laboratory investigation of the structural integrity of a similar mechanical wedge-type splice for 305 mm square precast prestressed concrete piles. In a series of structural tests, Venuti showed that the splice can develop the full capacity of the pile in tension, compression, and bending, and has a shear capacity in excess of that required for most severe loading conditions.

4 OBSERVATIONS DURING PILE INSTALLATION

The precast concrete piles were driven with a Kobe K25 diesel hammer having a rated energy at maximum ram stroke of 68.7 kJ to a final penetration resistance of 15 blows/25 mm in the dense sand and gravel deposit or on bedrock. In the early part of the pile driving work, several minor problems were encountered during installation of the splice, as follows.

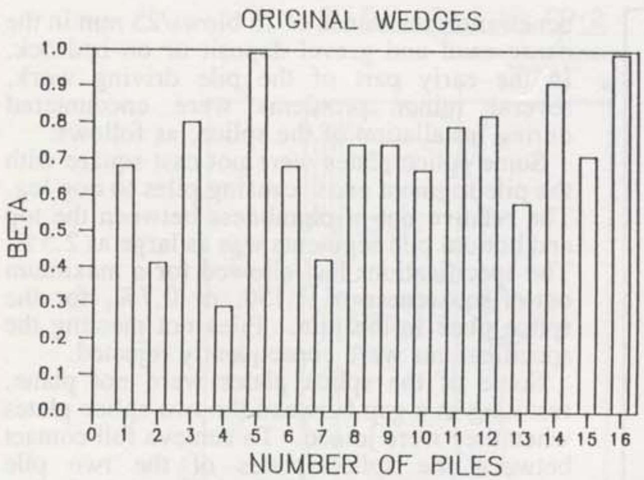
Some splice plates were not cast square with the pile segment ends, causing piles to dog-leg. The relative out-of-plumbness between the top and bottom pile segments was as large as 2.5%. The specifications had allowed for a maximum out-of-squareness of 1:150, or 0.7%, for the splice plate in the pile. Piles not meeting the specifications were consequently rejected.

Some of the splice plates were not plane, resulting in a gap between the two splice plates when they were joined. To achieve full contact between the splice plates of the two pile segments, high points on the splice plates had to be removed with hand grinders.

Another problem observed was that, in some cases, the four connecting bolts at the splice corners were not positioned accurately. When this occurred and the two pile segments were aligned for splicing in the field, three of the corner bolts would be in good alignment with the corresponding bolts in the other pile segments, while the fourth set of bolts would be misaligned. This makes installation of the connecting wedges difficult and sometimes impossible. Such piles were also rejected.

The major problem encountered with the splice, however, was related to the wedges. During installation of the splice, the four corner wedges had to be driven into the splice plates with a sledge hammer. It was observed that many of the wedges cracked or fractured under a few blows from the sledge hammer during installation. When the wedges were "successfully" installed and appeared sound by visual inspection, some wedges were yet found to be cracked or broken on subsequent retrieval even before resumption of pile driving. It was further found that some wedges cracked after resumption of pile driving following the splicing. This was again confirmed by retrieving the wedges for inspection after a metre or so of driving, while the installed splice was above or just below the ground surface. A total of 149 wedges were observed to be broken for the first 172 piles driven at Sites A and B.

The cast iron wedges as supplied by the manufacturer was examined by a metallurgist and found to be brittle. It was decided to heat treat or anneal the wedges in order to improve their ductility. Although annealing improved the ductility of the wedges, it also reduced the breaking strength of the wedges. The extent of the strength reduction was determined by testing four wedges, two original and two annealed wedges, in tension in the laboratory. The



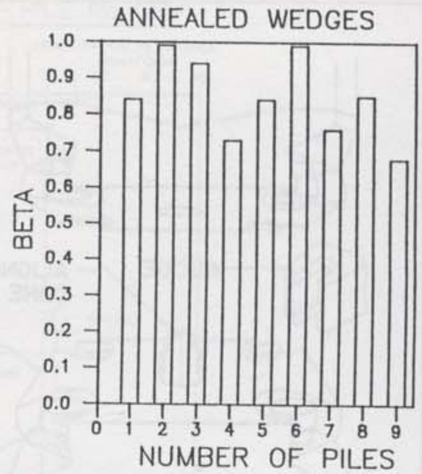


Fig. 4 Beta-factors for piles with original and annealed wedges

original wedges failed at an average load of 270 kN, while the two annealed wedges failed at an average load of 210 kN, i.e., a strength reduction of about 22%.

Annealed wedges were used for the remaining piles. Despite the lower breaking strength, only 7 annealed wedges were broken when driven in with the sledge hammer for the remaining 545 piles. No annealed wedges were found to be broken during pile driving while the splices were still above the ground surface and could be visually observed or retrieved for examination.

5 DYNAMIC MONITORING

The integrity of the splice in the ground was further evaluated by dynamic monitoring of the piles using the Pile Driving Analyzer (Goble et al., 1980). Monitoring was conducted on several occasions during construction to help evaluate and resolve the splice problem. The monitoring was performed during initial driving as well as during restriking. The Pile Driving Analyzer (PDA) measures strain (to determine force) and acceleration for each hammer blow, integrates the acceleration to velocity, and computes quantities relating to hammer performance, driving stresses, pile integrity and soil resistances. The PDA also calculates a beta-factor which is an indicator of the cross-sectional reduction in the pile and is commonly used to evaluate pile damage. The damage can be a crack or a series of cracks, a void, a spalling, or other loss of pile material. Damaged piles typically have beta-factors in the range of 0.5 to 0.8 or lower (Rausche and Goble, 1979).

A compilation of the beta-factors calculated for the monitored piles with original and annealed wedges are shown in Fig. 4. It was found that the beta-factor provided an approximate indication of damage in the pile. Comparison of PDA results for piles with original and annealed wedges showed that 75% of the 16 monitored piles with original wedges have beta-factors of 0.8 or less, while only 33% of the monitored piles with annealed wedges have beta-factors smaller than 0.8.

The stress wave measurements can also be used to detect the location of the damage in the piles. When the force and velocity during impact are plotted as wave traces in a diagram, the incident force and velocity are proportional via the pile impedance, EA/c (i.e., the product of the Young's modulus times cross-sectional area over the wave speed). Therefore, when plotted to the scale of the ratio of the pile impedance, the force and velocity initially plot on top of one another. However, when the impact wave meets soil resistance, compression wave reflections occur resulting in a separation of the force and velocity traces recorded at the pile head, or specifically, an increase of force and decrease of velocity. The greater the separation of the two traces, the greater is the shaft resistance. On the other hand, when the impact wave meets a cross-section reduction at some point in the pile, a reflected tensile wave will result at that location, which will be measured at the pile head as a drop in force trace and an increase in velocity trace, commonly referred to as a "blip". This principle of wave mechanics was successfully used by Authier and Fellenius (1980) to detect damage in precast concrete piles.

Fig. 5 shows the recorded wave traces from a series of blows during driving of Pile 10 at Site A. Pile 10 was driven to a final depth of 25 m, and consisted of two segments, an upper

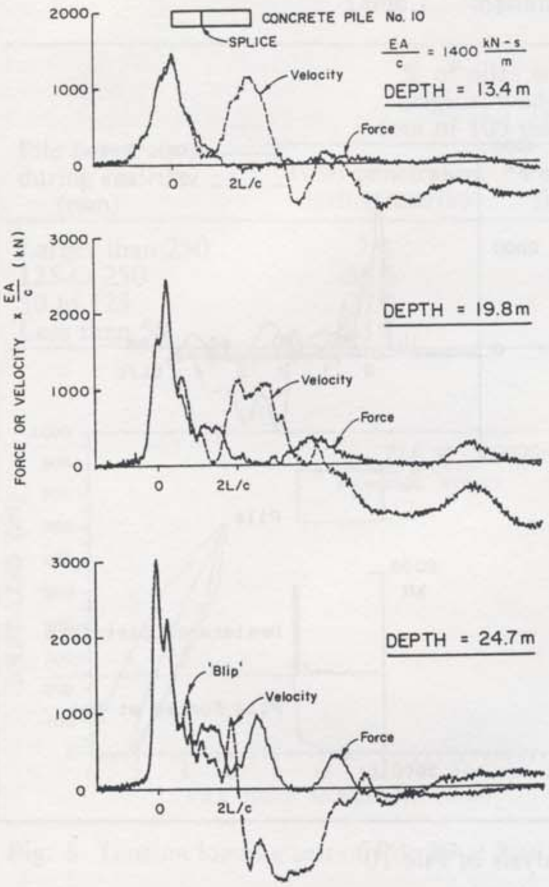


Fig. 5 Progressive damage of splice in Pile 10

segment 13.7 m long spliced with original wedges to a bottom segment of 14.0 m length. Initially, there was no sign of distress in the pile as shown in the top trace in Fig. 5. However, at about 15 m, the visual appearance of the traces changed and a "blip" started to appear on the traces originating from the location of the splice in the pile. The damage was fully evident at 19.8 m and remained evident to the end of initial driving, as shown in Fig. 5. The pile had a computed beta-factor of 0.7 or slightly lower toward the end of driving. Signal matching by the CAPWAP analysis (Rausche et al., 1972) confirmed the damage indicated by examination of the wave traces and low beta-factor. CAPWAP is a computer program which uses the force and velocity traces obtained with the PDA to evaluate the pile and soil boundary conditions through a trial and error process of signal matching. The boundary conditions are the pile impedances, soil resistance distribution, soil quake and damping characteristics. The results of the CAPWAP analysis for Pile 10 are shown in Fig. 6 and clearly illustrate the impedance reduction required at the location of the splice to achieve the signal match. This pile was subsequently subjected to an uplift loading test as discussed below.

6 TENSION LOADING TESTS

Static uplift tests were performed at Site A on Piles 10 and 207. Both piles had comparable lengths (25.0 m and 23.8 m, respectively) and had splices at about the same depth (11.0 m and 10.1 m, respectively). Pile 10, however, was spliced with the original wedges, while Pile 207 had annealed wedges. The PDA indicated both piles had beta-factors of about 0.7. The CAPWAP analyses on both piles also indicated cross-section reductions at the splices, although the reduction at Pile 207 was smaller. The results of the tension tests are shown in Fig. 7. Each pile was subjected to a maximum uplift load of 710 kN. The test confirmed that the splice with original wedges at Pile 10 was damaged, as indicated by the large deflection measured under the maximum uplift test load. The wedges were probably broken. The tension test on Pile 207 indicated a stiffer response to the uplift load. However, the movements were still larger than expected for a full length undamaged pile. Furthermore, the CAPWAP analysis indicated that the splice plates were not in full contact during driving.

At Site B, a static uplift test was performed on Pile 89. This pile was 47.2 m long and contained two splices, both with original wedges, at 4.0 m and 17.4 m depths. The CAPWAP analysis indicated cross-section reductions at both splices, with a much larger reduction in the lower splice than in the top splice. The tension test result is shown in Fig. 8, which suggested that the top splice was at least capable of resisting an uplift load of 710 kN. The test, however, was insufficient to stress the lower splice which the dynamic monitoring indicated was more severely damaged than the top splice.

7 HEAVE AND RESTRIKE

The ground surface at Site A heaved up to 600 mm after installation of the 460 piles within an area of 40 m by 60 m. Field observations indicated that essentially all piles in Site A heaved during driving of adjacent piles. No appreciable ground or pile heave was measured at Site B. All piles at Site A were subsequently redriven at least once. It was found that piles which achieved significant penetration during

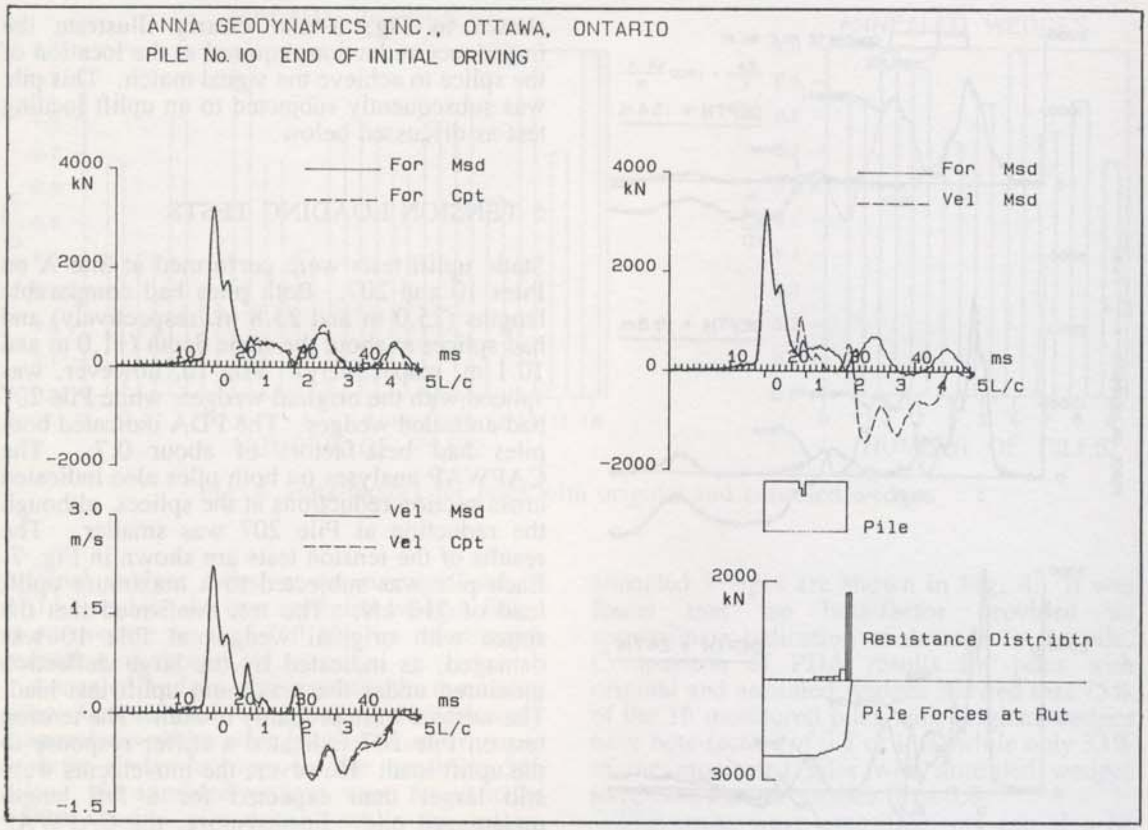


Fig. 6 CAPWAP analysis of Pile 10

Fig. 7 Tension loading tests of Piles 10 and 207 at Site A

restrike had also heaved significantly. It was also found that the large heave was mostly associated with separation of the pile segments at the splice location. The restrike records, therefore, provided an indication of piles with splice separation. The restrike records for the piles with original and annealed wedges are compared in Table 1. As shown in the table, while 42% of the piles with original wedges penetrated 125 mm or more during the restrike, only 10% of the piles with annealed wedges moved the same amount during restrike.

Also shown in Table 1 are the penetrations for the first 15 blows during restrike. As

Table 1. Summary of pile restrike data

Pile penetration during restrike (mm)	% of piles with original wedges (out of 105 piles)		% of piles with annealed wedges (out of 201 piles)	
	Total penetration during restrike	Penetration for first 15 blows	Total penetration during restrike	Penetration for first 15 blows
Larger than 250	7%	0%	0%	0%
125 to 250	35%	18%	10%	2%
50 to 125	27%	48%	46%	23%
Less than 50	31%	34%	44%	75%

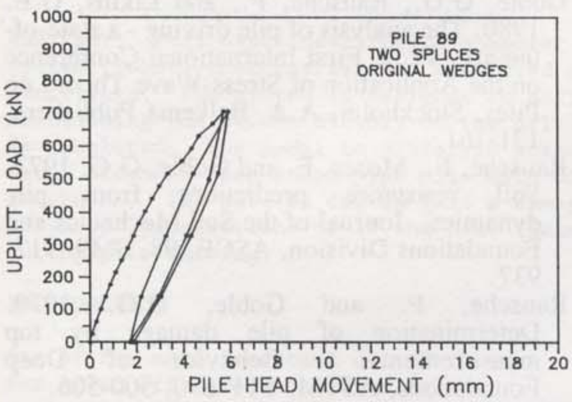


Fig. 8 Tension loading test of Pile 89 at Site B

shown, 66% of the piles with the original wedges penetrated 50 mm or more under the first 15 blows, while the percentage was only 25% for the piles with annealed wedges. When the pile penetrated a significant amount under the initial 15 blows during restrike, the conclusion was that the upper segment was being redriven initially to achieve contact with the bottom segment. This conclusion was confirmed by dynamic monitoring of the piles during restrike.

In order to check the compressive capacity of those piles with defective splices which separated due to ground heave and which were brought back into contact, a static compression test was carried out on Pile 79 at Site A. Pile 79 was driven with the original wedges to a final depth of 25.5 m, but it had heaved 240 mm due to driving of adjacent piles. The PDA confirmed that the upper and lower pile segments had separated at the splice location, but were brought back to contact when the pile was restruck. Fig. 9 shows the recorded wave traces from several blows of restrike with a drop hammer. The wave traces clearly show that the two pile segments closed after some

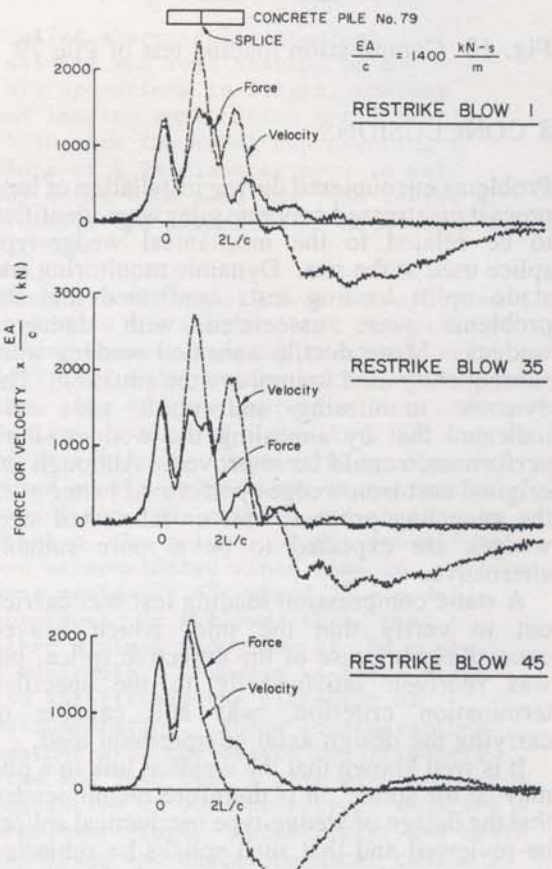


Fig. 9 Wave traces during restrike of Pile 79

35 blows of restriking. The results of the subsequent axial compression loading test on Pile 79 are shown in Fig. 10. The pile did not reach failure. A Davisson offset limit load of 2.9 MN can be determined from the load-movement data. The Young's modulus used to calculate the elastic compression line in Fig. 10 is 32.3 GPa.

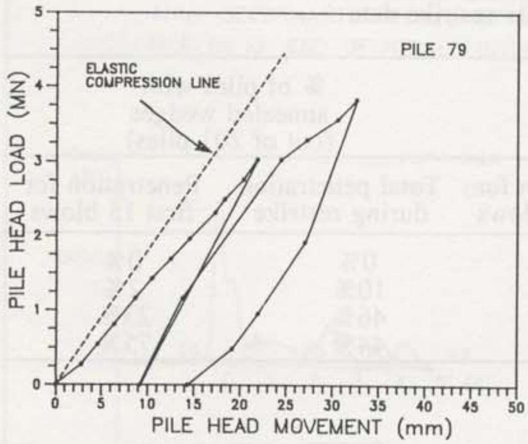


Fig. 10 Compression loading test of Pile 79

8 CONCLUSIONS

Problems encountered during installation of long precast prestressed concrete piles were identified to be related to the mechanical wedge-type splice used at the site. Dynamic monitoring and static uplift loading tests confirmed that the problems were associated with damaged wedges. More ductile annealed wedges were subsequently used to improve the situation. The dynamic monitoring and uplift tests also indicated that by annealing the wedges, their performance could be improved. Although the original cast iron wedges performed better after the annealing process, cast or fabricated steel wedges are expected to be a more suitable alternative.

A static compression loading test was carried out to verify that the pile, which heaved excessively because of the defective splice, but was redriven satisfactorily to the specified termination criterion, was still capable of carrying the design axial compression load.

It is well known that the weakest link in a pile may be the splice. It is therefore recommended that the design of wedge-type mechanical splices be reviewed and that such splices be subjected to extensive dynamic field tests before being accepted with confidence for use on engineering projects.

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