

7TH Annual

INTERNATIONAL BRIDGE CONFERENCE

PITTSBURGH, PENNSYLVANIA

JUNE 18, 19 & 20, 1990

FOUNDATION REHABILITATION OF THE POCOMOKE RIVER BRIDGE, MD
USING HIGH CAPACITY PRELOADED PIN PILESSM

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ABSTRACT

PIN PILESSM HAVE BEEN USED TO UNDERPIN TWO piers of the Pocomoke River Bridge, Maryland. The project had several unique features, including the use of pile preloading techniques to prevent further structural settlements. An intensive series of tests was run on preproduction piles. They confirmed the practicalities of the preloading concept, and the elastic response of the piles at twice working load (200 kips). Thereafter one of these piles was loaded to failure. A maximum load of 390 kips was recorded before failure in the bearing stratum. Analysis and telltale data confirm that the structural pile section performed elastically during the entire program.

1. INTRODUCTION

Throughout urban and industrial America, the use of small diameter bored and grouted piles (Pin PilesSM) is increasing dramatically^[1]. Pin PilesSM can be constructed through all types of rock and soil conditions, in restricted access areas, and at virtually any angle. Special drilling

techniques are used to minimize vibrational or other damage to soil or structure. They are typically six to ten inches in diameter and may be as much as 200 ft. deep^[2,3]. By varying the nature of the composite grout and steel section, they can be designed to resist compressive, tensile or bending stresses, or combinations of all three. Due to the mode of construction, with special drilling and pressure grouting techniques, they have outstanding load-deflection characteristics, as load is primarily transferred to the bearing stratum by skin friction^[4].

All of these characteristics make them an ideal choice for underpinning existing structures whose foundations have become inadequate or where additional load carrying capacity is desired. Such a case is the Pocomoke River Bridge, carrying Maryland Route 675 on its eastern shore in Pocomoke City about 25 miles southwest of Ocean City. The 275' long bridge consists of two 85' long approach spans on pile bents and a 105' long double leaf bascule center section providing a clear 65' channel width^[5]. This movable bascule structure (Figure 1) was built for the Maryland State Road Commission in 1921. In August of 1988, the northerly approach

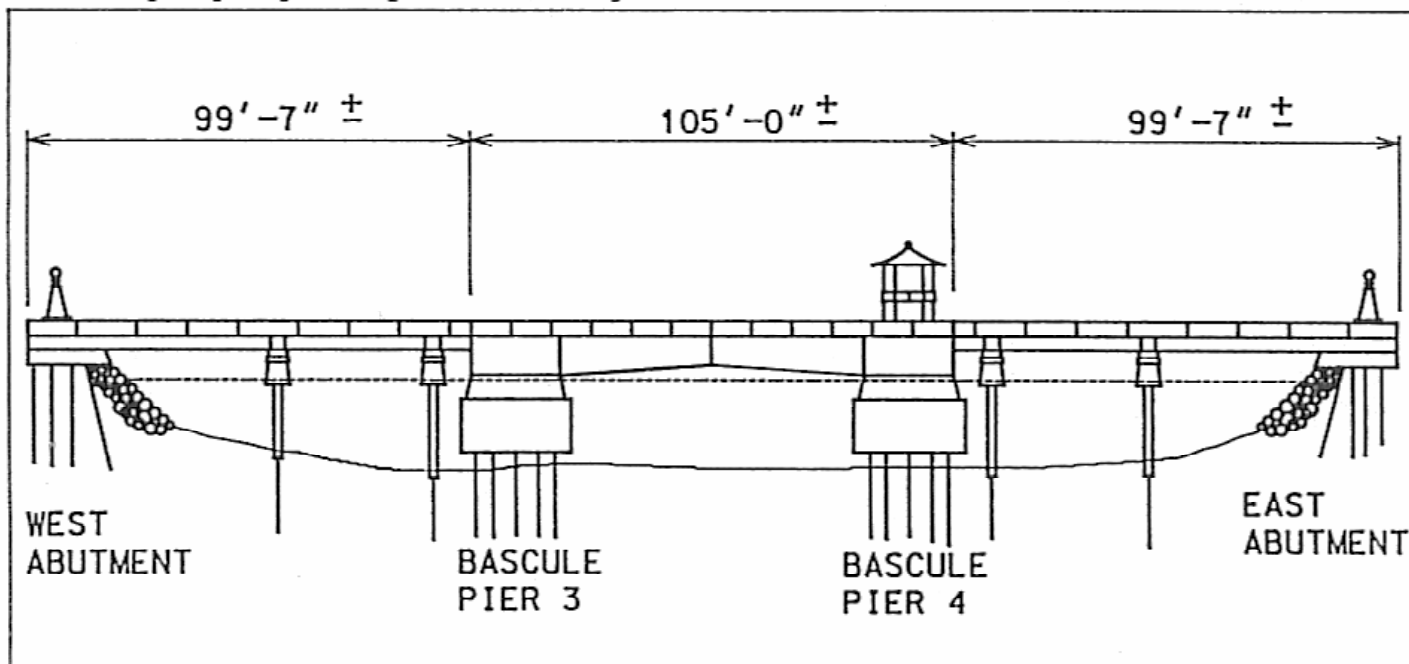


Figure 1. General Configuration of Bridge

span collapsed, promoting an investigation of the entire structure, a major rehabilitation design and a contract for the construction. The rehabilitation included full replacement of the approach spans.

Bascule piers 3 and 4 were originally supported on wooden piles driven through the soft riverbed muds into the underlying compact sand. The support offered by these piles had been compromised by river scour which had exposed them in several locations. Additional support was therefore required. Concurrent with the overall rehabilitation, the Pin Pilessm were installed. This underpinning had to be capable of being installed through the scour zone and of providing support without allowing any additional settlement of the structure.

The Pin Pilessm solution therefore featured steel casing through the scour zone, and a preloading technique involving a concentric strand tendon through each pile.

2. DESIGN

In each of Bascule Piers 3 and 4, a total of 24 piles were drilled from the deck of the bridge (Figure 2). In addition, a further 4 piles were installed from the restricted access of the Control House of Pier 4. Each pile had a design working load of 100 kips.

2.1 GROUND CONDITIONS - The river bed materials into which the underpinning was installed comprised a Recent sequence of alluvial sediments. In summary the stratigraphy was as follows:

0 to -15' Water
 -15' to -60' Soft alluvial riverbed sediments
 Below -60' Dense medium to coarse sand
 Blowcounts from Standard Penetration testing ranged from 25 to 30 blows in the sand.

This sand horizon was selected as an appropriate bearing horizon, given its density, thickness, uniformity, and ability to accept grout under pressure. These factors contribute to excellent frictional bond.

2.2 PIN PILESsm

2.2.1 Internal Composition - Each Pin Pilesm (Figure 3) comprised a 7 inch outside diameter, 1/2 inch wall thickness N-80 pipe casing with a yield strength of 80 ksi installed to the top of the sand. A #11 (1-3/8 inch diameter) Grade 60 epoxy coated reinforcing bar was used in the bond zone from the bottom of the pile to 5 feet above the bottom of the pipe casing. The 4000 psi neat cement grout performed structurally in the high strength pipe, as well as in the bond zone in conjunction with the reinforcing bar.

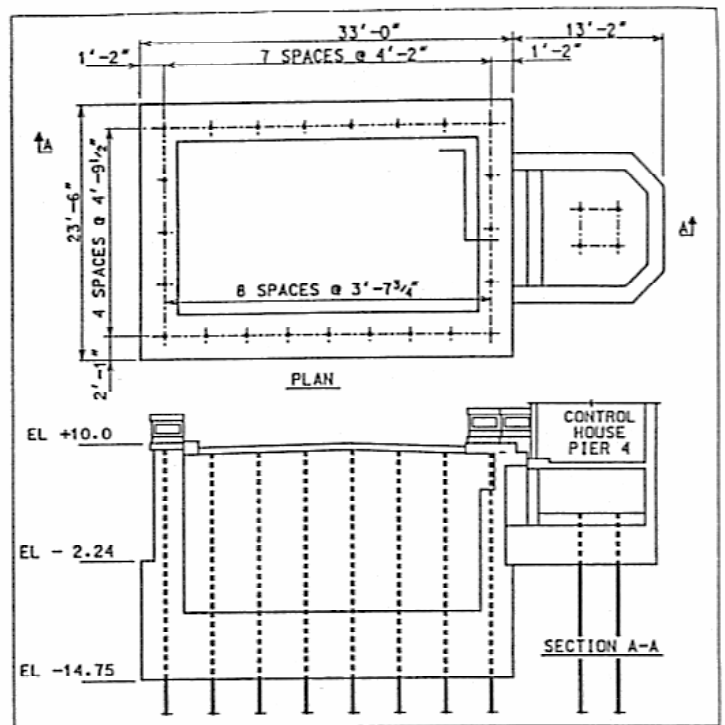


Figure 2. Plan and Section of Bascule Pier #4 Showing Pin Pilesm Locations

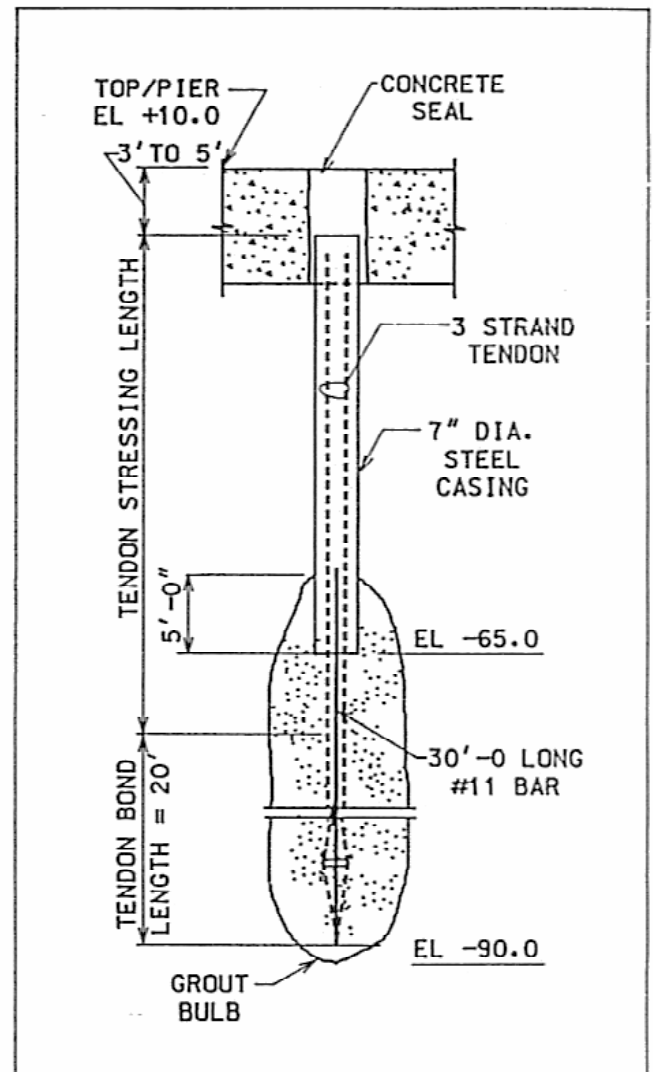


Figure 3. Typical Detail of Pin Pilessm

The allowable stresses used in this particular design were 30 percent of the unconfined compressive strength of the grout and 40 percent of the yield strength in both the pipe casing and reinforcement. In other typical designs, the allowable stress in the steel may range up to 50 percent, and in the grout, up to 45 percent. Research, however, is needed to evaluate and account for the true effective strength of the grout in place, bearing in mind its confinement by the steel casing. The allowable stresses currently in use would appear to be conservative for a confined material.

To achieve the preloading in compression, a tendon consisting of three 0.6 inch diameter low-relaxation prestressing strands was placed through each pile (Figure 3). The lower 20 feet of the tendon consisted of bare strand while the remainder was coated with grease and an extruded plastic sheathing to prevent bond with the grout. These are commonly referred to as the bond length and free length, respectively, in prestressed ground anchors. Each strand had a net area of about 0.215 in², a minimum guaranteed ultimate tensile strength (GUTS) of 58.6 kips and an elastic modulus of about 28,500 ksi. Working loads were equivalent to 46% GUTS.

2.2.2 External Design Factors - Due to the permeation effects of pressure grouting, the bond zone was assumed to have an actual diameter of 9 inches. This value was undoubtedly conservative, since the drilling with a 7 inch pipe casing created at least an 8 inch diameter neat hole, prior to pressure grouting.

The piles were designed for bond into the sand layer using a maximum allowable friction value of 30 psi along a theoretical 9 inch diameter by 20 foot long bond zone. The actual required bond stress at the design load of 100 kips was 15 psi. The actual tested ultimate bond stress subsequently proved to be about 57 psi.

To transfer the pile load into the structure, bond is mobilized at two interfaces - grout to pipe, and grout to existing concrete. Design allowables for the grout to steel interface range typically from 175 to 275 psi^[6]. For the minimum embedment of 15 feet in the pier, the working steel to grout bond stress was 25 psi. Typical design allowable stresses for grout to the roughened concrete interface are dependent upon the shear strength of the existing concrete assuming that a grout of higher strength is used to fill the annulus around the pipe. Common rules of thumb dictate that the ultimate shear strength of the base concrete is about 10 percent of its unconfined compressive strength^[6]. The allowable shear values for design would commonly be 1/2 to 2/3 the ultimate shear strength. Therefore, with a 3000 psi concrete the allowable grout

to concrete stress would be from 150 to 200 psi. For the minimum 15 feet embedment in the pier, the concrete to grout bond working stress was only 21 psi. Both interfaces, therefore, had high safety factors at working load.

3. CONSTRUCTION

The general construction sequence was as follows:

- o Place grout filled geotextile bags around the base of the piers and, using these as form work, inject any voids below the piers with viscous sand-cement grout. (Voids as much as 6' high were encountered.)
- o Predrill 8-3/4 inch diameter holes through the pier structure concrete and the grouted void infill using rotary drilling methods. The rotary head was mounted on a diesel hydraulic self-propelled crawler mounted rig and used 800 c.f.m. of compressed air at 100 psi.
- o Advance 7 inch diameter steel casing of 1/2 inch wall thickness to elevation -90 feet (i.e. approx. 100 feet below deck elevation), using rotary methods and water flush.
- o Tremie casing full of neat cement grout with water:cement ratio of about 0.45. Prepare the grout in a high speed colloidal mixer.
- o Place 30' of #11 bar plus 3 ea. 0.6" diameter strands to full depth. Then pressure grout to a maximum of 100 psi through the drill head while extracting the casing 25 ft (i.e. from -90' to -65'), and then pushing it back down 5 ft. into the bond zone (i.e. to -70').
- o Once the grout has reached 3500 psi, typically in 5 to 7 days, preload the pile to 82 kips.
- o Tremie grout the annular space between the pin pile and pier concrete.
- o After a further 5 to 7 days, release the preload by cutting the strands at the stressing head, thereby transferring the load to the pier.
- o Remove the 7 inch casing to 3' to 5' below the top pier concrete surface and fill the hole with a structural concrete.

Note that for the four piles under the Control House, a 13-1/2" outside diameter, 1/2" wall thickness steel pipe was first placed to elevation -35', through which the 7" diameter drill casing was drilled and grouted. These Pin PilesSM were also preloaded to 50 kips before being grouted into the larger pipe. A structural concrete beam system was then tied into the old Control House base to act as the new pile cap. The Control House measured 8' x 8' in

plan, with 14' of headroom obtained by removing the bridge level floor, and so all drilling equipment and pile components had to be handled and installed manually.

4. TESTING AND PERFORMANCE

4.1 TESTING ARRANGEMENT AND PROCEDURES - Two special preproduction test piles were installed for intensive testing, 16' apart on the adjacent west bank, 145' north and 55' west of the west abutment. The general set up is shown in Figure 4. Each pile had an outer casing 8-5/8" outside diameter, preplaced from the surface to -30'. The 7" casing of the pile was then installed in standard fashion through this large casing, but without being bonded to it in any way. This arrangement was intended to simulate in the test the lack of resistance afforded by the river and the very soft soils on its bed as well as the portion of the pile which was within the confines of the bascule pier. Each of the identical piles had 25' of pressure grouted bond zone (maximum grouting pressure 100 psi), 30' of #11 rebar, and 70' of 7" casing (from surface to 5' into the bond zone). Soil anchors were installed to provide reaction to the test loads. This is a common and attractive feature of testing pin piles in restricted access locations.

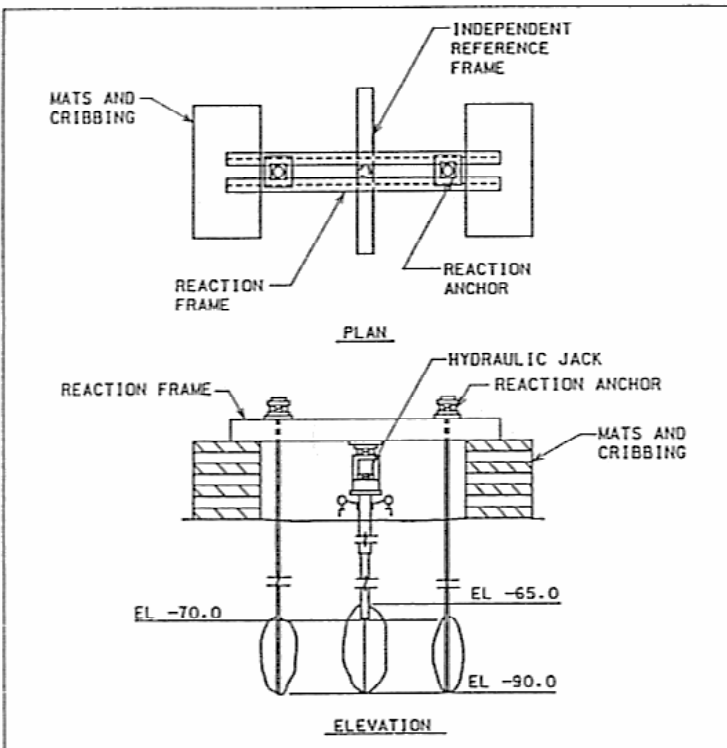


Figure 4.

Configuration for Testing of Pin Pilessm

The test was in these phases:
Phase I: a "Preload-Unloading" test designed to verify the performance efficiency of the preloading system (Figure 5).

Phase II: a conventional pile load test to establish load-deflection performance (Figure 6) within the scope of the specification (i.e., progressively to twice working load). Details of the Pile Testing Procedures for Phase I and II are provided in Appendix 1.

Phase III: on one pile loading to failure.

The test was heavily instrumented, with load being measured independently by load cell and by hydraulic jack gauge, and deflection monitored by dial gauges supported from an independent reference beam and by piano wire and mirror scale. Dial gauges were also used to indicate movements of fixed end extensometers (telltales) located at elevation -70' (i.e. at top of bond length) and at elevation -90' (i.e. at bottom of bond length). These telltales were intended to directly indicate displacement and at these two points in the bond zone, relative to the pile cap.

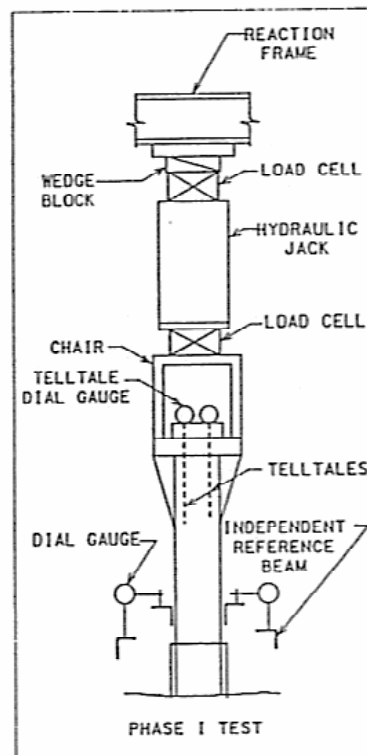


Figure 5.
 Specific Jacking
 Detail - Phase I
 (Rebound) Test

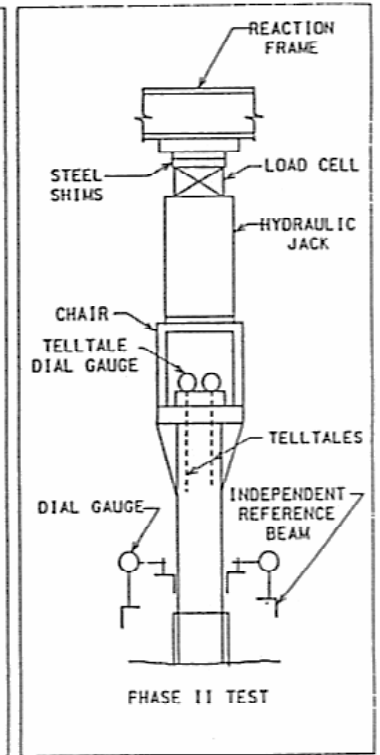


Figure 6.
 Specific Jacking
 Detail - Phase II
 (Vertical Compression)
 Test

4.2 RESULTS

4.2.1 Phase I Tests (Preload - Unloading Tests) - The anchor tendon in each pile was loaded to 82 kips, creating an elastic shortening of the piles of 0.123" and 0.137" respectively. Upon unloading to zero (i.e. releasing the prestressing load), the pile cap rebounded totally elastically, indicating no measurable permanent shortening.

	Deflection at 200 Kips (inches)	Creep in 24 hrs at 200 Kips (inches)	Permanent Displacement Upon Unloading from 200 kips Instantaneous/After 2 hrs (inches)
<u>Pile 1</u>			
Pile Cap	0.442	0.038	0.044/0.020
Upper Telltale	0.344	0.028	0.024/0.021
Lower Telltale	0.374	0.031	0.095/0.093
<u>Pile 2</u>			
Pile Cap	0.437	0.059	0.047/0.027
Upper Telltale	0.385	0.033	0.041/0.037
Lower Telltale	0.420	0.021	0.067/0.063

Table 1. Highlights of Load/Deflection Data, Test Piles 1 and 2.

As the procedure was demonstrated to work, and since the performance was elastic, this Phase of testing was accepted as being successful.

4.2.2 Phase II Test (Load/Deflection Test to Twice Design Working Load) - Each pile was loaded progressively to 200 kips in 20 kip intervals, each with a 5 minute hold period. Details are summarized in Table 1. Major points are as follows:

- * Performance of the piles was very similar, being virtually elastic, linear, and with minimal creep at intermediate holds
- * The total pile deflections (anticipated and observed) at 200 kips were less than one half inch, and the permanent deflections upon unloading were around 0.04 inches at 2 hours after final load release. After 12 more hours, the piles had returned to full extension (i.e., no measurable permanent shortening).
- * The performance of the telltales was wholly consistent. They reflected the internal elastic performance of the piles, and, so provided movements less than the total pile displacement (i.e. elastic plus permanent). Predictably, the upper telltale, monitoring a shorter length, provided the smaller movements. These data compare closely with the net elastic deflection obtained by subtracting total cap movement (at 200 kips) from the residual (at zero), as shown in Table 2.

- * Total creep at 200 kips ranged from 0.038" to 0.059" over 24 hours. However, the amount of "internal" creep was smaller and more uniform (0.021" to 0.033", Avg. = 0.028").
- * There was a time related "rebound" evident in all points of measurement after unloading. Overall, this was 0.020" to 0.027" at the pile cap including 0.002" to 0.004" of "internal" pile rebound.

Pile Number	Net Elastic Deflection* at 200 kips (Inches)	Measured Elastic Deflection+ at 200 kips (Inches)
1	0.398	0.374
2	0.390	0.420
Average	0.394	0.397

Table 2. Comparison of net and measured elastic pile performance

- * total deflection at 200 kips less permanent deflection at subsequent zero.
- + from Lower Telltale.

Load Cycle	Max Load	Total Deflection at Max Load	Creep at 15 mins at Max Load	Permanent Displacement after Unload	Reason for Test Termination
A	360 kips	1.241"	0.025"	Not recorded	Reached Limit of Reaction System
B	360 kips	1.242"	Not recorded	0.217"	Reached Limit of Reaction System
C	360 kips	1.297"	Not recorded	0.237" (0.210" after 16 hours)	Reached Limit of Reaction System
D	337 kips	1.144"	0.001"	0.113"	Load Cell Nonfunctioning

Table 3. Phase III tests, first four load cycles on Test Pile 2.

4.2.3 Phase II Tests (Load/Deflection Test to Failure, Test Pile 2) - Once the required test to two times design load (200 kips) was satisfied, an attempt was made to determine the ultimate skin friction available to the Pin Pilessm. The hold down reaction system was sized for about 360 kips which was initially felt to be sufficient to plunge the pile. Surprisingly, after four successive cycles to about 360 kips as shown in Table 3, the pile had not yet failed.

Thereafter, the test set up was overhauled, and the test rerun. As shown in Figure 7, a maximum load of 390 kips was reached before plunging of the pile was recorded.

Again the evidence of the telltales was that of virtually perfect elastic performance within the pile structure. The difference, at maximum load, between overall elastic performance (Lower telltale) and total deflection was 2.874" - 1.128" = 1.746". This compares very closely with the measured permanent set at zero load, of 1.712". The difference is probably due to the fact that the telltale was not exactly at the pile tip. Creep values were only significant from about 340 kips onwards.

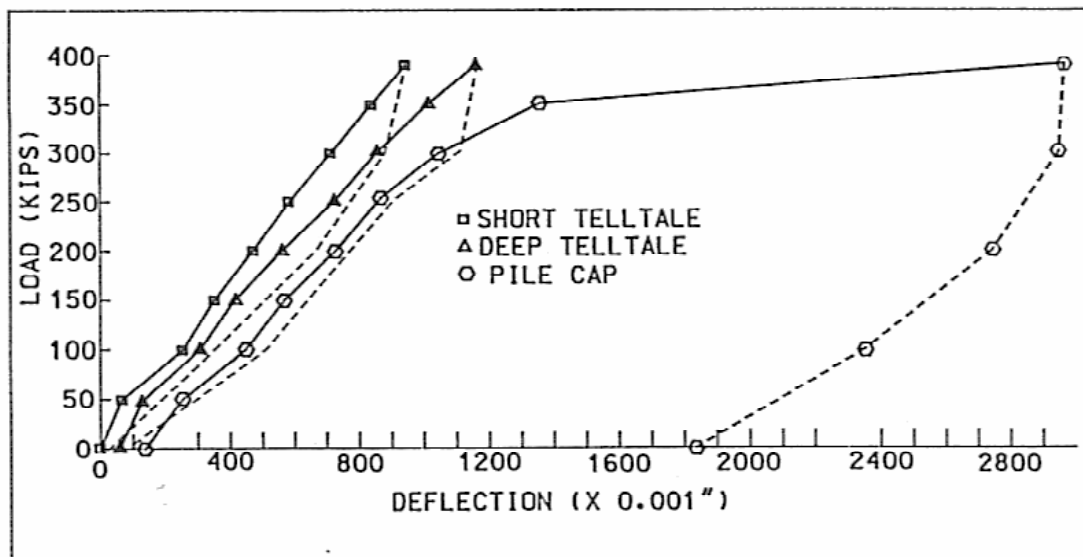


Figure 7. Load vs. Deflection Plot for the Test to Ultimate Load

The intensive preproduction test program confirmed many attractive aspects of the performance of Pin Pilessm in this unique application.

1. The preloading system worked perfectly.
2. At a test load of 200 kips (twice working load) the anticipated and observed total cap deflection was less than 1/2 inch, with no measurable permanent displacement at 12 hours after release of load. This excellent elastic performance was confirmed by internal telltales within the pile.
3. A maximum load of 390 kips was recorded at ultimate pile plunge. Analysis of the load deflection data and the telltale information confirms that the pile itself worked internally perfectly elastically during this load cycle, and failure occurred exterior to the pile, i.e. at the grout/soil interface, or in the soil mass itself.

ACKNOWLEDGEMENTS

The Maryland State Highway Administration (MSHA) was responsible for the funding of the program. The consulting engineer for the MSHA was Greiner Inc. of Timonium, Md., and the general contractor performing the overall rehabilitation work was Empire Construction Co. of Baltimore. Nicholson Construction, Inc. of Frederick, Md. installed the Pin Pilessm and substructure fabric bag and bulk grouting, for a total subcontract value of about \$1.2 million. National Foundation Co. (d.b.a. Nicholson Construction Inc.) received an award from the Md. State Highway Administration for "...their superb job in underpinning the bascule piers of the Pocomoke River Bridge." Greiner, Inc. was selected to receive an Award of Merit in the "1990 CEC/MD Engineering Excellence Awards Competition" for design of the rehabilitation.

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APPENDIX 1

PILE TESTING PROCEDURE

Phase I (Preload - Unloading Test)

1. Step 1: Load reaction frame to 120 tons (min.)
 - A. Incrementally load reaction anchors (A & B) to 60 tons each.
 - 1) Load anchor A to 20 tons, then anchor B to 20 tons; back to anchor A to 40 tons, then anchor B to 40 tons; back to anchor A to 60 tons, then anchor B to 60 tons.
 - 2) Keep checking reaction frame for excessive differential settlement.

2. Step 2: Set up for preloading of pile.
 - A. Place small chair on top of test pile plate.
(chair provides room for telltale dial gauges).
 - B. Place load cell on top of small chair.
 - C. Place transition plate on top of load cell.
 - D. Place jack (150 ton) on top of transition plate.
 - E. Place high chair over jack and load cell set-up.

3. Step 3: Preload test pile.
 - A. Place an alignment load on pile as close to one (1) ton as possible.
 - 1) Note: Reading on load cell beneath the jack will give the most accurate reading to determine this alignment load.
 - B. Set all dial gauges as indicated in Figure 5.
 - C. Pull and set piano wires across scales.
 - D. Record all initial readings on:
 - 1) Load cells (3).
 - 2) Dial gauges (6) (1 on each telltale = 2, plus 4 around pile).
 - 3) Piano wire reference scales (5).
 - 4) Hydraulic jack gauge pressure (1).
 - E. Incrementally preload pile in 25% increments (10 tons, 20 tons, 30 tons, 41 tons).
 - 1) Maintain each load increment until the rate of settlement is not greater than 0.01 in. per hour but not longer than 2 hours.
 - 2) Movement and load readings shall be taken and recorded at 5 minute intervals at the applied incremental loads.

4. Step 4: Preload applied - prepare for measuring release of preload.
 - A. Place a load cell (No. 4) between the high chair top plate and the reaction beam.
 - B. Secure a tight connection between top load cell (No. 4) and the reaction frame with steel wedges. (Place "snug-up" load of approximately 1 ton in load cell with wedges.)
 - C. Take and record all readings:
 - 1) 4- load cells
 - 2) 6- dial gauges
 - 3) 5- piano wire reference points.
 - 4) 1- jack pressure gauge.

5. Step 5: Unload the preload test piles.
 - A. Incrementally unload the preload (41 ton) in 25% increments (41 ton to 30 ton, to 20 ton, to 10 ton, to 1 ton)
 - 1) Release gradually at a rate not to exceed 1 ton per minute.
 - 2) Hold decrement loads 15 minutes.
 - 3) Take and record all readings at 5 minute intervals during holding period.
 - B. After the preload has been released, movement and load readings shall be taken and recorded at 10-minute intervals for 1 hour, 20 minute intervals for the next 2 hours.
 - 1) Readings can be terminated after 2 hours if the rate of movement is less than .01 inches/hr.
 - 2) If after 2 hours the rate of movement (rebound) is greater than .01 inches/hr. continue recording readings in 1 hour intervals for the next 10 hours or until rebound movement is less than .01 inches/hr., whichever occurs first.
 - C. Unload the remaining load in the pile.
 - 1) Load cell #4 (located between high chair and load beam) now reads the load remaining in the pile.
 - 2) Apply sufficient load to tendons to free wedges between load cell and load beam.
 - 3) Remove wedges.
 - 4) Release remaining preload in pile in 50% decrements of the remaining load at intervals of 1 hour.
 - 5) Take and record all readings before and after each load decrement and at 20 minute intervals.
 - a) 1 - load cell
 - b) 6 - dial gages

- c) 2 - piano wire reference points
- d) 1 - jack pressure gage

6) After all preload has been released take readings in accordance with 5-B above.

Phase II (Load/Deflection Test)

1. Step 1: Rearrange test set up and reaction frame to accomodate test set-up shown in Figures 4 and 6.

- A. Again incrementally load reaction anchors (A&B) to 60 tons each.
 - 1) Load anchor A to 20 tons, then anchor B to 20 tons; back to Anchor A to 40 tons, then Anchor B to 40 tons; back to Anchor A to 60 tons, then to B to 60 tons.
 - 2) Keep checking reaction frame for excessive differential settlements.

2. Step 2: Set up to do pile loading test.

- A. Place small chair on top of test pile plate to provide room for telltale gauges.
- B. On top of jack place a transition plate and load cell.
- C. Place transition plate and shim plates between load cell and reaction frame.

3. Step 3: Loading of test pile.

- A. Place an alignment load on pile. A 5 ton (5%) alignment is suggested; however, this can be determined in the field based on the inspection engineer's preference.

B. Set up, take readings and record all readings from measuring devices:

- 1) 3- load cells.
- 2) 6- dial gauges.
- 3) 5- piano wire reference scales.
- 4) 1- hydraulic jack pressure gauge.

C. Load test pile in increments of 10 tons.

- 1) Increments: AL (Alignment load)

10T	(Tons)	60T
20T		70T
30T		80T
40T		90T
50T		100T
- 2) Hold increments for 5 minutes minimum.
 - a. Take and record all readings at start, half way and end of each hold period.
- 3) Hold 100 ton (maximum load) for 24 hours.
 - 1) When maximum load has been applied, take and record readings when the jacking is stopped.
 - 2) Repeat taking and recording readings at 2.5 minutes and again at 5 minute intervals for the first 1 hour of hold.
 - 3) Take and record readings at 20 minute intervals for the next 2 hours.
 - 4) Take and record readings each hour for the remaining 21 hours.

4. Step 4: Unloading of test pile:

- A. Remove test load in 4 equal decrements.
- B. Hold each decrement for a 15 minute (minimum) time period.
 - 1) Take and record instrument readings immediately before and at the end of each 15 minute decrement period.
- C. Upon release of load take and record instrument readings in the following manner: at 10 minute intervals for 1 hour, 20 minute intervals for the next 2 hours, and 1 hour intervals for the next 10 hours.
 - 1) Readings can be terminated after 2 hours if the rate of movement is less than .01 inches per hour.
 - 2) Once the load has been released, the only instruments that require monitoring would be:
 - a. 6- dial gauges.
 - b. 2- piano wire scales attached to pile.