

by

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The benefits of pinpiling have been exploited in the USA for the last two decades. The demands and skills of American practice have led to developments of sufficient importance to now distinguish it sharply from aspects of the original European technology. This paper provides details from 20 recent projects and reviews in greater depth three major case histories. These particular projects illustrate major features about new developments in design, construction and performance in unique applications.

## INTRODUCTION

1. A number of papers in the late eighties (1-4) described the state of practice in pinpile technology in the United States. These papers demonstrated that pinpiles - known elsewhere in the world as minipiles or micropiles, but defined generically as small diameter cast-in-place bored piles - had arrived relatively late in the United States, probably some 20 years or so after their European genesis in the early fifties. However, the growth in their use had been dramatic, as the specialist geotechnical construction industry responded to the demands of renovation, remediation and redevelopment, largely in urban and industrial locales. These reviews also detected that the pinpile market in the United States was generating a special identity for itself as a result of its ongoing pursuit of progressively higher unit capacities in a wide and complex range of subsurface conditions.

2. This paper presents brief reviews of three selected case histories of pinpile projects conducted by the author's company alone, in the few years since the first survey. To begin with, it should be noted (Table 1) that 20 pinpile projects can be cited in the two years from early 1989, compared to the total of 25 projects listed in the previous 11 years (1). This is clear evidence of the rapidly increasing use of the technique of pinpiling, especially in the older cities of the East Coast (5). Equally noteworthy is the routine use of individual pile working loads appreciably higher than in earlier years, or elsewhere in the world. The test loads of 220 tons in fine sands in Brooklyn or 350 tons in poor rock in Pittsburgh (Table 1) are, admittedly, exceptional: normal working loads of 75-110 tons in soils, and higher in rock are now fairly routine.

3. This expansion in application and growth in working loads has been supported by evidence from numerous exhaustive test programs, executed in a consistently routine and scientific manner as an integral part of each

commercial project. In most cases the test piles have been loaded in incremental, cyclic fashion, so permitting movements to be partitioned into elastic and permanent components. Analysis of the former provides indications of load transfer mechanisms within the pile, whereas examination of the latter essentially permits the behavior of the founding medium to be investigated. This approach has significantly aided the understanding of how the ancillary ground treatment technique of post grouting (6) works to enhance pile performance - although space prevents a detailed account of the fundamental work at Augusta, Brooklyn and Seattle. More experience is also being obtained with the concept of preloaded piles wherein the service load is preapplied and locked into the pile before the pile is connected to the structure. In this way the possibility of later structural movement is eliminated. The potential of this particular pinpile variant, the NCA-Pile (Nicholson Compressed Anchor-Pile) in seismic areas is outlined.

4. In addition, it should be recorded that much recent attention has been focused on the performance of pinpiles when subjected to lateral loading. The impressive results from laboratory testing (7) have been supported by recent field test programs (Table 1) at Brooklyn and Augusta: provided casing is left in place through the upper, typically "soft" horizons, pinpile performance is excellent.

## REVIEW OF ILLUSTRATIVE CASE HISTORIES

5. Table 1 summarizes data from the 20 projects most recently conducted. Those three case histories selected for more detailed review to illustrate new developments are: (i) Postal Square, Washington, D.C. - A classic application of pinpiles as an advantageous construction alternate to hand underpinning or large diameter caissons during the redevelopment of an old, massive structure, and which demonstrates the benefits of cyclic loading.

LOCATION	APPLICATION	GROUND CONDITIONS	INSTALLATION CONDITIONS	LOAD (TONS) WORKING/TEST	PRODUCTION PILES	LENGTH INSTALLED	TYPICAL PILE LENGTH	DRILL DIA OF BOND ZONE	INTERIOR PILE COMPOSITION	NOTES
Cleveland OH	Foundations for new electric furnace in existing building	Slag fill, soft silty clay over shale bedrock	Low headroom	35/-	12	1500'	125'	5-1/2"	5-1/2" casing plus 1-1-3/4" rebar in bond zone	-----
Boston, MA	Underpinning of existing building being redeveloped	Fill and soft clay over bouldery till	Restricted access, with 8' minimum headroom	60/120	37	4850'	50'	7"	2" high yield rebar	Two tests to 128 tons: total def. = 0.223" permanent def. = 0.050"
Cleveland OH	Support to spread footings of existing pipe bridge, already settled 18"	Stiff clay	Very difficult access to and under bridge	12-1/2 / 25	4	200'	70'	5-1/2"	5-1/2" casing	-----
Montgomery Co., PA	Foundations for new bridge abutment	Silty soil over Karstic limestone	Overhead power lines	11/235	48	1025'	23-80'	8-1/2"	3-5/8" casing to rock, 1" casing full length	At test load: total def. = 0.298" permanent def. = 0.010"
Rome, GA	Support for foundations in operational paper mill	Fills over shales with quartzitic seams	Access through doorways; minimum 12' headroom	95/110	33	1320'	40'	5-1/2"	5-1/2" casing	-----
Apollo, PA	Support for column foundations to permit excavation of hazardous waste	Low level radioactive fill and silty clay with rock fragments over siltstone and shale	Interior of operating steel mill, minimum 10' headroom	60/-	20	800'	40'	5-1/2"	5-1/2" casing	-----
Orangeburg NY	Foundations for exterior stairway for existing psychiatric center	loose fill overlying very compact glacial till	14x14' access to interior courtyard	5-33/ 20-75	103	2760'	26-32'	8"	1/2"-1" rebar	At test load: total def. = 0.110" permanent def. = 0.028"
Huddleston VA	Foundations for new river bridge	15' of alluvials and weathered rock over granite/gneiss	Good access, unutilized headroom	10/140	72	1901'	25'	7"	7" casing plus 1-3/8" high yield rebar in bond zone	-----
Pocono City, MD	Replacement foundations for 10-year-old delicate bascule bridge	River bed silts and clays over 30' dense fine-medium sands	Most from bridge deck, 4 from very limited access/headroom	50/100	52	5200'	100'	7"	7" casing plus 1-3/8" high yield rebar in bond zone	See text.
Augusta, GA	Underpinning of footings subjected to additional loads in operational detergent factory	23' clay over various medium-fine sands with interbedded clays	Very restricted access, minimum 8-10' headroom	50/140	143	5231'	31'	7-5/8"	1-3/8" dia high yield rebar, plus 3" casing in upper 16' for lateral resistance	Routine use of postgrouting to enhance soil/grout bond.
Baltimore, MD	Intensive underpinning of historic 5 story building threatened by deterioration of original wood piles	Peats and clayey silt over silty fine sands	Very restricted access, 8-10' headroom	10/250	121	4500'	35-40'	7"	15-20" of upper 7" casing with 20-25" of 1-3/8" rebar in bond zone	Described in "Civil Engineering" in December 1990.
Seattle WA	Test program for underpinning of historic building	Sands and silts over fine and silty dense sands	Through concrete footings in old structure, headroom as low as 8'	70/135-150	4 (test)	140	30-40	5-1/2"	10-30" casing, bond lengths with full length 1-1/4" rebar	Excellent test data, including use of post-grouting.
Pittsburgh PA	Supporting existing columns of operating hospital to permit adjacent and ulterior excavation	Siltstone, shale, claystone	Interior of very sensitive building with 10' headroom	125/325	18	775'	43'	7"	7" casing	See text.
Brooklyn NY	Temporary and permanent piles to support overhead roadway	Fine-medium glacial sands with silts and clays	Reasonable access, 16' headroom	60-100/ 120-250	77	4250'	50-60'	7"	7" casing	Excellent vertical and lateral testing with postgrouting.
Covington VA	Foundations for pipe bridge foundations for mill expansion	20' soils over 15' shale and limestone	Through and around existing foundations	100/-	172	6020'	35'	8"	7" casing to rock, 3 ea 1-3/4" rebar in bond zone	-----
State College, PA	New column foundations for fire damaged church	Clay over karstic limestone	Difficult access, low headroom	20-35/-	50	1750'	35'	5-1/2"	5-1/2" casing to rock, 1" rebar in bond zone	-----
Memphis, TN	Test pile for underpinning of major transport facility	Clayey fill over sanitary landfill over loose sand and stiff clay	Unrestricted	-/40	1	130'	130'	5"	5" casing to top of bond zone, 3 ea 5/8" dia rebars below	At failure load: total def. = 1.069" permanent def. = 0.094"
Washington D.C.	Underpinning for new and existing foundations for historic, massive building being refurbished	Fill over various alluvial fine-medium sands with cobbly/clayey horizons	Existing basement with 8-10' headroom in 3 areas	75/150	603	37500'	51-58'	7"	25-30" casing plus 25' of 1-3/8" rebar in bond zone	See text.
Pittsburgh PA	Foundation for pedestrian bridge	Backfill over claystone	20' headroom within 18' of existing structure	75/-	12	540'	45'	6-1/2"	5-1/2" casing	-----
Baltimore MD	Foundation for temporary highway bridge	25' of alluvials and weakened material over schist	Unrestricted	75/-	4	100'	25'	7"	7" casing	-----

**Table 1.** Details from pinpile projects completed since late 1988 by Nicholson Construction

Note: Grouting conducted with or without excess pressure with neat cement grouts, Type I or II, at w/c = 0.45

(ii) Presbyterian Hospital, Pittsburgh, PA - High capacity pinpiles installed in very difficult access conditions, subsequently partially exposed, to operate without lateral restraint.

(iii) Pocomoke River Bridge, MD - High capacity, temporarily preloaded pinpiles installed to underpin an existing bridge unable to accept further structural movements.

6. Each case history is presented in the same, standard format.

POSTAL SQUARE, WASHINGTON, D.C.

7. Background. The original portion of the massive Old Postal Building, Postal Square, was completed in 1911. A major extension followed in 1931. For many years it served as the main Post Office for Washington, D.C., being located adjacent to the Union Station on Massachusetts Avenue, a few blocks north of the Capitol Building.

8. The developer (Gerald D. Hines Interests), acting for the Federal Government, planned to remodel the existing structure by adding new office floors in the center court area and constructing mechanical space below the existing lowest basement elevation of +23'. This meant that existing foundations had to be upgraded and new columns added to support new interior framing. The existing supports are steel and concrete columns on large concrete footings, and 14" square caissons end bearing on dense sands.

9. Originally a very cumbersome underpinning scheme was considered, involving hand dug support, massive spread footings and large diameter caissons, both hand excavated and mechanically drilled. However, the hand work would probably have caused significant undermining of the existing footings, leading to settlement, whereas drilled caisson work would have been inhibited by the very restrictive access, and low headroom. Both techniques would have been unattractively time consuming and costly. The pinpile alternate resolved both concerns.

10. Site and Ground Conditions. The work was conducted underground in three main areas in the basement of the existing structure:

- B2 Level (EL 6'): large level area with about 12' headroom. Piles reached EL -45'.
- B2 Level (EL 11'): most restricted area, headroom 8'. Piles reached EL -45'.
- B1 Level (EL 23'): Open access with 15-20' headroom. Piles reached EL -35'.

11. Under the concrete footings and a few feet of fill, the natural soils comprised recent alluvials, ranging from coarse to fine sands, laterally and vertically variable. Some gravel and mica were found sporadically, together with thin layers of cobbles or stiff clayey silt and silty sand in lower reaches. Typically the sands were dense to very dense. Natural groundwater level was at about EL -5'.

12. Design and Construction. Past experience and standard texts (8) were used to design 390 piles in the B2 levels and 310 piles in the B1 level, each with a nominal working load of 75 tons. About 25% of the piles were

installed in groups of 4 or 6 through 15 existing B2 (EL 6) footings comprising 7-14' of concrete. Pile centers were within 20" of existing columns.

13. Totals of 21 new caps were created in B2 (EL 6), 17 in B2 (EL 11) and 53 in B1. These featured standard (and several non-standard, specially designed) plan geometries from 5'4" x 4'8" (3 piles) to 7'6" square (9 piles). The minimum pile separation was 26" center to center, but was typically 30".

14. Custom built, short mast diesel hydraulic track rigs were used to rotate 7" dia. 0.5" wall N80 casing with water flush, to target depth. Type I grout of w/c = 0.45 was injected under excess pressures of 80-110 psi during progressive extraction of the casing for 25'. The casing was then reinserted 5' into this pressure grouted zone as permanent support. The lowermost 25' of pile was reinforced by Grade 60, 1-3/8" dia. rebar in 3 coupled lengths.

15. For those holes through existing footings, a 8-7/8" down-the-hole hammer was used to penetrate until significant steel was encountered. Thereupon, the hole would be completed with a 8" core bit. Load transfer between casing of the pile and the footing was ensured by the use of a special non-shrink, high strength grout. For the new pile caps, the pinpile casing was extended 4" up into the subsequent concrete, the horizontal reinforcing of which was fixed 2" above the top of casing.

16. Testing and Performance. Four special test piles (TP) were installed prior to constructing the production piles (Table 2).

17. TP1 and 2 were tested cyclically, yielding the analysis provided in Figure 1. TP3 and 4 were also tested incrementally but progressively to maximum load in accordance with ASTM-D1143-81 (9). TP1 clearly failed at the grout-soil interface, the founding horizons being on average finer and less dense than those for the other piles. Figure 1 also shows that the elastic compressions of TP1 and 2 were similar at the failure load of the former. This shows that load must have been transferred to similar depths in both piles, despite the nominal difference in "free length" upon construction. The elastic performance of TP3 and 4 was likewise similar, supporting the observation.

18. Table 2 also highlights higher total creep amounts in TP1 and 2: simply a reflection that there were far more creep monitoring

Table 2. Test pile data, Postal Square, D.C.

	TP1	TP2	TP3	TP4
Area Length (ft)	B2 (EL 6) 36	B2 (EL 6) 51	B2 (EL 11) 36	B1 (EL 23) 58
Bond (ft)	25	25	25	25
Max. test load (kips)	187.5 (failed)	300	300	300
Elastic extension at max. load (x0.001")	173	461	383	374
Permanent deflection at end of test (x0.001")	313	291	187	113
Total cumulative creep (0.001") during test	96	174	59	61

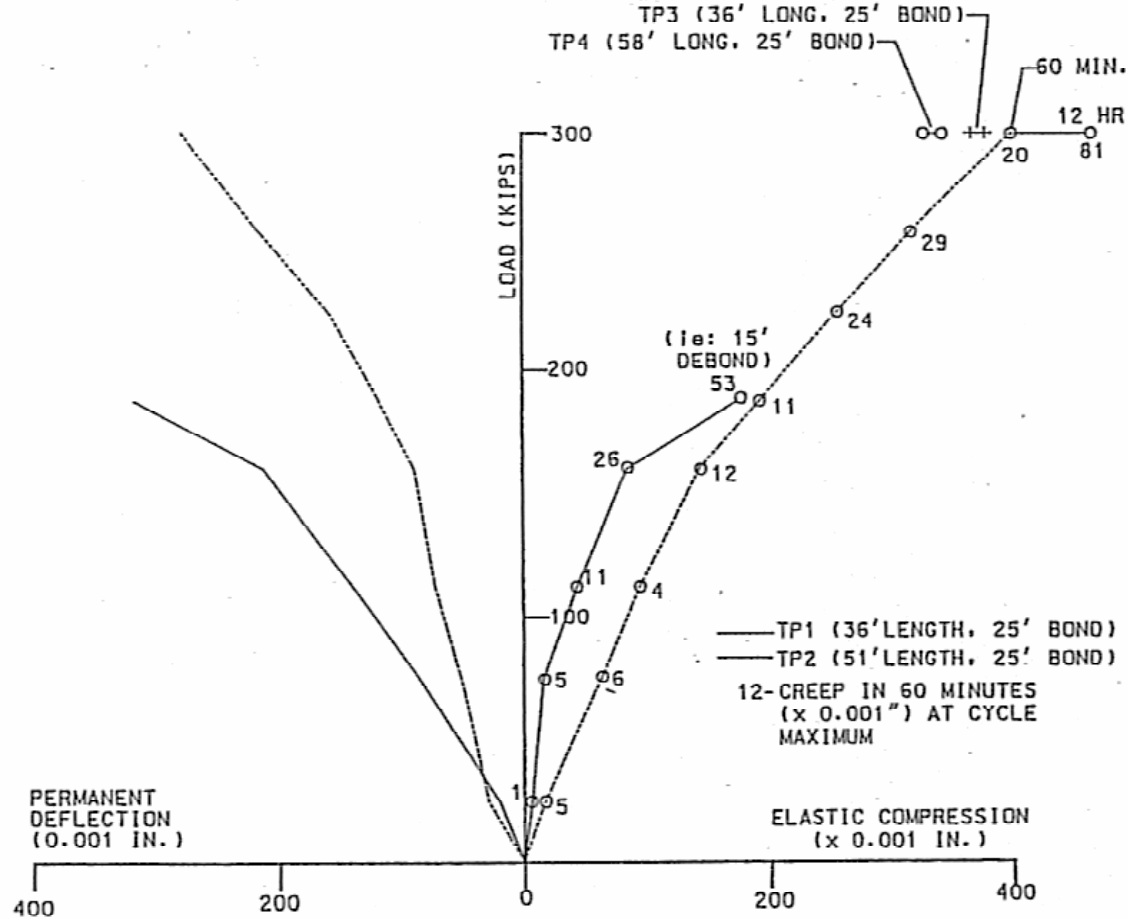


Figure 1. Elastic/permanent set performance of TP1, TP2, Postal Square, D.C.

periods in the cyclic loading that in the progressive loading. This clearly impacts overall permanent displacement and is an important point to bear in mind when judging pile performance by this criterion.

19. A separate pullout test was conducted in an existing column footing in the B2 (+6) level to explore concrete/grout bond. A special element was grouted 4'6" into an 8-7/8" hole drilled through the concrete. A special high strength, non shrink grout was used. After repeated cyclic loading to 525 kips (79% GUTS), and equivalent to 350% design load), the maximum uplift recorded was 0.005", reducing to 0.001" upon destressing. Assuming uniform bond distribution, an average grout-concrete bond of almost 350 psi had therefore been safely mobilized.

20. Following installation of the pinpiles, the structural renovation has progressed, and the foundations have performed perfectly.

PRESBYTERIAN UNIVERSITY HOSPITAL, PITTSBURGH, PA

21. Background. The Presbyterian University Hospital complex already occupies two extremely congested city blocks, and so when the need for more facilities became apparent, the decision was made to vertically extend and laterally link several existing operational structures. Overall, 1.6 million square feet of new facilities are being built in four major additions.

22. This highly delicate operation, conducted within a fully functional facility,

has necessitated some equally complex and innovative foundation engineering solutions involving excavation support and structural underpinning. One of the most dramatic operations has been associated with the completion of a new Magnetic Resonance Imaging Center. The construction of a new elevator pit called for a 30' deep excavation directly underneath three exterior column footings of the adjacent 13-story hospital structure. The pit, 60' x 32' in plan was further bounded on two sides by five additional footings, and so these sides required anchored lateral support.

23. Historically, the support of columns in such circumstances has been achieved by conventional underpinning pits and needle beams. However, in this instance, the difficult access conditions, and the specified requirement to limit downwards movement of the columns to less than 1/8" demanded a special solution, featuring high capacity pinpiles in rock.

24. Site and Ground Conditions. As noted, the access was very restricted laterally and vertically (as low as 12'), and the work had to be conducted within the confines of a fully operational medical facility. The piles were installed through 3'6" of existing nearby reinforced concrete footings cast directly on fractured, fissile medium hard-hard siltstone, occasionally calcareous or limy.

25. Design and Construction. At each existing footing, six pinpiles (4 working, plus 2 redundant) were installed in 8-1/2" holes drilled vertically by rotary percussive methods

and air flush to the target depth (43' below the footing). Each had a design working load of 250 kips. The reinforcing element consisted of a 7" dia., 1/2" wall N-80 casing placed full depth in each hole which was then tremied full of neat cement grout of w/c = 0.45. The upper 23' of each pipe was greased on the outside to debond it from the surrounding grout in that region and so permit load transfer into the 20' long bond zone. The suitability and security of this design had been proved in the earlier test program, described below.

26. A structural steel jacking frame was then erected over the top of the piles and fastened to the existing steel column. Each of the steel columns - supporting an occupied hospital building - were then sequentially lifted off their existing spread footing by a distance of 1/16 - 1/8". This effectively preloaded the piles to prevent any later settlement of the building, and transferred the column loading into the bedrock, but 23' below. Excavation then proceeded, supported laterally by beams, shotcrete lagging and prestressed rock anchors. As the excavation deepened, cross frames were welded to the pinpiles to limit the unbraced lengths of these piles, now exposed as steel columns.

27. Testing and Performance. By the end of excavation, the foundations of the existing structure could be seen resting on the pinpile groups, 22' off the bottom of the excavation. During and after excavation, absolutely no movement of the structure could be measured.

28. One of the most common problems foreseen for pinpiles (10) is their potential for buckling or bending, as an inferred consequence of their high slenderness ratio. This unique project - featuring pinpiles with no surrounding ground to offer any lateral restraint - is proof irrefutable that correctly designed and constructed pinpiles can operate with surprising efficiency not only in the axial sense.

29. Clearly, testing of production piles was not possible in this instance, and so a full scale test pile was installed beforehand. Using identical construction methods, a pinpile with 20' bond was formed in the same geological stratum. The total length was 50', including, therefore, 30' of debonded "free length". Reaction to the test load was provided by a pair of prestressed rock anchors. The casing was preassembled in the workshop and consisted of 5 separate lengths, hand tightened together. Two "telldatales" were incorporated - one each at the top and bottom of the bond zone. A thick, soft wood plug was attached to the bottom of the reinforcing pipe to eliminate any possible end bearing contribution and to so allow only side shear to be mobilized. As part of the contract requirement, the pile was then tested to twice design working load (500 kips), according to ASTM-D1143-81 (9) (modified to allow cycles at 25%, 50%, and 75%). Results are summarized in Table 3. At 250 kips, the elastic compression of 0.227" was exactly that predicted, while the permanent displacement of 0.052" was proved by the telldatales to be due to some inelastic compression of the steel casing itself. While loading from 400 to 425

Table 3. Summary of test pile performance, Presbyterian University Hospital, Pittsburgh. (All movements in thousandths of an inch.)

Load cycle maximum (kips)	Total butt movement at maximum (A)	Apparent permanent butt movement at subsequent zero (B)	i.e. Elastic deflection at maximum		Bottom telltale movement (relative to butt)
			(A) - (B)	(A) - (B)	
125	127	42	85		37
250	279	52	227		44
375	448	77	371		63
300	663	329	334		295
500	1020	517	503		471

kips, a "bump" was recorded and the load dropped to 300 kips. Load was then increased to 500 kips when a further "bump" occurred. However, when the data from the cyclic loading and the telldatales were analyzed it became clear that:

(i) the pile elastic deflection at 500 kips was exactly as predicted, and  
(ii) the apparently large permanent movement (Table 3) was due to an irreversible "one off" shortening of the steel pipe. The assembly records of the pile were then reviewed. It transpired that there had been several "unshouldered" hand tightened joints between adjacent casing sections. It was suspected that each joint was unshouldered about 1/4 - 1/8". Thus, the sudden 0.471" permanent compression of the pile material was readily explained, and when subtracted from the permanent set of 0.503", gave a true movement of the pile tip into the rock of 0.032" at 500 kips - an outstanding performance. There was negligible creep at all load increments.

30. Thereafter, the pile was tested to a maximum load of 675 kips before it became clear that material failure of the steel casing was occurring. At this load, the steel had compressed 3.084" (from telldatales), compared with the measured butt permanent displacement of 3.224". Thus, at 675 kips, a true permanent movement of the pile of 0.140" had been recorded, while analysis proved the perfect elastic performance of the pile with a calculated debonded length a few feet into the bond zone.

31. This project was therefore highly significant from several viewpoints:  
(i) the excellent lateral and vertical performance of pinpiles was proved again;  
(ii) the value of telldatales in aiding understanding of internal pile performance was demonstrated, and  
(iii) the warming fact that the boundaries of pinpile design are now those of the constituent materials - i.e., independent of the surrounding ground properties - was massively underlined.

POCOMOKE RIVER BRIDGE, MD (11)

32. Background. This 275' long movable bascule pier drawbridge (Figure 2) was built over the Pocomoke River in 1921. 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 places. The pinpiles designed to stabilize the



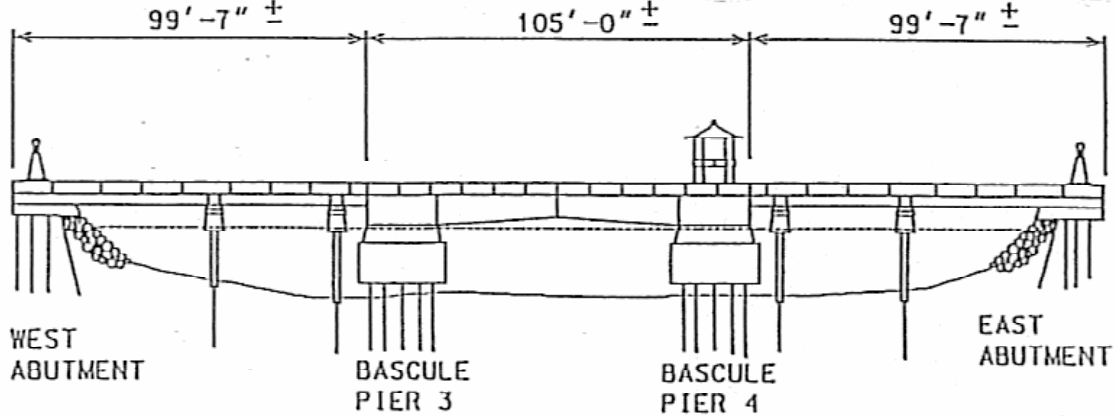


Figure 2. General configuration of Pocomoke River Bridge, MD

structure were remarkable on three counts: (i) they had to be installed through the structure and through the scour zone, (ii) they had to provide support without allowing any additional structural settlement so necessitating the use of preloading techniques, and (iii) a very intensive test program was required on special test piles to verify the concept and the performance.

33. Site and Ground Conditions. In each of Piers 3 and 4 a total of 24 piles were drilled from the bridge deck. In addition, a further 4 piles were installed from the restricted access of the Control House of Pier 4 (8x8' plan, 14' headroom) (Figure 3). The riverbed materials into which the underpinning was installed comprised a Recent sequence of alluvial sediments. The founding horizon was dense medium to coarse sand, beginning about 60' below river surface level.

34. Design and Construction. Each pile was installed as shown in Figure 4. The allowable stresses used in the design were 30% U.C.S grout and 40% of the yield strength in both the

casing and the epoxy coated rebar. To permit preloading of the pile a tendon comprising 3 ea. 0.6" dia seven wire strands was also installed through each hole, its 20' bond zone extending to 25' below the toe of the pinpile casing. Prior to drilling, grout filled bags had to be placed around the bases of the piers as framework for void filling grouting.

35. After the neat cement grout had reached 3500 psi, the tendon was stressed against the top of the steel casing, to the design load of 82 kips. The annulus between casing and structure was then grouted with special high strength grout. About 5-7 days later, the prestress was released at the tendon head, thereby allowing full structural load transfer to the pile but without obviously causing

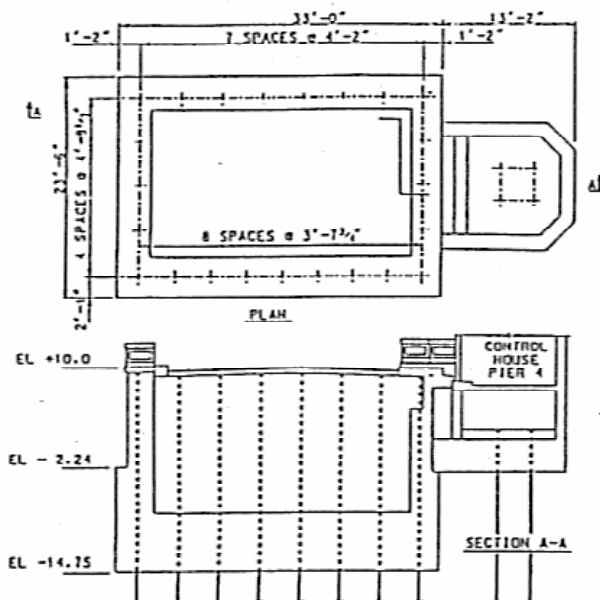


Figure 3. Plan and Section of Bascule Pier #4, Pocomoke River Bridge, MD, showing pinpile locations

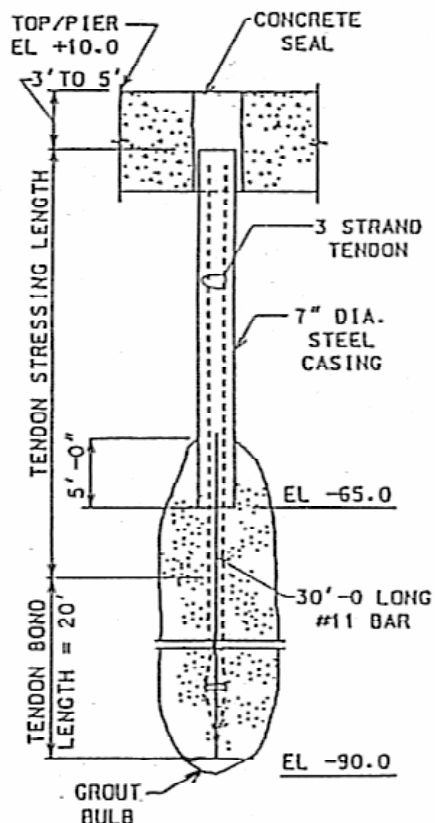


Figure 4. Typical detail of pinpile, Pocomoke River Bridge, MD

further pile compression. Slightly amended procedures had to be adopted in the restricted access of the Control House, but the same basic principles were followed, resulting in the perfect installation and functioning of all 52 pinpiles on the project.

36. Testing and Performance. 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. Each pile had an outer casing of 8-5/8" outside diameter, predrilled 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.

37. The test was in three phases:  
 Phase I: a "Preload-Unloading" test designed to verify the performance efficiency of the preloading system.  
 Phase II: a conventional pile load test to establish load-deflection performance within the scope of the specification (i.e., progressively to twice working load).  
 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 pressure 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' (bottom of bond length). These telltales were intended to directly indicate displacement at these two points in the bond zone, relative to the pile cap.

38. Phase I Tests (Preload - Unloading Test). The anchor tendon in each pile was loaded to 82 kips, creating an elastic shortening of each pile by 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. As the procedure was demonstrated to work, and since the performance was elastic, this Phase of testing was accepted as being successful.

39. 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 4. Major points are as follows:

- Performance of the piles was very similar, being virtually elastic, linear, and with minimal creep at intermediate holds.

Table 4. Highlights of load/deflection data, test piles 1 and 2, Pocomoke River Bridge, MD.

	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

- 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" 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 5.

Table 5. Comparison of net and measured elastic pile performance, Pocomoke River Bridge, MD

File 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

- \* total deflection at 200 kips less permanent deflection at subsequent zero.
- + from lower telltale.

- 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.

40. Phase III Test (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. 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 6, the pile had not yet failed.

41. Thereafter, the test set up was overhauled, and the test rerun: a maximum load of 390 kips was reached before plunging of the pile was recorded.

42. 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.

Table 6. Phase III tests, first four load cycles on test pile 2, Pocomoke River Bridge, MD.

Load cycle	Max load	Total deflection at	Creep at 15 mins at	Permanent displacement after unload	Reason for test termination
		max load	max load	after unload	
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

43. As a final point, this project represented the second time that this particular preloaded pinpile concept had been used. Some years before a structure at the Mid Orange Correctional Facility, NY, had been saved by preloaded 55 kip piles (1). In both instances, the subsequent structural movements in service have been of the order of a few thousands of inches.

44. These successes have recently encouraged the possible use of permanently preloaded piles in California to underpin transmission tower footings threatened by "rocking" actions produced by seismic events. The basic requirements of the problem can be readily satisfied with this technique:

- resistance to uplift forces in the range of 100-400 kips
- no additional compressive loads are imposed upon a new or existing footing
- no structural movement within the design capacity of the system in service
- each pile installed is tested (during application of the prestress).

#### FINAL REMARKS

45. The recent case histories described in this paper clearly underline the advances being made in pinpile technology in the United States. One can cite the unusually high load capacities, the use of preloading and postgrouting, the quality of the testing, and the growing understanding of lateral loading behavior. It is questioned if the same rate of advance is being sustained anywhere else in the world. It is queried if there is the same growth in market potential anywhere else in the world.

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