CURRENT PRACTICE IN STRUCTURAL UNDERPINNING USING PIN PILES^{sm}

by

D.A.Bruce, R.Gemme

Presented at

NY Met. Section ASCE Seminar
New York, NY

April 21-22, 1992

NICHOLSON CONSTRUCTION
P.O. BOX 308
BRIDGEVILLE, PA 15017 (412) 221-7977

92	02	70

CURRENT PRACTICE IN STRUCTURAL UNDERPINNING USING PINPILES

by

Dr. D.A. Bruce* and R. Gemme+

ABSTRACT

Pinpiles - small diameter cast-in-place bored piles - are now accepted features of many structural underpinning works in the cities of the East Coast. Pinpiles evolved in Europe forty years ago and were first used in this country only in the mid-70's. However, it is really within the last ten years or so that engineers have had the confidence to exploit widely the extraordinary advantages of this technique. This paper uses a series of recent case histories to illustrate pinpile characteristics and advantages, and the developments which are continually being made to advance the technology for common benefit.

1. INTRODUCTION

To many structural or piling engineers, the concept of a small diameter pile being required to accept an axial load in excess of 200 kips is still risible. Indeed, throughout the history of pinpiling, which dates from 1952 in Italy, it has taken a great deal of persuasive effort - and financial investment - by specialty contractors to persevere with this excellent

* Vice President, Nicholson Construction of America, P.O. Box 308, Bridgeville, PA 15017

+ Associate Soils Engineer, New York State Department of Transportation, Soil Mechanics Bureau, 1120 Washington Avenue, Albany, NY 12232

underpinning method (Lizzi, 1985; Bruce, 1988a, 88-89, 92). However, once potential clients are convinced of the practicality and advantages of the method, they are then often equally active themselves in promoting and developing the potential even further.

Small diameter, cast-in-place, bored piles are typically known in North America as pinpiles. Their use in this country dates from the mid-70's but early applications were relatively scarce due to the typical combinations of skepticism, ignorance and restricted opportunities characteristic of the introduction of many "new technologies". Since the mid-80's, however, with knowledgeable contractors, receptive owners, and market opportunities afforded by urban and industrial rehabilitation and redevelopment (Bruce, 1988a), the demand for pinpiling has grown dramatically. For example, Bruce (1992) cites, for one contractor, a total of 25 pinple projects executed up to 1988 (Table 1) but a total of 20 further projects (Table 2) executed in the two years immediately following. This upward trend has since continued.

The purpose of this paper is to illustrate current typical practice in design, construction and performance, by reference to three projects recently conducted in the New York area. In addition, details from two other projects from wider afield are described to highlight some newer developments which may be of considerable benefit to future schemes in the city.

2. LOCAL PROJECTS TYPICAL OF STANDARD PRACTICE

Details from three case histories are reviewed:

1. Coney Island, where low-medium capacity pinpiles of traditional concept were installed.
2. Brooklyn-Queens Expressway, where higher capacity pinpiles

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
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 1-1-3/8" rebar in bond zone	-------
Underpinning of existing building being redeveloped	Fill and soft clay over bouldery till	Restricted access, with 8' minimum headroom	60/120	97	4850'	50'	7"	2" high yield rebar	Two tests to 120 t total def. = 0.223" permanent def. = 0.050
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	280'	70'	5-1/2"	5-1/2" casing	-------
Foundations for new bridge abutment	Silty soil over Karstic limestone	Overhead power lines	77/235	48	1026'	29-80'	8-1/2"	9-5/8" casing to rock, 7" casing full length	At test load: total def. = 0.290" permanent def. = 0.020
Support for foundations in operational paper mill	Fills over shales with quartzitic seams	Access through doorways; minimum 12' headroom	95/190	33	1320'	40'	5-1/2"	5-1/2" casing	-------
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	-------
Foundations for exterior stairway for existing psychiatric center	loose fill overlying very compact glacial till	14x14' access to interior courtyard	5-38/ 20-75	103	2760'	26-32'	8"	1/2"-1" rebar	At test load: total def. = 0.110" permanent def. = 0.020"
Foundations for new river bridge	15' of alluvials and weathered rock over granite/gneiss	Good access, unlimited headroom	70/140	72	1901'	25'	7"	7" casing plus 1-3/8" high yield rebar in bond zone	-------
Replacement foundations for 60-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.
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	50/100	143	5291'	37'	7-5/8"	1-3/8" dia high yield rebar, plus 7" casing in upper 10' for lateral resistance	Routine use of postgrouting to enhance soil/grout bond.
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	70/250	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 Dec. 1990.
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.
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.
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 lateral testing, with postgrouting.
Foundations for pipe bridge foundations for mill expansion	20' soils over 15' shale and limestone	Through and around existing foundations	100/-	172	6020'	35'	6"	7" casing to rock, 3 ea 1-3/4" rebar in bond zone	-------
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	-------
Test pile for underpinning of major transport facility	Clayey fill over sanitary landfill over loose sand and stiff clay	Unrestricted	-/80	1	130'	130'	5"	5" casing to top of bond zone, 3 ea 5/8" dia rebars below	At failure load: total def. = 1.060" permanent def. = 0.094"
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 8-17' headroom in 3 areas	75/150	609	37500'	51-58'	7"	25-30' casing plus 25' of 1-3/8" rebar in bond zone	See text.
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	-------
Foundation for temporary highway bridge	25' of alluvials and weakened material over schist	Unrestricted	75/-	4	100'	25'	7"	7" casing	-------

Table 2. Some pinpile projects executed in the U.S. by Nicholson, 1988-1990 (Bruce, 1992). (NOTE: Grouting conducted with or without excess pressure with neat cement grouts, Type I or II at w/c = 0.45).

Location	Location/Application for foundations being underpinned	Ground conditions	Installation conditions	Load (tons) working/test	Number of production piles	Total length installed (ft)	Individual length (ft) typical/range	Nominal drilled dia. in bond zone (inches)	Construction data: Reinforcement & casing	Grouting	Test performance/special notes
Appoila, PA	New tank in existing wastewater treatment plant	Loose fill with concrete obstructions over clay over med. to v. dense sands with silt and gravel	Plant measured 38' × 48' in plan. Maximum headroom 18'	10/20	45	1350	30	5	#11 rebar in lower 20' + 5" casing in upper 15'	Type I w = 0.5 Maximum press 100 psi	Test data on 2 piles: Total displacement at 20 tons – 0.049", 0.077" Permanent displacement after – 0.008", 0.022" resp.
Brookgreen Gardens, SC	Supported masts of suspended net forming 'natural' aviary in swamp, with minimal damage to environment	Loose sands & organics over medium-dense sand	Natural cypress swamp	55 generally (15 for centre pile)	25	1174	30 to 35 for verticals 55 for rakers	5	#9 rebar full length 5" casing in upper 20'	Type I w = 0.5 Maximum press. 120 psi	Award winning solution to unique set of problems
Neville Island, PA	Existing dust collector structure on rapidly compacting soil	Loose fill over compact sand and gravel	10' to 16' headroom	30/60	32	928	29	5	#9 rebar in lower 16' 5" casing in upper 20'	Type I w = 0.5 Maximum press. 100 psi	Test data on 1 piles: Total displacement at 60 tons – 0.078" Permanent displacement after – 0.01" (Allowable 0.60")
Providence, R.I.	Test to assess viability of underpinning existing granite block seawall	Quay, bearing on silt, sand and till overlying sandstone bedrock	Open air	55/110	1 (Test)	65	65	6	5" casing for 57'	Type I w = 0.45 Gravity fill	Test data on the one pile. Total displacement at 110 tons – 0.70" Permanent displacement after – 0.03"
Trafford, PA	New printing press in existing building	Loose cinder fill over silty clay and weathered shale bedrock	14' headroom	10/20	20	720	36	5	5" casing full length	Type II w = 0.5 Maximum press. 100 psi	Test data on 1 pile: Total displacement at 20 tons – 0.065" Permanent displacement after – 0.005"
Warwick, NY	Existing gymnasium building (use of preloaded piles)	Loose sandy silt and glacial till becoming denser with depth	Minimum headroom 20'	27.5/55	62	4030	65	5	2 No 0.6" dia. strands (for preloading 5" casing in upper 40'	Type I w = 0.45 Maximum press. 120 psi	Test data on 2 piles: Total displacement at 55 tons – 0.188" and 0.249" Permanent displacement after – 0.002" and 0.005" resp.
Monessen, PA	Existing operating coke battery, emission control facility	Fill over clayey sand and gravel	19' to 25' headroom	50/100 (comp) 35 or (tension) 45	102	6330	55 and 65	5	#7 rebar full length 5" casing for all except lower 10'	Type II w = 0.45 Maximum press. 100 psi	Test data on 1 pile: Total displacement at 100 tons – 0.312" Permanent displacement after – 0.008"
Mobile, AL	Two existing sodium hydroxide storage tanks under which wood piles had failed	Soft organic silt and clay over dense sand with gravel	Very restricted access. 8' to 15' headroom. Caustic chemical spills	34 54	171 7	9600 400	56 Range 46 to 60	5 6 5/8	5" or 6 5/8" for full length except lower 8'	Type I w = 0.5 Maximum press. 80 psi	Piling part of major overall structural repair.
Burgettstown, PA	Existing gantry runway	Slag, silty sandy clay & shales over sandstone and limestone	Maximum headroom 24'. Soil saturated with sulphuric acid	10	20	640	32	4 (for 3' rock socket)	3½" casing full length	Type II w = 0.45 Maximum press. 40 psi	—
Dunbar, PA bedrock.	Addition to water treatment plant	Fill over fine sand and sandstone	Open air	45	7	179	26 (Range 25 to 26)	5	#6 rebar for lower 10' 5" casing for upper 20'	Type III w = 0.45 Gravity pressure	—
Pittsburgh, PA	Existing structure adjacent to deep excavation	Fill and fine alluvials over dense sands & gravels with trace silt	Open air	50	21	630	30	5	5" casing for upper 20'	Type I w = 0.5 Maximum press. 60 psi	Piles installed in conjunction with subhorizontal soil nails for excavation stability.
Pittsburgh, PA	Existing parking garage	Fill and alluvials over sandstone/siltstone bedrock	8' to 10' headroom	55	46	1980	43 (Range 38 to 44)	5	5" casing to rock head	Type I w = 0.45 Gravity pressure	—
Aliquippa, PA	New emission control building at existing coke battery	Slag fill over dense sand & gravel	25' headroom	50/100 (comp) 75/150 (tension)	31 8	2170 600	70 75	5 5	#6 rebar for lower 25' 5" casing for upper 50'	Type I w = 0.45 Maximum press. 120 psi	Test data on 1 pile: Total displacement at 100 tons – 0.2" Permanent displacement after – 0.02"
Jeanette, PA	New machine in existing building	Fill, silts and clay over bedrock	20' headroom	Total of 150 tons of structural weight supported	27	945	35	5½	5½" casing full depth	Type I w = 0.45 Gravity pressure	—
Appoila, PA	New nuclear power structure in existing building	Loose fills with clay over medium sands with gravel	20' headroom	10	24	552	23	5½	#7 rebar full depth 5½" casing for upper 18'	Type III w = 0.45 Maximum press. 150 psi	—
Marion, IN	Existing body stamping plant	Silty sand over rock	18' headroom	60	24	1680	70	7	7" casing for upper 50' #11 rebar for lower 25'	Type I w = 0.45 Maximum press. 50 psi	—
Alcoa, TN	Data not available										
Washington DC	Existing structure at Castle Building, Smithsonian Institute	Fill over dense sands with gravel	Very restrictive access and hole entry conditions	50/100	21	1580	75 (range 69 to 77)	5½	#11 rebar full depth 5½" casing between footing and bond zone	Type I w = 0.5 Maximum press. 140 psi	1 Piles combined with subhorizontal soil nails to stabilize excavation adjacent to structure. 2 Data on Test Pile 2: Total displacement at 100 tons – 0.653" Permanent displacement after – 0.078"
Pittsburgh, PA	Restoration of existing Timber Court Building	Sands and gravels, over sandstone bedrock	10' headroom	50	15	1050	70	5½	5½" casing full length	Type I w = 0.45 Gravity pressure	—
Warren Co., NJ	New bridge pier	Karstic limestone with voids and gouge	Open air, small area	100/224	24	1889	78 (Range 44 to 200)	8½	7" casing full length	Type III w = 0.5 Maximum press. 50 psi	Test data on 1 pile: Total displacement at 205 tons – 0.40" Permanent displacement after – 0.07"
Kingsport, TN	New storage tank in existing building	Silts and sands over limestone	11' headroom	40/80	115	4025	35	5½	#8 rebar in lower 15' 5½" casing to bedrock	Type I w = 0.45 Gravity pressure	No measureable permanent displacement after testing to 80 tons.
Boylston St. Boston, MA	Existing building being redeveloped	Soft fills and organics over medium dense sand	Minimum headroom 8' in very restrictive basement conditions	40/92 (comp) 12/27 (tension)	262	7070	27	5½	#8 rebar full length 5½" casing in upper 19'	Type II w = 0.5 Maximum press. 60 psi	Test data on 2 piles: Total displacement at 92 tons – 0.44" and 0.34" Permanent displacement after – 0.25" and 0.16"
Ann Street Pittsburgh, PA	To support new soldier beams for new retaining wall	Weathered shale & sandstone over competent sandstone	Open air	45/68 (comp) 8/12 (lateral)	86	1000	11.5	6	#11 high strength rebar full length	Type I w = 0.45 Gravity pressure	1 Piles subjected to vertical, lateral and moment testing. 2 Compression test data on 6 piles: Total displacement at 68 tons – 0.059 to 0.097" Permanent displacement after – 0.006 to 0.020"
Coney Island, NY	Rehabilitation of existing repair shop	Fill and organic silt over dense sands	Minimum headroom 8'. Very difficult access in fully operational facility	15/30 and 30/60	2300 1900	80 500 85 500	35 45	6 5/8 7 5/8	#6 rebar full length #9 rebar full length	Type I w = 0.45 Maximum press. 60 psi	Extensive test programme (see text):
Cleveland, OH	New addition to existing control building	Slag fill and soft silty clay over shale bedrock	Open air but difficult access due to ongoing steel plant operations	60	45	6390	142	6½ (for 5' rock socket)	7" casing to rock head #8 rebar for 5' rock socket & 10' into casing.	Type I w = 0.45 Gravity pressure	—

Table 1. Some pinpile projects executed in the U.S. by Nicholson, 1978-1988 (Bruce, 1988).

of more contemporary concept were installed in soils, and

3. Miller Highway, where similar elements were installed to rock.

2.1. Coney Island

• Background

The Coney Island Main Repair Facility of the New York Transit Authority has been in operation for over 70 years and is the largest of its kind in the world. It encompasses, including the rail yards, about 100 acres, of which 12 acres are covered building space. Constructed on the former Coney swamp, the repair shop was built on a loose fill surface with no pile support for the floors. The steel frame, columns and outside walls were supported on piled foundations.

Settlement has produced major underfloor voids which have led to many floor collapses such as an 18" drop in the main shop in 1980. During the original construction, the swamp filling had apparently created mud waves causing uneven thicknesses of the soft organics underlying the structure. The subsequent settlement of the ground surface due to the loading by the fill and the structure has thus been irregular in magnitude across the site.

After "Years of Band-Aids" (Munfakh and Soliman, 1987) a $100 million repair program was initiated in 1984 coincident with the installation of new equipment, the weight of which alone have accelerated the settlement problem. Foundation repair had to be carried out in a fashion guaranteeing minimum disruption to continuing shop operation, as well as constituting a proven, compatible and cost effective solution.

Remedial options under consideration included compaction grouting, chemical grouting, and concrete filled steel shell piles. However, conventional pinpiles proved to be the most attractive solution from all viewpoints, and a contract was let in early 1987 to install over 4200 piles. The general contractor was A.J. Pegno Construction Corp., the construction manager was the Greiner/MKE Team and Parsons Brinckerhoff Quade & Douglas acted as the architect/engineer.

• Site and Ground Conditions

Four distinct soil layers were identified under the slabs: fill, peat with organic silt, grey sand and brown sand. Short and long term consolidation testing confirmed the organic layers to be the cause of the settlement. These strata experienced long term secondary consolidation and peat/organic degradation, either from oxidation or micro organisms. Typically the medium dense, fine sands recognized as being adequate load bearing materials commenced 10' to 25' below the surface. The piezometric level was at about -4'.

Access and headroom conditions were always restrictive and frequently obstructive, being as little as 8'. In addition, as the work was to be carried out in a busy, fully operational facility, in collaboration with other major structural repairs, it had to be executed in restricted "packages" in a piecemeal fashion.

• Design

Approximately 2300 number 15 ton working load piles and 1900 number 30 ton piles were required. The Engineer's conventional pinpile design allowed for the load to be taken on #6 bars, without the addition of sacrificial steel casing in the upper zones wherein resistance to buckling was analyzed and judged adequate.

Standard grout-soil design procedures (Littlejohn, 1980), based on $\phi = 30^\circ$ were used to arrive at total lengths of 35' and 45' for 15 ton and 30 ton piles respectively, i.e. 10' or 20' into the load bearing sand.

- <u>Construction</u>

Before installing the piles, the existing voids were filled with a lightweight foamed concrete. It was intended that its light weight would inhibit additional settlement and corresponding downdrag forces to the piles. The fill would also protect against erosion by blocking water flow through such voids.

The access and headroom restraints over much of the site demanded the use of specially constructed drilling equipment featuring short masts and remote power units. Whenever possible, conventional crawler mounted units were employed with special care having to be taken in all cases with exhaust fumes and drilling spoil disposal.

The 15 ton piles were drilled and cased to 6-5/8" nominal diameter and the 30 ton piles to 7-5/8" nominal diameter. Water flush was used. This casing was completely withdrawn during the pressure grouting of the bond length (using neat Type 1 cement with w/c = 0.50) to a maximum of 60 psi, following the placing of the reinforcing bar (#6 or #9 rebar full length).

Load transfer to the existing slab structure was provided by an underreamed supporting zone formed under the slab (<u>Figure 1</u>).

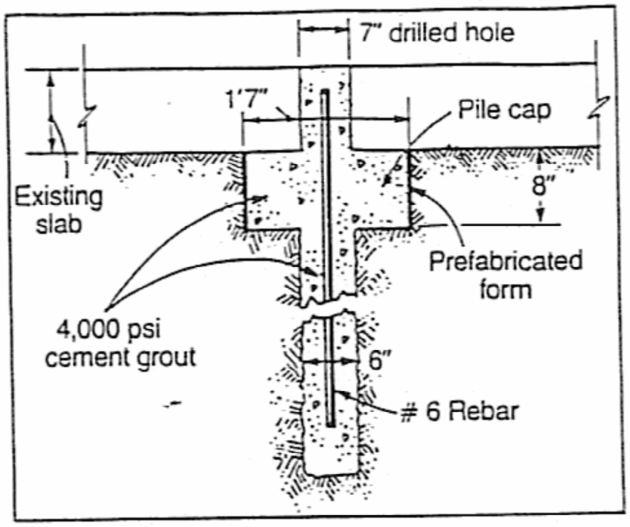


Figure 1. Schematic arrangement of 35' long, 15 ton pinpile and existing base slab. Coney Island, N.Y. (Munfakh and Soliman, 1987).

- Testing and Performance

Ten full scale test piles were used to verify assumptions regarding design and performance. PVC liners were provided from the slab to the top of the sands to ensure transfer of load only in the lower horizons. In the first three compression tests the load was applied directly to each pile via a beam/reaction anchor system. Munfakh and Soliman (1987) reported that the high concentration of stress crushed the top portion of each pile. The remaining test piles were given an enlarged cap providing better load transfer to the grout and reinforcement. Load tests were run to twice working load in compression, and to 50 tons in tension.

The steel casing was left in place in one pile (number A/8) so that a performance comparison with the standard pile (number A/9) could be obtained (Figure 2).

The first four piles (Table 3) experienced significant creep at maximum load (up to 0.35" in 4 hours) whereas those tested through the cap had less (0.032" to 0.064" in 4 hours at 30 tons). The cased pile had less than half this amount of creep in 5 hours at 30 tons.

Such performances were acceptable to the structural designers and the benefits of the cased pile were not required in the production piles subsequently installed.

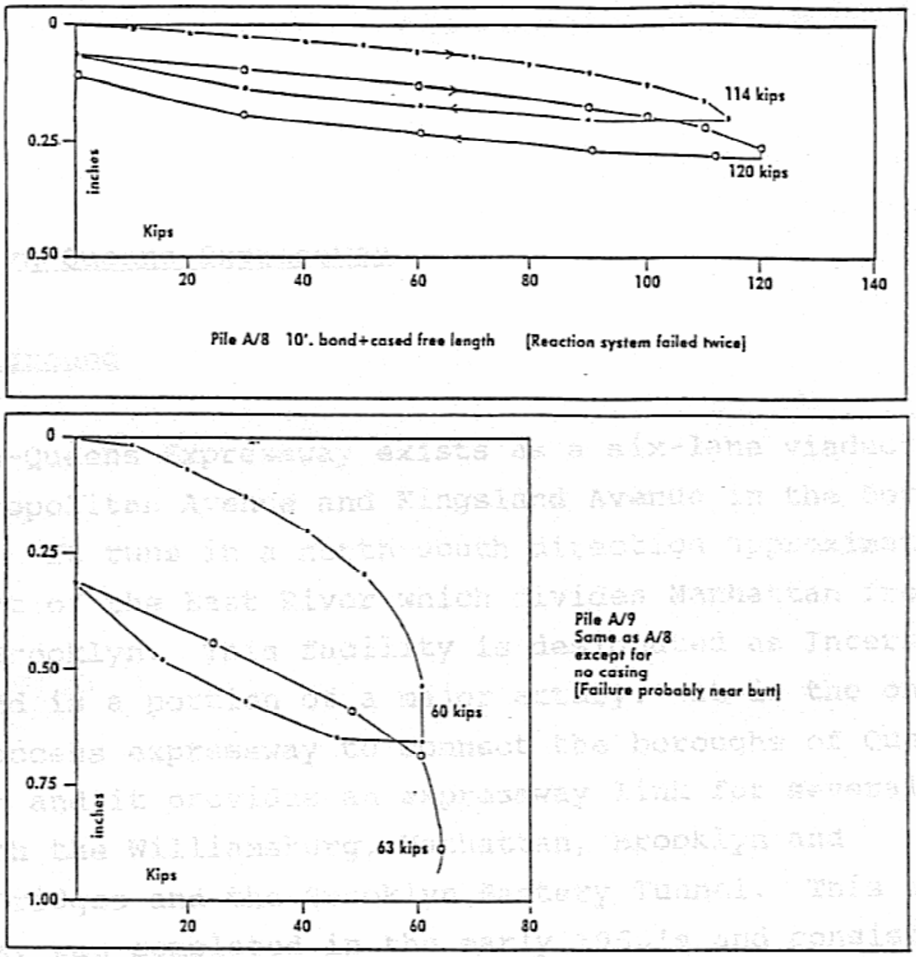


Figure 2. Comparative performances of two 35' long test piles, in identical geological conditions, but with and without upper casing. Coney Island, N.Y. (Bruce, 1988).

Pile #	Description	Ratio of grout volume to hole volume	Stiffness in *lineal part (tons/inch)	Max. load & total accum. deflection (tons)	(inches)	Notes
A/3 A/4	Loaded annulus only	1.2 3.7	80 85	20 (F) 31 (F)	1.25" 0.65"	Failure premature and most probably due to crushing of pile head.
A/5 A/9	Loaded full section	2.5 2.9	95 72	29 (F) 31 (F)	0.75" 0.85"	Failure possibly due to soil/grout failure although distress at head also noted.
A/7	Includes original conc. slap in cap	2.9	303	70 (F)	0.90"	Soil-grout failure likely.
A/10	Excludes conc. slab in cap	3.4	178	56	0.42"	Test suspended upon failure of pile cap.
A/8	With sacrificial casing for 25'	7.7	385	60	0.30"	Test suspended when reaction pile pulled.

* A measure of pile stiffness obtained by dividing the maximum load over which displacement is relatively linear, by the displacement at that load.

Table 3. Comparative performance of 15 ton piles, Coney Island, N.Y. (All piles were 6-7/8" in diameter, 35' long including 10' bond, and had a full length #6 reinforcing bar.) (Bruce, 1988).

2.2. Brooklyn-Queens Expressway

- Background

The Brooklyn-Queens Expressway exists as a six-lane viaduct between Metropolitan Avenue and Kingsland Avenue in the Borough of Brooklyn. It runs in a north-south direction approximately one mile east of the East River which divides Manhattan from Queens and Brooklyn. This facility is designated as Interstate Route 278 and is a portion of a major artery. It is the only controlled-access expressway to connect the boroughs of Queens and Brooklyn and it provides an expressway link for several counties with the Williamsburg, Manhattan, Brooklyn and Verrazzano Bridges and the Brooklyn Battery Tunnel. This section of expressway was completed in the early 1950's and consists of a series of simply supported spans resting on pile supported bents.

A major improvement program was put into place to replace the deck of the viaduct and to add a new center lane and several new entry-exit ramps. These were needed to correct access, geometric and safety deficiencies which were exacerbated by severe traffic congestion experienced particularly during rush hours.

Due to this viaduct being a portion of a major arterial highway with high traffic volumes, maintaining traffic became a fundamental criterion for project approval. In order to maintain a minimum of two (out of three) lanes of traffic in each direction during construction, a temporary viaduct, adjacent to the existing structure had to be constructed prior to lane closures in each direction to accommodate rehabilitation.

Small diameter (approximately 12 inch) bored piles were specified for the permanent viaduct and ramps, and larger diameter (24 inch, 30 inch and 36 inch) bored piles were specified for the temporary viaduct. Bored piles were specified for this project due to the otherwise adverse vibration effects that pile driving impact hammers would have on the many adjacent old and sensitive buildings.

This was the first New York State Department of Transportation project where small diameter bored cast in place piles (pinpiles) were specified to be designed by a prequalified contractor to meet predetermined design capacities. The general contractor was Yonkers Contracting Company, Inc. who sublet the installation of most of the pinpiles to Nicholson.

• Site and Ground Conditions

The general foundation conditions at the site were highly variable, but generally consisted of

- 10 to 15 feet of loose to medium compact miscellaneous fill containing silt, sand and gravel with bricks and the like;
- up to 30 feet of layers and lenses of loose silt, sand and clay (organic near surface);
- up to 50 feet of compact silty sand, occasionally gravelly;
- stiff varved silty clay and clayey silt (Gardiners clay).

Bedrock was not encountered to the maximum explored depth of about 100 feet. Generally the compact silty sand and lower reaches of the lenses of silt, sand and clay were recognized as being adequate load bearing materials commencing at a depth of about 50 feet below the existing ground surface. The piezometric level was encountered between 10 and 15 feet below the existing ground surface.

There were some access and headroom restraints particularly where drilling had to be performed under the existing viaduct (about 17' headroom). In addition, construction had to accommodate traffic control, protection of buried and overhead utilities and noise and vibration impact mitigation.

• Design

Approximately 120 new pile caps, each with between 2 and 10 piles per cap were proposed as follows in different stages to accommodate maintenance and protection of traffic:

CONSTRUCTION STAGE	NUMBER OF NEW PILE CAPS	PILE DIAMETER	DESIGN CAPACITY (TONS)
1 (eastbound permanent viaduct)	Approx. 30	Approx. 12"	50, 80, 100
2 (temporary viaduct)	Approx. 60	24", 30", 36"	Variable 60 to 145
5 (westbound permanent viaduct)	Approx. 30	Approx 12"	50, 80, 100 (to be installed)

All piles were to be designed without batter. The small diameter piles were specified to be designed as friction piles by a prequalified contractor to meet the design capacities. The prequalification consisted of requiring the contractor performing the work to submit proof of: 1) two projects on which he had successfully designed and installed similar bored piles or tiebacks, using non-displacement methods under similar site conditions; and 2) the foreman having supervised the successful installation of the same on at least two projects in the past two years. The specifications indicated that the grout mix, steel casing and/or reinforcement had to meet specified minimum requirements and included general provisions concerning shop drawing submittals, drilling, casing removal, post grouting, and construction tolerances (Appendix 1). The contractor's proposal to found the temporary viaduct on small diameter bored in piles in lieu of the larger diameter piles was approved by the State with the provision that the piles be battered where permitted by right of way and utility conditions and the pile caps be tied to the permanent viaduct in the direction where piles could not be battered to provide the necessary lateral restraint.

The specification called for completing several successful static pile load tests as a basis for pile acceptance:

	On non-production piles prior to installation of production piles	On production piles
Permanent Viaduct	4	Bents 19, 26, 31, 37, 44 and at 3% of remaining at locations designated by the Engineer.
Temporary Viaduct	2	Bents 27, 37, 50, 69, and at 1% of remaining at locations designated by the Engineer.

The following pile designs and general installation procedures were submitted by Yonkers and Nicholson and were approved by the state contingent upon obtaining successful pile capacities from the static pile load tests.

YONKERS

9.05" O.D. casing is advanced using duplex drilling with air flush. Klemm drill counter-rotates the casing and 4" inner drill rods. (Double Head Duplex: Bruce (1989))

Install #8 or #18 grade 60 rebar after drill rods are removed.

Place grout hose to bottom and place 5000 PSI neat cement grout.

Pressurize grout with air to 100 PSI as casing is withdrawn about 35 feet.

Inject the top section of pile under moderate pressure as casing is removed.

NICHOLSON

7" O.D., 1/2" wall grade N 80 casing is drilled in using water flush.

Place grout tremie to bottom and place 5000 PSI neat cement grout.

Pressurize grout to about 80 PSI as casing is withdrawn 15 feet.

Readvance casing to bottom to serve as reinforcement.

- Construction

Piles were installed as foreseen, and subsequent testing (below) showed that, when founded in silty sands, the 100 ton design capacity was readily attained. However, when the bond zone predominantly consisted of looser deposits of silt, sand and clay, the guaranteed minimum design load was estimated as 60 tons. This inability to reach high loads reflected the fact that the soils would not naturally "seal" around the drill casing, so preventing the application of the target grouting pressures. A test with postgrouting techniques (Section 3) did raise grout/soil bond capacities to a level capable of providing the original design load. However, the contractor, given the overall site and project restraints, proposed instead to derate the pile design capacity in these areas to 60 tons and to install more piles (at no extra cost to the State). This proposal was found acceptable, and so piles typically varied from 50-60 feet, although in one area they were taken deeper so as to avoid extra loading on existing subway tunnels.

All piles were loaded cyclically, incrementally in order to provide data on both elastic and permanent displacements (Bruce 1991). This was done basically in conformance with Soil Control Procedure SCP-4/77. To date a total of 18 tests have been performed (9 non-production and 9 production piles) as summarized in Table 4.

Regarding permanent movements, Figures 3 and 4 summarize the performance of piles founded in the silty sands, and clayey silts respectively. Figures 5 and 6 summarize the creep data for the same groupings of piles.

As shown in Table 4, the pile at Bent 27C was postgrouted after initial testing (Figure 7). One may compare the original maximum achieved load of 175 tons (permanent deflection 0.7"), to the subsequent, easily attained load of 200 tons (0.2" permanent deflection). After postgrouting, the creep was 0.012" during the last 4 hours at 200 tons.

One lateral loading test to 20 kips gave a total deflection of 0.76", and a permanent displacement of 0.14".

TABLE 4

SUMMARY OF PILE TESTS, BQE, N.Y.

LOCATION	BETWEEN BENTS 35 & 36	BETWEEN BENTS 38 & 39	BETWEEN BENTS 38 & 39	BETWEEN BENTS 30 & 31	BETWEEN BENTS 15 & 16	BENT 34	BENT 31	BETWEEN BENTS 12 & 13	BETWEEN BENTS 12 & 13	BENT 16	BENT 26	BENT 11B–C	BENT 26C	BENT 28C	BETWEEN BENTS 9 & 10	NEAR BENT 1	BENT 27C	BENT 82
LOAD TEST NO.	TP–1	TP–2	TP–3	TP–4	TP–5	P–1	P–2	1	2	3	4	7	5	6	–	–	8	T–1
ORDER NO.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
PERM. OR TEMP.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	TEMP
PROD. OR NON–PROD.	NON.	NON.	NON.	NON.	NON.	PROD.	PROD.	NON.	NON.	PROD.	PROD.	PROD.	PROD.	PROD.	NON.	NON.	PROD.	PROD
AGE AT TEST (DAYS)	37	13	N/A	14	29	N/A	N/A	5	6	5	7	19	7	7	N/A	N/A	7 3 POST GRT.	8
NOMINAL DIA. (INCHES)	9.05	9.05	9.05	9.05	9.05	9.05	9.05	7	7	7	7	9.05	7	7	9.05	9.05	7	7
NOMINAL BOND LENGTH (FEET)	35+/–	35+/–	35+/–	35+/–	35+/–	35+/–	35+/–	15	18	7	6	20	20.5	21	35+/–	35+/–	15	30
TOTAL LENGTH (FEET)	57	59	N/A	65	78	N/A	N/A	40.5	50	50.5	50	50.5	62	66.5	61	61	50	50
ORIGINAL DESG. LOAD (TONS)	50	100	100	100	100	50	75	100	100	100	100	100	100	100	100	100	60	60
MAX. TEST LOAD (TONS)	186	146	125	175	275	100	150	165	200	200	125	150	160	125	175	200	175 200 POST GRT.	120
TOT. DEFLEC. @ MAX.LOAD (INCHES)	N/A	N/A	0.51	0.86	0.36 @200T	0.29	1.2	1.7	0.51	0.42	1.85	0.77	0.43	0.38	2.7	0.58	1.24 0.72 POST GRT.	0:1
PERM. DISPL. AFTER MAX LOAD (INCHES)	N/A	N/A	FAILED	FAILED	0.14 @200T	0.10	FAILED	FAILED	0.13	0.13	FAILED	0.44	0.12	0.13	2.2	0.10	0.74 0.20 POST GRT.	0.03

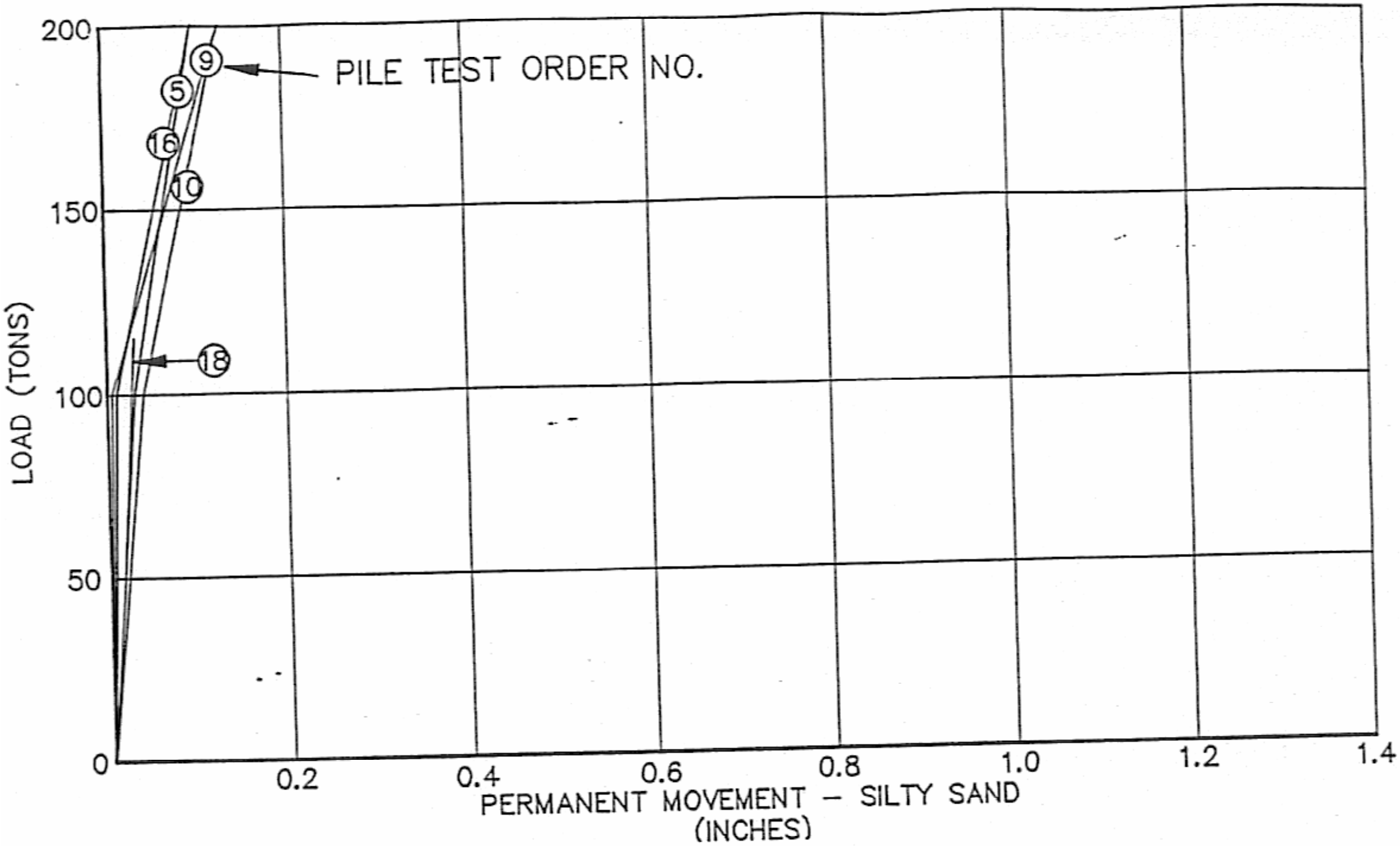


Figure 3. Permanent displacements of test piles founded mainly in silty sand. BQE, N.Y.

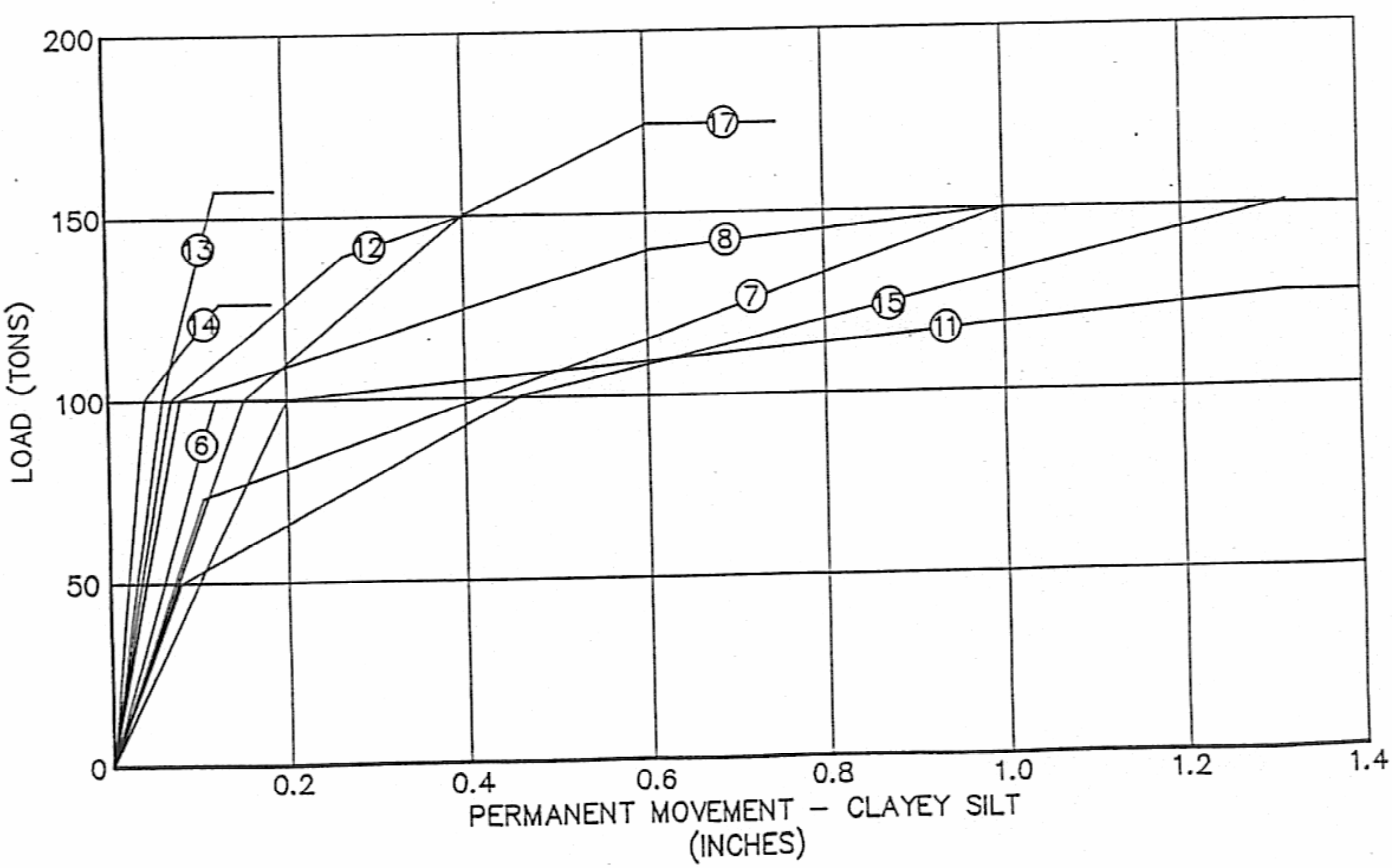


Figure 4. Permanent displacements of test piles founded mainly in silts. BQE, N.Y.

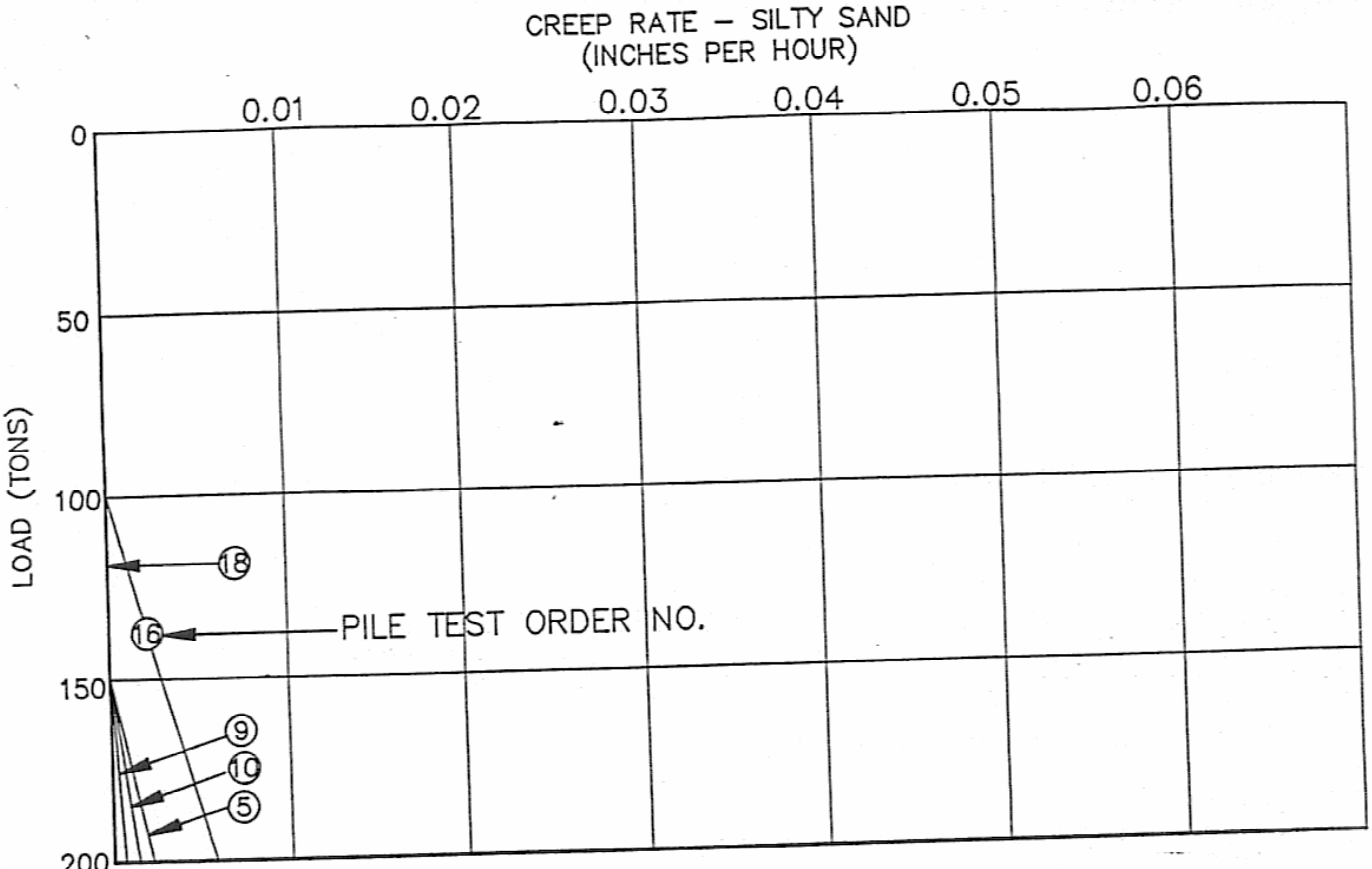


Figure 5. Creep rates, test piles in silty sands. BQE, N.Y.

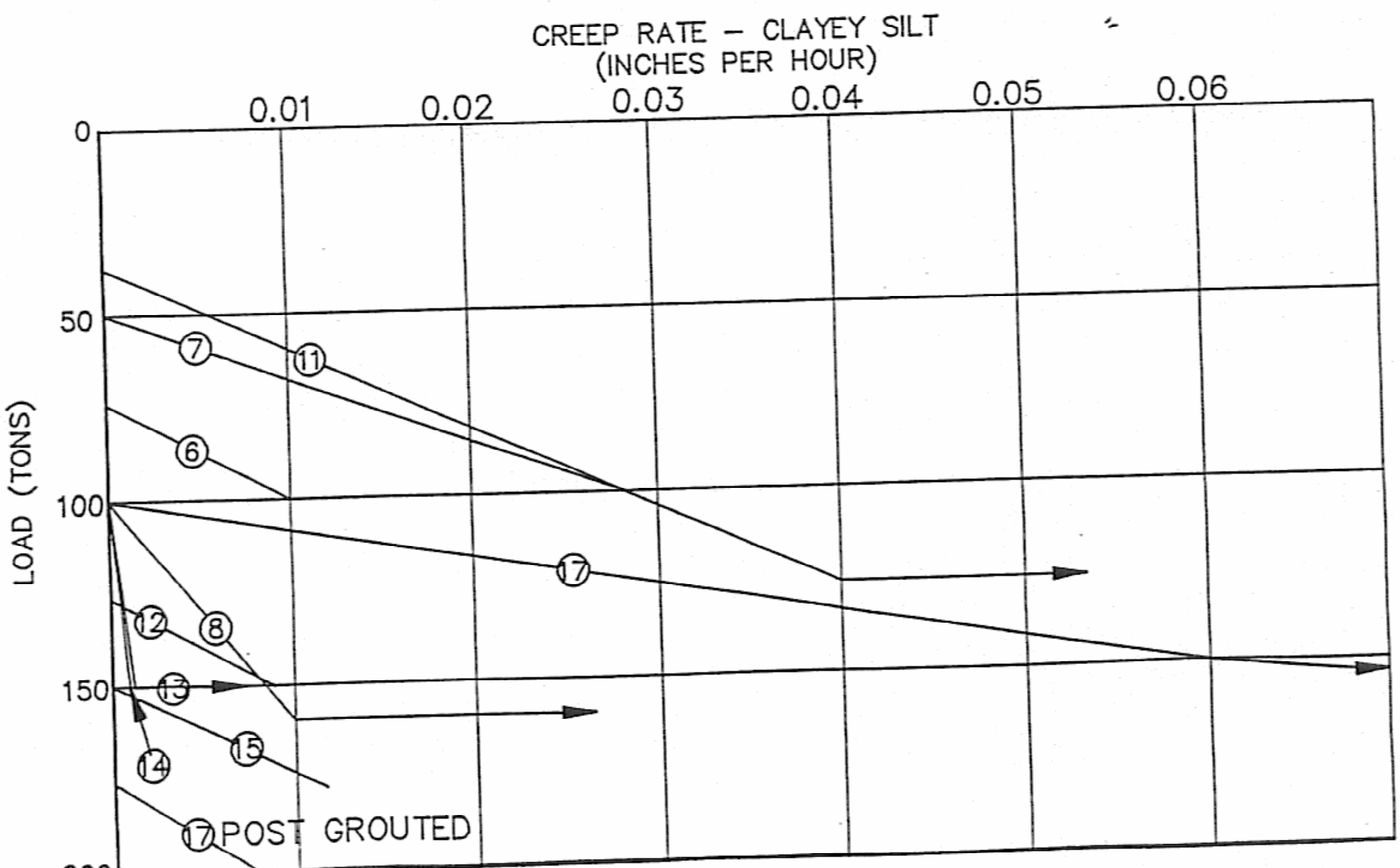


Figure 6. Creep rates, test piles in clayey silts. BQE, N.Y.

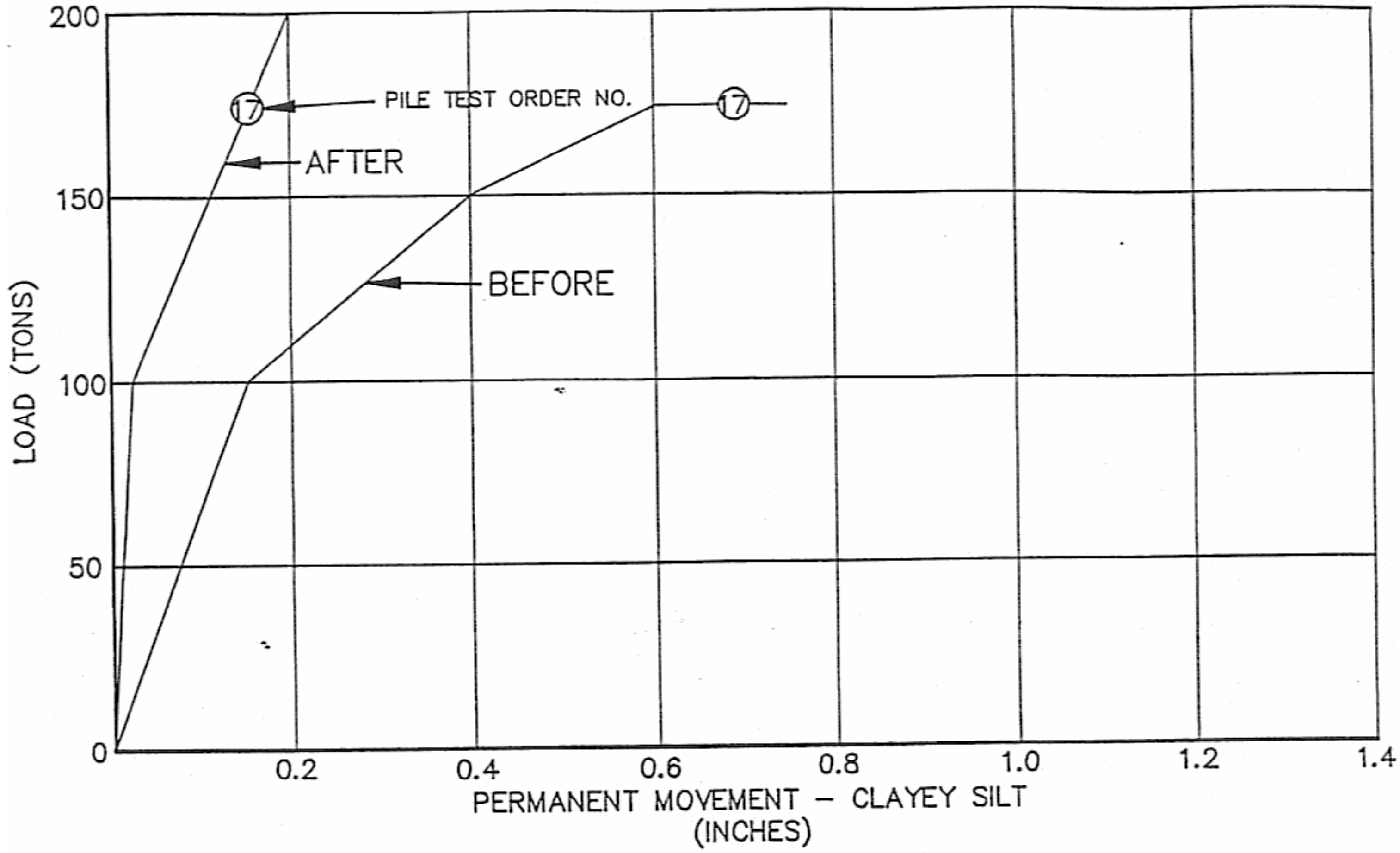


Figure 7. Performance of test pile at Bent 27C, in clayey silts, before and after postgrouting, BQE, NY.

2.3. Miller Highway

• Background

The Miller Highway Viaduct is a southern extension of the Henry Hudson Parkway and runs adjacent to the Hudson River, west of Central Park. It was built in the 1930's and is a rivetted plate girder structure resting on columns. Very heavy traffic and natural corrosion had caused a severe deterioration.

A major improvement program was therefore put in place to rehabilitate the viaduct between West 57 and West 72 Streets. This involved demolition, widening (an extended 7'7" cantilever on the deck), straightening of the route (at the southern end) and two new entry/exit ramps.

Pinpiles were specified for the ramp abutment structures, and were the preferred option - as opposed to 42" dia caissons - to underpin new pile caps and piers.

The general contractor was Yonkers Contracting Company, Inc.

- Site and Ground Conditions

Although none of the work had to be conducted under existing structures, construction had to accommodate the logistical and spatial restraints typical of such work: traffic control, construction impact mitigation, close phasing of the various trades. Special care had to be taken with locating and protecing buried and overhead services. Ground conditions were extremely variable and their foreseen difficulty to penetrate was a prime argument in deciding against the use of the large diameter caissons.

Normal urban fills with bricks, concrete and massive granite seawall blocks were usually found over stone filled timber cribs and timber piles. These conditions extended as far as 35' below the asphalt/concrete/block rubble surface layers. Various thicknesses and types of alluvial deposits (including sands, silts and clays) overlaid the rockhead which, dipping to the west and north, varied in depth from less than 40' to over 80' across the site (most commonly around 50'). The rock consisted of typical Manhattan schist lithologies (including pegmatites and gneisses) and was often distinctly weathered in the upper 5-10'.

- Design

The 17 new caps each had four pinpiles, battered outwards at 1:6, each with nominal individual design loads of up to 138 kips. The six piers each had 4 or 8 battered piles of similar working loads. The 20 piles for the Northbound Entry Ramp Abutment and the Southbound Exit Ramp Abutment required design working loads of 200 kips per pile (vertical and battered).

The pile design incorporated a 7" o.d., 1/2" wall Grade N 80 casing to rockhead, and a 5-1/2" dia socket into fresh rock below. For the 138 kip piles, the embedment was 3'-6" (based on an allowable grout/rock bond of 200 psi and the reinforcement consisted of one 1-1/4" dia plus one 1" dia Grade 150 reinforcing bar. The larger capacity piles had a rock embedment of 5' and were reinforced by a pair of 1-3/8" dia Grade 150 bars. The bars in both designs extended a minimum of 10' up into the casing.

Using Type 2 cement, and a w/c ratio of about 0.45, the target grout strength of 4000 psi at 28 days was substantially surpassed.

The foreseen quantities of piling were 775 l.f. for the abutments and 6400 l.f. for the other work.

- Construction

Holes were rotary drilled with a high torque truck mounted drill rig. Various methods were experimented with in order to reach rock head but the most successful procedures involved

opredrilling with a 7-7/8" dia roller bit (with or without water flush) to penetrate the obstructed fill;

ofollow with casing and shoe (wet or dry) before switching to duplex (wet);

odrill on into fresh bedrock with the rock roller.

Grout was tremied into the casing and the reinforcement placed. Overall total individual hole lengths varied from 40 to 92'.

- Testing and Performance

No pile load testing was conducted: each pile was accepted on the basis of satisfactory construction, and especially the founding of the socket in sound rock, as determined by the New York State DOT geologist.

3. PROJECTS FURTHER ILLUSTRATING NEW DEVELOPMENTS

Most recently, attention has been focused within the research and development departments of pinpile specialists into three main subjects:

(1) Improved understanding of load transfer mechanisms within pinpiles;
(2) Enhancing grout-soil bond by postgrouting; and
(3) Preloading pinpiles to improve pile stiffness and response.

Regarding item (1), a very recent project in Mobile, AL (Bruce, et al, 1992) has shown that when sufficient bond can be developed between grout and soil, the "weak link" in the pile can be its internal strength. In other words, the load carrying potential of certain high capacity pinpiles (over 500 kips) is being limited by the material strengths of the grout and the steel, and their interaction. Research is continuing into this phenomenon (Kenny, 1992) to better understand the mechanisms, and so permit logical designs and selections of more efficient elements.

The principle of enhancing grout-soil bond by postgrouting has long been expounded (Jones and Truner, 1980; Herbst, 1982; Mascardi, 1982; Bruce, 1991). Grouts are injected through a separate grouting tube (Figure 8) or through the steel core pipe itself (Figure 9) at regular intervals by a double packer. The

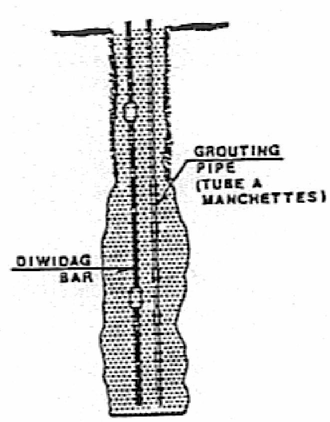


Figure 8. GEWI pinpile (After Mascardi, 1982).

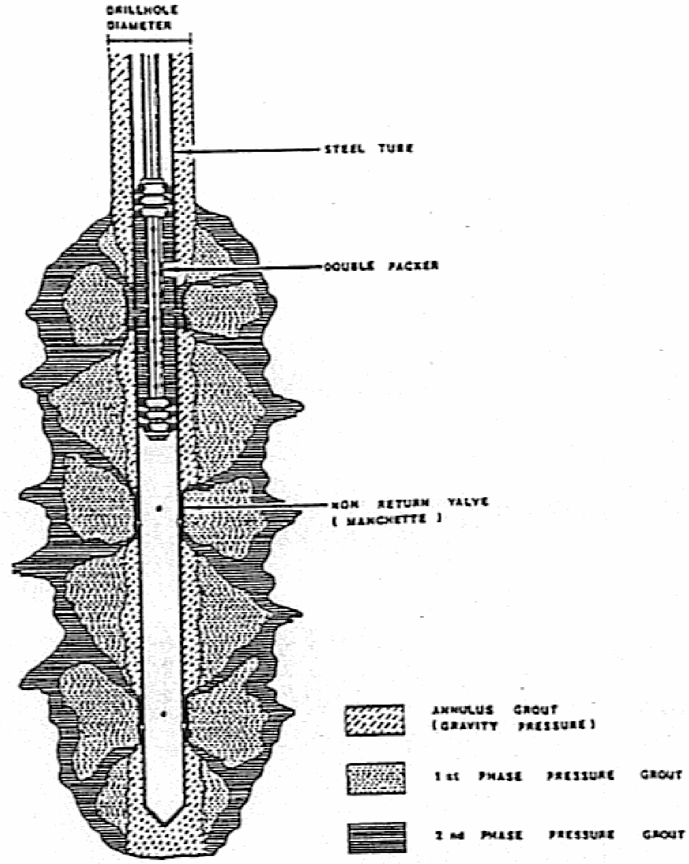


Figure 9. Concept of repeated postgrouting through the steel core to enlarge effective grouted diameter (Mascardi, 1982).

grouting improves soil-grout bond (Figure 10), and may increase the nominal pile cross section, especially in weaker soil layers. PTI (1986) indicates for ground anchors an enhancement potential of 20-50% in both cohesive and cohesionless soils. Postgrouting can

- allow higher loads to be sustained for similar pile dimensions;
- allow equal loads to be sustained for reduced pile dimensions;
- allow "failed" piles to be "repaired" to safely reach target working loads.

The project recently completed at Augusta, GA, (Section 3.1.) is a clear example of the benefits of the method, and compliments the result obtained at Brooklyn Queens Expressway.

With respect to preloading, the concept is to cause compression of the pile (elastic and permanent) prior to connecting it to the

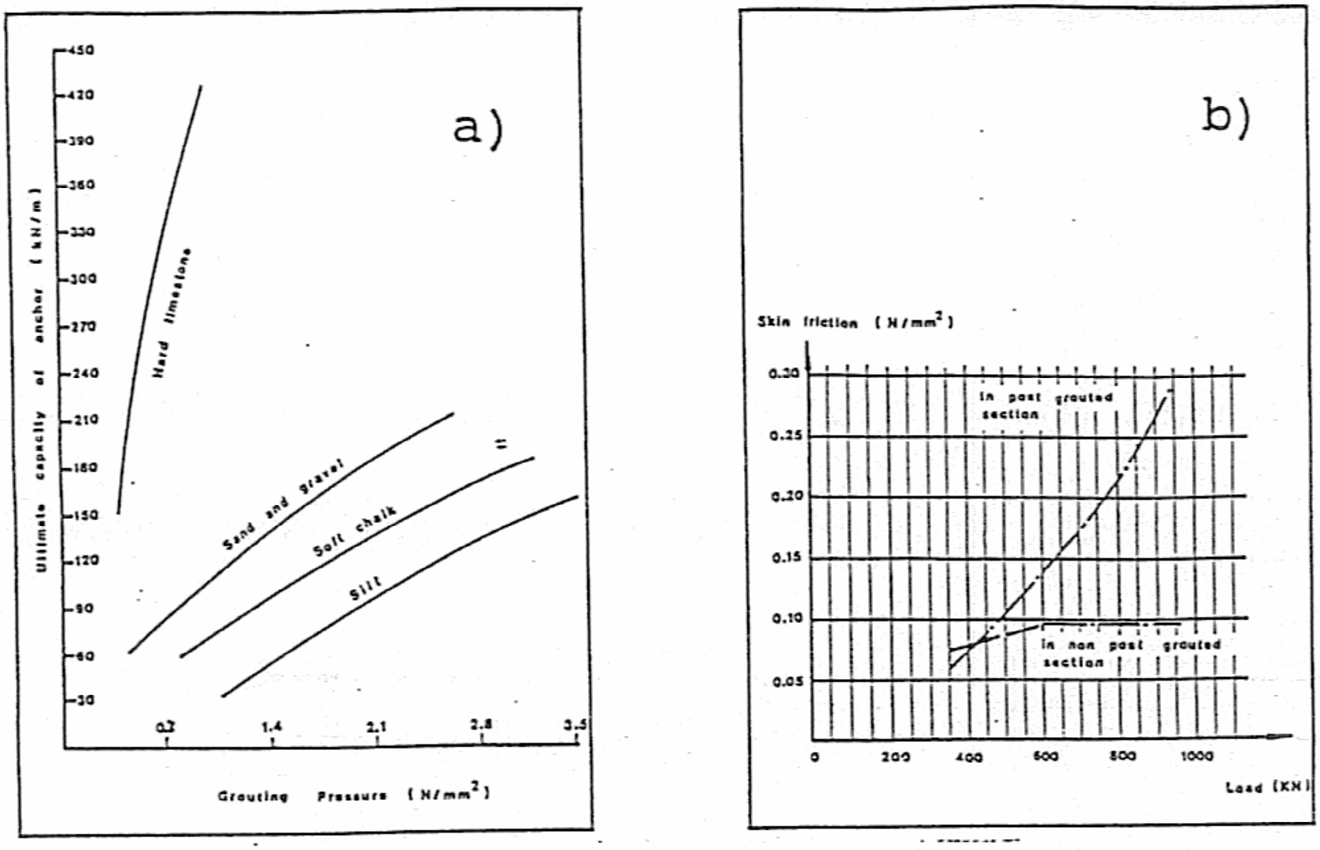
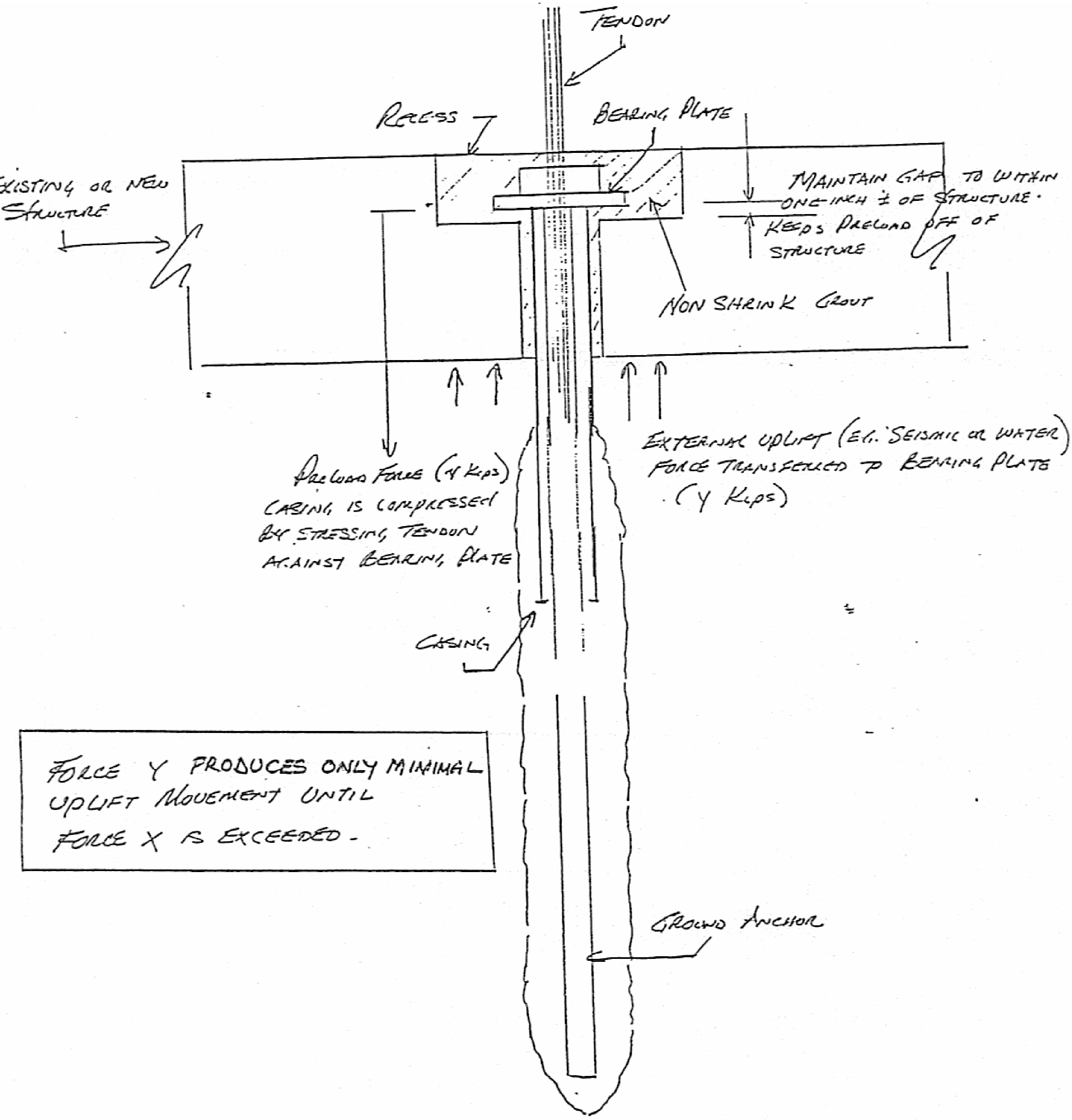


Figure 10. a) Influence of grouting pressure on ultimate load holding capacity (Littlejohn and Bruce, 1977). b) Effect of postgrouting on skin friction (Herbst, 1982).

structure to be underpinned. In this way, further settlement of the structure is not required to "activate" the pile, and so this technique is well suited to the problems of supporting extremely delicate structures. An example (Section 3.2) is provided which describes the underpinning of the Pocomoke River Bridge, MD. Incidentally, this principle is also of extreme potential in the seismic retrofit of bridge structures. Pier footings are often supported on soft soils by piles. Applying extra vertical load through conventional prestressed rock anchors bearing on the footing may therefore cause overloading of these pile foundations. On the other hand, prestressing has the benefit of reducing the amplitude of any "rocking" motions which may develop during a seismic event. The recently developed NCA-Pile^{SM} solves both problems (Figure 11), by providing a very stiff prestressed pile/anchor system which will act equally efficiently in tension and compression, without putting any extra vertical load on the existing foundation system.



<u>Figure 11</u>. Sketch showing principle of Nicholson Compressed Anchor-Pile^{SM}.

3.1. Augusta, GA

• Background

An existing soap manufacturing factory was founded on both spread footings and driven pile foundations. Due to the planned heightening of the facility, certain footing capacities had to be upgraded in both the vertical and lateral senses. This necessitated the installation of 143 pinpiles of nominal working load 100 kips through and adjacent to existing footings. Roughly half were vertical, with the others having a batter of 2 vertical to 1 horizontal.

• Site and Ground Conditions

The work had to be conducted within the fully functional facility where site cleanliness was paramount, and access was restrictive. Apart from a few inches of silty fill under the footings, the founding stratum consisted of a fine to coarse sand ($N \approx 30$) with lenses of clay, and underlain by a clay layer that dipped across the site. The three test piles did, however, penetrate two feet into this underlying clay layer. The elevation of the water table was below the pile tips.

A typical drilling log (variable by only one foot across the site) was:

0	-	3'	Red Clay
3'	-	20'	Sandy Clay
20'	-	23'	White Sandy Clay
23'	-	34'	Competent Dense Coarse Sand
34'	-	35'	Pink Sandy Clay
35'	-	37.5'	Slick Wet Clay

• Design

The pin piles were designed to be 38 feet long within a 7-1/4" O.D. hole. A 1-3/8" dia., 150 ksi reinforcing bar was later specified as standard. Due to design changes prior to construction, however, the first two test piles were reinforced by a #18 Grade 60 bar. Each pile also had a 5-1/2" O.D. steel casing installed in the top ten feet of the pile to resist a 12 kip lateral design load (Figure 12). The grout was designed to provide a 28-day strength of 4000 psi. A maximum vertical deflection of 0.5 inches at 50 tons (after ultimate test load was reached and held) was specified and a maximum lateral deflection of 0.5 inches at 12 tons was anticipated.

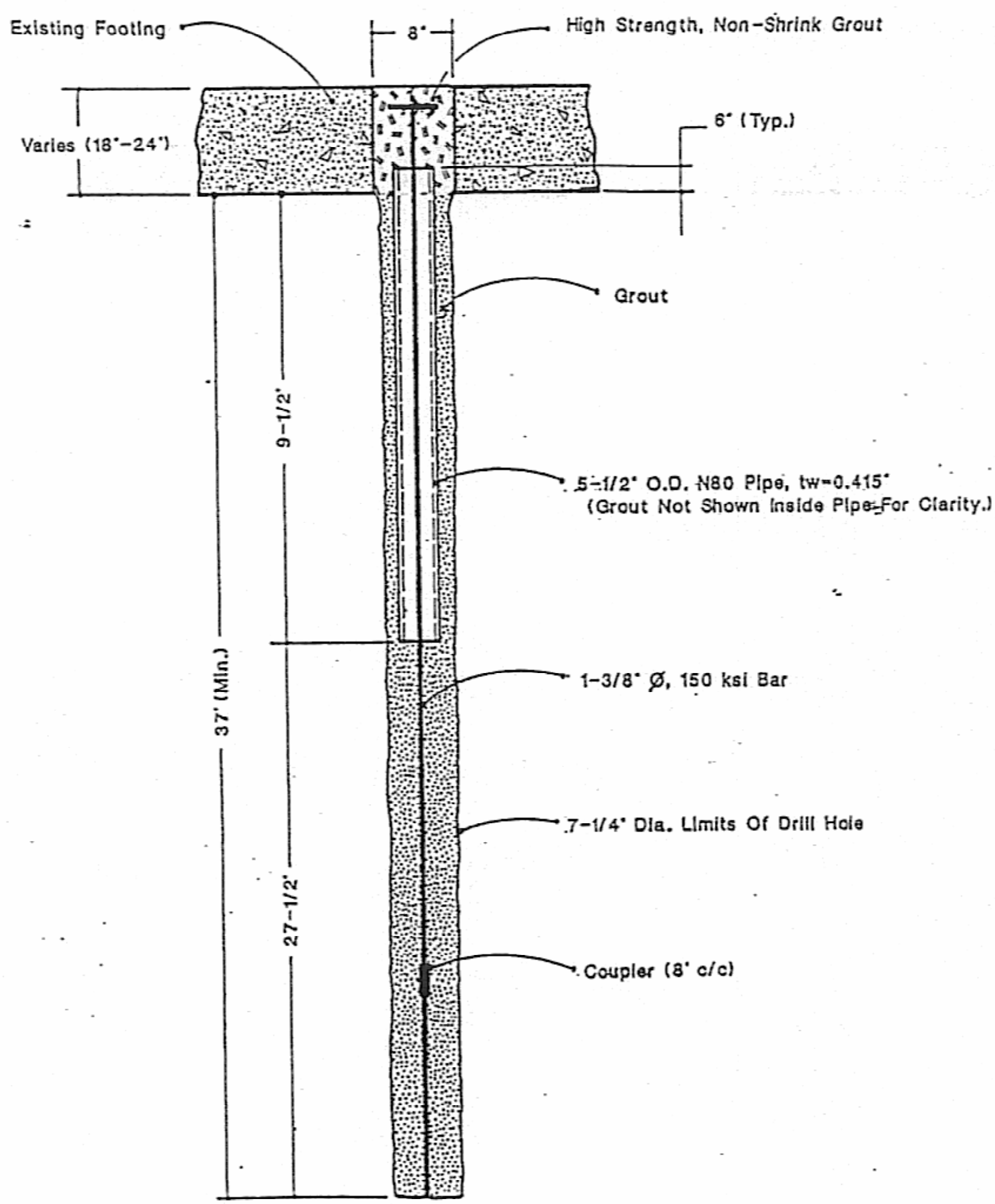


Figure 12. Typical pinpile arrangement. Augusta, GA.

- Construction

To minimize the handling of drill spoils in the manufacturing facility, drilling was conducted with hollow stem augers. Each reinforcing bar was placed in 8' long sections to which were attached the 1" dia. regrout tube. Primary grouting through the auger was conducted but only to a maximum pressure of about 40 psi due to leakage between the auger sections and around the flights. This was a considerably lower grouting pressure than could have been applied with the typical rotary casing with water flush method of drilling.

- Testing and Performance

The three test piles were generally tested cyclically using the ASTM D1143-81 Quick test procedures to a target of 200 kips.

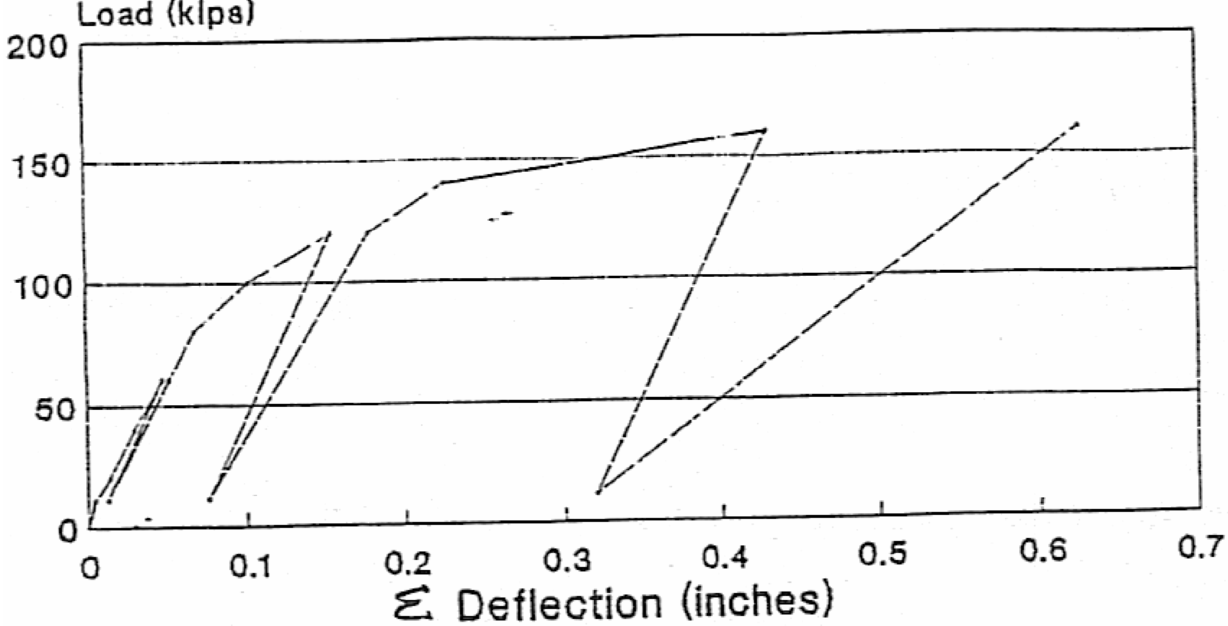
TP1 Plunged at 160 kips (Figure 13). This was felt to be an atypically low value in the prevailing ground conditions, and thought to be due to the inability to exercise higher grout pressures during installation. Could not be regrouted due to blockage in tube. Test discontinued.

TP2 Plunged at 160 kips in identical fashion. Regrouted via the tube à manchette and retested after four days to 200 kips with excellent performance (Figure 14).

TP3 Regrouted one day after installation. Plunged at 180 kips. Regrouted and retested successfully to 200 kips with excellent performance (Figure 15).

TP3 deflected elastically almost four times as much as the other two piles at equivalent loads. The only substantial difference between them was that TP3 had smaller diameter reinforcing.

Analysis of the loading data confirmed the mode of failure to be at the grout/ground interface. Postgrouting that interface



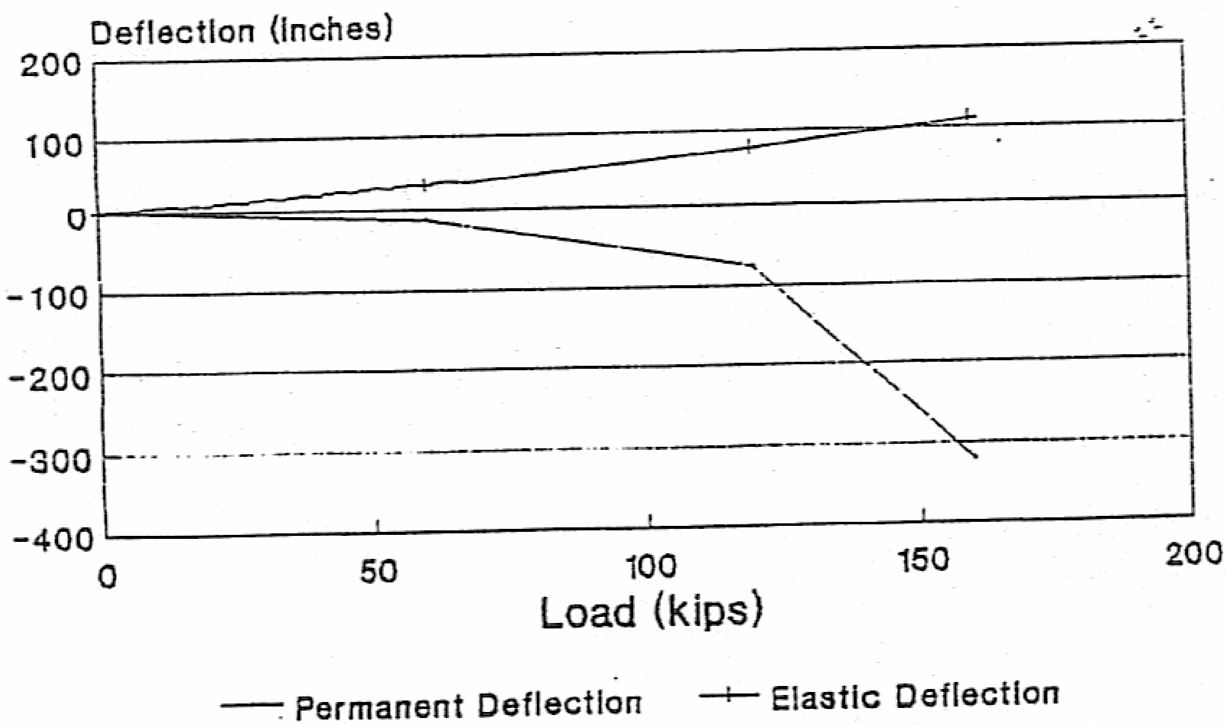


Figure 13. Performance of Test Pile 1. Augusta, GA. (No postgrouting possible.)

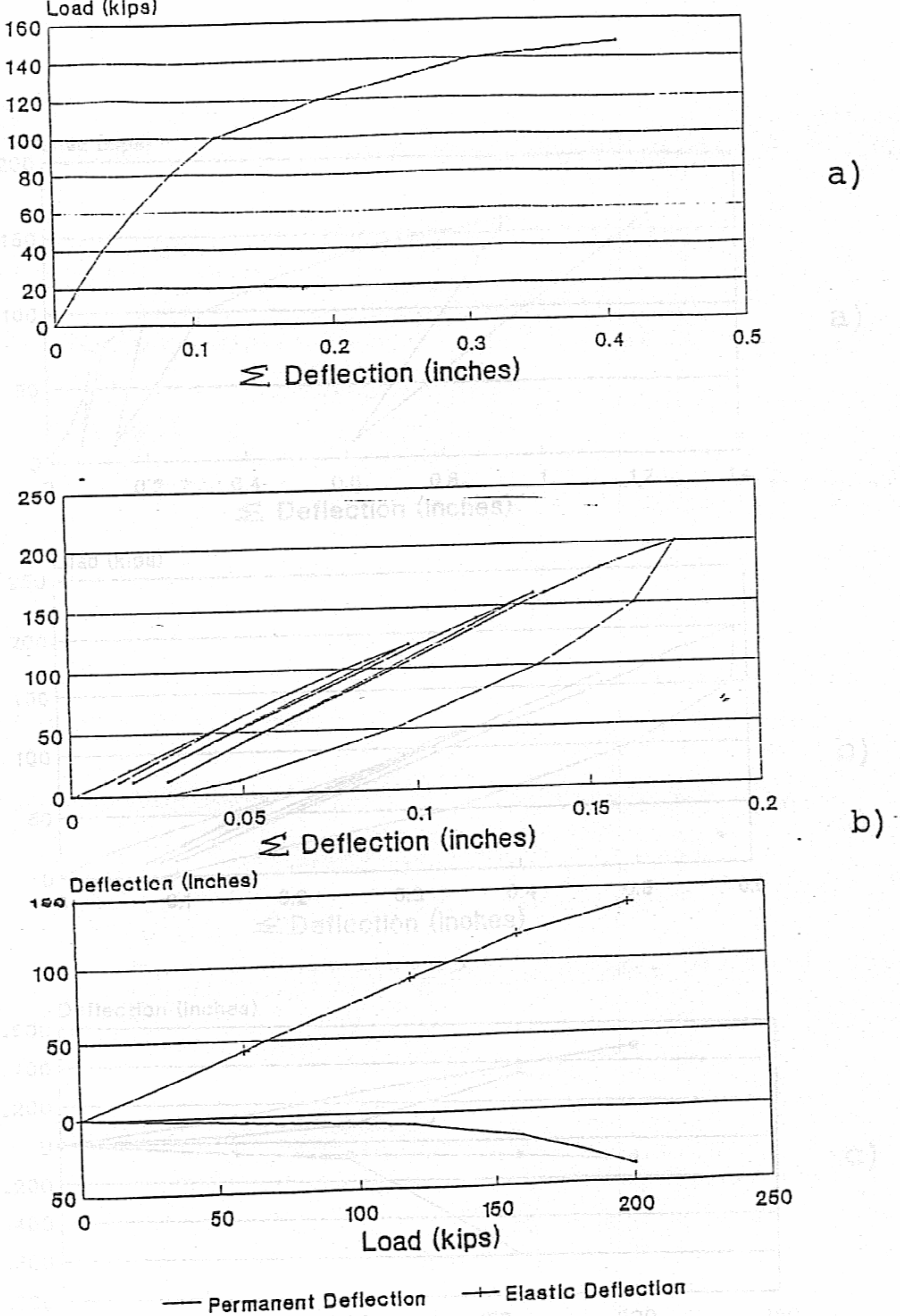


Figure 14. Performance of Test Pile 2. Augusta, GA.

a) Before, b) After Postgrouting

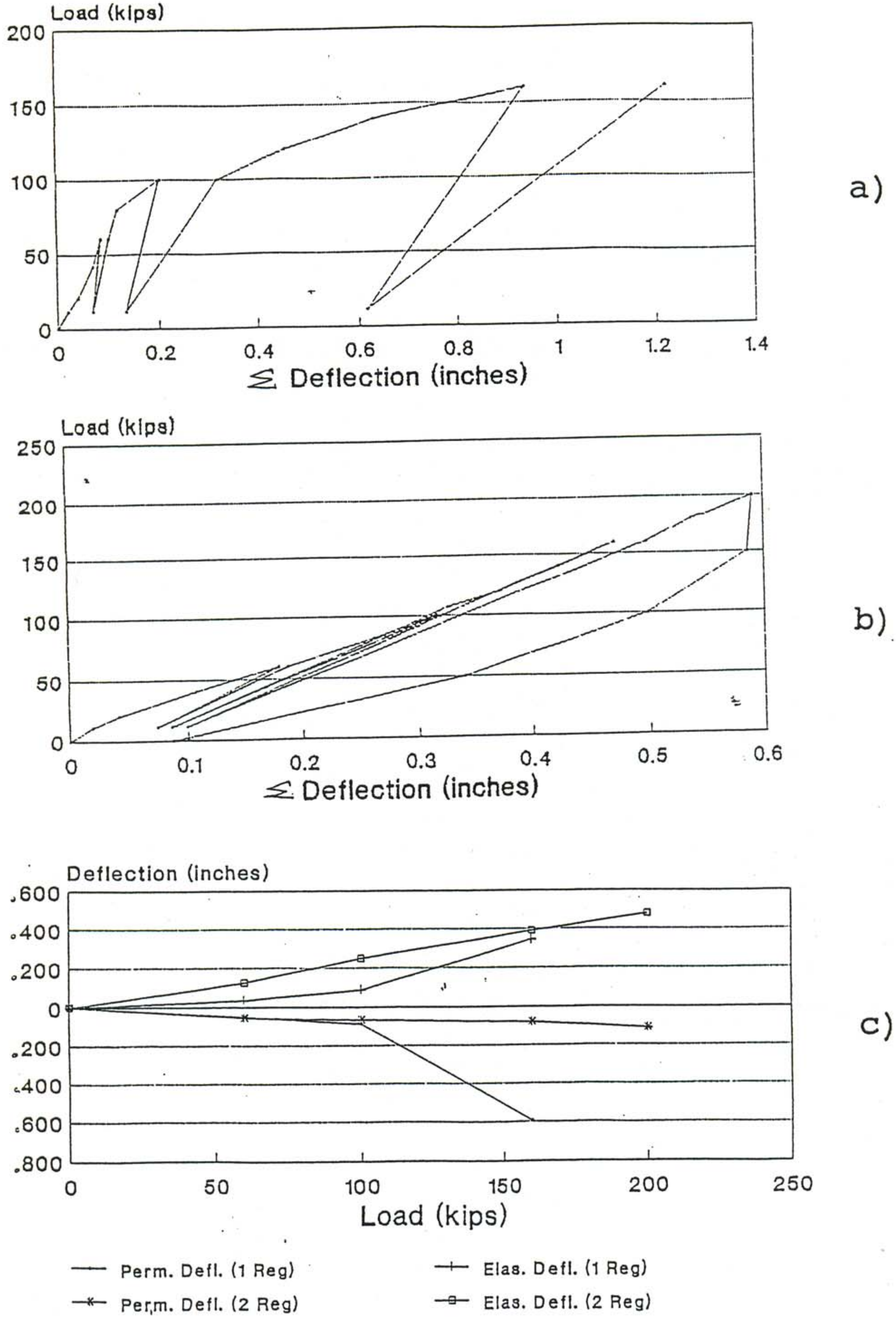


Figure 15. Performance of Test Pile 3, Augusta, GA. a) Total deflection after one regrout; b) Total deflection after two regrouts; c) Elastic and Permanent Analysis.

appeared to impact performance in two ways:

- it reduced permanent movements at equivalent loads by a factor of about 5 times.
- it reduced the amount of creep at equivalent loads substantially (Table 5).

Since the soil was judged to be impermeable to cementitious grouts (being too fine grained), it can be argued that this local improvement was due to a recompression, or compaction, of the soil, making it denser and so capable of sustaining higher intergranular and soil/grout contact stresses.

TP3 was tested laterally in accordance with ASTM D3966-81 using a cyclical quick method. The deflections were completely recoverable and totalled 0.247" and 0.293" at design and test loads respectively.

LOAD	CREEP (inches)				
(Kips)	TP1 No Regrouts	TP2 No Regrouts	TP2 1 Regrout	TP3 1 Regrout	TP3 2 Regrouts
13.6	0.000	0.000	0.000	0.001	0.000
20	0.000	0.001	0.002	0.003	0.004
40	0.000	0.000	0.002	0.002	0.001
60	0.001	0.001	0.002	0.003	0.001
80	0.002	0.005	0.000	0.004	0.000
100	0.004	0.005	0.000	0.004	0.000
120	0.010	0.015	0.001	0.028	0.002
140	0.017	1.032	0.002	0.032	0.002
160	0.045	Failed	0.000	Failed	0.002
180	Failed		0.004		0.004
200			0.012		0.007

Table 5. Creep data from the five tests, Augusta, GA.

3.2. Pocomoke River Bridge, MD (Bruce et al, 1990)

• Background

This 275' long movable bascule pier drawbridge (Figure 16) 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 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.

• 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

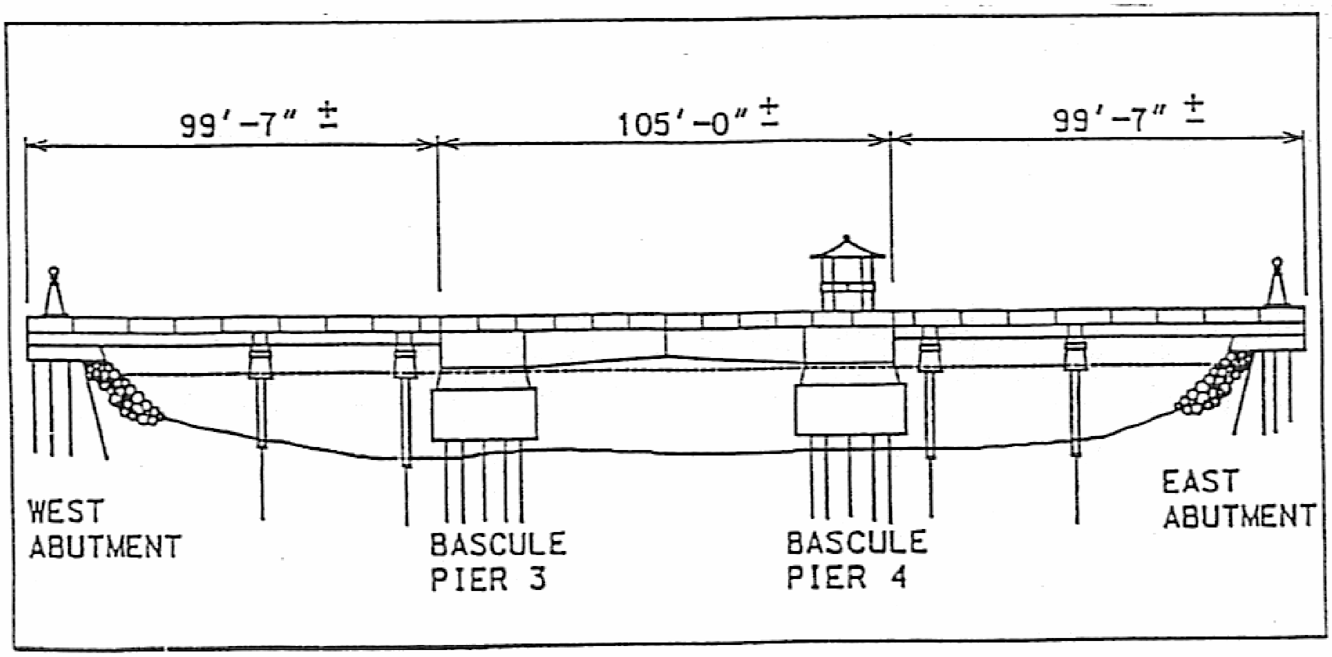


Figure 16. General configuration of Pocomoke River Bridge, MD. (Bruce et al., 1990).

from the restricted access of the Control House of Pier 4 (8x8' plan, 14' headroom) (Figure 17). 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.

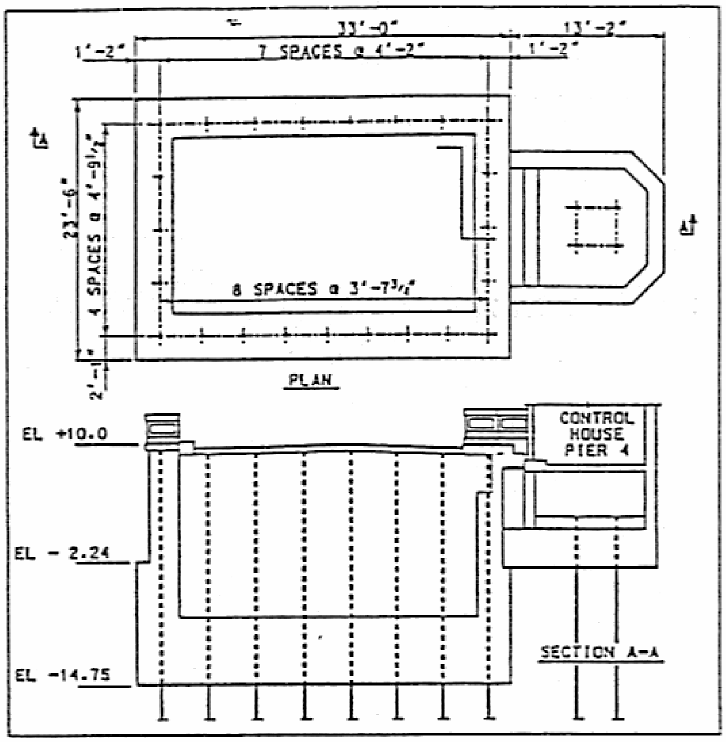


Figure 17. Plan and Section of Bascule Pier #4 showing pinpile locations, Pocomoke River Bridge, MD (Bruce et al., 1990).

Design and Construction

Each pile was installed as shown in Figure 18. 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 reinforcing bar. To permit preloading of the pile a tendon comprising 3 ea. 0.6" dia seven wire prestressing 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.

After the neat cement grout had reached 3500 psi, the tendon was stressed against the top of the steel casing, to the design load

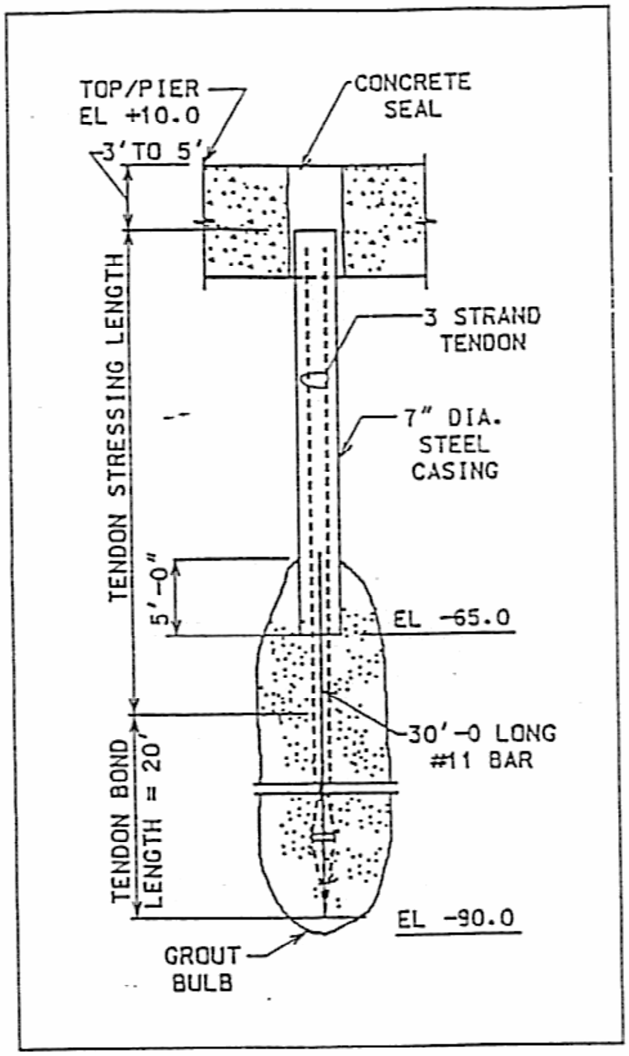


Figure 18. Typical detail of preloaded pinpile, Pocomoke River Bridge, MD (Bruce et al., 1990).

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 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.

- 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 8-5/8" in 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.

The test had three phases:

<u>Phase I:</u> a "Preload-Unloading" test designed to verify the performance efficiency of the preloading system.

<u>Phase II:</u> a conventional pile load test to establish load-deflection performance within the scope of the specification (i.e., progressively to twice working load).

<u>Phase III:</u> 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' (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.

<u>Phase I Tests (Preload - Unloading Tests)</u> - 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.

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 6. Major points are as follows:

	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)
Pile 1			
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
Pile 2			
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 6. Highlights of Load/Deflection Data, Test Piles 1 and 2, Pocomoke River Bridge, MD

- 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.
- 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" 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 7.

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

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

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 fail the pile. Surprisingly, after four successive cycles to about 360 kips, the pile had not yet failed, despite a cumulative permanent displacement of 0.567". 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.

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": very close to 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.

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 repaired by preloaded 55 kip piles (Bruce, 1988, 89). In both instances, the subsequent structural movements in service have been of the order of a few thousands of an inch.

4. FINAL REMARKS

Pinpiles are generally accepted in the New York area as a very attractive underpinning option for major structures, under certain prevailing factors. These factors may include restrictive access and headroom, difficult soil conditions, the need for "stiff" support, and scheduling restraints. All these factors in turn impact cost considerations. Meanwhile research and development continues along several paths with a common goal, which can be summarized as "more, cheaper, safer, stiffer kips".

Acknowledgements

The authors wish to thank their colleagues in Nicholson Construction, New York State DOT, and the various construction and engineering firms cited for their cooperation.

REFERENCES

ASTM - D1143-81 (1981). "Method of Testing Piles Under Static Axial Compressive Load". Section 04, Vol. 04.08.

ASTM - D3966-81 (1981). "Method of Testing Piles under Lateral Loads". Section 04, Vol. 04.08.

Bruce, D.A. (1988a). "Developments in Geotechnical Construction Processes for Urban Engineering". Civil Engineering Practice, 3 (1), Spring, pp. 49-97.

Bruce, D.A. (1988, 1989). "Aspects of Minipiling Practice in the United States". Ground Engineering 21 (8), pp. 20-33 and 22 (1), pp. 35-39.

Bruce, D.A. (1989). "Methods of Overburden Drilling in Geotechnical Construction - A Generic Classification". Ground Engineering, 22 (7), pp. 25-32.

Bruce, D.A., Pearlman, S.L. and Clark, J.H. (1990). "Foundation Rehabilitation of the Pocomoke River Bridge, MD, Using High Capacity Preloaded Pin Piles", 7th International Bridge Conference, Pittsburgh, PA, June 18-20, 1990.

Bruce, D.A. (1991). "The Construction and Performance of Prestressed Ground Anchors in Soils and Weak Rocks: A Personal Overview". DFI Conference, Chicago, IL, October 7-9, 1991.

Bruce, D.A. (1992). "Recent Progress in American Pinpile Technology". Proc. ASCE Conference, "Grouting, Soil Improvement and Geosynthetics", New Orleans, LA, February 24-28. 13 pp.

Bruce, D.A., Hall, C.H. and Triplett, R.E. (1992). "Structural Underpinning by Pinpiles, Near Mobile, Alabama". Proc. DFI

Annual Meeting, New Orleans, LA, October 21-23.

Herbst, T.F. (1982). "The GEWI Pile - A Solution for Difficult Foundation Problems", Symp. on Soil and Rock Reinforcement Techniques, AIT, Bangkok, Nov. 29 - Dec. 3, Paper 1-10.

Jones, D.A. and Turner, M.J. (1980). "Post-grouted Micro Piles", Ground Engineering, 13 (6), pp. 47-53.

Kenny, J.R. (1992). "Behavior and Strength of Composite Tubular Columns in High Strength Steel". MSc Thesis, University of Pittsburgh. Unpublished.

Littlejohn, G.S. (1970). "Soil Anchors", Ground Engineering Conference, Institution of Civil Engineers, London, pp. 33-44.

Lizzi, F. (1985). "Pali Radice (root piles) and Reticulated Pali Radice", Underpinning, Surrey University Press, pp. 84-151.

Mascardi, C.A. (1982). "Design Criteria and Performance of Micropiles", Symposium on Soil and Rock Improvement Techniques Including Geotextiles, Reinforced Earth and Modern Piling Methods, Bangkok, December, Paper D-3.

Munfakh, G.A. and Soliman, N.N. (1987). "Back on Track at Coney", Civil Engineering, 59 (12), pp. 58-60.

Post Tensioning Institute (1986). Recommendations for Prestressed Rock and Soil Anchors, Phoenix, 41 pp.

ITEM 17551.9902 - BORED-IN PILES (DESIGN CAPACITY: 50 TONS OR LESS)
ITEM 17551.9903 - BORED-IN PILES (DESIGN CAPACITY: 51 TO 100 TONS)
ITEM 17551.9904 - BORED-IN PILES (DESIGN CAPACITY: 101 TO 150 TONS)

DESCRIPTION

General

Under this work, the Contractor shall design bored-in piles and furnish all labor and materials and perform all operations necessary to install bored-in piles at the locations and to the capacities shown on the plans, or as modified by the Engineer in writing. At his option, the Contractor may post-grout the piles to achieve the required capacities.

Static Pile Load Tests will be paid for under a separate item.

The piles shall be designed to perform satisfactorily from both structural and geotechnical considerations. The structural design shall be performed according to AASHTO standards.

The Contractor shall design the pile, including diameter, length, grout and concrete strengths, and grouting pressures, and shall select the equipment, procedures and methods so that the ultimate capacity of the pile-soil system for each pile is at least twice the pile design capacity shown on the plans.

If the Contractor selects a pile size that is not compatible with the pile caps shown in the plans, he shall redesign the pile caps at his own expense.

The Contractor shall submit his design and his method of installation to the Deputy Chief Engineer Technical Services (D.C.E.T.S.) and the Deputy Chief Engineer Structures (D.C.E.S.) for review and approval. The D.C.E.T.S. and D.C.E.S. will require 30 working days to approve the submission. No work shall begin prior to approval by the D.C.E.T.S. and the D.C.E.S. Approval by the D.C.E.S. and the D.C.E.T.S. does not constitute a guarantee of satisfactory performance. Successful performance is the responsibility of the Contractor.

The Contractor performing the work described in this specification shall submit proof of: 1) two projects on which he has successfully designed and installed bored-in piles or soil tiebacks, using non-displacement methods under similar site conditions as shown on the plans, and 2) the foreman for this work having supervised the successful installation of bored-in piles or soil tiebacks on at least two projects in the past two years.

MATERIALS

A. Concrete

The concrete mix shall be designed by the Contractor. Concrete shall be capable of being placed underwater. The concrete shall conform to the requirements of Section 501 - Portland Cement Concrete - General.

ITEM 17551.9902 - BORED-IN PILES (DESIGN CAPACITY: 50 TONS OR LESS)
ITEM 17551.9903 - BORED-IN PILES (DESIGN CAPACITY: 51 TO 100 TONS)
ITEM 17551.9904 - BORED-IN PILES (DESIGN CAPACITY: 101 TO 150 TONS)

The concrete shall meet the material requirements of Subsection 501-2 except that:

1. A set retarding, water reducing admixture shall be added.
2. The minimum slump shall be seven (7) inches.
3. The minimum air content shall be three (3%) percent.

The concrete shall have a minimum temperature of 50 degrees F., a maximum temperature of 90 degrees F. and be placed on areas cleaned of all debris, mud or other foreign material to the satisfaction of the Engineer. The water in which the concrete is placed shall have a minimum temperature of 35 degrees F. and a maximum of 90 degrees F.

B. Steel Casing

The casing shall be of appropriate thickness to withstand the stresses associated with advancing it into the ground, in addition to the stresses due to hydrostatic and earth pressures. The maximum outer diameter of the cutting shoe, if a cutting shoe is used, shall not exceed the outer diameter of the casing plus 0.25 inches. Casing shall be of the flush joint type.

C. Grout

The grout mix shall be designed by the Contractor, and shall consist of materials meeting the following Specification requirements:

Material	Subsection
Portland Cement, Type 1 or 2	701-01
Grout Sand	703-04
Water	712-01

The grout mix shall be pumpable.

D. Reinforcement

Bar reinforcement shall meet the requirements of Subsection 709-01, Bar Reinforcement Grade 60, or continuously threaded "Uncoated High-Strength Steel Bar for Prestressing Concrete" - ASTM A722.

If a steel core is used, it shall be a new and unused HP or pipe section conforming to the requirements of Subsection 715-01, Structural Steel, ASTM Designation A36.

CONSTRUCTION DETAILS

GENERAL

The Contractor shall fully examine the existing site conditions to ensure that his equipment can operate without removing or relocating existing utilities, structures or structural members.

Non-production piles shall be installed using the same methods as for the production piles.

A. Submittals

Shop drawings shall be submitted to the Engineer for approval at least forty-five (45) days prior to starting the work. The drawings shall show:

1. Details of the pile configuration for each capacity, including nominal diameter, length, reinforcement and grouting pressures.

2. Details of equipment and procedures for pile installation, including consecutive steps for pile installation and the approximate time required for each step.

3. Procedures for advancing through boulders and other obstructions.

4. Procedures for control and removal of all spoil.

5. Drawings which show that the specified work can be performed in low headroom conditions and as close to obstructions as site conditions warrant to install the piles at the locations shown on the plans, if such conditions are expected to be encountered.

6. Drawings which show the method, procedure and equipment for post grouting, if used.

7. The Contractor shall submit layout drawings showing the proposed sequence of pile installation. This sequence shall be coordinated with the proposed phasing and scheduling requirements stated in the contract plans.

The mix designs for the concrete and grout, and documentation from an independent laboratory showing that the mix designs conform to this specification, and meets the strength requirements set by the Contractor shall be submitted to the Director, Materials Bureau for approval at least 45 calendar days prior to use.

At least forty-five (45) days prior to concrete or grout placement, the Contractor shall submit his proposed methods and equipment for placing concrete or grout to the Engineer for approval.

B. Drilling and Excavation

Pile installation, including removal of casing, shall be by non-vibratory and non-displacement methods. In the vicinity of subway tunnels, and at locations shown on the plans, drilling shall be performed by rotating or oscillating a casing and applying a static vertical load.. In other areas, bentonite slurry may be used in lieu of casing to hold the hole open. Drilling and excavation shall be performed in such a manner to minimize collapse of the hole.

If obstructions are encountered during excavation for a pile, the obstructions shall be penetrated by coring. In no case shall impact hammers or blasting be used.

The Contractor shall control his methods to prevent the soil at the bottom of the hole from blowing in. The water or bentonite slurry level inside the hole shall be kept above the ground water level at all times during installation and cleaning out.

It may be necessary for the Contractor to have equipment capable of pumping water into the hole in order to maintain the water level. An auger, if used, shall be removed slowly to avoid suction.

All excavation spoil and waste materials shall be transported to a containment area off-site. The Contractor shall conduct his operations so that waste and spoil is not deposited on city streets.

A hole for a bored-in pile shall not be progressed and post grouting (if any) shall not be performed within five (5) pile diameters of another bored-in pile unless casing has been temporarily left in place prior to concreting or until the concrete and grout for a bored-in pile within this zone has cured for at least one (1) day. The casing for a bored-in pile shall not be removed within five (5) pile diameters of another bored-in pile until the concrete for a bored-in pile within this zone has cured for at least one (1) day.

C. Concrete Placement and Casing Removal

Concrete shall be placed so as to avoid segregation by means of a tremie pipe from the bottom of the pile upward. The casing shall be slowly removed while the concrete is still fluid. The rate of removal of the casing shall be closely controlled so that the concrete level is at least ten feet higher than the casing tip, or at the existing ground elevation, whichever is lower.

If the Contractor anticipates pumping the concrete, he shall design the mix accordingly.

The Contractor may substitute grout for concrete for filling the pile as long as the required strength is met.

D. Post Grouting

Post grouting may be performed by the contractor to increase the bored-in pile capacity.

Initial post grouting shall be performed between one and two days after the tremie concrete or grout for the bored-in pile has been poured.

The work shall consist of pressure grouting the soil around the pile, or pressure grouting the soil around the pile plus the soil below the tip of the pile. Pressure grouting of the soil around the pile shall be performed from within the pile.

The Contractor's equipment and methods shall permit him to regrout several times if necessary to achieve the required capacity.

E. Construction Tolerance

The center of the bored-in pile shall not vary from the plan location by more than three (3) inches. The bored-in pile shall not vary from the vertical or established batter by more than 1/8 inch per foot, as measured above ground.

The top of the pile shall be cut off within 2 inches of the elevation shown on the plans, or where ordered by the Engineer.

If the soil at the pile tip is post grouted, the elevation of the pile top shall be monitored during grouting. Pile uplift shall not exceed ¼ inch.

F. Unacceptable Piles

Piles that are rejected by the Engineer because of inadequate capacity, damage, mislocation, misalignment, or failure to meet other installation criteria shall be cut off and additional pile (s) shall be installed as ordered by the Engineer.

For piles that the Engineer finds, unacceptable based on the results of the Static Pile Load Test, the Contractor shall submit a written proposal containing a suggested course of action. The written proposal shall include:

1. Details of the proposed modifications;

2. A procedure, including any additional testing, describing how the performance of the modified method will be verified;

3. An explanation of what will be done with the piles already installed in similar soil conditions using the same method as the pile that was found unacceptable based on the results of the Static Pile Load Test, to ensure that the design loads called for in the plans are met.

For every pile that fails the Static Pile Load Test, the Contractor shall perform two additional Static Pile Load Tests at his expense, at locations chosen by the Engineer.

The proposed action to be taken will be subject to written approval by the Engineer.

ITEM 17551.9902 - BORED-IN PILES (DESIGN CAPACITY: 50 TONS OR LESS)
ITEM 17551.9903 - BORED-IN PILES (DESIGN CAPACITY: 51 TO 100 TONS)
ITEM 17551.9904 - BORED-IN PILES (DESIGN CAPACITY: 101 TO 150 TONS)

Whenever an additional pile installation or pile repair is required, such additional installation(s) or repair(s) shall be at no cost to the State. The pile cap shall be modified, where necessary, at no cost to the State.

METHOD OF MEASUREMENT

The quantity to be paid for under this item shall be the number of bored-in piles furnished and installed in accordance with the Contract Plans and Specifications and accepted by the Engineer.

BASIS OF PAYMENT

The unit price bid per pile shall include the cost of furnishing all labor, materials, and equipment necessary to complete the work.

Furnishing equipment to install the bored-in piles shall be paid for under a separate item.

No payment shall be made for piles rejected by the Engineer.