FUNDAMENTAL TESTS ON THE PERFORMANCE OF HIGH CAPACITY PIN PILES

by

D.A. Bruce, R. Bjorhovde, J.R. Kenny

Presented at

DFI Conference
Pittsburgh, PA

October 18-20, 1993

NICHOLSON CONSTRUCTION COMPANY
P.O. BOX 98
BRIDGEVILLE, PA 15017 (412) 221-4500

93	2	93

FUNDAMENTAL TESTS ON THE PERFORMANCE OF HIGH CAPACITY PIN PILES[SM]

Donald A. Bruce*, Reidar Bjorhovde†
and John R. Kenny°

ABSTRACT

Pin Piles are small diameter, bored, cast-in-place piles used to underpin structures. They were first employed over 40 years ago in Italy, and have been used in the U.S. since the late 1970's. However, it is only in the last few years that their full potential in terms of load holding capacity has been exploited: whereas 100 kip working loads typified former practice, ultimate loads approaching 700 kips have recently been recorded for Pin Piles founded in sand.

In order to better understand the actual performance of these piles, fundamental research has been conducted by a consortium of industry and academia. This paper describes the advances made in the laboratory and full scale field testing programs. In particular, the extensive field tests have been conducted in a variety of materials in which different pile configurations have been cyclically tested to failure. By using this test method, the elastic performance of the piles has been examined and so the progressive interfacial debonding phenomenon has been studied. This has led to the development of the Elastic Ratio concept which is proving extremely useful in analyzing and predicting Pin Pile performance. Data are provided from the full scale Test Pile Programs conducted at Mobile, AL and Port Vancouver, WA.

Given that Pin Piles are usually installed in areas of low headroom or restricted access, and through difficult, occasionally contaminated soils, the paper also describes key aspects of construction.

1. INTRODUCTION

The last decade has seen a significant growth in the use of Pin Piles[sm] in the United States. Generically, these piles may be classified as small diameter, bored, cast-in-place elements, and they owe their origins to developments by specialty contractors in Italy over 40 years ago. As a result of the kind of research and development activities described below, their safe working load range has been extended from 50-100 kips to up to 300 kips, while special test piles have yielded ultimate loads of around 700 kips in certain conditions.

Initially, these advances were made as a result of the careful execution and analysis of full scale field test programs, and such experiences have been widely published (References 1-8). However, within the last few years it has become apparent that extra dimensions of research efforts were necessary to explore and understand fundamental aspects of Pin Pile behavior, and especially those related to the performance of the component materials in resisting and transferring axial load.

---

* Nicholson Construction Company, Pittsburgh, PA 15017 (412-221-4500)
† University of Pittsburgh, Professor of Civil Engineering and Director, Bridge and Structures Information Center, Pittsburgh, PA 15261 (412-624-9876)
° Gannett Fleming, Inc., Pittsburgh, PA 15220 (412-922-5575)

This research was funded by Nicholson Construction Company and conducted jointly with the University of Pittsburgh. The laboratory work comprised three major phases:

> Phase 1, where single grout filled steel casings, simulating the upper (free) section of a typical high capacity Pin Pile, were compressed to failure, to establish their composite strength and elasticity.
>
> Phase 2, as Phase 1 but including connected sections with threaded ends, and
>
> Phase 3, where similar tests were conducted on internally reinforced grout columns simulating the lower (bonded) section.

In parallel, the opportunity was taken to run full scale field tests at two major contemporary underpinning projects, one at a petrochemical facility near Mobile, AL, the other at a grain silo complex near Port Vancouver, WA. The data from these four sources constitute the essence of this practice oriented paper, and clearly demonstrate the value of the research as a way of analyzing and predicting pile performance. In particular, the elegance of the Elastic Ratio (ER) concept is underlined.

Firstly, and in order to introduce the terminology, a brief review is made of construction techniques.

2. CONSTRUCTION

Pin Piles are most commonly used to underpin existing structures settling, or liable to settle as a result of changes in loading or foundation conditions. Construction methods have therefore been developed to accommodate the gamut of ground and structure types, while causing the minimum of damage to either, or the environment. Also Pin Piles operate principally in side shear and so these techniques have been honed to enhance bond capacity at the grout/soil interface.

A common contemporary method of installing Pin Piles in shown in Figure 1. Older variants using compressed air to pressurize the grout, or a vibrated mandrel (displacement pile) are described by ASCE (Reference 9) but are rarely if ever used in developed countries. Likewise, the "expanded base" pile (Reference 10) and the Menard inflatable cylinder pile (Reference 11) are never seen nowadays.

The successive steps of relevance to the content of this paper (and therefore excluding consideration of Connection to Structure and Corrosion Protection) are:

- Drilling: A drilling method is chosen to ensure the minimum practical disturbance or upheaval to the structure or the soil. Frequently a different system may be necessary to penetrate through any existing structure from that to be used in the soils below. For soil drilling, some type of duplex method (Reference 12) is common, although in certain conditions the use of a single casing is permissible. Water or foam flush (Reference 7) is typical: air flushing is typically disallowed. In certain soil conditions (e.g. clays) or where fluid spoils cannot be tolerated within the structure being underpinned for environmental reasons, a hollow stem auger can be used, although subsequent grout/soil bond capacity may be impacted adversely as a result of lateral

- Pressure Grouting: The casing or auger is then withdrawn, while grout is continually injected through the drill head. This grout is pressurized (50-150 psi) to enhance subsequent performance characteristics, with the maximum pressure reflecting:
  - the need to avoid soil hydrofracture or heave;
  - the nature of the drilling system (only relatively low pressures are possible in augers due to leakage at joints and around the flights);
  - the ability of the soil to form a "seal" around the casing during extraction;
  - the "groutability" of the soil.

Pressure is maintained only over the bond zone length: the rest of the pile is filled with grout at gravity head.

In most countries, this drill casing is fully extracted (as the auger must always be) during this process. However, in the United States, it has been proven that by leaving the casing in place through the zones above the pressured zone, the Pin Pile performance is greatly enhanced, both vertically and laterally. This option also prevents wasteful travel of grout into these often permeable upper horizons while it also provides excellent corrosion protection to the interior of the pile in what is usually the most vulnerable zone. A useful subclassification of Nicholson Pin Pile types, based on the geology of the founding zone, and the internal composition (and the mode of action of the pile) is provided in Figure 2. (Reference 5):

- Type S1 - A steel pipe is rotated into the soil using water to externally flush the cuttings up around the pipe annulus. A neat cement grout is tremied from the bottom of the hole to displace the water. The reinforcing element is then placed to the bottom of the hole. As the pipe is withdrawn over the length of the bond zone, additional grout is pumped under sufficient excess pressure to create the bond zone. The pipe is then seated into the grouted bond zone for 5 to 10 feet. In granular soils, a certain amount of permeation and replacement of loosened soils takes place. In cohesive soils, some lateral displacement or localized improvement of the soil around the bond zone is accomplished with the pressure grouting. Postgrouting (see below) may be used later to further enhance soil/grout bond.

- Type S2 - The Type S2 pile is installed in the same fashion as the S1 pile except that:
  - the centralized reinforcing element is not needed;
  - the steel pipe is installed to the full length of the bond zone after pressure grouting is completed,
  - post-grouting is not typically used in this type of installation.

- Type R1 - The Type R1 pile uses the same technique for advancing the steel casing as Type S1, except that the depth of penetration is limited to the top of rock. Once the pipe is seated into the rock, a smaller diameter drill string is advanced through its center to drill the rock bond zone of diameter slightly less than the inside diameter of the pipe. Neat cement grout is then tremied from the bottom, and a reinforcing element is placed in the rock bond zone to complete the pipe installation. A minimum transfer length is required for the reinforcing to develop inside the pipe (typically 5 to 10 feet).

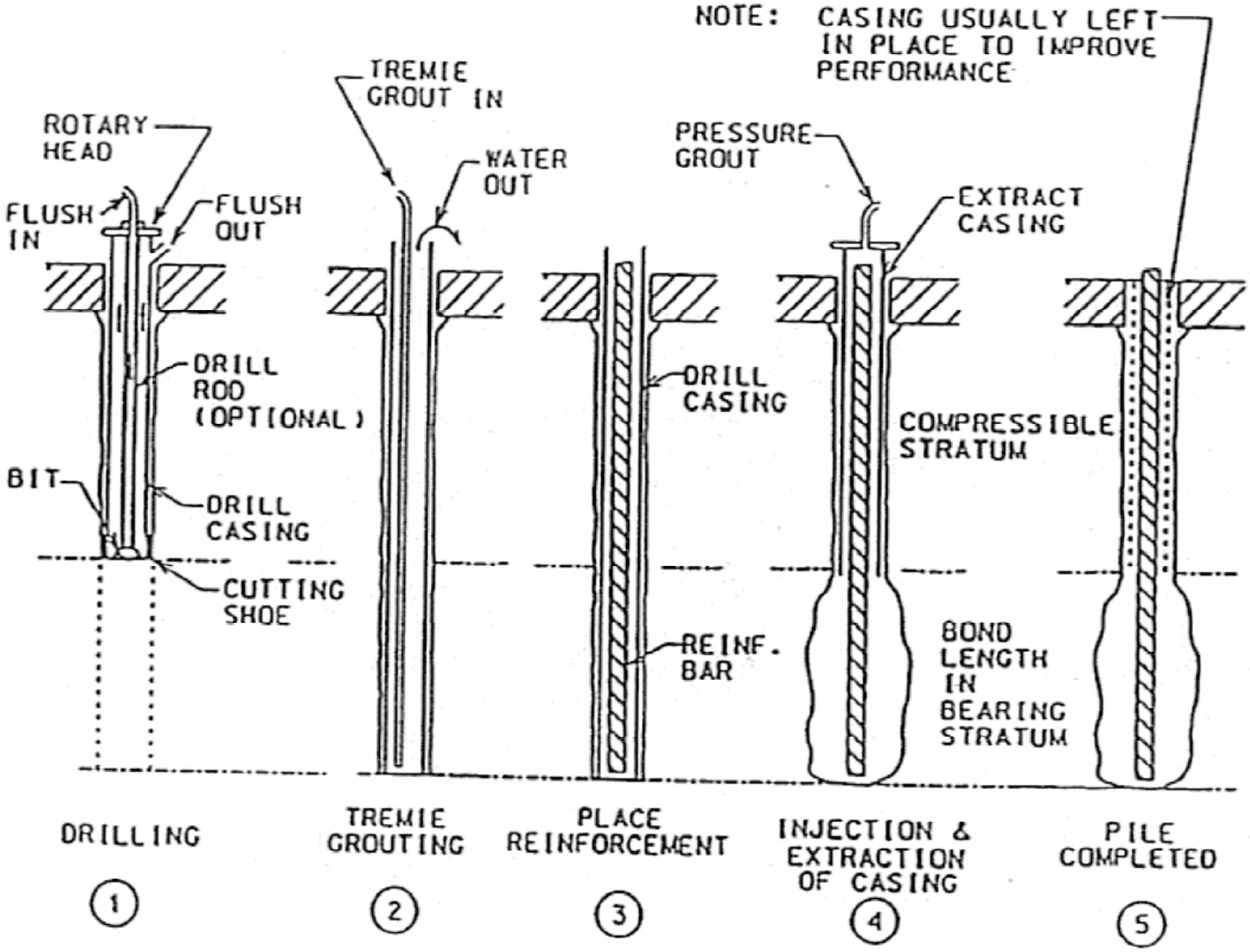


Figure 1 Stages in the construction of a typical Pin Pile in soil.

decompression of the surrounding soil. This is a clear reminder that the critical design aspect of pile-soil bond capacity is highly sensitive to constructional method, and especially the drilling and grouting techniques.

Contemporary drilling rigs for such work are diesel hydraulically or electrohydraulically powered, track mounted and extremely powerful for their compact size. Many have dimensions allowing them to pass through very narrow openings and operate in less than 10 feet of headroom. Such rigs are highly maneuverable and capable of drilling at any angle through rock, soil and obstructions. They can commence drilling within 1 foot of existing structures.

- Placing of Reinforcement, and Tremie Grouting: After the casing (or auger) has reached full depth, it is tremied full of grout. This grout is typically a neat cement mix prepared in a high speed colloidal mixer. The reinforcement, suitably centralized, is then placed. This may consist of a cage of reinforcing bars, a high strength bar (or group of bars) or a steel pipe, depending on the design requirements and the purpose of the pile.

Spec #	Dia. (in)	Length (in)	Wall (in)	Max Load (kips)	Elastic Ratio*
1	7	36	0.502	1181	(Equivalent to 0.30 for 10' length)
2	7	36	0.502	1242	(Equivalent to 0.30 for 10' length)
3	7	120	0.502	969	0.32
4	5.5	36	0.363	685	(Equivalent to 0.70 for 10' length)
5	5.5	36	0.363	584	(Equivalent to 0.54 for 10' length)
6	5.5	120	0.363	450	0.53

Table 1 Summary of Phase I Laboratory Tests (Single Casings)

*Calculated for each specimen a compression (in thousandths of an inch) divided by load (in kips), in the elastic response field.

Spec #	Dia. (in)	Length (in)	Wall (in)	Max Load (kips)	Elastic Ratio*
1	7 B	36	0.502	1300	0.49
2	7 U	36	0.502	1160	0.44
3	5.5 B	36	0.363	630	1.52
4	5.5 U	36	0.363	545	1.06

Table 2 Summary of Phase 2 Laboratory Tests (Coupled Casings)

B = Banded
U = Unbanded
* 10 ft. equivalent length

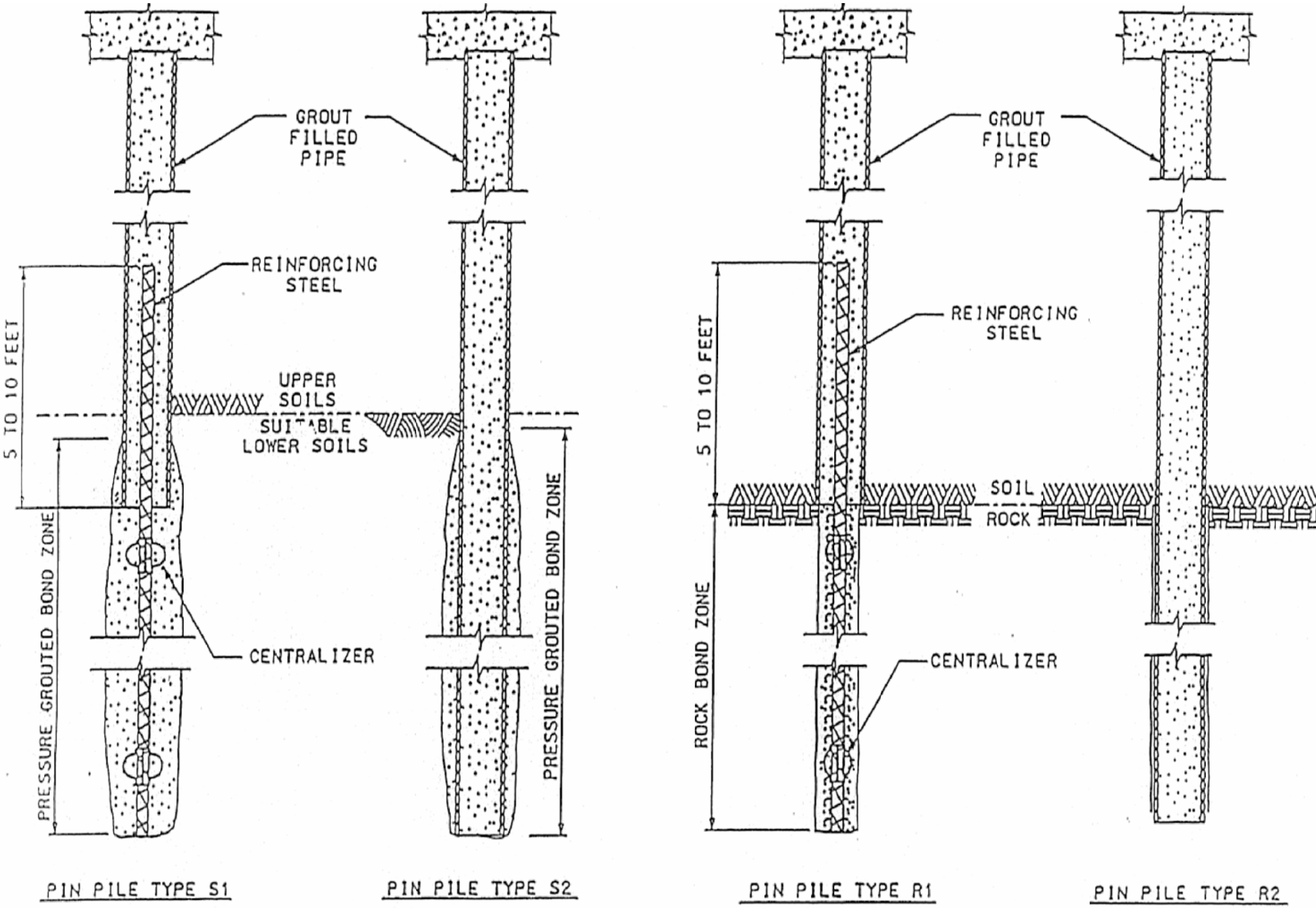


Figure 2 Pin Pile types in soil and rock (Reference 5)

- Type R2 - The Type R2 pile differs from the R1 pile in that it uses a full length steel pipe. Centralized reinforcement is optional. In order to advance through both the overburden and the rock, a permanent drill bit is used on the end of the casing with a diameter somewhat greater than that of the casing. There are grout ports in the bit, and once the hole is advanced to the desired depth, grout is tremied from the bottom, and additional grout is pumped to ensure full grouting of the rock bond zone. This grout may not flow completely to the surface in some conditions. However, once the level inside the pile has stabilized, the final grout level on the outside of the pile can be verified.

- Postgrouting (Optional)

By injecting discrete volumes of cement grout into the bond zone after the initial, or primary, grout has set, a significantly improved load bearing performance can be provided. The cement grouts are injected through a separate grouting tube (i.e., sleeved pipe or tube à manchette (Figure 3), as in the Gewi pile system of Herbst, (Reference 13) or through the steel reinforcement itself (Tubfix and Ropress piles). In the latter case, the double packer is introduced into the steel core pipe (Reference 11), and the grout is ejected through the rubber sleeved ports at regular intervals (Figure 4). Postgrouting greatly improves the grout/soil bond, but in addition it may increase the nominal pile cross section, particularly in weaker soil layers or near ground level where natural in-situ horizontal stresses are small. Postgrout pumping pressures of over 600 psi are not uncommon.

Mascardi (Reference 11) also noted that in cases of repeated postgrouting, an effective pile diameter in the range of 12-30 inches can be achieved. Postgrouting tends to be most effective in ground where displacements can be imparted relatively quickly, such as sands and gravels, residual soils, shales, and some weaker sedimentary and low grade metamorphic formations. Jones and Turner (Reference 14) also noted a favorable response in stiff clay. No service records of good behavior in very soft non consolidated clay or soft peat has been recorded to date, although recent tests conducted in the Bay Mud of San Francisco have yielded encouraging results. (Reference 15).

3. LABORATORY RESEARCH

3.1 Phase 1

Testing of composite members has been conducted for decades, worldwide, and the results of 68 tests of axially loaded concrete filled tubes were addressed in a Steel Structures Research Council (SSRC) report. (Reference 16). Actual steel yield stress varied from 38 to 88 ksi, and concrete compressive strengths from 2.9 to 9.6 ksi. A table of data comparing these test loads with the theoretical allowable loads, based on the proposed modifications to the AISC allowable stress equations, was prepared to give an indication of actual safety factors. These ratios varied from 1.28 to 3.68, average 2.26, standard deviation 0.45 and a coefficient of variation of 20%.

The tests in Phase 1 were run with composite tubular members of uniform, high strength steel (nominal 80 ksi) and grout (minimum 4 ksi), such as would comprise certain sections of Pin Piles. (Reference 17). Specimen lengths were selected to provide a set of slenderness ratios that would be consistent with previous experiments. Details are summarized in Table 1, and proved consistent with the earlier tabulated work of SSRC.

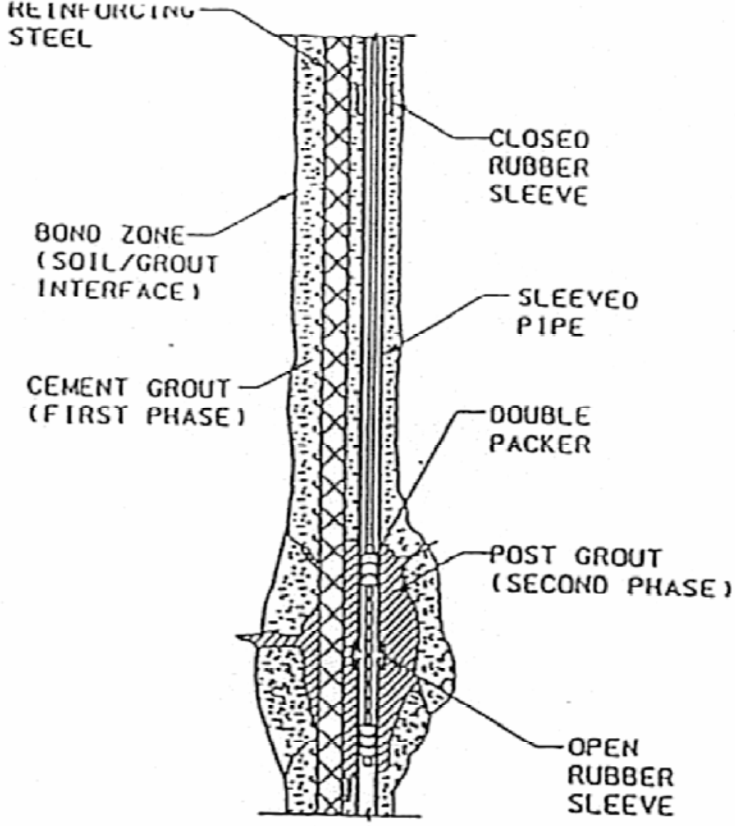


Figure 3 Representation of Type S1 Pin Pile during post grouting procedure (Reference 13)

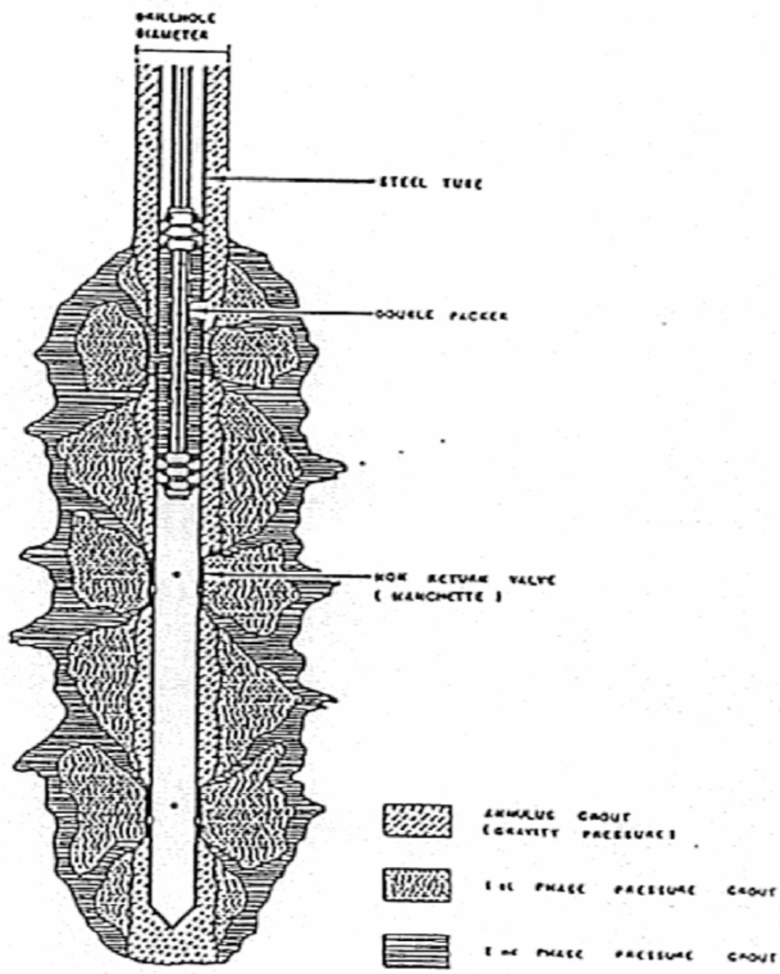


Figure 4 concept of repeated post grouting to enlarge effective grouted diameter (Reference 11)

Separate tests on material properties confirmed the specified minimum yield strengths to be 86 ksi (7"), and 99 ksi (5 1/2"), and grout strengths (28 day compressive) averaging about 5.6 ksi. Each column responded similarly throughout the loading range - initial local yielding at the ends followed by gradual bending. No evidence of buckling was observed. The shorter specimens exhibited a linear load deflection relationship to about 75% maximum load, while the longer casings were linear almost to maximum load.

Of particular interest in Table 1 is the Elastic Ratio (ER) for each configuration. ER is calculated as the quotient of resultant displacement and applied load, and is therefore a simple indicator of the effective composite elastic modulus of the grout filled casing. This directly determined value can then be used to ascertain the seat of load transfer during the cyclic loading of Pin Piles, as demonstrated in Section 4 below.

3.2 Phase 2

The typical Pin Pile casing joint consists of mating the male and female ends of successive lengths of casing. This joint is typically flush, with little or no resulting space between sections, and its strength is dependent on many factors including material yield strength, thread pitch, root size, length of splice, shoulder contact and the confining effects of pipe and grout. Tests were conducted (in tension also, but not detailed herein) on the typical Nicholson casing thread, with 3 ft. long samples with and without external banding reinforcement around the female end. (Table 2).

A comparison of the data of Table 1 (single casings) and Table 2(coupled casings) shows that for stub columns (simulating the fully braced pile configuration), no significant difference exists in the magnitudes of the ultimate loads or the ultimate failure modes. However, the ER values recorded for the Phase 2 tests were higher for two main reasons:

a) "Slop" in joints, creating higher total displacements, coupled with
b) the relatively short test lengths being more sensitive to these displacements.

Parallel tests, run in tension, showed the joints to have about 60% less capacity than in compression. Also, the failure mode in tension was explosive (i.e. the thread experienced sudden failure).

3.3 Phase 3

Clearly the structural capacity of Pin Piles with full length casing (S2, R2) can be designed conservatively by using the composite strength of the grout filled casing, ignoring the confining contribution of the annular grout. However, if the grout is neglected in the design of an internally reinforced bond zone (S1, R1), the resultant design would be significantly over-conservative. A series of tests were therefore run on such simulated bond zone configurations, as detailed in Table 3.

Each specimen was cast and tested in a plastic mold the lateral confining properties of which approximated that of a medium dense sand. The specimens were all 36 inches long and 10 3/4 inches in diameter. The length was selected to create a stub column, so allowing ignoring of slenderness effects, while the diameter reflected a typical effective pressure grouted bond zone diameter in situ.

Spec #	Reinforcement Configuration	Cross Section Steel (in^2)	Max Load (kips)		Equivalent 10 ft. Elastic Ratio
1A	None - Plain Grout	zero	209	212	2.20
1B	None - Plain Grout	zero	214		
2A	1 # 10	1·23	315	308	1.66
2B	1 # 10	1·23	300		
3A	1 # 14	2·40	390	365	1.53
3B	1 # 14	2·40	340		
4A	1 # 18	3·97	490	470	1.45
4B	1 # 18	3·97	450		
5A	2 # 10	2·45	367	393	1.68
5C	2 # 10	2·45	418		
6A	2 # 14	4·81	563	572	0.71
6B	2 # 14	4·81	580		
7A	1 # 14 + Simulated Spiral cage	2·40+	500	500	1.53
7B	1 # 14 + Simulated Spiral cage	2·40+	500		
8A	11 ea. 0.6" strand*	2·39	394	355	2.20**
8B	11 ea. 0.6" strand*	2·39	315		

Table 3 Summary of Phase 3 Laboratory Tests (Bond Zones)

* Strand fy = 270 ksi; all other rebar 60 ksi.

** Strand columns exhibited similar stiffness as plain grout columns. Strand stiffness in compression is suspect in these tests.

Linear behavior was noted over an average of 84% of the ultimate load capacity, and failure was characterized by axial crushing.

The relationships between ultimate load, and ER, and the cross sectional area of steel in the sample are shown in Figures 5 and 6 respectively. The benefit of the simulated spiral reinforcement is clearly demonstrated - an improvement in ultimate load of 135 kips (37%) over the comparable specimen without a confining cage.

4. FIELD RESEARCH

Concurrent with this laboratory research, Nicholson Construction was the design-build contractor on two significant Pin Pile projects. The larger was at an operational grain export facility on the Columbia River at Vancouver, Washington, where certain major structures were threatened by settlement as a result of deterioration of the original 4050 timber piles driven in 1934-1939. (Reference 18). Prior to installing the 840 replacement high capacity Pin Piles (300 kip working load), about half of which were to be located in the cramped basements of the three silo structures, an extensive test program was conducted, involving six full-scale special test piles.

The other project involved Pin Piles to arrest settlement of a Caustic Evaporator structure within a major active chemical plant near Mobile, Alabama. (Reference 7). As an enhancement of the Specifications, a total of six test piles were installed and tested to failure, before commencing work on the 123 production piles, each with a 200 kip design working load. At the end of the job, an additional three "research" piles were similarly tested to failure to provide further data on load transfer mechanisms.

Although each project in itself constitutes an excellent case history of the application, design, construction and performance of Pin Piles, the following description focuses solely on the special test programs.

4.1 United Grain Facility, Vancouver, WA

The design foresaw each pile to be drilled with 7" casing to a depth of approximately 70' from grade and so a minimum of 30' into a very dense gravelly and cobbly bed. The upper portion was to be reinforced by the casing, with the lower pressure grouted portion reinforced by a central reinforcing bar, in a standard S1 type configuration. (Figure 2). The Specification called for an underpinning system to ensure additional differential settlements of less than 1.5" in 100', and additional uniform settlements of less than 6".

The test program required the successful loading of three piles to 200% working load, held for a minimum 12 hour period. The working load was considered to be 300 kips for the test program, although final calculated individual pile loads were 252 to 292 kips, depending on location. These initial three piles (TP1-3) all reached the 600 kip maximum but all exhibited what appeared to be internal (structural) failure prior to the end of the hold period. As a result, and after structural adjustments, a second group of three piles successfully passed the test, and subsequently attained ultimate loads of up to 750 kips. These piles established the production pile structural detailing and criteria for minimum embedments into the bearing stratum. As for the Mobile piles, test loads were applied in cycles of increasing load, so permitting the partition of total pile settlements into elastic and permanent displacements at each maximum load attained. The elastic component therefore permitted the calculation of Elastic Ratio for each load maximum.

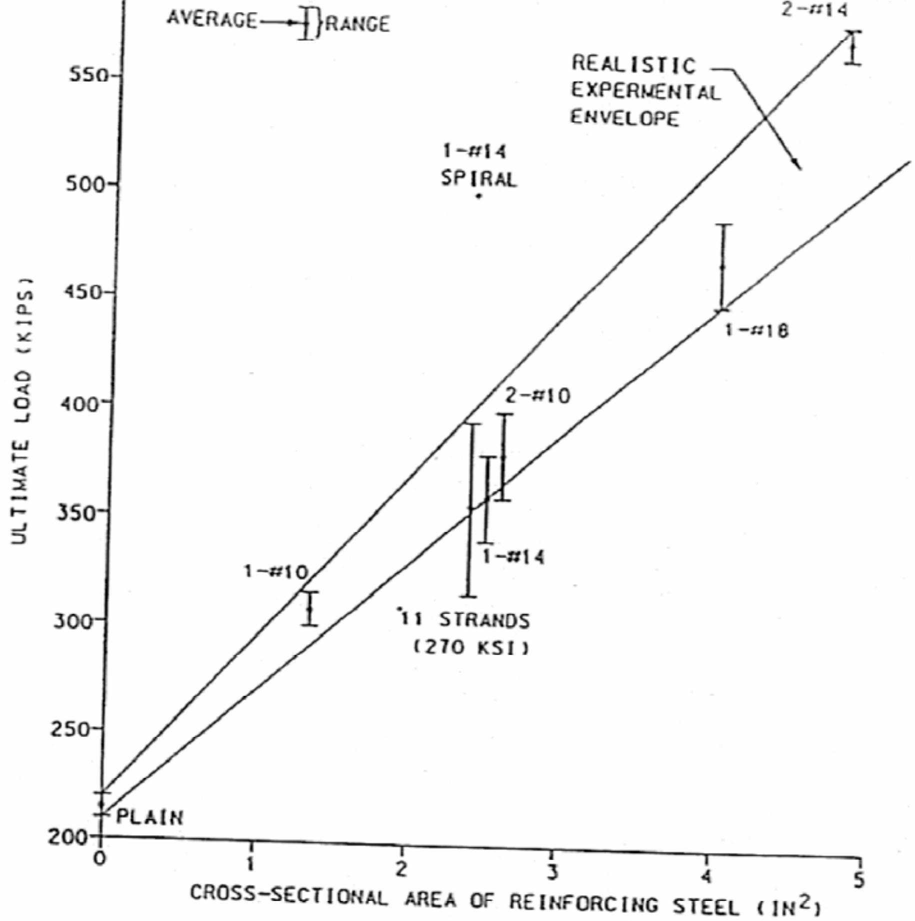


Figure 5 Ultimate compressive load vs. reinforcing steel area, Phase 3 Test:
All steel = 60 ski except for strand

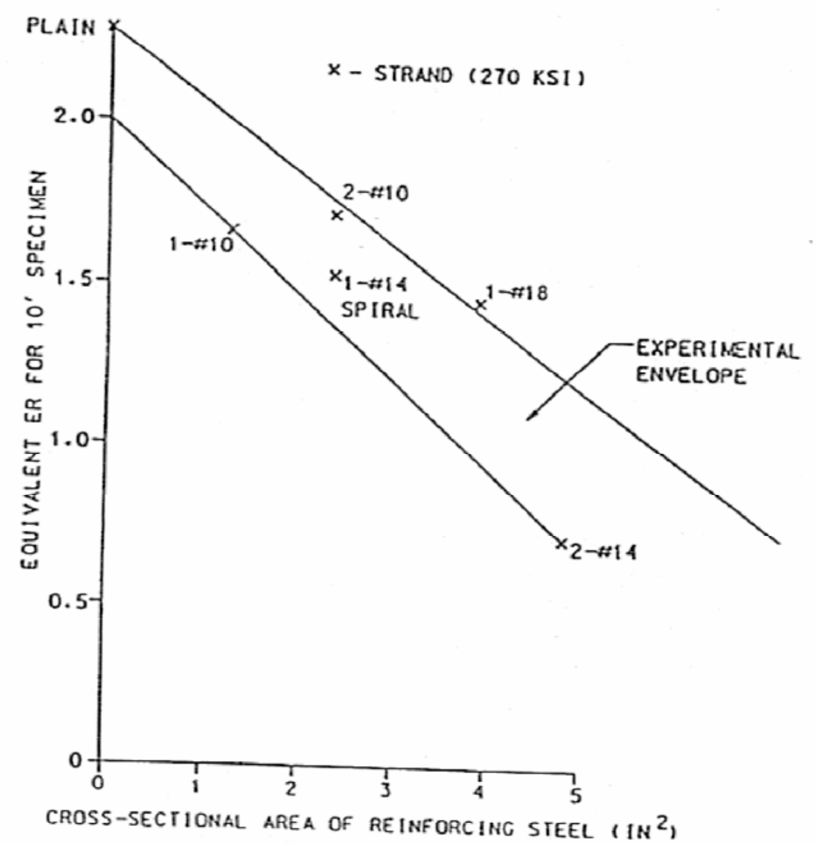


Figure 6 Equivalent ER vs. reinforcing steel area, Phase 3 Tests.

Summaries of the soil strata encountered, and the pile installation details are provided in Tables 4 and 5 respectively. The testing history of each pile was as follows:

- TP-2 - After 3 minutes at the 600 kip hold, load dropped explosively to 355 kips. Gained 1.40" of additional residual movement this cycle. Attained a maximum load of 300 kips with additional testing.

- TP-1 - At 5 minutes into the 75 kips step 30 minute hold, load dropped 11 kips. At 13.5 minutes into 600 kip hold, load dropped 11 kips, and dropped an additional 26 kips 15 seconds later. Reduced to alignment load. While reloading to 600 kips, at 588 kips, load dropped 14 kips. Reduced to alignment load. Reloaded to 600 kips. Between 4 and 5 hours into the load hold, load dropped explosively to 300 kips. Gained 0.992" of additional residual movement this cycle. Attained a maximum load of 300 kips with additional testing.

- TP-3 - At 45 minutes into 600 kips hold, load dropped explosively to 417 kips. Gained 0.578" of additional residual movement this cycle. Attained a maximum load of 450 kips with additional testing.

- TP-4 - Successfully tested pile at 525 kip load step with a 12 hour hold. Successfully tested pile at 600 kip load with a 12 hour hold. Pile supported 675 kip load for two hours, with 0.031" of creep during the last hour of the load hold. At 4 minutes into 750 kip hold, load dropped explosively to 372 kips. Gained 1.026" of additional residual movement this cycle. Attained a maximum load of 430 kips with additional testing.

- TP-5 - Successfully tested pile at 525 kip load step with a 3 hour hold. Successfully tested pile at 600 kip load with a 12 hour hold. Pile supported 675 kip load for two hours, with 0.030" of creep during the last hour of the load hold. While loading to 750 kips, at 740 kip test load, load dropped (not explosively) to 534 kips. Gained 0.721" of additional residual movement this cycle. Attained a maximum load of 574 kips with additional testing.

- TP-6 - Successfully tested pile at 525 kip load step with a 100 minute hold. Successfully tested pile at 600 kip load with a 12 hour hold. At 25 minutes into 675 kip load hold, load dropped explosively to 150 kips. Gained 0.721" of additional residual movement this cycle. Attained a maximum load of 574 kips with additional testing.

The creep performance of the piles at 600 kips was as follows. The specified acceptance criterion was a value less than 0.010" over the final hour. Pile TP-1 failed at between 4 and 5 hours into the hold, with a creep rate of 0.009" per hour prior to failure. Pile TP-2 failed after 4 minutes hold with a creep of 0.022" for the first 3 minutes. Pile TP-3 failed at 45 minutes into the hold with a creep of 0.022" for the 15 minutes prior to failure. Piles TP-4, 5, and 6 settled to creep rates of approximately 2 to 3 thousandths of an inch per hour after 6 hours into the 12 hour hold. (Table 6).

The hold duration at 525 kip load were increased for piles TP-4, 5, and 6 to verify the piles' acceptability at 525 kips. The creep rates reduced to near zero after 1 to 5 hours of hold. For TP4-6, which proved completely acceptable in every aspect, the total displacement at 300 kips averaged 0.339", and at 600 kips, an average of 1.247".

Test Pile #	Upper Length Soil Strata Sand Fill (ft)	Silt (ft)	Medium Dense Sand (ft)	Bond Length Soil Strata Medium Dense Sand (ft)	Very Dense Sand (ft)	Very Dense Gravels (ft)
TP-1	22	16	5	13	12	5
TP-2	20	15	5	7	9	14
TP-3	10	21	8	4	14	12
TP-4	10	21	6	6	14	5
TP-5	10	21	6	6	14	5
TP-6	10	21	6	6	14	5

Table 4 Soil Strata Thickness Encountered, United Grain.

Test Pile #	Installation Order	Total Pile Depth (ft)	Bond Length (ft)	Casing Insertion Into Bond Zone (ft)	Casing Length From Grade (ft)	Rebar Size	Rebar Length (ft)
TP-1	2	73	30	5	48	#14	30
TP-2	1	70	30	5	45	#14	30
TP-3	3	69	30	10	49	#14	32
TP-4	4	62	25	10	47	#18	27
TP-5	5	62	25	10	47	#18	27
TP-6	6	62	25	10	47	#18	27

Table 5 Pile Installation Details, United Grain.

Pile #	Creep @ 525 kip Load		Creep @ 600 kip Load				Creep @ 675 kip Load		Max Test Load Attained	Hold Duration @ Max Load	Failure Description
	0-10 min (in)	10-30 min (in)	0-10 min (in)	10-100 min (in)	100-240 min (in)	240-720 min (in)	0-10 min (in)	10-100 min (in)			
TP-1	.024	.018	.031	.048	.022	-----	-----	-----	600 kips	Approx 270 min	Explosive Drop to 300 kips
TP-2	.022	.016	-----	-----	-----	-----	-----	-----	600 kips	3 min	Explosive Drop to 355 Kips
TP-3	.028	.012	.028	.031 *	-----	-----	-----	-----	600 kips	45 min	Explosive Drop to 417 kips
TP-4	.024	.010	.022	.032	.012	.037	.025	.059	750 kips	4 min	Explosive Drop to 372 kips
TP-5	.022	.013	.040	.071	.026	.027	.054	.070	750 kips	120 min	Plunging Drop to 534 kips
TP-6	.021	.013	.032	.036	.010	.038	.035	.061 *	675 kips	25 min	Explosive Drop to 150 kips

Table 6 Test Pile Displacement Creep and Failure Behavior, United Grain.

Figures 7 and 8 show the changes in ER with increasing load for each pile. It was always recognized that this progressive increase was indicative of progressive debonding down the pile: if debonding were not occurring (i.e. if the pile were acting as a strut with fixed ends) then the ER would be constant, since deflection would be directly proportional to load. However, the question was the relation between ER and effective pile length, and until the Phase 1 laboratory tests this had not been satisfactorily resolved.

These tests showed that the ER for a 10' length of grout filled 7" casing was approximately 0.32. Thus, for a recorded pile ER of, say 2.0, it can be calculated that the effective elastic pile length would be (2.0/0.32) x 10' = 62.5'.

Table 7 summarizes ER values, and equivalent elastic lengths for each pile. The calculated free lengths are generous: no allowance is made for the decreased ER value of the pile in the casing/rebar overlap section. It is clear that explosive, structural failure occurred when the load had been fully shed to within a few feet of the bottom of the cased length. (It also is apparent that at 300 kips, the casing had debonded only about 30' below the ground surface, highlighting the true load holding capability of the poorer upper soils). The table also shows that the change from a #14 bar (TP1-3) to a #18 bar (TP4-6) gave the (centrally reinforced) bond zone an extra 75-150 kip capacity to resist explosive bursting failure. This compares with the 60-150 kip range identified in the Phase 3 Laboratory Tests. (Table 3).

4.2 Caustic Evaporator Structure, Mobile, AL

The underpinning was selected to stop settlements which had ranged from 0.038 to 0.178 inches per month per column over a 44 month period. (Reference 7). The design loads exerted by the 60 existing columns varied from 24 to 810 kips, and a standard S1 pile of 200 kips design working load was selected. A total of 123 piles were installed in groups of 1 to 6 piles per footing. The 7" casing through the upper zone combined with a 40' long #14 bar in the 37' of pressure grouted bond zone. (Figure 9). To prevent downdrag, the upper 47' of casing was coated with a low friction resin.

The first 6 piles (TP1-6) were installed about 200' east of the later three (TP7-9). At the former site, the very dense load bearing sands and gravels were encountered 12-18' below the surface, whereas at the latter they commenced about 30' down. Overlying these were variable fills, clays and silts. Construction details are provided in Table 8.

Cyclic load testing to a minimum specified target of 400 kips was conducted and the piles performed as follows:

- TP1: loaded in 50 kip cycles to 300 kips. While reloading, a plunging failure occurred at a maximum of 340 kips. The 7' casing was noticeably "wobbling" during loading. (Performance was quite linear to 300 kips.) A second loading program reached a maximum load of 300 kips before plunging resumed.

- TP2: loaded in 50 kip cycles to 400 kips and held for 12 hours (0.050" creep). Reloaded directly in 30 kip increments to 590 kips at which load the reaction beam had to be reset. Reloaded similarly, but a plunging failure occurred at 570 kips. (Performance linear to 550 kips.)

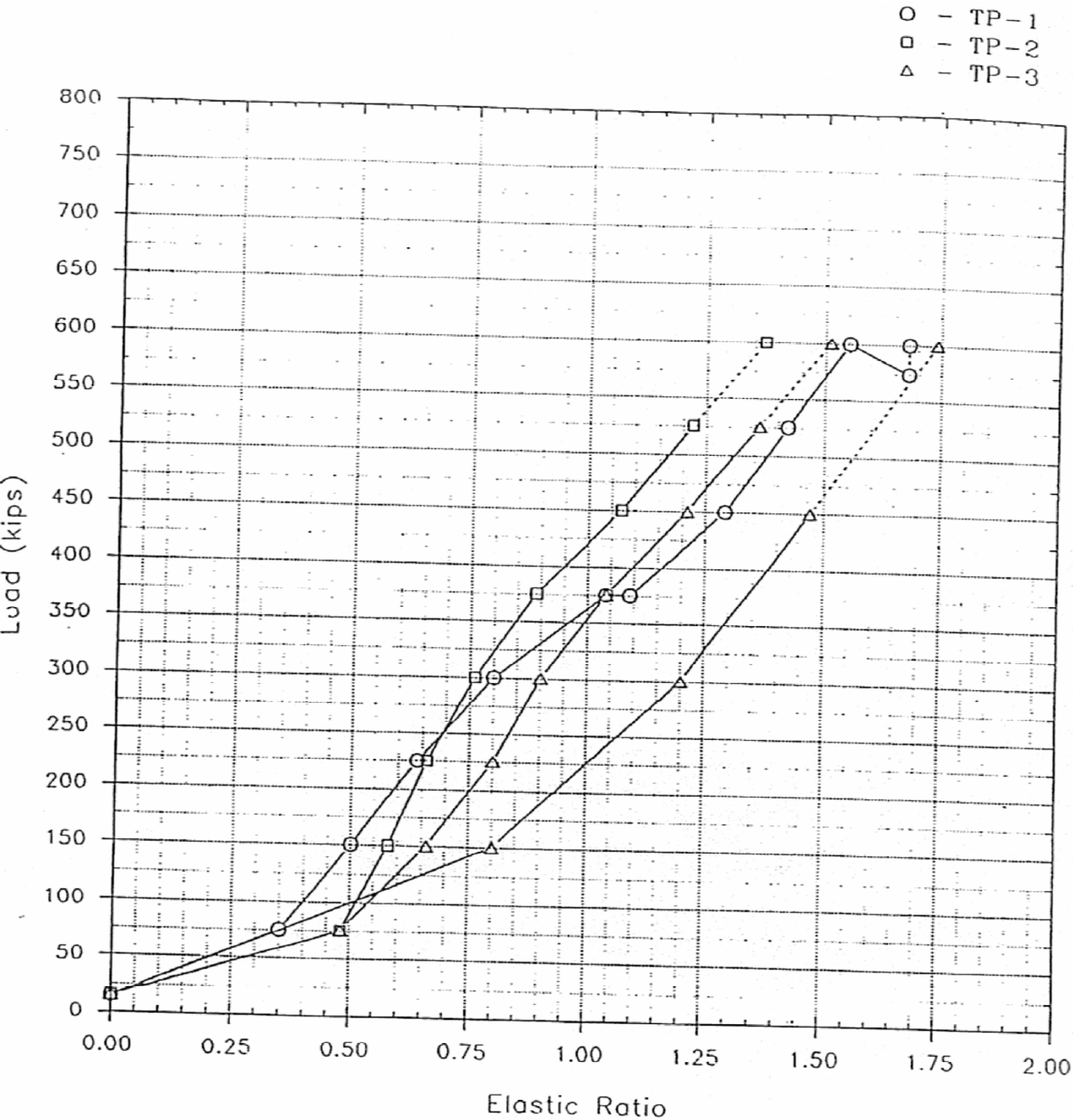
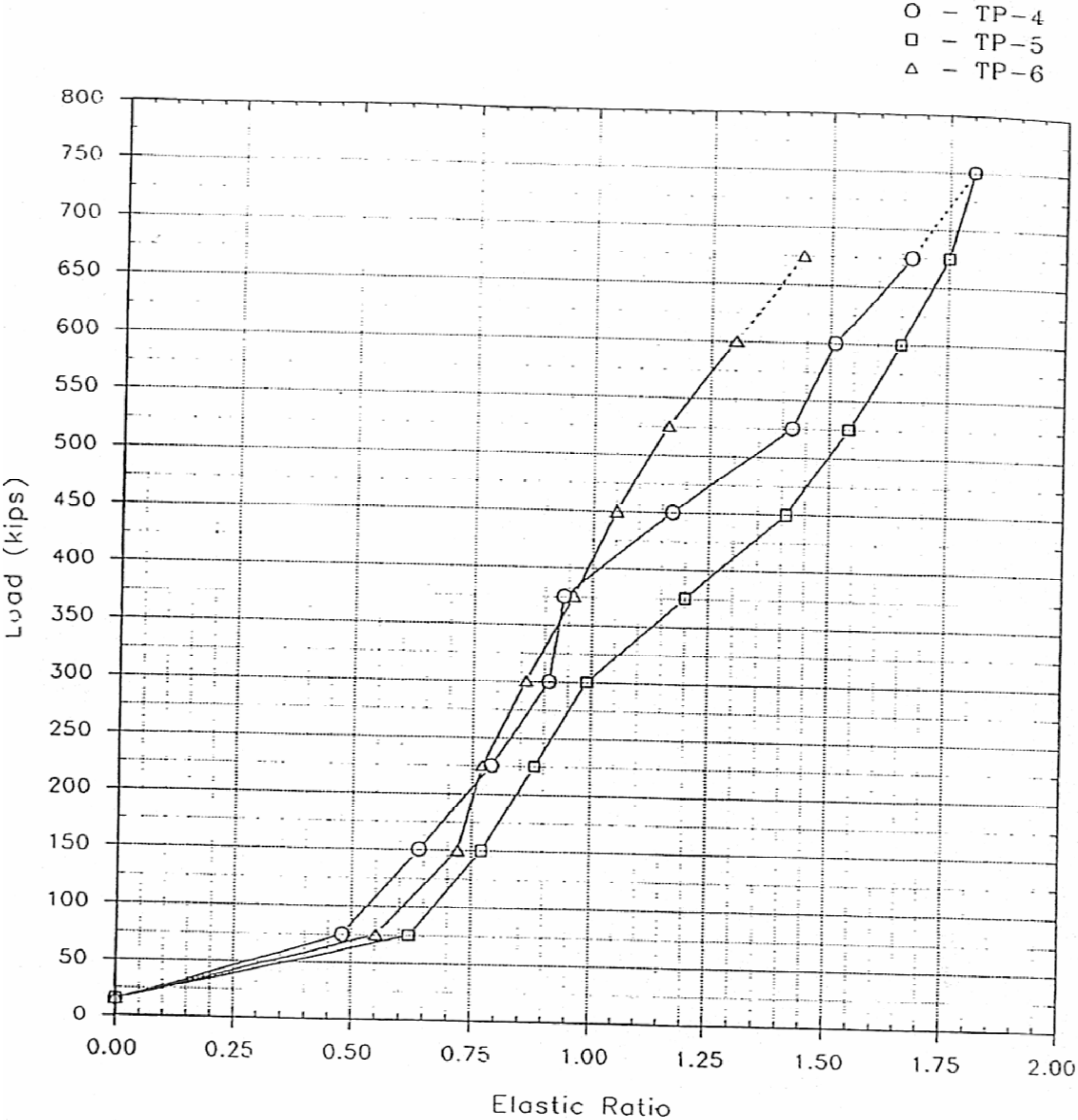


Figure 7 Elastic Ratio Comparison, Piles TP-1, 2, 3, United Grain.



<u>Figure 8</u> Elastic Ratio Comparison, Piles TP-4, 5, 6, United Grain.

Pile #	Casing Length (ft)	Total Pile Length (ft)	300 kip Load		450 kip Load		525 kip Load		600 kips		Failure Load		
			Elastic Ratio	Elastic Length (ft)	Elastic Ratio	Elastic Length (ft)	Elastic Ratio	Elastic Length (ft)	Elastic Ratio	Elastic Length (ft)	Failure Load (kips)	Elastic Ratio	Elastic Length (ft)
TP-1	50	75	0.80	25.0	1.29	40.3	1.42	44.4	1.68 *	52.5	600	1.68 *	52.5
TP-2	47	72	0.76	23.8	1.07	33.4	1.22	38.1	1.37 *	42.8	600	1.37 *	42.8
TP-3	51	71	0.90	28.1	1.21	37.8	1.36	42.5	1.51 *	47.2			
TP-3	51	71	1.20	37.5	1.47	45.9			1.74 *	54.4	600	1.74 *	54.4
TP-4	49	64	0.91	28.4	1.17	36.6	1.42	44.4	1.51	47.2	750	1.80 *	56.3
TP-5	49	64	0.99	30.9	1.41	44.1	1.54	48.1	1.65	51.6	750	1.80	56.3
TP-6	49	64	0.86	26.9	1.05	32.8	1.16	36.3	1.30	40.6	675	1.44 *	45.0

Table 7 Test Pile Elastic Ratio and Length, United Grain

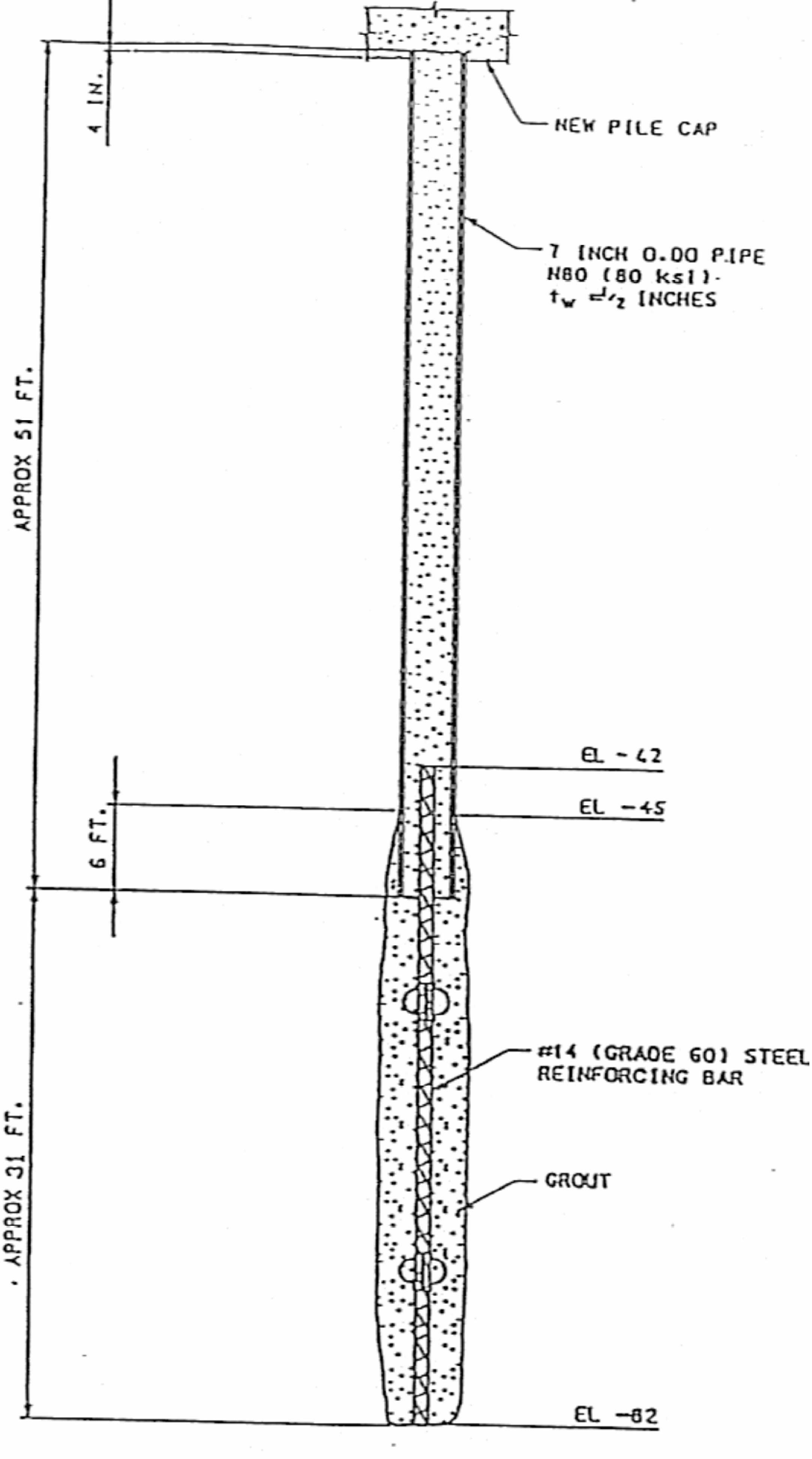


Figure 9 Standard Pin Pile configuration, Mobile.

Test Pile #	Total Depth Drilled	Grouting	Depth Pressure Grouted	Casing Left in Place	Reinforcement	Notes
TP1	82' (Drill Fluid 'A')	17 bags tremie + 10 bags pressure	82 to 54 i.e. 28'	60'	82 to 52' i.e. 30' of #14 rebar	o Upper 40' surrounded by 9-5/8" casing (open annulus)
TP2	82' (A)	20 bags + 13 bags	82 to 54 i.e. 28'	60'	82 to 48' i.e. 34' (#14)	• Ditto except pea gravel placed in annulus for lateral support
TP3	82' (A)	18 bags + 6 bags	82 to 52 i.e. 30'	57' (40' debonded)	82 to 52' i.e. 30' (#14)	• Problem during grouting; o Annulus clear for min. 12'; • Very low grout strengths.
TP4	82' (A)	27 bags + 15 bags	82 to 51 i.e. 31'	57' (40' debonded)	82 to 47; i.e. 35' (#14)	• Flush connected to TP3 from depth of 30' (Age 10 days at time.)
TP5	82' (Drill fluid 'B')	30 bags + 20 bags	82 to 45 i.e. 37'	51'	82 to 42; i.e. 40' (#14)	o High grout pressures. • Gravel near tip.
TP6	82' (B)	20 bags + 16 bags	82 to 45 i.e. 37'	51'	82 to 42; i.e. 40' (#14)	Ditto as for TP5
TP7	82' (B)	18 bags + 18 bags	82 to 45 i.e. 37'	51'	None	• Relatively low grout pressure
TP8	82' (B)	18 bags + 27 bags	82 to 45 i.e. 37'	51'	82 to 42; i.e. 40' of #18 bar	o Slightly oversize bit (9"). o Relatively low grout pressure
TP9	82' (B)	18 bags + 14 bags	82 to 45 i.e. 37'	51'	11 strands ea 0.6" dia, full length	• Lower grout pressure

Table 8 Summary of Test Pile Construction Data, Mobile.

- TP3: loaded in 50 kip cycles to a creep/plunge failure at the 300 kip cycle maximum. (Performance linear to that point.)

- TP4: loaded in 50 kip cycles to 400 kips (linear to about 300-350 kips) and held for 2 hours (0.858" creep). Reloading gave maximum of 286 kips with large permanent movement. Reloaded directly to a maximum of 252 kips with plunging failure. (Linear to 240 kips.)

- TP5: loaded directly in 50 kip increments to 220 kips and zeroed. Loaded directly in 100 and 50 kip increments to 400 kips (0.028" creep in 2 hours) and then directly up to 440 kips (0.106" creep in 12 hours) (Linear). Reloaded in 100 kip cycles to 400 kips (0.043" creep in 1 hour) and then to 450 kips when, 15 minutes into the hold (already 0.090" creep) a sudden explosive failure occurred, to a 290 kip residual. (Linear to 440 kips.) Reloading achieved 335 kip maximum .

- TP6: loaded directly in 50 kip increments to 220 kips and zeroed. Loaded directly in 100 and 50 kip increments to 400 kips (0.048" creep in 2 hours), but with a bump at about 350 kips (linear to there). Continued directly to 440 kips (0.220" creep in 12 hours), with bump 58 minutes into the hold. Reloaded (linear) in 100 kip cycles to 450 kips (0.100" creep in 1 hour) and destressed. During next cycle, at 500 kips, after 5 minutes hold (0.097" creep), an explosive failure occurred, to a 300 kip residual.

- TP7: loaded cyclically in 50 kip increments to maximum of 400 kips (very linear to 375 kips). Immediate explosive failure to residual of 140 kips and permanent movement of about 1.3 inches.

- TP8: loaded cyclically in 50 kip increments to 500 kips, very easily. Reloaded directly in 50 kip increments to 550 kips where a substantial shock occurred. Reloading only reached 280 kips.

- TP9: loaded cyclically in 50 kip increments to 490 kips when one strand on a reaction anchor broke. Upon reloading, a maximum of 450 kips was reached when a steady plunge failure occurred. Subsequent loads would not exceed 280 kips.

This is summarized in Table 9.

Figures 10 and 11 show the development in ER values for each group of piles, while Table 10 summarizes the ER values and loads at which permanent settlements and creep became significant and failure occurred. It was apparent that permanent sets became significant when the load reached within 5 to 10' of the bottom of the casing; creep became significant when the load appeared to reach or exceed the bottom of the casing; failure then occurred soon after (either by overloading the soil-grout interface, or by structural failure in the bond zone).

Debonding is envisioned as occurring around the free length at the interface between the steel casing and the surrounding grout and ground. Once ruptured it is not fully recovered. The rate at which debonding occurs (and so, to a large extent the ultimate pile capacity) is dictated by (i) the ground itself, (ii) the surface properties of the outside of the casing, and (iii) the efficiency and vigor of the grouting, especially the option to continue injecting until grout emerges around the casing at the surface.

TEST PILE	MAX RECORDED LOAD (kips)	PRESSURE GROUTED LENGTH (feet)	MODE OF FAILURE	AVERAGE GROUT/SOIL BOND AT MAX RECORDED LOAD (kips/foot)	COMMENT
TP1	337	28	Grout-soil	12	o Failed at 300 kips, second load cycle.
TP2	590	28	Grout-soil	21, apparently, but realistically 13	o Failed at 570 kips, second cycle. o Estimate 40% load resisted permanently in free length.
TP3	300	30	Grout-soil	10	o Possibly poor grout. o Failed on first cycle.
TP4	400	31	Grout-soil	13	o Failed on first cycle.
TP5	450	37	Internal	12 OK	o Failed on first cycle.
TP6	500	37	Internal	14 OK	o Failed on first cycle.
TP7	400	37	Internal	11 OK	o Failed on first cycle.
TP8	550	37	Internal	15 OK	o Failed on first cycle.
TP9	490	37	Grout-soil	13	o Failed on second cycle at 450 kips.

Table 9 Summary of Test Pile Performance, Mobile

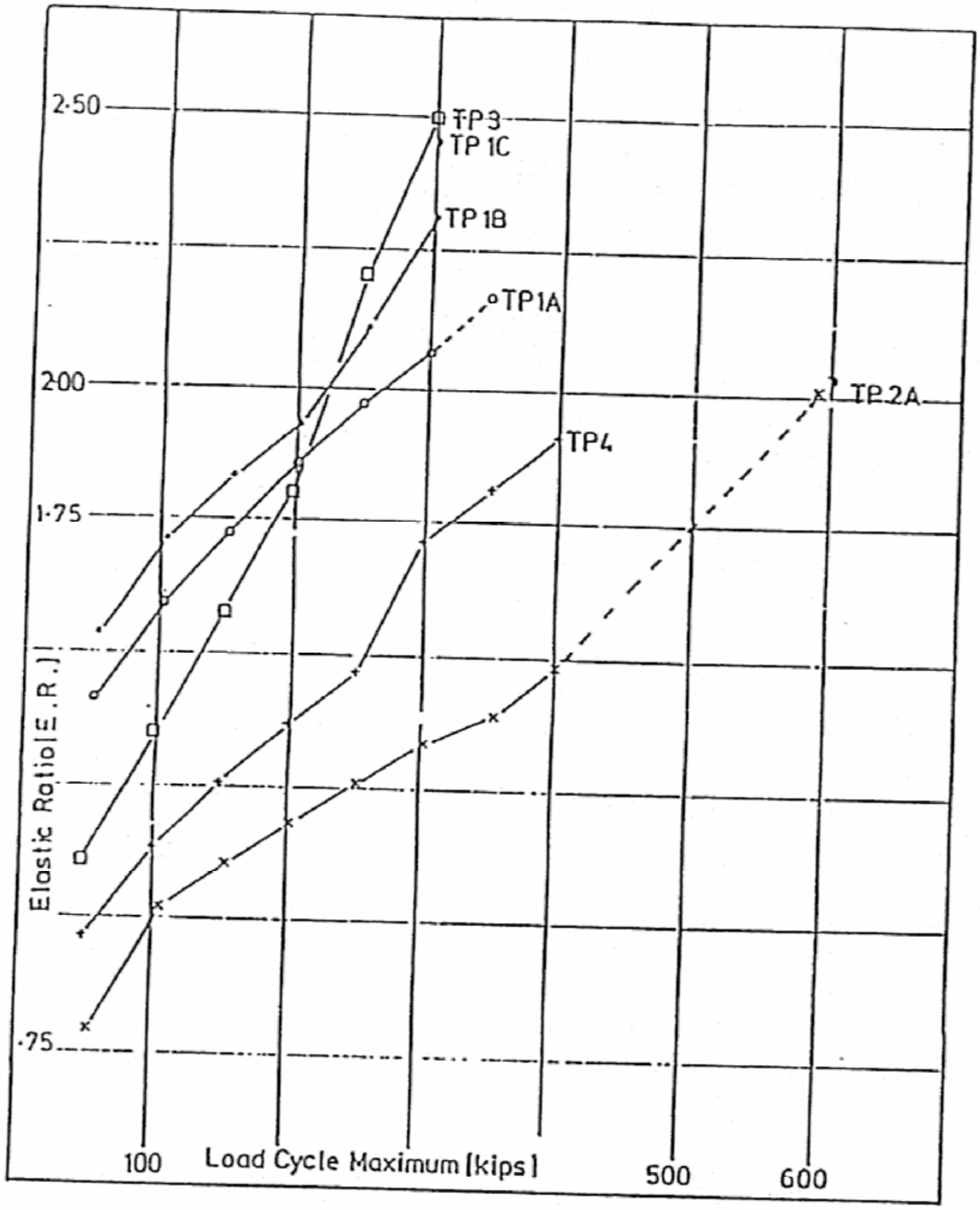


Figure 10 Elastic ratios for TP1-4, Mobile.

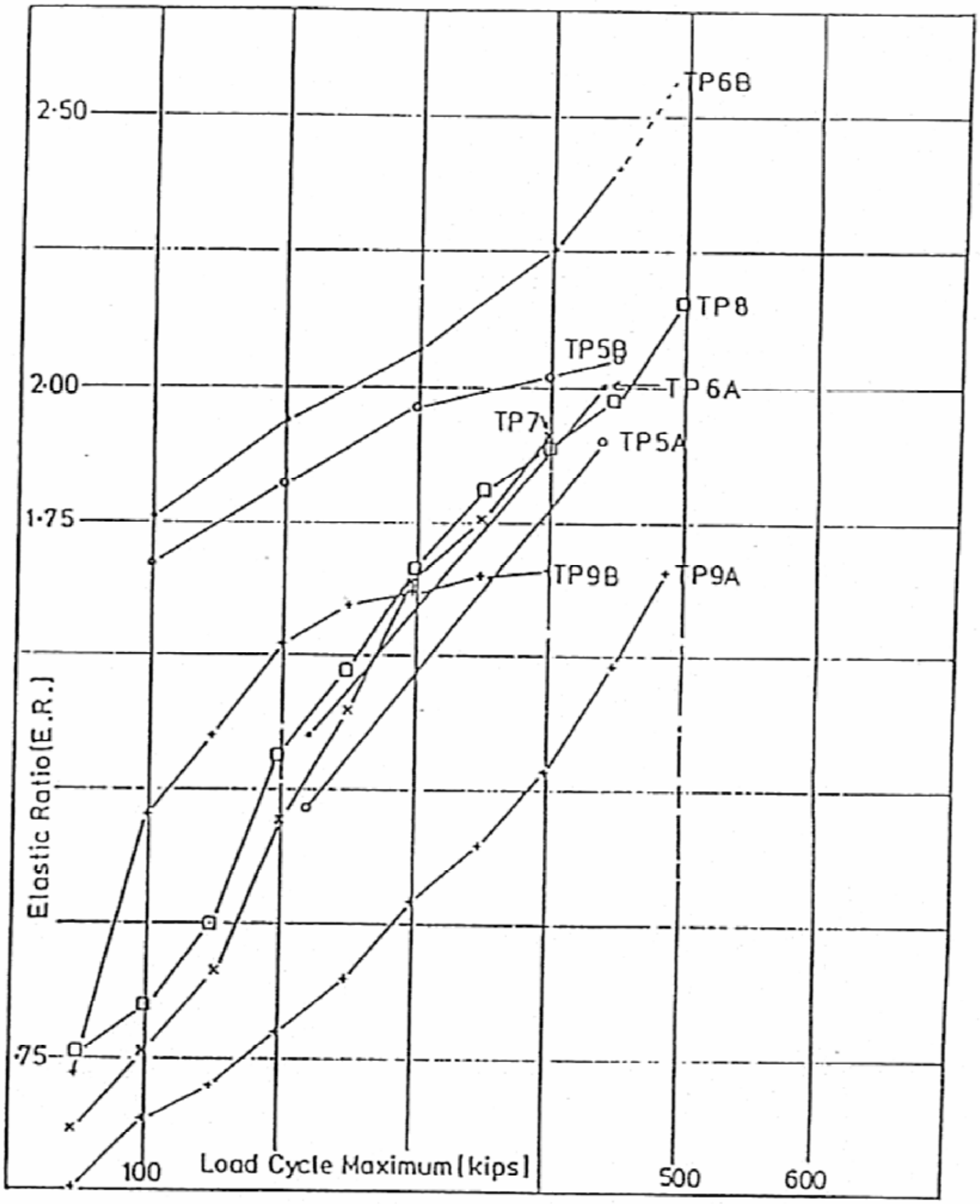


Figure 11 Elastic ratios for TP5-9, Mobile.

TEST PILE	(A) INFERRED ER WHEN PERMANENT MOVEMENTS EXCEEDED 0.100" (Load)	(B) INFERRED ER WHEN CREEP EXCEEDED 0.040" in 60 Minutes (Load)	(C) ESTIMATED ER AT FAILURE (Load)	CALCULATED FREE LENGTH FROM COLUMN (A) (Feet)	CALCULATED FREE LENGTH FROM COLUMN (B) (Feet)	CALCULATED FREE LENGTH FROM COLUMN (C) (Feet)	ACTUAL CASING IN GROUND PLUS ABOVE GRADE (Feet)
TP1	1.9 (200)	2.1 (250)	>2.2 (337)	58	64	67+	62
TP2	1.3 (300)	1.5 (>400)	2.0 (590)	40	45+	61	62
TP3	1.8 (200)	2.2 (250)	2.5 (300)	55	67	76	59
TP4	1.5 (>250)	1.8 (>350)	1.9 (400)	45	55	58	59
TP5	1.7 (350)	1.8 (400)	1.9 (450)	52	55	58	53
TP6	1.7 (350)	1.9 (400)	>2.4? (500)	52	58	73?	53
TP7	1.6 (300)	1.9 (400)	1.9 (400)	48	58	58	53
TP8	1.7 (300)	2.4 (550)	2.4 (550)	52	73	73	53
TP9	1.5 (450)	1.5 (450)	1.7 (450)	?	?	?	53

Table 10 Inferred and Estimated ER's and Equivalent Effective Free Lengths at Significant Intervals, Mobile.

By comparing piles TP1 to 4, it appears that the free length composition of TP1 (debonded from the soil by outer casing), and TP3 (40' debonded by an epoxy coating) each ensured that load was transferred down to the bond zone very readily. They then experienced a grout/soil failure in the bond zone at very similar loads under very similar elastic, permanent and creep conditions. TP2 (gravel packed annulus), and TP4 (presumably the coating had been abraded off during installation) were both able to "use up" more load in the free length and so reduce the amount of load requiring to be resisted in the bond zone below. Therefore, they had significantly higher total load capacities than TP2 and 3, although in reality the average grout-soil bond mobilized in the bond zone was most likely very similar.

This highlights again the fact that fully grouted pin piles may often transfer significant portions of their load well above the bond zone, and that therefore this can be promoted (or avoided) by appropriate choice of casing protection and grouting processes. This fact also explains the relative stiffness of pin piles in general especially when founded through the blocky or permeable materials characteristic of site fill preparations.

The results from the other five test piles were wholly consistent - appreciable load being transferred in upper reaches, progressive and irreversible debonding, and the links between depth of load transfer and key failure phenomena. The most interesting information relates to the composition of the bond zone and the mode and magnitude of failure. Table 11 summarizes the performance of each of these piles (each with 51' of casing and 37' of pressure grouting).

TP7 may be regarded as the "starter" unit. It debonded steadily and progressively, apparently from about 20 feet beneath the surface to within 5 feet of the bottom of the casing. At this point it is inferred that the load was then transferred progressively from the grout filled casing into the upper part of the unreinforced bond zone below, by end bearing (Figure 12). Simplistically, and assuming an effective bond diameter of, say, 10 inches, this grout would provide a resistance of $\pi \times 10^2/4 \times 5$ ksi = 392 kips before rupturing in a brittle fashion. This is very close to the maximum load recorded.

Regarding the other related piles, namely TP5, 6 (#14 bar reinforcement), and TP8 (#18 bar), it is reasoned that the presence of reinforcement permits their bond zones to accept more load before an explosive failure situation is achieved. Table 10 also indicates that the load was able to be transferred deeper into the bond zone in proportion to the weight of the reinforcement. In the case of the former pair (ultimate load 450-500 kips), this reinforcement may have contributed a further 50-100 kips, while in the latter case, the bigger bar provided a 150 kip benefit. In each of these cases, the grout-soil bond capacity was sufficient to force an internal, structural failure, as opposed to a geotechnical, grout-soil failure. (Table 11). Again these data are consistent with the laboratory Phase 3 tests.

TP≠	BOND ZONE COMPOSITION	MAXIMUM ER RECORDED AND LOAD	RESIDUAL LOAD (Or Maximum Subsequently Achievable)	MODE OF FAILURE
5	≠14 bar	2.0 - 450 kips	335 kips	Explosive
6	≠14 bar	>2.4 - 500 kips	300 kips	Explosive
7	No reinforcement	1.9 - 400 kips	140 kips	Explosive
8	≠18 bar	2.4 - 550 kips	280 kips	Explosive
9	11 strands (also in casing)	1.7 - 490 kips	280 kips	Plunge

Table 11 Summary of Performance of Second Group of Piles, Mobile:

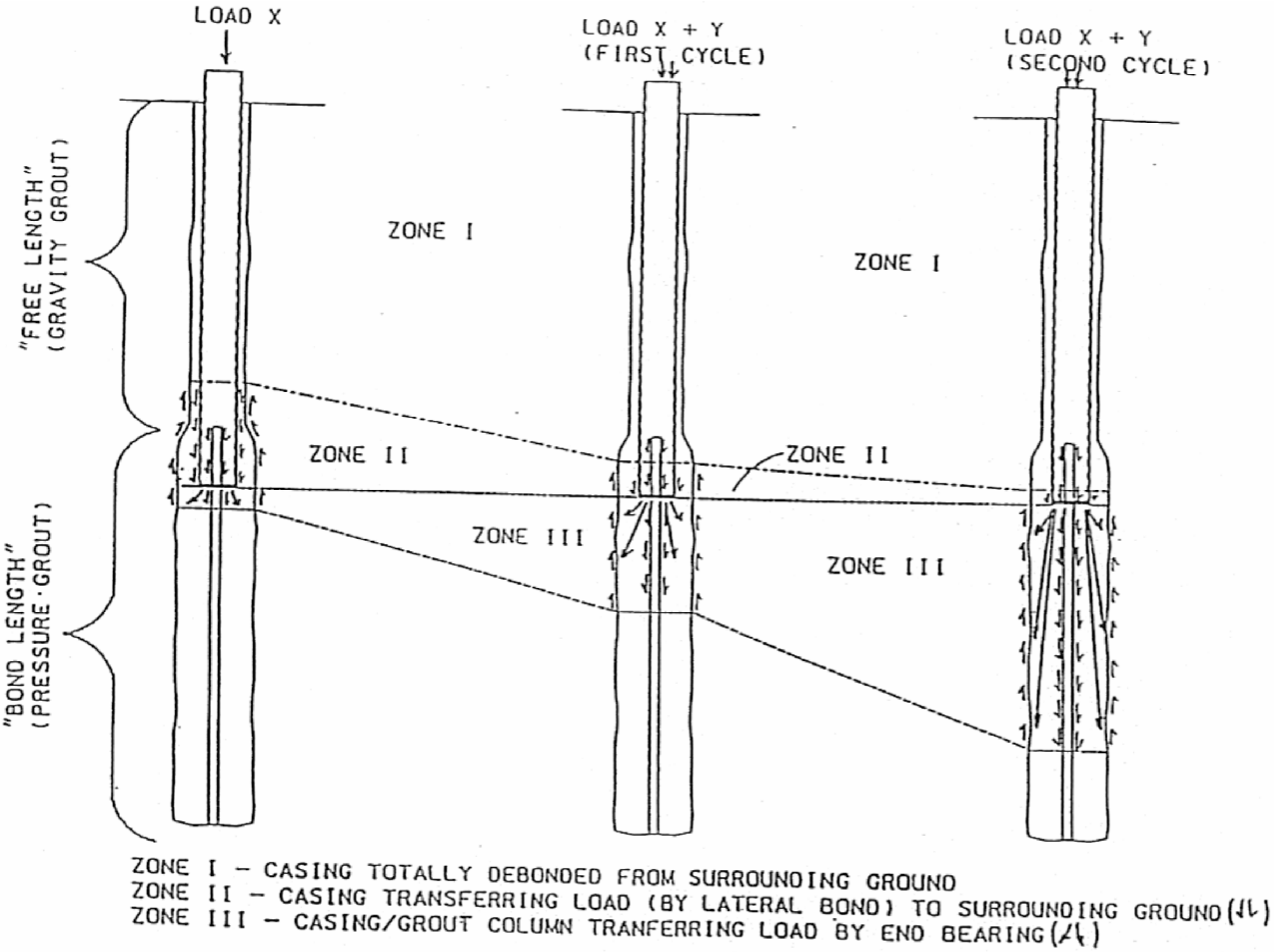


Figure 12 Conceptual illustration of load transfer mechanisms at increasing loads, and at repeated load.

TP9 was, of course, reinforced full length, and so the free length behaved more stiffly, although the debonding characteristics were again clear. In this pile it is interpreted that the bond zone remained structurally intact and that the total applied load was sufficient to cause grout-soil failure. It was noted (Table 8) that the grouting pressures in this pile were lower, and so it is reasonable to expect that the grout-soil bond capacity was therefore less than in its companion piles. Generally, however, the results indicate that a geotechnical failure was close in each of these piles which reached structural failure first.

Reverting to TP1-4, it is clear that the bond zone was capable of structurally resisting the applied load, but that the total geotechnical bond available (reflecting both unit friction and length) was exceeded. TP2 gives an anomolously high value, but it must be noted that the ER analysis confirms that perhaps as much as 40% of the applied load continued to be resisted by the upper 40 feet of casing during the latter stages of loading, resulting in a true bond zone load of perhaps only 570 x 60%, i.e., 342 kips at grout/soil failure. This would place its performance far closer to that of its similar piles TP1, TP3, and TP4.

5. SUMMARY

The laboratory tests have provided the key to determining effective debonding lengths in high capacity Pin Piles: the breakthrough is the Elastic Ratio concept. The Phase 3 Tests, on simulated bond zones, clearly demonstrated the range of capacities which can be expected, and this range was confirmed in both the field test programs. Both these field programs also highlighted the fact that large amounts of load may be shed in the upper reaches of Pin Piles, into soil which is usually neglected as having load transfer potential at the design phase. This explains the surprising stiffness of Pin Pile systems in their lower range of capacity.

The field tests also remind us that in certain favorable geotechnical conditions, the grout-soil bond which contemporary drilling and grouting methods promote can be so large that it is the internal load carrying capacity of the pile, i.e. its structural strength, which is the limit over ultimate capacity. The ER approach to field analysis of Pin Pile testing offers a precise analytical and predictive tool, especially when combined with creep data: when the extent of apparent casing debonding reaches to within a few feet of the end of the casing, explosive failure may be expected shortly. At such times, the creep monitored may be more a result of grout/steel interfacial phenomena rather than grout/soil, as conventionally assumed.

This analytical method opens the door to Pin Pile acceptance criteria similar to those used for prestressed ground anchors where elastic performance and creep patterns are used: this would be more rigorous than current "geometrical construction" type methods.

Two related questions remain to be addressed, namely the puzzle of why a failure load can be recorded in a pile, lower than a load previously reached, and why failure can occur during a creep test at constant load. The first case is simply explained by reverting to the concept of non-recoverable bond: once the virgin interface around the casing has been disrupted, it cannot sustain the same level of bond stresses. Therefore, when reloaded, the load must pass below that point to be resisted (Figure 12). This means that progressively less of the casing is available for bond, and so progressively higher proportions of the load must be resisted in the bond zone. This bond zone has a finite capacity (internal and external), and when this is exceeded, failure results.

The second riddle has a similar explanation. As less of the casing becomes capable of resisting load as a result of progressive debonding the average peripheral bond stresses increase. This increase accelerates the rate of interfacial creep, which reflects a continuing, accelerating progressive debonding. At lower loads, this creep tendency is low, and soon stabilized: at higher loads, this creep rate will be higher and may reflect a rate of debonding so relatively fast that the underlying bond zone is being required to accept a substantially and progressively higher proportion of the load over a time interval within the period of the creep test. Again, when the critical amount of load is transferred at the bond zone, a failure will occur. This time of transfer may vary from almost instantaneous to many minutes.

REFERENCES

1. Bruce, D.A., Ingle, J.L. and Jones, M.R. (1985). "Recent Examples of Underpinning Using Minipiles." 2nd International Conference on Structural Faults and Repairs, London , April 30 - May 2, pp. 13-28.

2. 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.

3. Bruce, D.A. (1989). "American Developments in the Use of Small Diameter Inserts as Piles and In Situ Reinforcement." International Conference on Piling and Deep Foundations, London, May 15-18, pp. 11-22.

4. Bruce, D.A. (1992). "Recent Progress in American Pinpile Technology." Proc. ASCE Conference, "Grouting, Soil Improvement and Geosynthetics", New Orleans LA, Feb. 25-28, pp. 765-777.

5. Pearlman, S.L. and Wolosick, J.R. (1992). "Pin Piles for Bridge Foundations". 9th Annual International Bridge Conference, June 15-17, 8 pp.

6. Bruce, D.A. and Gemme, R. (1992). "Current Practice in Structural Underpinning Using Pinpiles." Proc. NY Met. Section ASCE Seminar, New York, April 21-22, 46 pp.

7. Bruce, D.A., Hall, C.H. and Triplett, R.E. (1992). "Structural Underpinning by Pinpiles." Proc. DFI Annual Meeting, New Orleans, LA. October 21-23, 30 pp.

8. Pearlman, S., Wolosick, J., Groneck, P. (1993). "Pin Piles for Seismic Rehabilitation of Bridges". 10th Annual International Bridge Conference, June 14-16, 1993, 12 pp.

9. ASCE Committee on Placement and Improvement of Soils (1987). "Soil Improvement, A Ten-Year Update". Proceedings of Symposium at ASCE Convention, Atlantic City, New Jersey, April 28.

10. Lizzi, F. (1982). "The Pali Radice (Root Piles)". Symposium on Soil and Rock Improvement Techniques Including Geotextiles, Reinforced Earth and Modern Piling Methods, Bangkok, December, Paper D1.

11. 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 D3.

12. Bruce, D.A. (1989). "Methods of Overburden Drilling in Geotechnical Construction - A Generic Classifiction." Ground Engineering 22 (7), pp 25-32. Also published in "Drill Bits - The Official Publication of the International Drilling Federation", Fall 1989, pp 7, 8, 10, 11, 13, 14.

13. Herbst, T.F. (1982). "The GEWI Pile - A Solution for Difficult Foundation Problems". Symposium on Soil and Rock Improvement Techniques Including Geotextiles, Reinforced Earth and Modern Piling Methods, Bangkok, December, Paper D1-10.

14. Jones, D.A. and Turner, M.J. (1980). "Post-grouted Micro Piles". Ground Engineering, 11 (4), pp. 14-20.

15. DFI and Caltrans (1993). "Pile Load Test Results at a Deep Bay Mud Site Using Various Pile Types" . A DFI and Caltrans Specialty Seminar, Oakland, California, September 9 and 10, 1993.

16. SSRC Report. Task Group 20, Structural Stability Research Council. "A Specification for the Design of Steel-Concrete Composite Columns," Engineering Journal, American Institute of Steel Construction, Vol. 16, No. 4 (Fourth Quarter, 1979), pp. 101-115.

17. Kenny, J., Bruce, D.A., and Bjorhovde, R. (1992). "Behavior and Strength of Composite Tubular Columns in High Strength Steel". Research Report No. ST-13, April 1992. Department of Civil Engineering, University of Pittsburgh, Pittsburgh, Pennsylvania.

18. Groneck, P., Bruce, D.A., Greenman, J., and Gingham, G. . "Foundation Underpinning at an Operating Grain Export Facility," Civil Engineering Magazine, (in print).