# RESEARCHES INTO THE BEHAVIOR OF HIGH CAPACITY PIN PILES[SM]

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## SYNOPSIS

Minipiles are known by many names, but are generically small diameter, cast-in-place bored piles. Although they have been used in Europe for over 40 years, in the United States, where they are commonly referred to as Pin Piles[sm], they have became a popular choice for underpinning only during the last 10 years. The paper describes fundamental laboratory and field researches recently conducted to better understand load transfer mechanisms. This work has led to the development of the Elastic Ratio concept which is now proving extremely useful in analyzing and predicting pile performance, and in particular the phenomenon of progressive debonding with increasing load.


## 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 allowable load range has been extended from 250-500 kN to up to 1500 kN, while special test piles have yielded capacities of around 3500 kN 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 high axial loads.

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 characteristics.
>
> Phase 2, as Phase 1 but including connected sections with threaded ends.
>
> 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, (Reference 7), the other at a grain silo complex near Port Vancouver, WA (Reference 9). The latter case history is summarized in this paper.

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

### 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.

The successive steps of pile construction are well known and documented (e.g. References 1-9). They comprise drilling and casing, placing of reinforcement and tremie grouting, and, typically, pressure grouting of the bond zone.

In most countries, the temporary casing is fully extracted (as the auger must always be) during the pressure grouting process. However, in the United States, it has been proved 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 often permeable upper horizons, and 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 1:

- Type S1 - A steel pipe is rotated into the soil using water to externally flush the cuttings up around the outside of the pipe . 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 excess pressure. The pipe is then seated back into the grouted bond zone. 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 may be used later to further enhance soil/grout bond (References 10 and 11).

- Type S2 - The pile is installed in the same fashion as the S1 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 - This 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

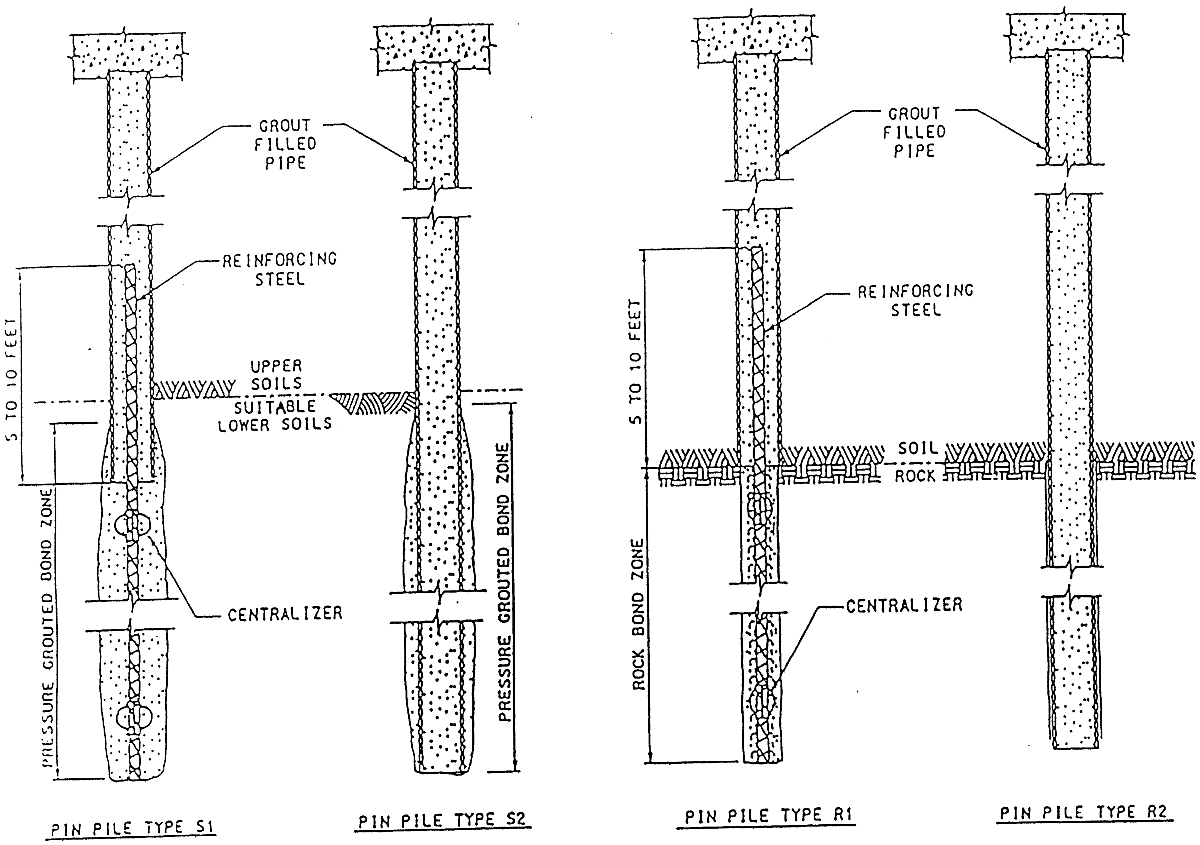


Figure 1 Pin Pile types in soil and rock (Reference 5).

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 bond inside the pipe (typically 1.5 to 3m).

- Type R2 - This type differs from the R1 pile in that it uses a full length steel pipe. Centralized reinforcement is dictated by internal pile capacity. 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. At the desired final depth, grout is tremied from the bottom, and additional grout is pumped to ensure full grouting of the rock bond zone.

## LABORATORY RESEARCH

### 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 12). Actual steel yield stress varied from 260 to 600 N/mm$^2$, and concrete compressive strengths from 20-66 N/mm$^2$. 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 given 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 550 N/mm$^2$) and grout (minimum 28 N/mm$^2$), such as would comprise certain sections of Pin Piles. (Reference 13). 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.

Separate tests on material properties confirmed the specified minimum yield strengths to be 593 N/mm$^2$ (178 mm dia.), and 583 N/mm$^2$ (140 mm dia.), and grout strengths (28 day compressive) averaging about 39 N/mm$^2$. 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 below.

### 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, threat 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 0.9m long samples with and without external banding reinforcement around the female end. (Table 2).

A comparison of the data of Table 1 (single casing) 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:

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

Table 1 Summary of Phase I Laboratory Tests (Single Casing Lengths). (Imperial Units)

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

| Specimen # | Dia. (in) | Length (in) | Wall (in) | Max Load (kips) | Elastic Ratio for 10 ft equivalent length |
|---|---|---|---|---|---|
| 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) (Imperial Units)

B = Banded protection around female end
U = Unbanded

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

The tension tests 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).

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 mould, the lateral confining properties of which approximated that of a medium dense sand. The specimens were all 0.9m long and 273mm 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.

Linear behavior was noted over an average of 84% of the ultimate 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 2 and 3 respectively. The benefit of the simulated spiral reinforcement is clearly demonstrated - an improvement in capacity (37%) over the comparable specimen without a confining cage.

## 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 14). Prior to installing the 840 replacement high capacity Pin Piles (140 tonnes service 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 design foresaw each pile to be drilled with 178mm casing to a depth of approximately 21m from grade and so a minimum of 9m 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 1). The Specification called for an underpinning system to ensure additional differential settlements of less than 38mm in 30m and additional uniform settlements of less than 150 mm.

The test program required the successful loading of three piles to 200% service load, held for a minimum 12 hour period. The service load was considered to be 140 tonnes for the test program, although final calculated individual pile loads were slightly lower, depending on location. These initial three piles (TP1-3) all reached the 280 tonnes 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 340 tonnes. These piles established the production pile structural detailing and criteria for minimum embedments into the bearing stratum. 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 the Elastic Ratio for each load cycle maximum.

Summaries of the soil strata encountered, and the individual pile installation details are provided in Tables 4 and 5 respectively. Table 6 summarizes the creep and failure behavior.

| 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 | .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) (Imperial Units)

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

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

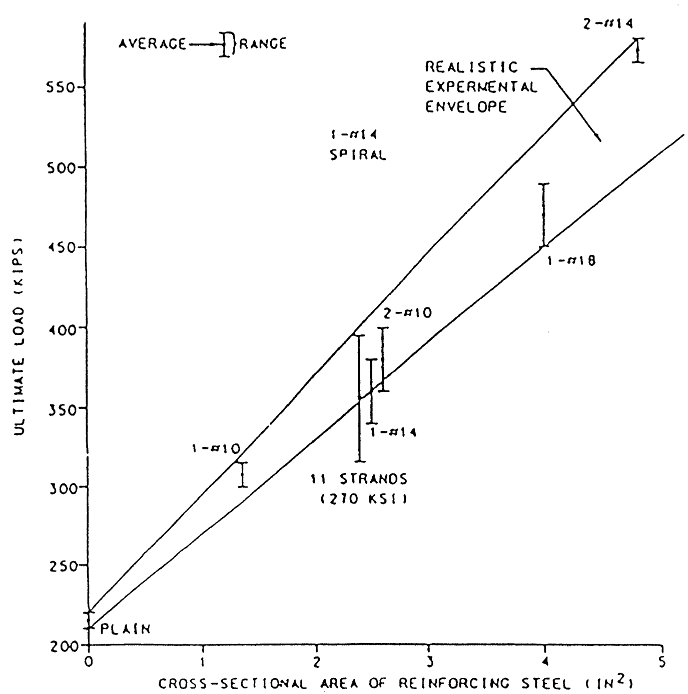


Figure 2 Ultimate compressive load vs. reinforcing steel area, Phase 3 Tests. All steel = 60 ksi except for strand

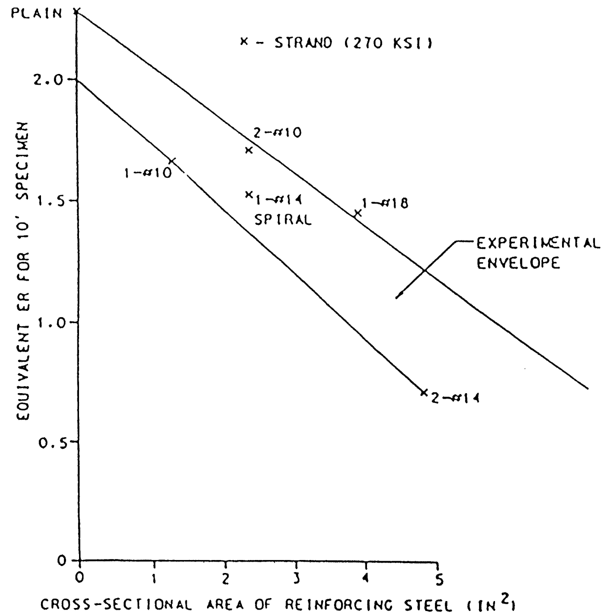


Figure 3 Equivalent ER vs. reinforcing steel area, Phase 3 Tests.

| Test Pile # | Upper Length Soil Strata | | | Bond Length Soil Strata | | |
|---|---|---|---|---|---|---|
| | Sand Fill (m) | Silt (m) | Medium Dense Sand (m) | Medium Dense Sand (m) | Very Dense Sand (m) | Very Dense Gravels (m) |
| TP-1 | 6.7 | 4.8 | 1.5 | 3.9 | 3.6 | 1.5 |
| TP-2 | 6.0 | 4.5 | 1.5 | 2.1 | 2.7 | 4.2 |
| TP-3 | 3.0 | 6.3 | 2.4 | 1.2 | 4.2 | 3.6 |
| TP-4 | 3.0 | 6.3 | 1.8 | 1.8 | 4.2 | 1.5 |
| TP-5 | 3.0 | 6.3 | 1.8 | 1.8 | 4.2 | 1.5 |
| TP-6 | 3.0 | 6.3 | 1.8 | 1.8 | 4.2 | 1.5 |

Table 4 Soil Strata Thickness Encountered, United Grain, Vancouver, WA

| Test Pile # | Installation Order | Total Pile Depth (m) | Bond Length (m) | Casing Insertion Into Bond Zone (m) | Casing Length From Grade (m) | Rebar Size | Rebar Length (m) |
|---|---|---|---|---|---|---|---|
| TP-1 | 2 | 21.9 | 9 | 1.5 | 14.4 | #14 | 9 |
| TP-2 | 1 | 21.0 | 9 | 1.5 | 13.5 | #14 | 9 |
| TP-3 | 3 | 20.7 | 9 | 3 | 14.7 | #14 | 9.6 |
| TP-4 | 4 | 18.6 | 7.5 | 3 | 14.1 | #18 | 8.1 |
| TP-5 | 5 | 18.6 | 7.5 | 3 | 14.1 | #18 | 8.1 |
| TP-6 | 6 | 18.6 | 7.5 | 3 | 14.1 | #18 | 8.1 |

Table 5 Pile Installation Details, United Grain, Vancouver, WA
Note: #14 rebar has 1 3/4 inch diameter (i.e. 44 mm)
#18 rebar has 2 1/4 inch diameter (i.e. 57 mm)

| Pilo # | Creep @ 234 tonnes 0-10 min (mm) | Creep @ 234 tonnes 10-30 min (mm) | Creep @ 268 tonnes 0-10 min (mm) | Creep @ 268 tonnes 10-100 min (mm) | Creep @ 268 tonnes 100-240 min (mm) | Creep @ 268 tonnes 240-720 min (mm) | Creep @ 301 tonnes 0-10 min (mm) | Creep @ 301 tonnes 10-100 min (mm) | Max Test Load Attained before failure | Hold Duration @ Max. Load before failure | Failure Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| TP-1 | 6 | 5 | 8 | 12 | 6 | --- | --- | --- | 268 tonnes | 270 min | Explosive Drop to 134 tonnes |
| TP-2 | 6 | 4 | --- | --- | --- | --- | --- | --- | 268 tonnes | 3 min | Explosive Drop to 158 tonnes |
| TP-3 | 7 | 3 | 7 | 8 | --- | --- | --- | --- | 268 tonnes | 45 min | Explosive Drop tp 86 tonnes |
| TP-4 | 6 | 3 | 6 | 8 | 3 | 9 | 6 | 15 | 335 tonnes | 4 min | Explosive Drop to 166 tonnes |
| TP-5 | 6 | 3 | 10 | 18 | 7 | 7 | 14 | 18 | 330 tonnes | --- | Plunging Drop to 238 tonnes |
| TP-6 | 5 | 3 | 8 | 9 | 3 | 10 | 9 | 15 | 301 tonnes | 25 min | Explosive Drop to 67 tonnes |

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

| Pile # | Casing Length (m) | Total Pile Length (m) | 134 tonnes Elastic Ratio | 134 tonnes Elastic Length (m) | 201 tonnes Elastic Ratio | 201 tonnes Elastic Length (m) | 234 tonnes Elastic Ratio | 234 tonnes Elastic Length (m) | 268 tonnes Elastic Ratio | 268 tonnes Elastic Length (m) | Failure Load (tonnes) | Failure Load Elastic Ratio | Failure Load Elastic Length (m) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| TP-1 | 15.0 | 22.5 | 0.80 | 7.5 | 1.29 | 12.1 | 1.42 | 13.3 | 1.68 | 15.8 | 268 | 1.68 | 15.8 |
| TP-2 | 14.1 | 21.6 | 0.76 | 7.1 | 1.07 | 10.0 | 1.22 | 11.4 | 1.37 | 12.8 | 268 | 1.37 | 12.8 |
| TP-3 | 15.3 | 21.3 | 0.90 | 8.4 | 1.21 | 11.3 | 1.36 | 12.8 | 1.51 | 14.2 | | | |
| TP-3 | 15.3 | 21.3 | 1.20 | 11.3 | 1.47 | 13.8 | | | 1.74 | 16.3 | 268 | 1.74 | 16.3 |
| TP-4 | 14.7 | 19.2 | 0.91 | 8.5 | 1.17 | 10.9 | 1.42 | 13.3 | 1.51 | 14.2 | 335 | 1.80 | 16.9 |
| TP-5 | 14.7 | 19.2 | 0.99 | 9.3 | 1.41 | 13.2 | 1.54 | 14.4 | 1.65 | 15.5 | 335 | 1.80 | 16.9 |
| TP-6 | 14.7 | 19.2 | 0.86 | 8.1 | 1.05 | 9.8 | 1.16 | 10.9 | 1.30 | 12.2 | 301 | 1.44 | 13.5 |

Table 7 Test Pile Elastic Ratio and Length, United Grain, Vancouver, WA

Figures 4 and 5 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 3m length of grout filled 178mm 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 3m = 18.8m.

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 meter of the bottom of the cased length. (It also is apparent that at 140 tonnes, the casing had debonded only about 9m 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 (TP 4-6) gave the (centrally reinforced) bond zone an extra 35-70 tonnes capacity to resist explosive bursting failure. This compares with the same range identified in the Phase 3 Laboratory Tests. (Table 3).

## CONCLUSION

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 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 determinant of ultimate pile 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 meter 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 and safely 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 the point to be resisted (Figure 6). 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 is 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.

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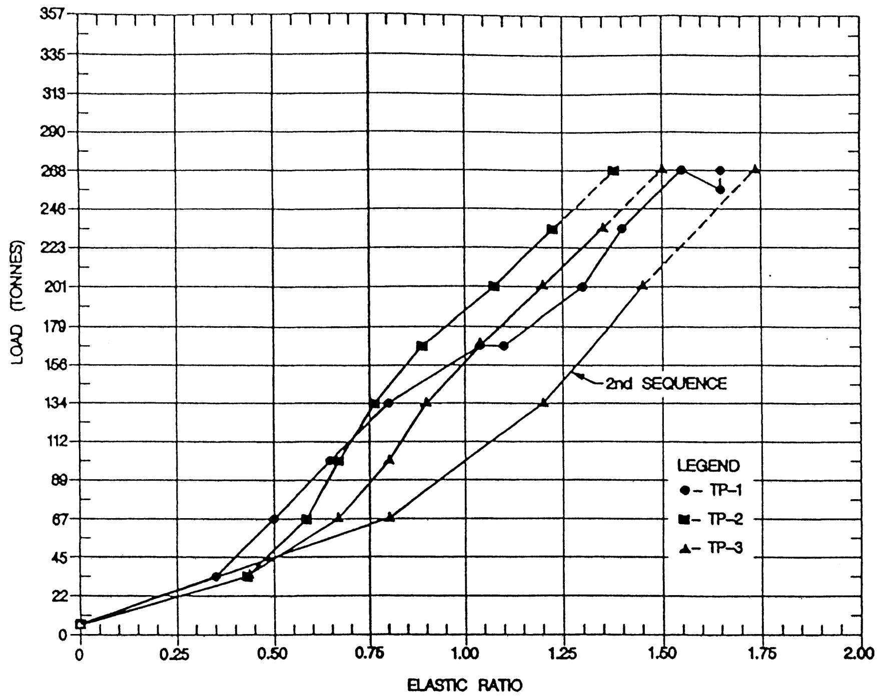


Figure 4 Elastic Ratio Comparison, Piles TP-1, 2, 3, United Grain, Vancouver, WA

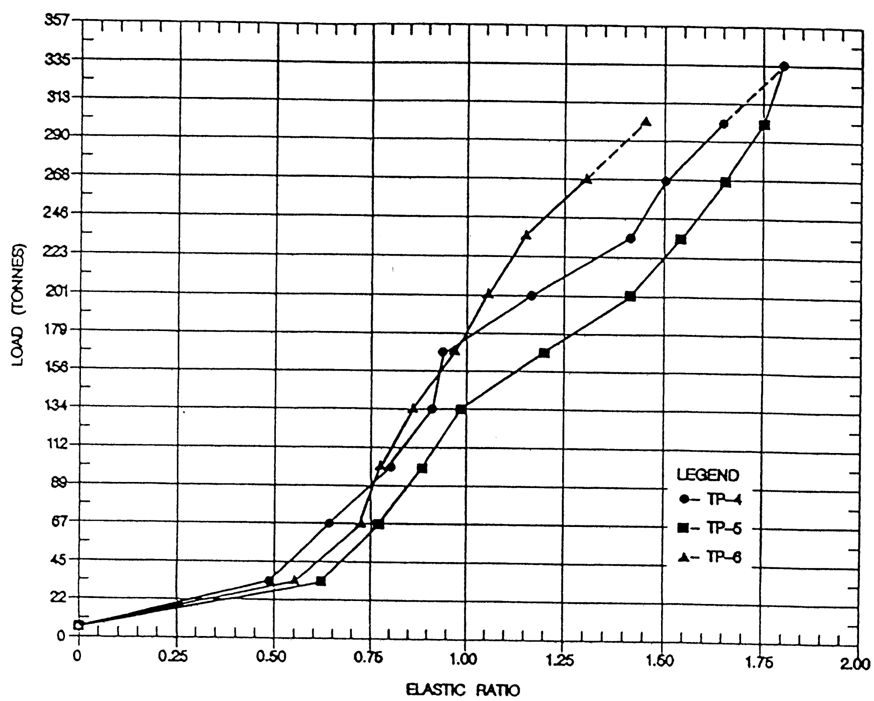


Figure 5 Elastic Ratio Comparison, Piles TP-4, 5, 6, United Grain, Vancouver, WA

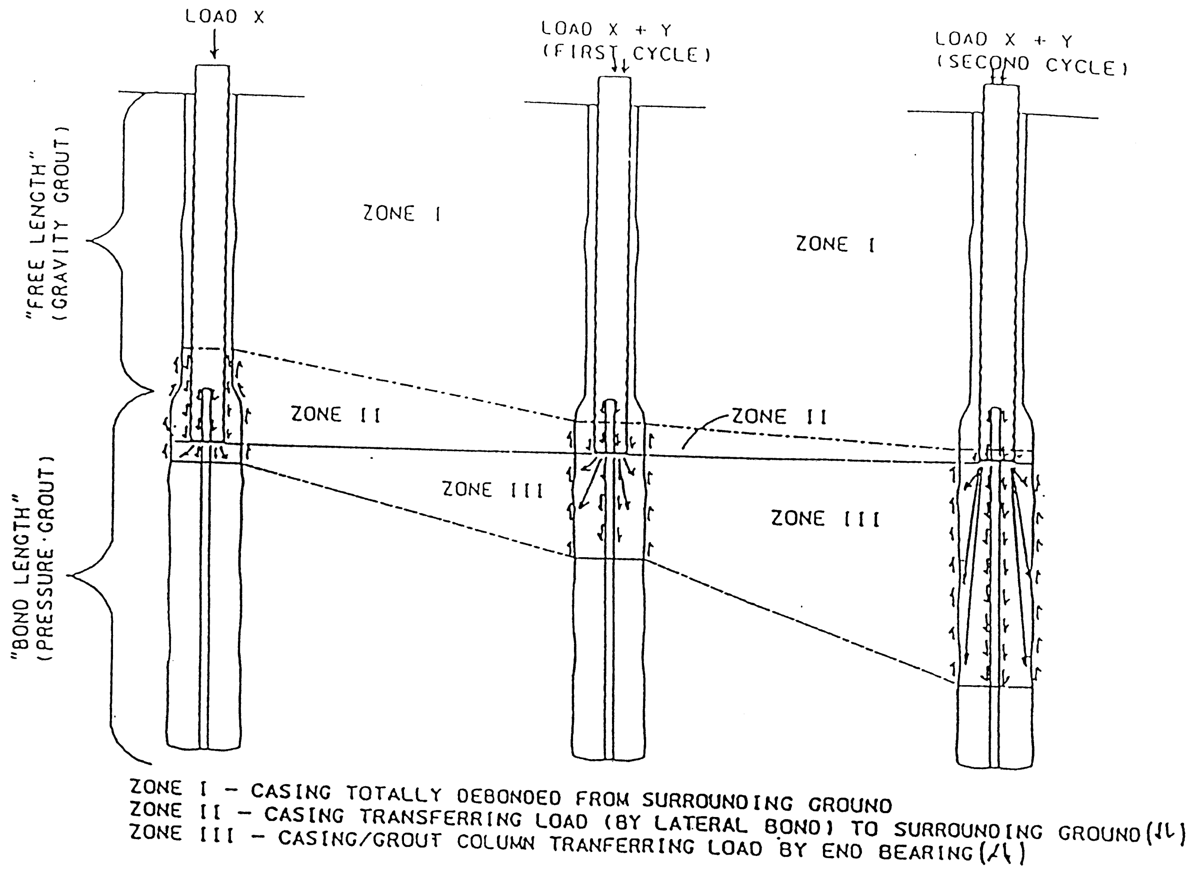


Figure 6 Conceptual illustration of load transfer mechanisms at increasing loads, and at repeated load.