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

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## ABSTRACT

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. Pin Piles are becoming more popular in applications for seismic rehabilitation of bridges, and this paper focuses on this aspect via recent case histories.

## 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 allowable load range has been extended from 50-100 kips 220-450 kN to up to 300 kips (1340 kN) while special test piles have yielded capacities of over 1000 kips (4450 kN) in favorable conditions.

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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 Phase1 but using connected casing 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.

## 2. CONSTRUCTION AND CLASSIFICATION

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 any 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 bond length, 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 casing 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 casing is withdrawn over the length of the bond zone, additional grout is pumped under excess pressure. The casing 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 casing is installed to the full length of the bond zone after pressure grouting is completed.
  - post-grouting is not feasible with this type of installation.

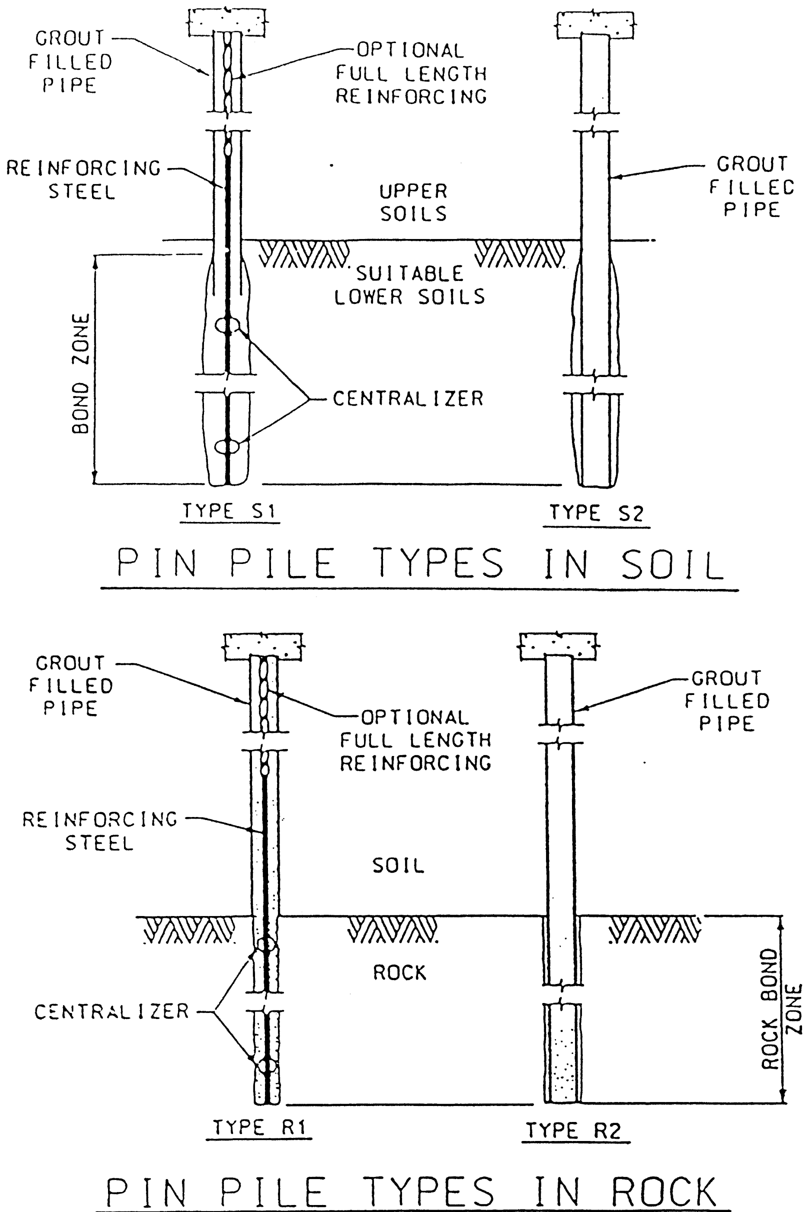


Figure 1 Generic Pin Pile Configurations in Soil and Rock

- 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 rock, a smaller diameter drill string is advanced through its center to drill the rock bond zone. Neat cement grout is then tremied from the bottom, and a reinforcing element is placed in the rock bond zone to complete the installation. A minimum transfer length is required for the reinforcement to develop bond inside the casing (typically 5 to 10 feet) (1.5 to 3.0m).

- Type R2 - This type differs from the R1 pile in that it uses a full length steel casing. The possible need for 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.

## 3. 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 38 to 88 ksi, (260 to 610 MPa), and concrete compressive strengths from 2.9 to 9.6 ksi (20 to 66 MPa). 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) (550 MPa) and grout (minimum 4 ksi) (28 MPa), 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. These data proved consistent with the earlier tabulated work of SSRC.

| 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 Casing Lengths)
1 in. = 25.4 mm; 1 kip = 4.45 kN; 1 ft. = 0.305 m
*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 Casing Lengths)

B = Banded
U = Unbanded
*10 ft. equivalent length
1 in. = 25.4 mm; 1 kip = 4.45 kN

Separate tests on material properties confirmed the specified minimum yield strengths to be 86 ksi (590 MPa) for the 7 inch (178 mm) dia. casing, and 99 ksi (680 MPa) for the 5.5 inch (140 mm) dia. casing. Grout strengths (28 day compressive) averaged about 5.6 ksi (39 MPa). 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 newly coined term Elastic Ratio (ER) for each configuration. ER is calculated by dividing the resultant displacement (in thousandths of an inch) by the applied load (in kips), 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 is formed by 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 36 inch (91 cm) 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:

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 tubular plastic mould, the lateral confining properties of which approximated that of a medium dense sand. The specimens were all 36 inches (91 cm) long and 10 3/4 inches (27.3 cm) in diameter. The length was selected to create a stub column, so allowing slenderness effects to be ignored, 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.

## 4. FIELD RESEARCH - UNITED GRAIN SITE

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

| 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 (1860 MPa); all other rebar 60 ksi (410 MPa).

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

1 in$^2$ = 645 mm$^2$; 1 kip = 4.45kN; 1 in. = 25.4 mm

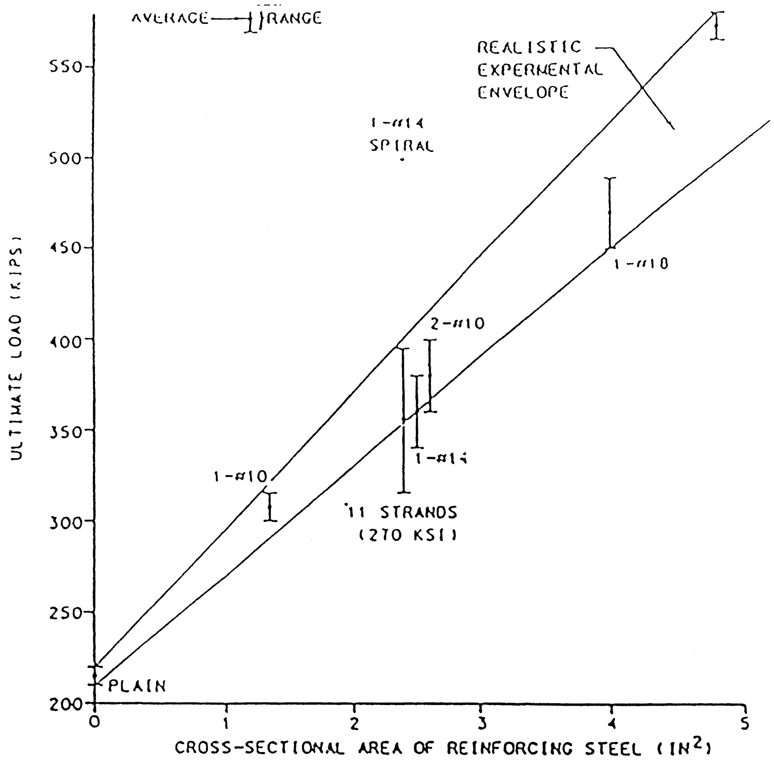


Figure 2 Ultimate Compressive Load vs. Reinforcing Steel Area, Phase 3 Tests. All steel = 60 ksi (410 MPa) except for strand.

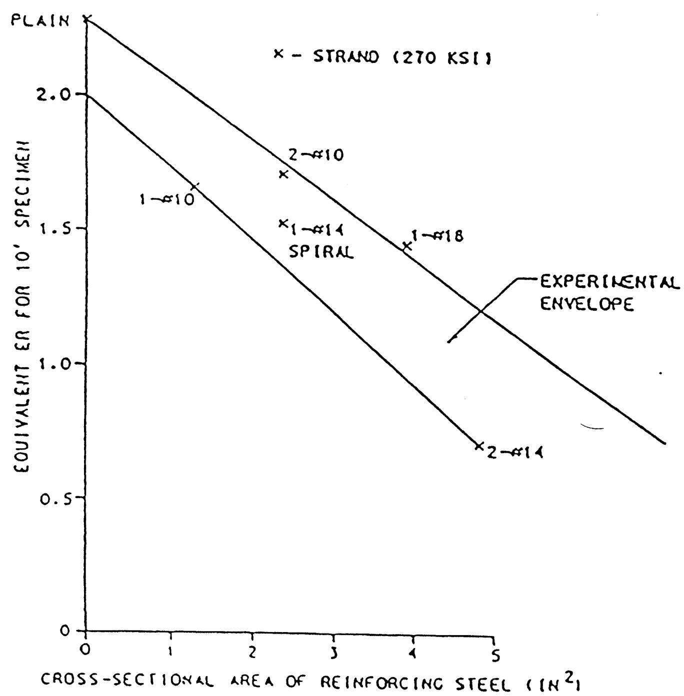


Figure 3 Equivalent ER vs. Reinforcing Steel Area, Phase 3 Tests.

replacement Pin Piles of 300 kip (1340 kN) 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 7 inch (178 mm) casing to a depth of approximately 70 feet (21 m) from grade and so a minimum of 30 feet (9 m) 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 1.5 inches (38 mm) in 100 feet (30 m) and additional uniform settlements of less than 6 inches (152 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 300 kips(1340 kN) 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 600 kips (2670 kN) maximum but all exhibited what appeared to be explosive 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 (3340 kN). 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.

Table 7 shows the progressive increases in ER with increasing load for each pile. It was always recognized that this progressive increase was indicative of progressive debonding: 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

| Test Pile # | Upper Length Soil Strata | | | Bond Length Soil Strata | | |
|---|---|---|---|---|---|---|
| | Sand Fill (ft) | Silt (ft) | Medium Dense Sand (ft) | 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 Project, WA
1 ft = 0.305 m

| 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 Project, WA
1 ft = 0.305 m

| 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 | .020 | .012 | .020 | .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 Project, WA
1 kip = 4.45 kN; 1 in. = 25.4 mm

| 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 Project, WA
1 kip = 4.45 kN; 1 ft = 0.305 m

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 foot (3 m) length of grout filled 7 inch (178 mm) 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 feet = 63 feet (19m).

The calculated free lengths in Table 7 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 (1340 kN), the casing had debonded only about 30 feet (9m) below the ground surface, highlighting the surprisingly high 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 70 - 150 kips (310-670 kN) capacity to resist explosive bursting failure. This compares with the same range identified in the Phase 3 Laboratory Tests. (Table 3).

## 5. OVERVIEW OF LABORATORY AND UNITED GRAIN SITE TESTS

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 proportions 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 neatly 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 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 conditions 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 safely reached in a previous cycle, 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 capable of resisting load, and so progressively higher proportions of the applied 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 surface area becomes capable of resisting load as a result of progressive debonding, the average bond stresses increase on the surface area remaining in virgin conditions. 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. So, when the critical amount of load is transferred to the bond zone, a sudden explosive failure will occur. This time of transfer may vary from almost instantaneous to many minutes.

## 6. SEISMIC RETROFIT APPLICATIONS - GENERAL INTRODUCTION

The California Department of Transportation (CALTRANS) recorded numerous bridge failures in the 1971 San Fernando Earthquake. The failures were linked to separations at bridge deck expansion joints and a lack of ductility in the supporting columns. As a consequence, CALTRANS retrofitted 1250 state bridges to provide deck continuity, from 1971 to 1989, although column ductility retrofitting was delayed until 1986 due to budget constraints. Column ductility improvements of course result in an increase of load demand on both superstructures and foundations.

Investigations into various bridge foundation repairs have been intensified in recent years as a result of the increased availability of funds following the 1989 Loma Prieta Earthquake. (15). One of the most common measures is to add tension/compression piles around the perimeter of an existing footing. Driven precast concrete and steel piles are typically used for foundation support of bridges. However, due to constraints including noise and vibration level limitations, installation difficulties presented by low overhead conditions, difficult drilling and driving conditions due to ground obstructions or high water tables, limited right-of-way access, the inability to extend the footings and higher tension capacity requirements, alternates to standard driven piles are becoming more desirable. Since varying project conditions may be more practically and economically favorable to certain construction techniques, there is no one single "best" solution.

## 7. CALTRANS TENSION PILE TEST PROGRAM SAN FRANCISCO, CALIFORNIA

In late 1991, CALTRANS initiated a full scale pile testing program as part of its seismic structural retrofit program. Foundation retrofits are the most costly element in the seismic retrofit program, fully justifying research into alternate construction techniques. This testing program was proposed as a joint effort between CALTRANS, the Federal Highway Administration, and contractors who could offer proprietary piling systems. CALTRANS tested traditional piling systems such as drilled shafts and driven steel H piles or pipe piles: proprietary systems offered alternatives. As part of this program, Nicholson

Construction installed six Pin Piles of three different types at the test site in San Francisco. (16).

A simplified stratigraphy of the test site was:

0 to 20 feet (0 to 6 m) Fill
20 to 110 feet (6 to 34 m) Clay (Soft Bay Mud deposits)
110 to 160 feet (34 to 49 m) Sand

During installation, actual pile lengths were varied in response to the actual conditions encountered. However, to limit test variables, many of the pile components and dimensions were held constant. The three types of piles installed were:

NCA-Pile. A high capacity multistrand tendon was installed within and below the 7 inch (178 mm) dia. steel casing. This tendon was stressed and locked off against the top of the casing prior to the pile test. A 35 foot (11 m) long pressure grouted bulb was formed in the sand, which included a 10 foot (3 m) embedment of the casing, a 5 foot (1.5 m) buffer zone, and a 20 foot (6 m) bond length for the tendon. This pile was similar to a type S-1 except that the prestressing tendon replaced the reinforcing bar.

Pin Pile. A 60 ksi (410 MPa) yield strength steel reinforcing bar was installed within and below the 7 inch dia. casing (Type S-1). No prestress loading was applied to this pile. The pile had a pressure grouted bulb 30 feet (9 m) long in the sand which included a 10 foot (3 m) embedment of the pipe, and 20 feet (6 m) of reinforcement extending below.

NFC-Pile. The 7 inch (178 mm) diameter steel casing was drilled full length into the lower sands and a 60 ksi (410 MPa) steel bar placed full length. (Type S-2). For these piles, the grout was introduced as drilling in the sand progressed.

Two examples of each pile type were installed at the site: one deep pile founded in the sand and one shallow pile founded in the Bay Mud. Deep pile lengths varied from about 140 to 155 feet (43 to 47 m). Shallow piles were installed to about 105 feet (32 m). The piles were tested to a 200 kip (890 kN) design load and then loaded to failure. The Nicholson pile load test results are shown in Table 8.

| PILE NO. | DESCR. | PILE CAPACITY (KIPS) Tension | Compression | ACTUAL ELASTIC DEFL. @ 200 KIPS Tension | Compression | ACTUAL TOTAL DEFL. @ 200 KIPS Tension | Compression |
|---|---|---|---|---|---|---|---|
| 10,A | NCA-Deep | 407 | * | .461" | - - - | .503" | - - - |
| 11,B | NCA-Shallow | 243 | 160 | .329" | N/A | .370" | N/A |
| 12,E | NFC-Deep | 500 | >400 | .302" | -.289" | .310" | -.348" |
| 13,F | NFC-Shallow | 195 | 220 | .302" | -.270" | .414" | -.420" |
| 73,A | Pin-Deep | 455 | >385 | .530" | -.516" | .631" | -.581" |
| 74,B | Pin-Shallow | 189 | 373 | N/A | .351" | N/A | .378" |

Table 8 Summary of Test Results for Nicholson Piles in CALTRANS Test Program

* Pile damaged during tension test loading. No Compression Test Results

N/A Not Applicable

1 kip = 4.45 kN; 1 in. = 25.4 mm

Piles in lieu of the 64 specified CIDH concrete piles. Detailed plans and calculations were prepared, submitted and approved by the CALTRANS Office of Structures.

## 8. FIELD RESEARCH - CALTRANS NORTH CONNECTOR OVERCROSSING - I-110 SITE, LOS ANGELES, CALIFORNIA

CALTRANS awarded a construction contract for the North Connector Overcrossing in Los Angeles in 1991. The original design involved retrofitting bents 2, 3, 5 and 6 by strengthening the existing footings. The design used sixteen 24-inch (61 mm) diameter cast-in-drilled-hole (CIDH) concrete piles placed around the existing footing at each single column bent.

An experienced and qualified drilled-shaft subcontractor attempted to install the specified piles. However, due to difficult drilling conditions, including concrete obstructions and water bearing (flowing) sand, and the installation difficulties caused by low overhead conditions, they were unable to complete the installation of any CIDH piles. CALTRANS was aware of the Nicholson Pin Pile through the San Francisco Test Program and subsequently, the general contractor engaged Nicholson to install 64 Pin Piles in lieu of the 64 specified CIDH concrete piles. Detailed plans and calculations were prepared, submitted and approved by the CALTRANS Office of Structures.

The project site was located in Los Angeles near Figueroa Street and the southbound on-ramp to I-5. The soils underlying the site consisted of loose to slightly compact fill in the upper 25 feet (8.0 m) and dense to very dense sands and gravels below. The ground-water table was approximately 25 feet (8 m) below grade.

The project site had been a dump location for a ready-mix concrete plant, and the upper fill zone contained large chunks of concrete and rubble. Three of the retrofitted footings were located adjacent to the Aroyo Seco drainage channel, and were accessible only by a graded road or from the edge of the Pasadena Freeway. The fourth footing was located in the middle of the Pasadena Freeway, creating very difficult access conditions. Overhead clearance under the freeway superstructure was approximately 20 feet (6 m).

The Type S-1 Piles for this project were required to support an ultimate compressive load of 500 kips (2225 kN) with a maximum pile head total deflection of less than 0.60 inches (15 mm). Each pile comprised:

- An upper pile length extending to 30 feet (9 m) below the bottom of the existing footing, consisting of a 7 inch (178 mm) o.d. 1/2" (12.7 mm) wall thickness steel casing, reinforced full length with two 1-3/8 inch (35 mm) diameter grade 150 threadbars and filled with neat cement grout.

- A pile bond length extending from 30 feet (9 m) to 60 feet (18 m) below the bottom of the existing footing, consisting of a pressure grouted bond zone, reinforced with the two 1 3/8 inch (35 mm) diameter threadbars, extending to the pile tip, and the 7 inch (178 mm) diameter steel pipe, extending 5 feet (1.5 m) into the top of the bond length.

- A specially designed connection between the pile and the cast-in-place extension to the structure footing.

The production test pile (Bent No. 3, Pile No. 3, selected by CALTRANS) was drilled with a high-torque, low-headroom drill rig. It was installed from existing grade to a depth of approximately 66.5' feet, (20 m) allowing testing to be performed before footing excavation. The casing was placed in 10 foot (3 m) lengths, and the threadbars were placed in 10 foot (3 m) and 20 foot (6 m) coupled lengths, centralized in the pile with plastic spacers. Maximum grout pressure attained during grouting of the pile bond length ranged from 100 to 140 psi (0.7 to 1.0 MPa) measured at the drill rig.

The pile test was conducted by representatives from the CALTRANS Office of Structures. The tension test was completed to the required 300 kip (1340 kN) load and the compression test to the required 500 kips (2225 kN). The pile was loaded in 100 kip (445 kN) cycles, with the load applied in 20 kip (89 kN) increments, and reduced in 20 to 100 kip increments. Each increasing load increment was held for 5 minutes the first time at that load, and for 2 minutes thereafter. Each decreasing load was held for one minute.

Figure 4 summarizes the load test data. The pile successfully resisted the required maximum tensile load of 300 kips (1340 kN) with a total displacement at maximum load of 0.304 inches (7.72 mm) and a permanent displacement of 0.050 inches (1.27 mm) at zero load after loading. Creep movement during the 5 minute hold at 300 kips was 0.006 inches (0.15 mm). The pin pile then successfully supported the required maximum compressive load of 500 kips (2225 kN) with a total displacement at maximum load of 0.392 inches (9.96 mm) and a permanent displacement of 0.068 inches (1.73 mm) at zero load after loading. Creep during the 5 minute hold at 500 kips was 0.007 inches (0.18 mm).

## 9. OVERVIEW OF CALTRANS TESTS

The Nicholson Pin Pile proved to be an excellent system for meeting the design load capacity and displacement requirements. The Pin Piles were also installed with relative ease at difficult access sites and ground conditions which prohibited the installation of conventional pile types. Pressure grouting techniques in the dense sands and gravels resulted in very high grout/soil bonds and small displacements. Even in the soft Bay Muds, surprisingly high skin friction values were mobilized. The response of the Pin Pile to test loads was essentially elastic, with very small permanent displacements. These observations offer real hope that the special demands imposed on pile performance by the particular demands of California can be adequately met by the appropriate proprietary option.

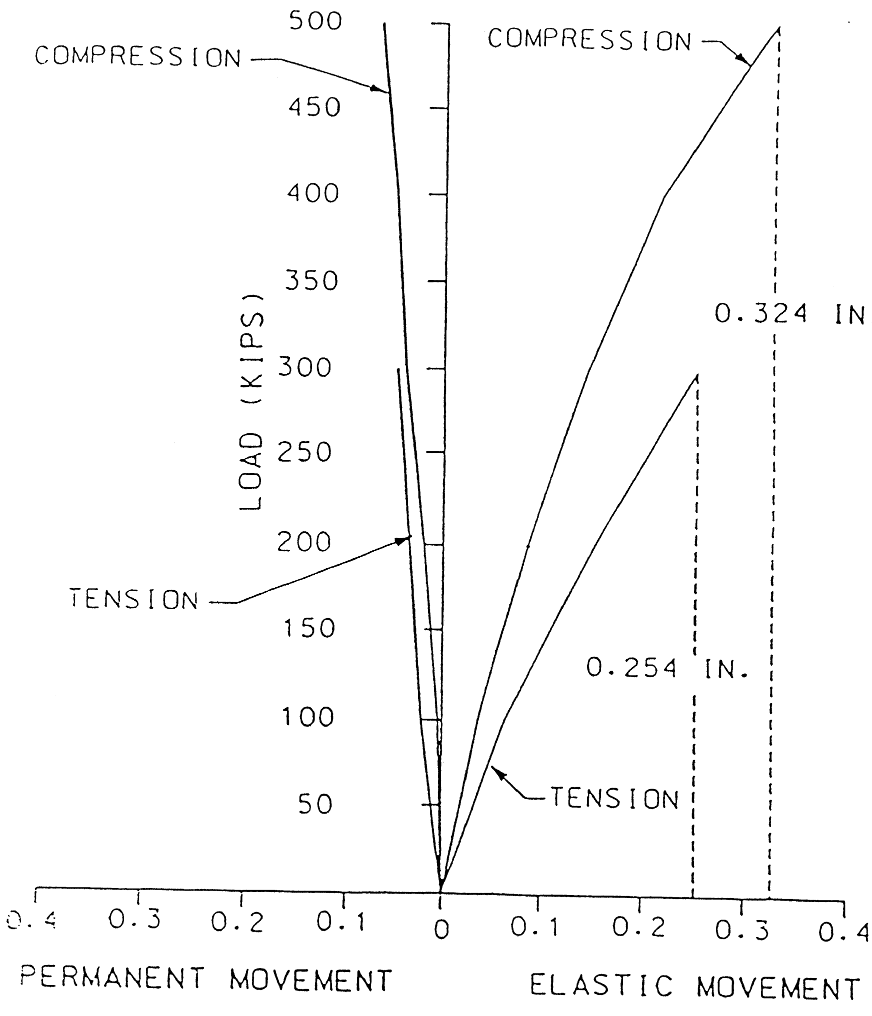


Figure 4 Permanent and Elastic Displacement Analysis
North Connector Overcrossing I-110, Los Angeles, CA

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