STRUCTURAL UNDERPINNING BY PINPILES

by

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ABSTRACT

Small diameter cast-in-place bored piles (pinpiles) have been installed throughout the world since 1952, but only more recently in the United States. The special features of their construction and performance makes them ideally suited for structural underpinning applications in difficult ambient and geological conditions. This paper describes the underpinning of part of a major industrial facility near Mobile, Alabama. In addition to being a most interesting and successful case history, per se, the paper also describes in detail a fundamental test pile program which has afforded a unique insight into the basics of load transfer mechanisms.

1. INTRODUCTION

Pinpiles have been used in Europe since 1952, and in the United States through two decades, although it is only in the last five years or so that they have become really popular (Bruce, 1988, 1989, 1992). Generically, a pinpile is a small diameter (4-10"), drilled, cast-in-place element. It can be as long as 200 feet although typically the range is from 50 to 100 feet. Given their geometry and their mode of construction, pinpiles transfer imposed loads to the surrounding ground by shaft resistance. When appropriately reinforced, and installed in material of suitable strength, pinpiles can be made to reach loads of over 600 kips - far in excess of the 100-200 kip capacity traditionally observed in other countries. These loads are resisted with extremely small head deflections: depending on pile length, elastic deformations are typically in the range of 1 inch, while permanent sets are recorded in tenths of an inch.

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Pinpiles are proving to be an excellent choice for structural underpinning where there are challenges posed by (i) limited access or headroom, (ii) "difficult" ground conditions, (iii) environmental impact restraints (especially the avoidance of piling induced vibrations) and/or (iv) the need to provide high load bearing capacity with minimal deflection. These factors typically conspire when considering the underpinning of old, delicate structures built on inadequate or deteriorating foundations, including timber piling, or when designing foundations for new structures within existing urban, industrial or transportation schemes (e.g., Bruce and Gemme, 1992; Pearlman and Wolosick, 1992).

This paper describes the underpinning work recently completed by the authors' company, at a chemical plant near Mobile, Alabama. In itself, it is a clear illustration of the major elements of applicability, design and construction. However, in addition, the paper describes the execution and analysis of a truly fundamental test pile program which has provided significant insight into load transfer mechanisms. This test program also clearly demonstrates the analytical tools which can be applied when attempting to understand the intricacies of pinpile performance.

2. PROJECT BACKGROUND: CHOICE OF UNDERPINNING PRINCIPLE

In order to prevent continuing settlement of a Caustic Evaporator structure within their major active chemical plant, the Owners called for a contractor designed underpinning system. The alternatives considered by Nicholson were:

1. Jet grouted columns (ASCE, 1987) extending from the underside of the existing footings into the dense sands and gravels of an underlying bearing stratum;

2. Pinpile support between the same limits; and

3. Pinpile support from the bearing stratum, but passing through the existing footings (approximately 10' below the surface) with at grade pile caps and welded steel connections. The existing footings would then be detached and isolated from the new pile cap, and its supporting pinpiles.

Careful analysis of the structural settlement data was a major determinant of the method. Average settlement rates of 0.038 to 0.178 inches per month per column had been recorded over the immediate 44-month period (Figure 1), and this demanded that special attention be paid to the connections between the piles and the structure. In effect, damage to the new underpinning system could potentially have occurred if their attachment to the structure was made before they achieved full design strength. This concern ruled out Option 1, jet grouting, despite its probable economic attractions. It was felt that the rate and total magnitude of the ongoing settlements would not have permitted proper undisturbed setting of the "soilcrete", especially in its upper part, immediately under the footing. In addition, Nicholson also believed that the disturbance to the soil, and its short term removal, inherent in the jet grout method would likely increase the local settlement rates.

These fears were supported by field observations on remedial concrete works recently completed: a concrete dampener installed at one group of columns, although well reinforced and constructed, had experienced significant distress due to settlement before its concrete had reached full strength. In another case, new concrete placed at another column had had to be repaired by epoxy resin injection for similar reasons.

The second option was also discounted. To achieve the foundation connection, difficult and costly excavation and shoring was anticipated to ensure safe access to the footings. In addition, ground or process water in-flows would have required continuous dewatering.

Option 3 - pinpiles with new pile caps and welded connections (pile cap to structure column) - was therefore chosen and developed.

3. SITE AND GROUND CONDITIONS

The working conditions were extremely awkward, having restricted headroom (6 feet typical minimum, occasionally as little as 54 inches), tight and difficult access, and very strict safety/procedural restrictions and regulations. The ambient weather conditions were typically very hot and humid.

A typical section through the site is shown in Figure 2. The principal soil groups were as follows:

- 0-8 feet Fill - fine to medium, tan-orange silty sand beneath shell cover material. From 1-26 blows per foot.

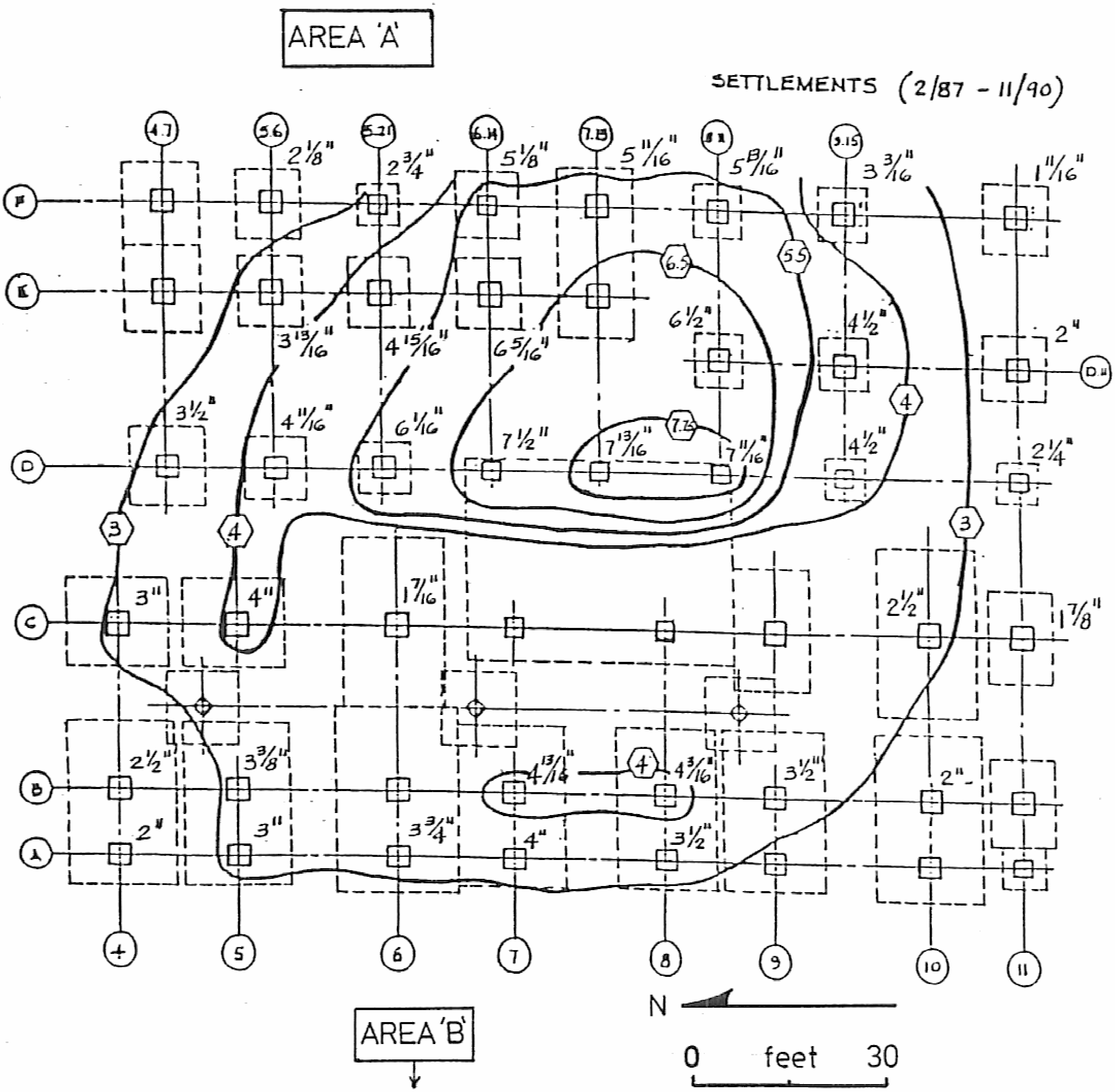


Figure 1. Cumulative settlements in period 2/1987 to 11/1990. Test Pile areas shown.

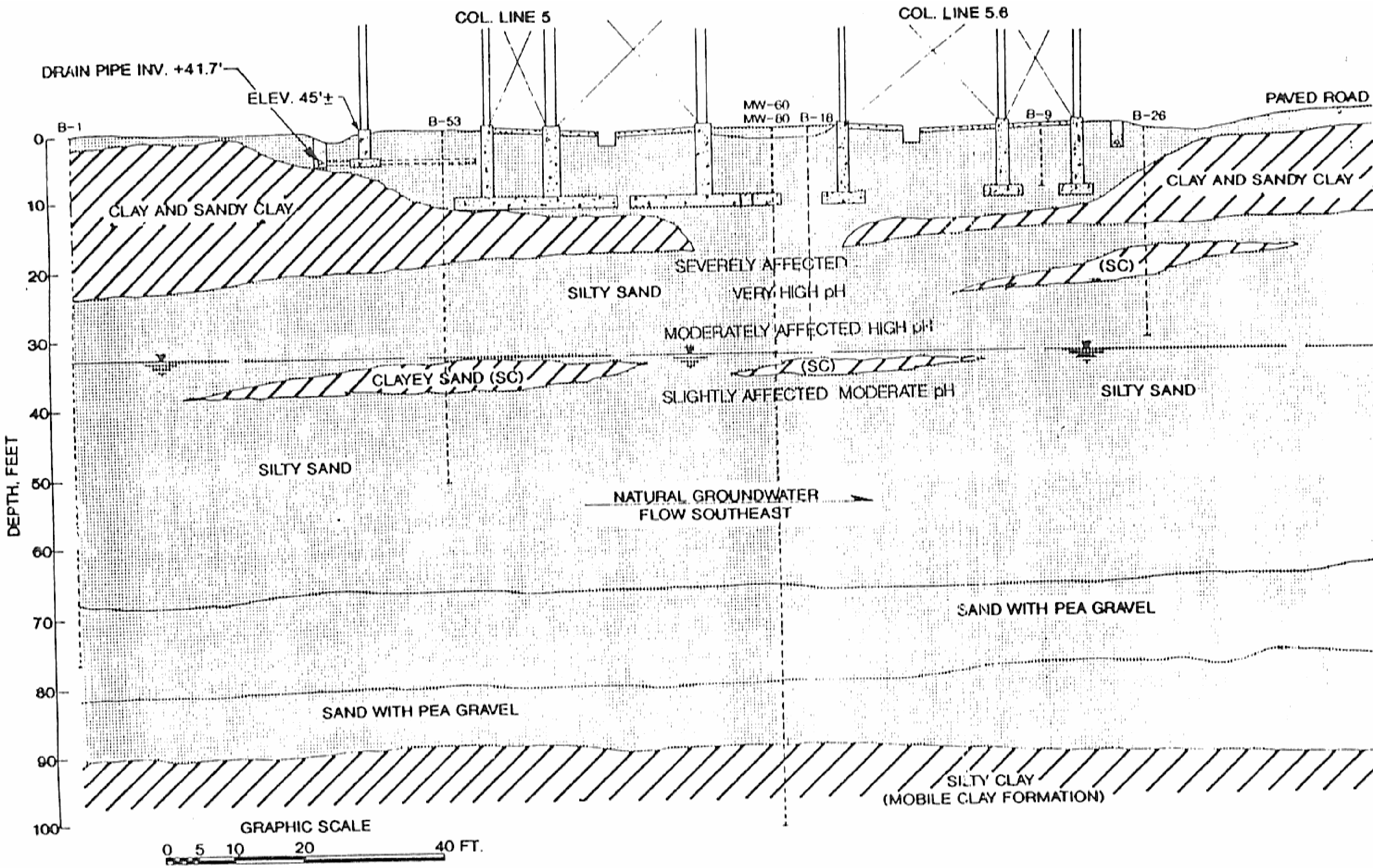


Figure 2. Typical geological section, looking north.

- 8-18 to 23 feet - Gray clay, gray and orange silty clays, light brown sandy clay over fine to medium tan-orange and gray silty sands, from 2-30 blows, increasing with depth.

- Thereafter to 90 feet - Fine to medium to coarse tan, brown sands, occasional clay lenses, and rounded quartz gravel beds. Typically single sized in any given bed. Dense to very dense (12 to 100+ blows, typically over 40 below 60'). Indication of more angular gravels below 65'.

The sands and gravels of this founding horizon were extremely permeable and the groundwater was within 5 feet of the surface, although this may have been perched, with the main table 25-30 feet down. The water was chemically contaminated with pH values being very high in the upper 20 feet or so.

4. DESIGN

The design loads exerted by the 60 existing columns varied from 24 to 810 kips. As described below, a standard pinpile of 200 kips design working load was selected throughout. So, when allowing for the geometries and structural symetries of the new footings, each new footing was supported on between 1 and 6 piles, with all but 16 of the footings having only 1 or 2 piles. Typical arrangements are shown in Figure 3, giving a total of 123 individual pinpiles.

The standard pinpile configuration is shown in Figure 4, and the more significant design details and assumptions were as follows:

- Casing: 0.50" wall, 80 ksi, with allowable stress of 0.4 Fyc.
- Rebar (bond zone only). Single #14 bar, 60 ksi, allowable stress of 0.5 Fy.
- Grout: 4 ksi UCS, allowable stress to 0.33 Fc.
- For the bond length calculation, the allowable bond stress at working load was (by convention) calculated as the average grouting pressure times tan ϕ (soil). Assuming an average grouting pressure of 75 psi, an effective bond zone diameter of 12", $\phi = 32^{o}$ for the sands and gravels and a factor of safety of 2, a bond zone length of 25 feet was chosen, conservatively.

This pile configuration was calculated to provide an elastic deformation at design load of 0.33 inches, with a total deflection of about 0.50 inches.

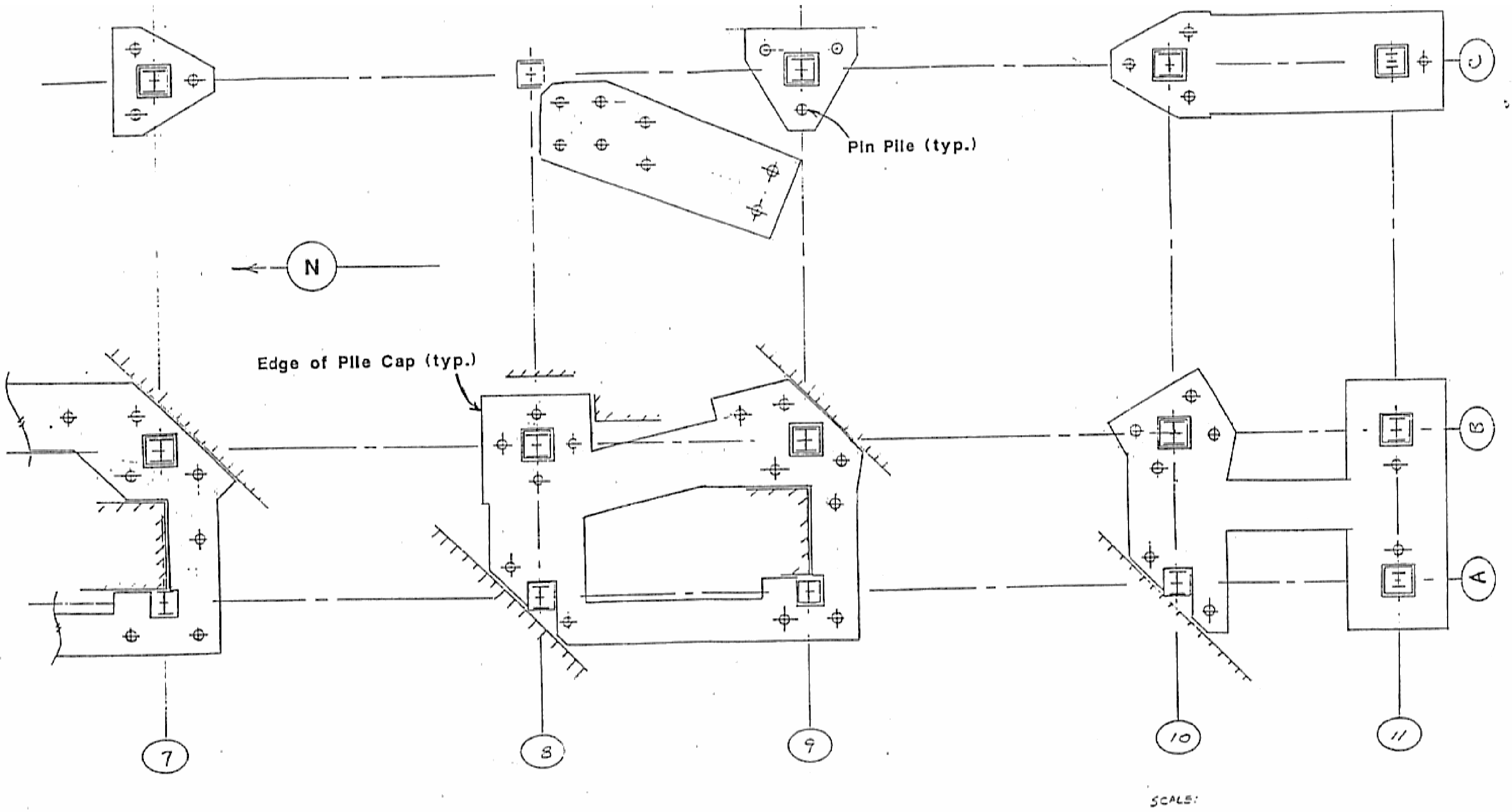


Figure 3. Example of typical pinpile and new cap arrangement.

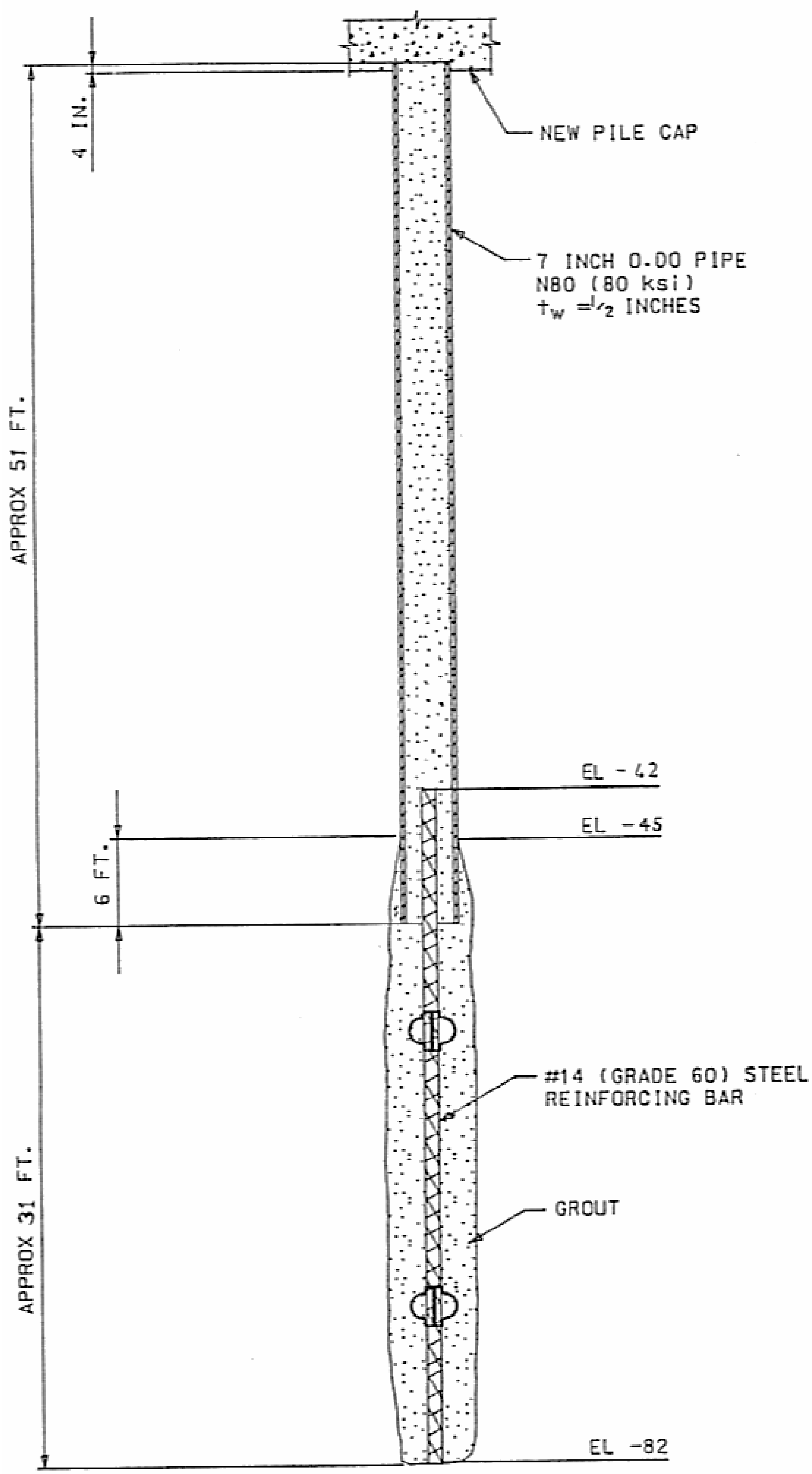


Figure 4. Standard pinpile configuration.

From the historical rate of settlement data, it was estimated that these pinpiles would be fully loaded to design load within one month of being connected to the structure.

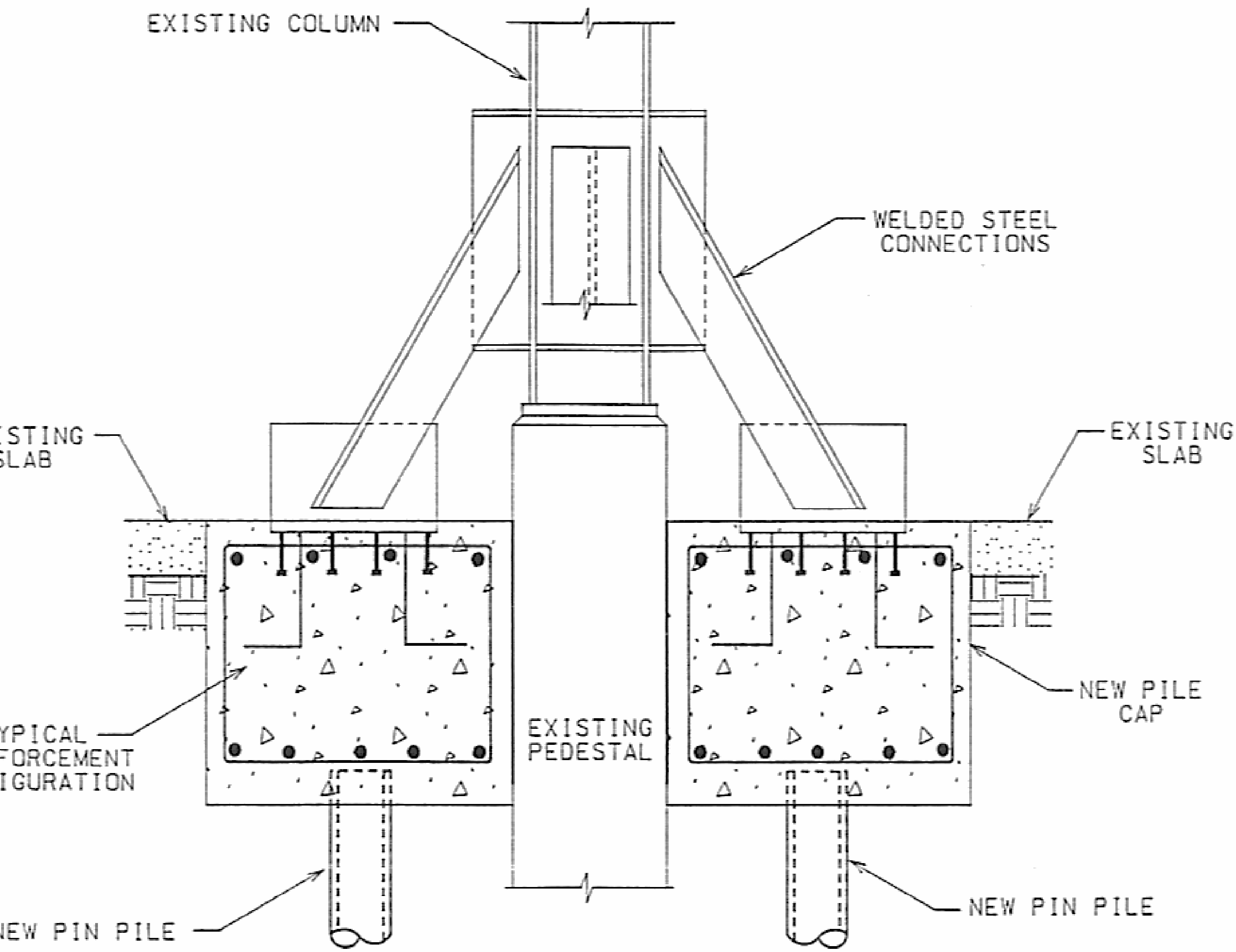
It was also necessary to accommodate the possible effects of negative skin friction (or downdrag). It was therefore decided to coat the upper 47 feet of the casing with a low friction, polyurethane resin. The lower 10 feet was not coated to ensure continuity of the load carrying system in the bond zone. This coating also provided corrosion protection to the casing against the caustic environment in the upper strata. Calculations indicated that the maximum possible downdrag would be 30 kips per pile. Consideration of the redundancy inherent at each pile cap as a result of "rounding up" the numbers of piles provided, confirmed that this extra load could be safely accommodated without danger of overstress.

Regarding other design implications, it was decided to install a PVC pipe into the existing footing after penetration, to act as a bond breaker between it and the pinpile.

As shown in Figure 4, the pile head was to be encased in the new reinforced concrete pile caps (46" thick, and poured around the existing pedestals to the same elevation as the existing slabs). The pile caps were then to be attached to the existing columns with welded steel connections (Figure 5). Due to the high structural settlement rates, this connection was to be made after the concrete reached its design strength (4000 psi). The caps and welded steel connections are not further detailed in this paper.

5. CONSTRUCTION

The existing concrete slabs adjacent to each column were sawn and removed, and in areas of minimum headroom, shallow excavations were prepared and shored to allow the drill rig mast to be placed. Where the pile had to be drilled through an existing footing, an oversized hole was first cored or hammered through it to accommodate the PVC bond breaker. Special electro-hydraulic track rigs with "mast-off" capability were then used to rotate the 7" casing to the target depth of 82 feet. The drilling of the very tough and permeable sands with such relatively small equipment was greatly facilitated by using biodegradable polymer drilling slurry. This material was completely displaced from the hole with water flush before tremie placement of the grout into the casing. The grout was prepared from Type I/II cement in a colloidal mixer, using a w/c ratio of about 0.45.



<u>Figure 5.</u> Typical connection between new cap and existing column.

Following placement of the 40 feet long rebar in coupled lengths, the casing was then rotated and withdrawn while grout was continually pumped through it at a pressure of about 50 psi. This phase of pressure grouting was conducted for 37 feet of withdrawal. Thereafter, the casing was plunged 6 feet down into the pressure grouted zone (for a total of 51 feet in the pile), and further pressure grouting commenced until grout leakage occurred up the outside of the casing, a pressure of 250 psi had been advanced, or nine bags had been injected. The head was then disconnected and the rig moved to the next location.

No pile installation was permitted within 30 feet or 1 day of a new adjacent pile to prevent possible disturbance of the setting grout.

6. TEST PILE PROGRAM

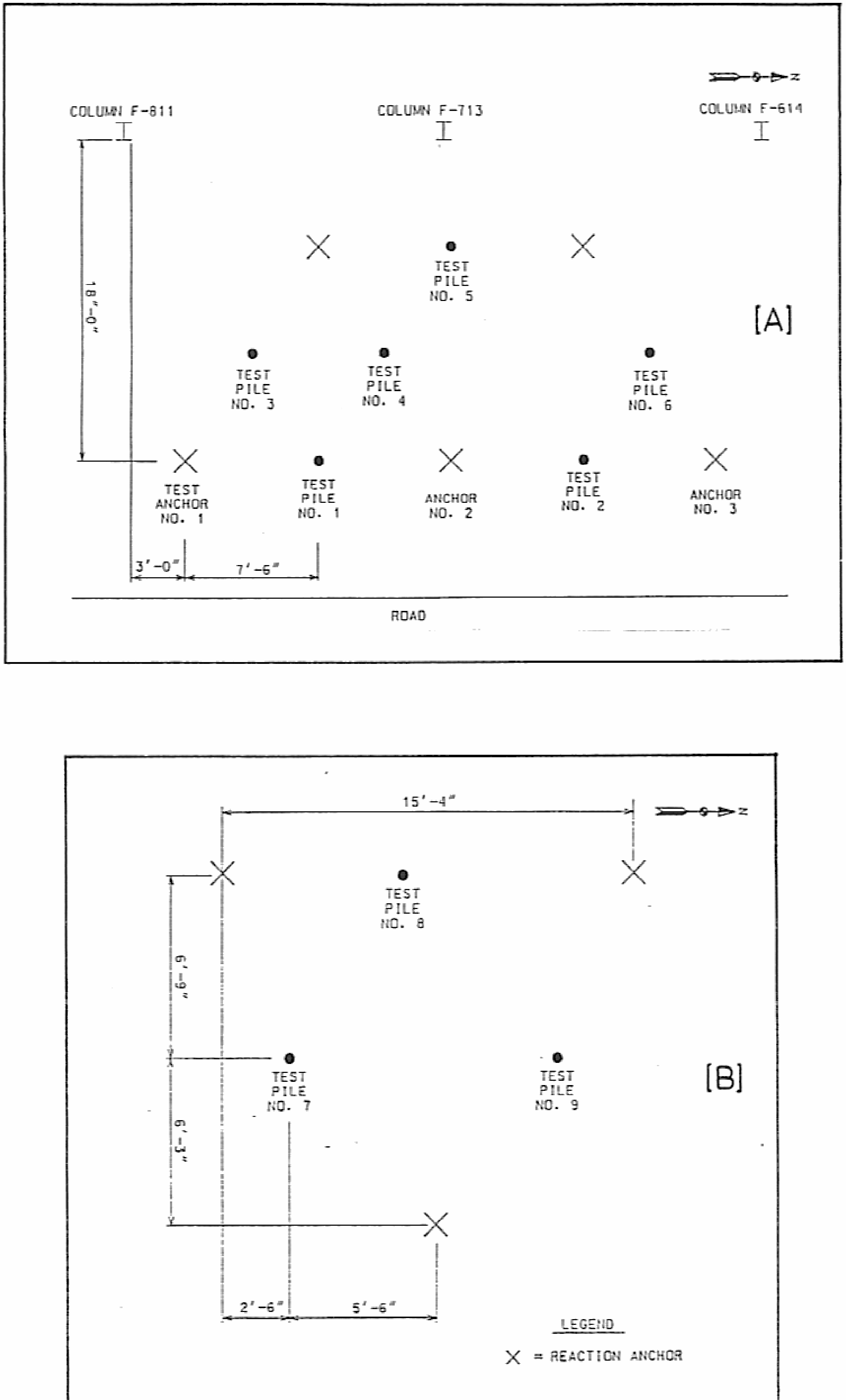
6.1 Background and Construction

The project documents called for two piles to be specially constructed and tested to twice the design working load (i.e., to 400 kips) to verify the suitability of the design assumptions and the construction methods. The chance was taken to enhance the value of this opportunity by installing six test piles in a preliminary phase, followed by a further three piles in a second area upon completion of the production works. Overall, therefore, this program provided both data of direct relevance to this project, but also fundamental information for Nicholson's ongoing research into the basics of pinpile load transfer mechanisms. The safe structural capacity of the test system was calculated as 630 kips, close to the elastic capacity (at 80% GUTS) of the strand anchors used for the reactions.

Test Site A (test piles 1 to 6: Figure 6A) was approximately 200 feet east of Test Site B (TP 7-9; Figure 6B). At the former site, the dense sands and gravels were encountered 12 to 18 feet below the surface, whereas at the latter they commenced about 30 feet down. Construction was generally as for the production piles, with specified details summarized in Table 1.

The change made from drill flush polymer 'A' to 'B' was to aid preparation and handling. The oversize casing used to isolate the pile casing in TP1 and 2 was not used elsewhere. The changes in the nature of the bond zone reinforcement in the latter group of piles was for experimental reasons.

Each pile was tested basically in accordance with ASTM - 1143D provisions except that intermediate cycles were inserted into the sequence. This



<u>Figure 6.</u> Plan of test piles; Area A (TP1-6) and Area B (TP7-9).

Test Pile #	Total Depth Drilled	Grouting	Total Depth Pressure Grouted	Total 7" 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	• 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; • 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)	• 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	• Slightly oversize bit (9"). • 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 1. Summary of Test Pile Construction Data

permitted the total head deflections to be partitioned into permanent and elastic components, for each load cycle maximum. Consideration of the latter permits conclusions to be drawn on load transfer depths within the pile.

6.2 Summary of Performance

In summary, 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.)

- TP3: loaded in 50 kip cycles to a creep/plunge failure at the 300 kips 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.

6.3 Principles of Data Analysis

Although the two groups of piles were installed in two phases in separate areas, there is more logic in assessing their performance on other grounds. Test Piles 1, 2, 3 and 4 were each installed using drill fluid 'A', had about 30 feet of pressure grouted length, 57-60 feet of casing, 30-35 feet of #14 rebar, and some form of debonding around the upper 40 feet of casing. Test Piles 5, 6, 7, 8 and 9, on the other hand, each were drilled with fluid 'B', had 37 feet of pressure grouting, 51 feet of casing (not debonded), a more aggressive pressure grouting program, and varied only in their bond zone reinforcement.

When illustrating pile load test data it is conventional to use a simple load/deflection graph as shown in Figure 7. However, when trying to analyze details of pile performance, it is much more useful to separate out the elastic, permanent and creep data into different formats.

Regarding the elastic data, the cyclic loading operaton permits, as noted above, separation of the elastic and permanent components, at each load cycle maximum (Figure 8). The magnitude of elastic deflection is obviously dictated by the applied load, and the length of pile over which this load acts. If the pile were acting purely in end bearing (i.e., as a strut), then the deflection would always be proportional to the load. However,

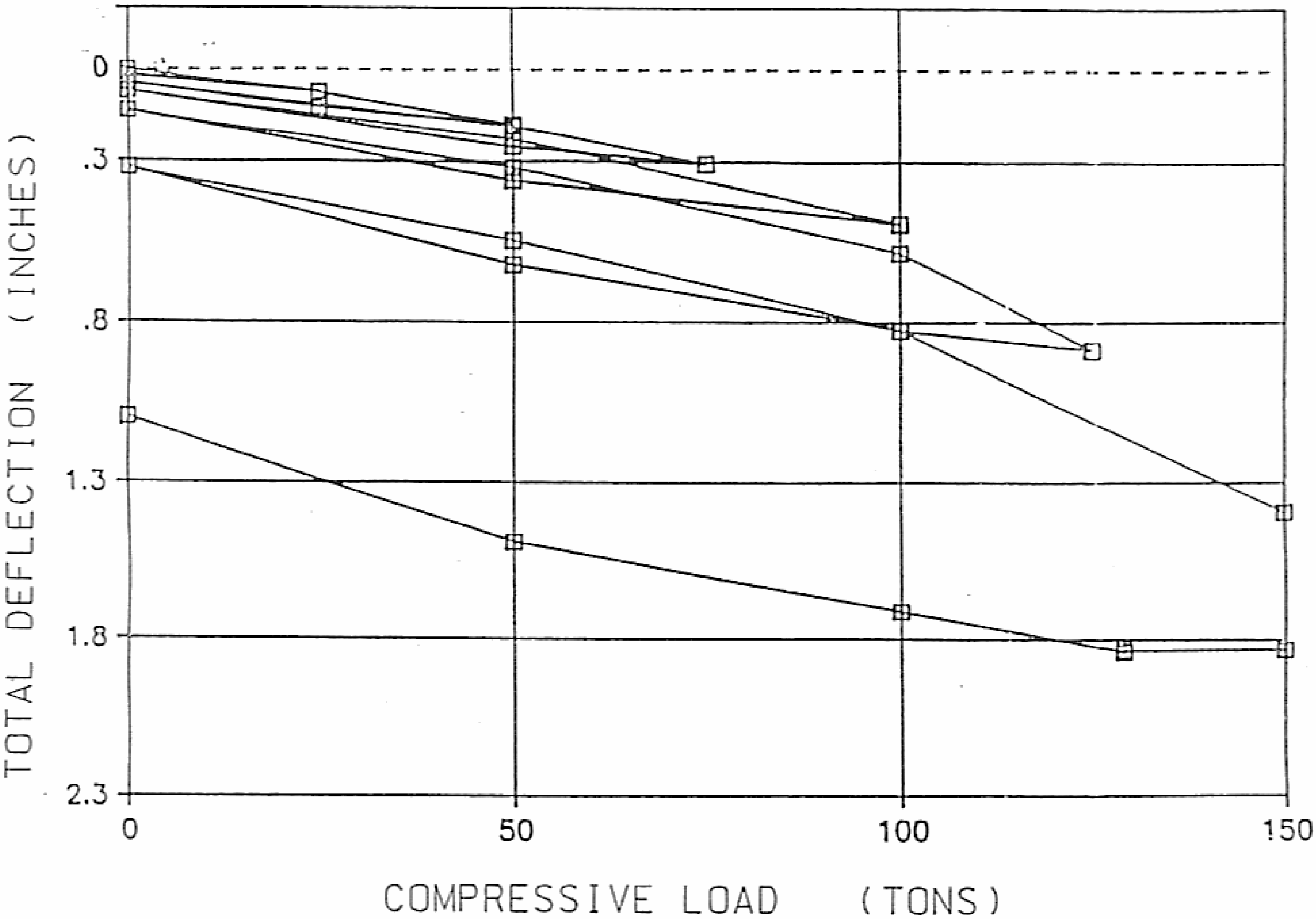


Figure 7. Typical load - total deflection graph (TP3).

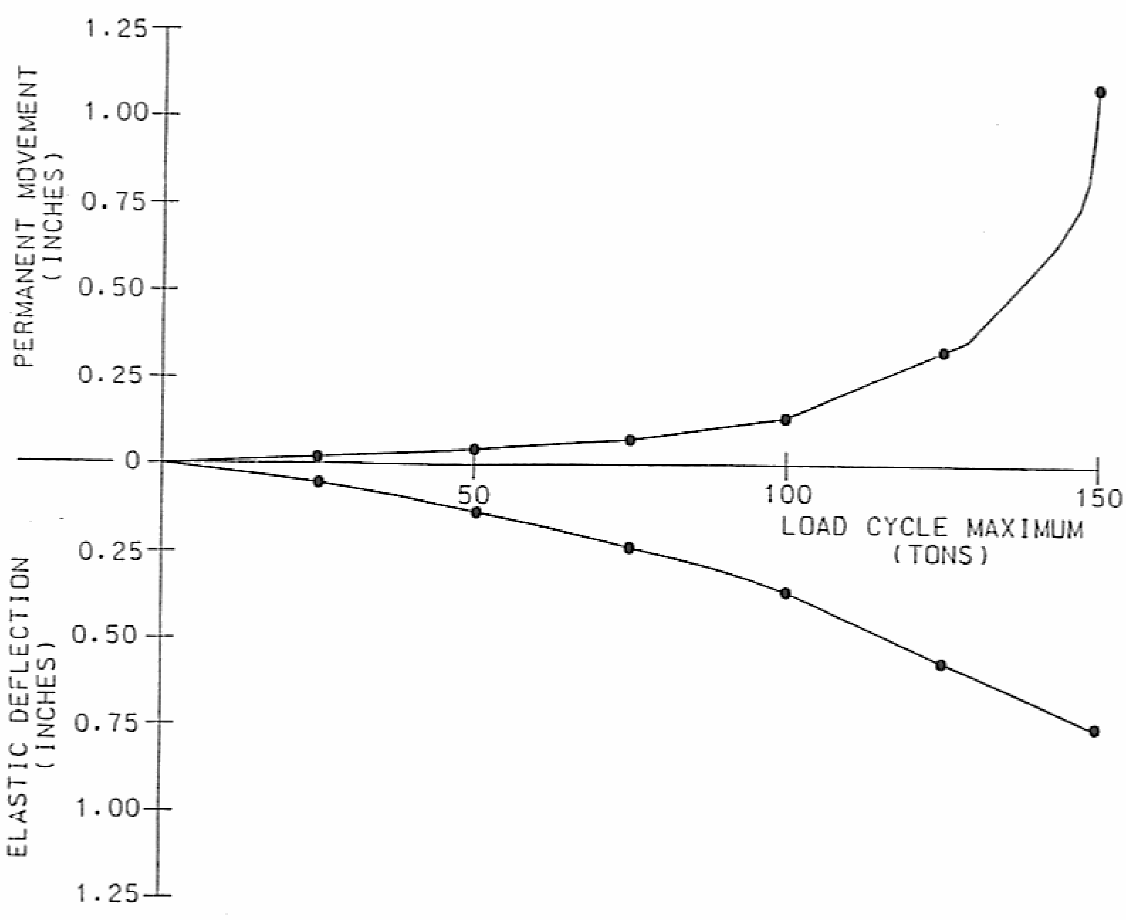


Figure 8. Typical permanent and elastic analysis (TP3).

pinpiles transfer load to the ground by skin friction, and this resistance is mobilized progressively downwards along the pile as the load is increased. Thus at load x, a certain length of the pile is energized, and acts elastically over that length. However, at higher load x + y, a greater length is energized, and so the resultant change in deformation is not simply in proportion to the change in load.

This fascinating phenomenon can be illustrated simply using the concept of the Elastic Ratio (ER). This is calculated by dividing the elastic deflection (in the thousandths of an inch), by the load (in kips). Thus, for an elastic extension of 0.500 inches at a load of 250 kips, the ER would be 500/250 = 2. Obviously the higher the ER, the further down the pile the load is being transferred.

When considering debonding characteristics in tensioned, prestressed rock anchors, one can easily translate elastic extensions into equivalent unbonded lengths through the PL/AE relationship. However, a pinpile has a complex, composite cross section, and is loaded in compression. It is therefore difficult to predict by calculation the elastic deformation characteristics, as various assumptions must be made about the confined performance of the grout, and so on. Thus it is equally difficult to relate an ER value to its equivalent effective load transfer length.

Recently, however, Nicholson has made a fundamental breakthrough in this regard. Kenny (1992) has conducted compressive load testing of grout filled steel drill casings which has shown that a 10 foot length of grout filled casing has an ER of approximately 0.32. Knowing this, piles of similar composition can therefore have their measured ER's converted directly to an effective load transfer length: if the ER is 1.8 at 400 kips, then we can conclude that the transfer length is 1.8/0.32 = 5.6 x 10 feet = 56 feet. This is the principle used in the following analyses.

6.4 Presentation of Data

Figures 9 and 10 show the ER calculations for the first group and second group of piles respectively and Figures 11 and 12 show the permanent movements similarly divided. Table 2 summarizes the creep data, while Table 3 summarizes the overall performance.

Figures 9 and 10 both show that the ER increases steadily with increasing load. Furthermore, the ER values recorded at the same load on successive

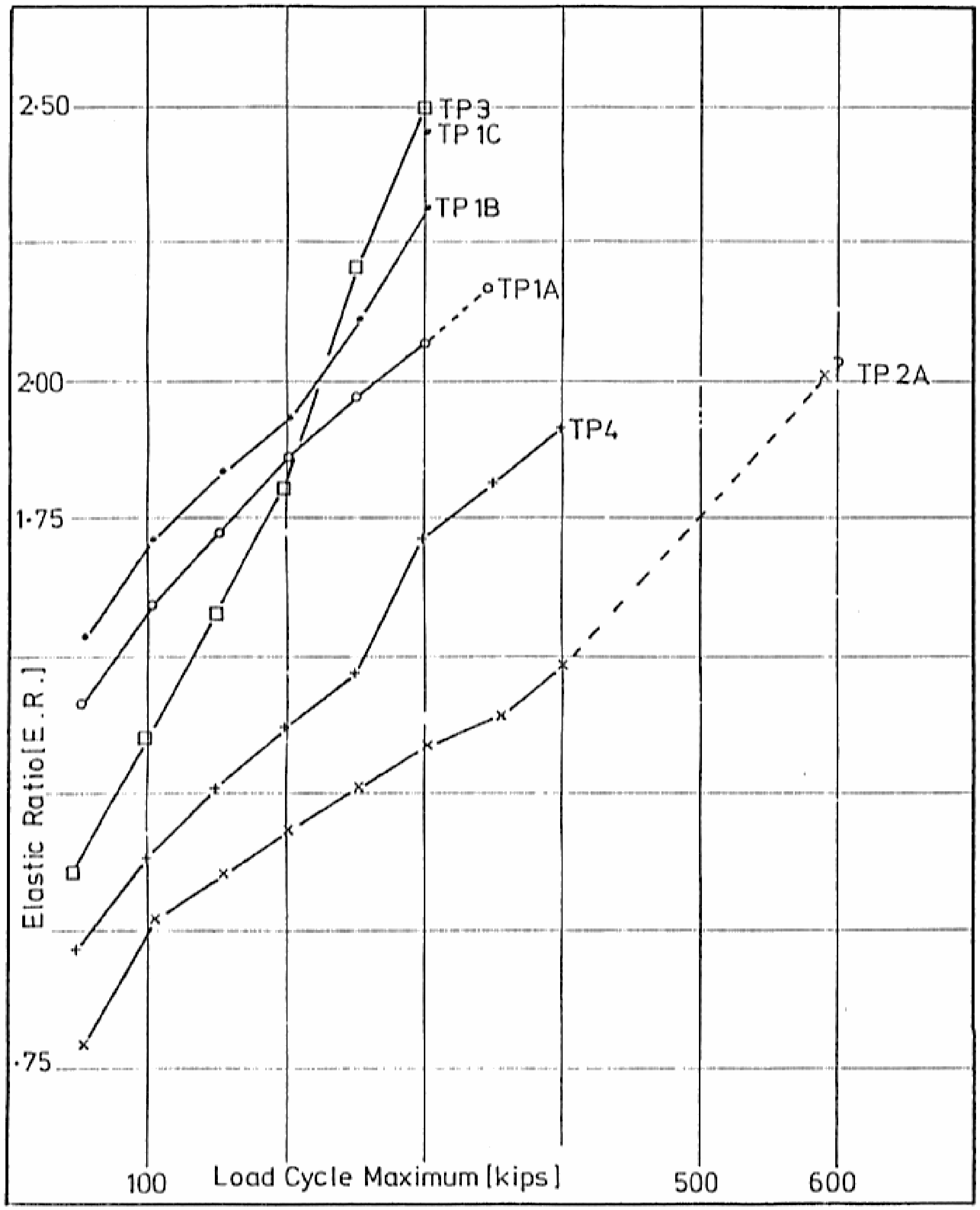


Figure 9. Elastic ratios for TP1-4.

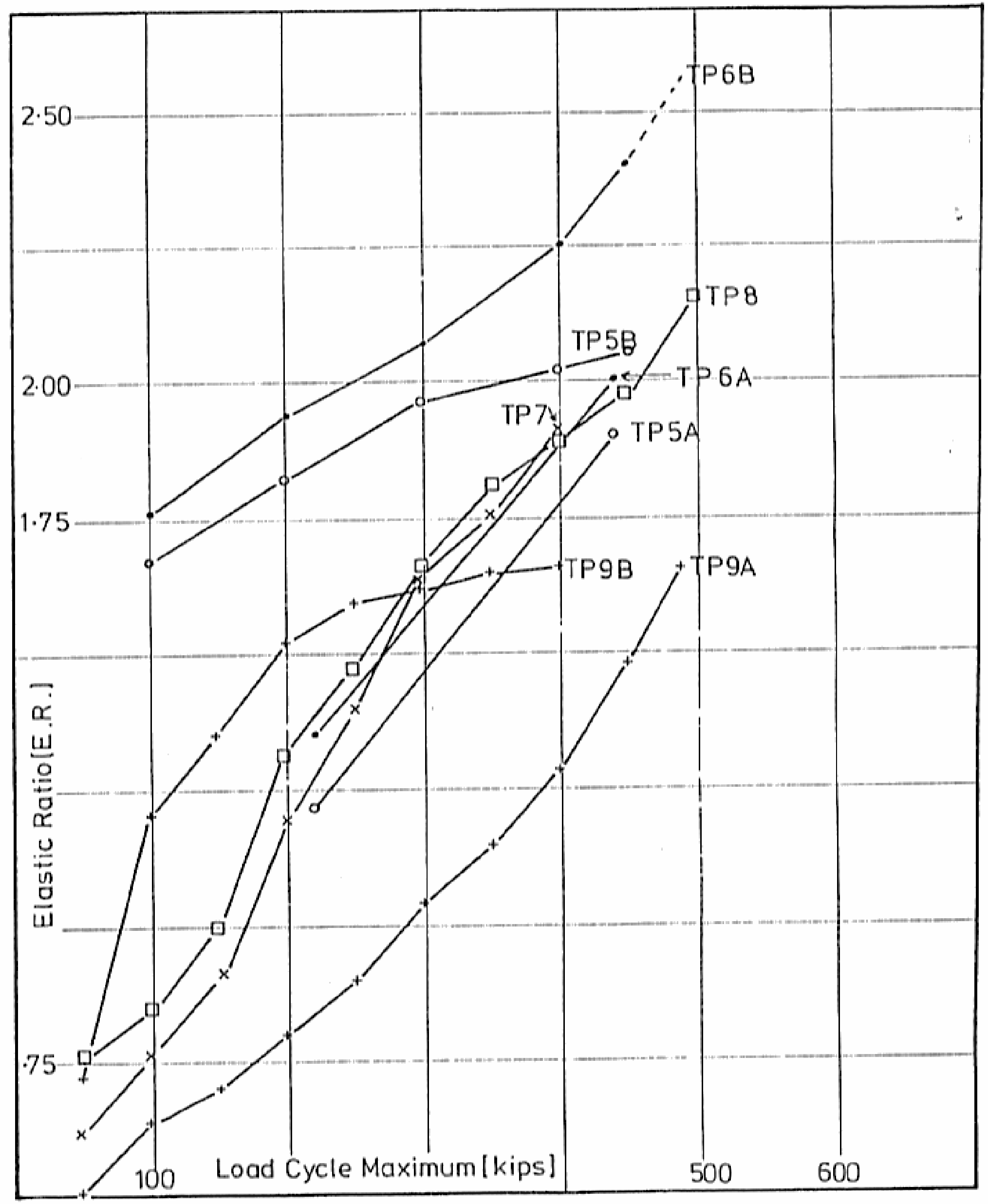


Figure 10. Elastic ratios for TP5-9.

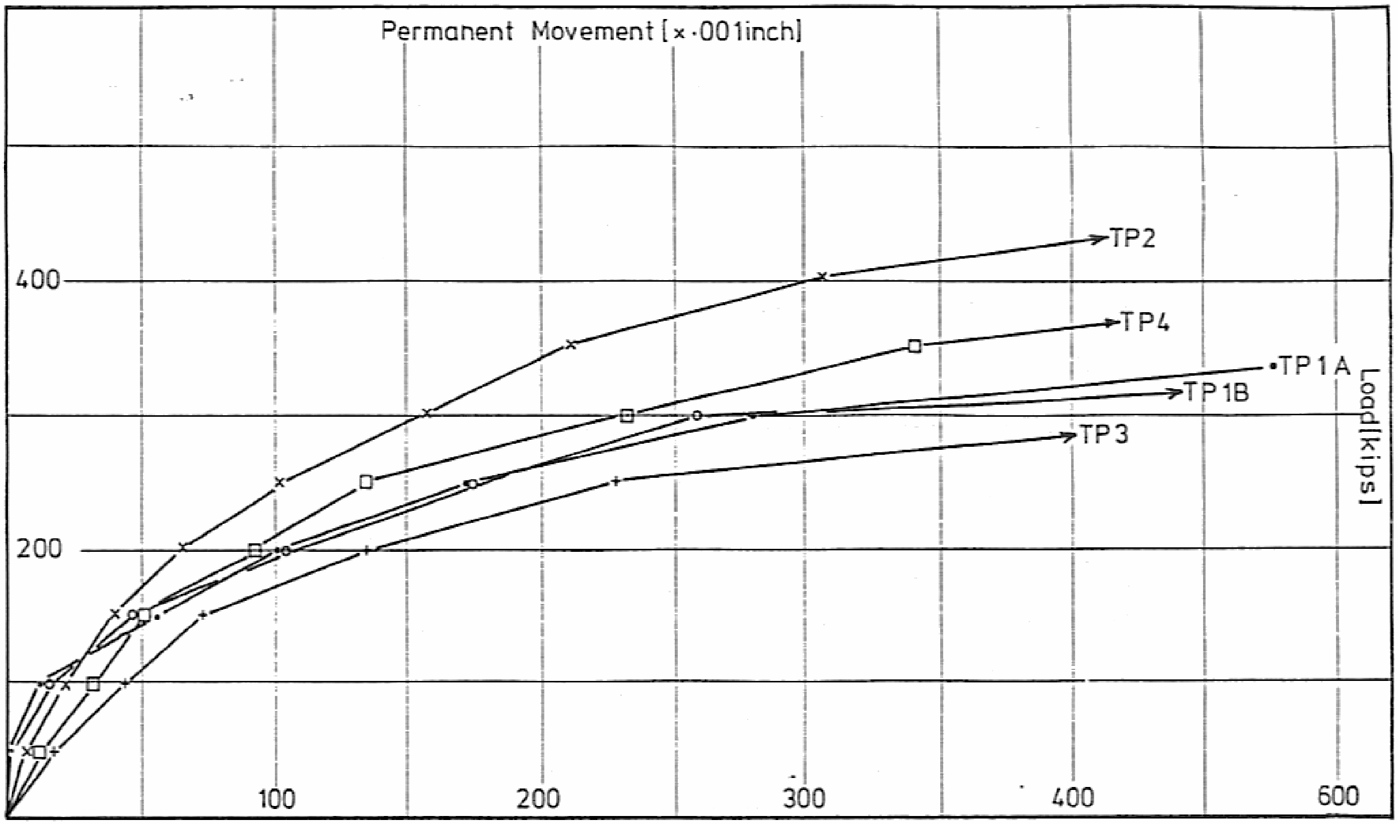


Figure 11. Permanent movements TP1-4.

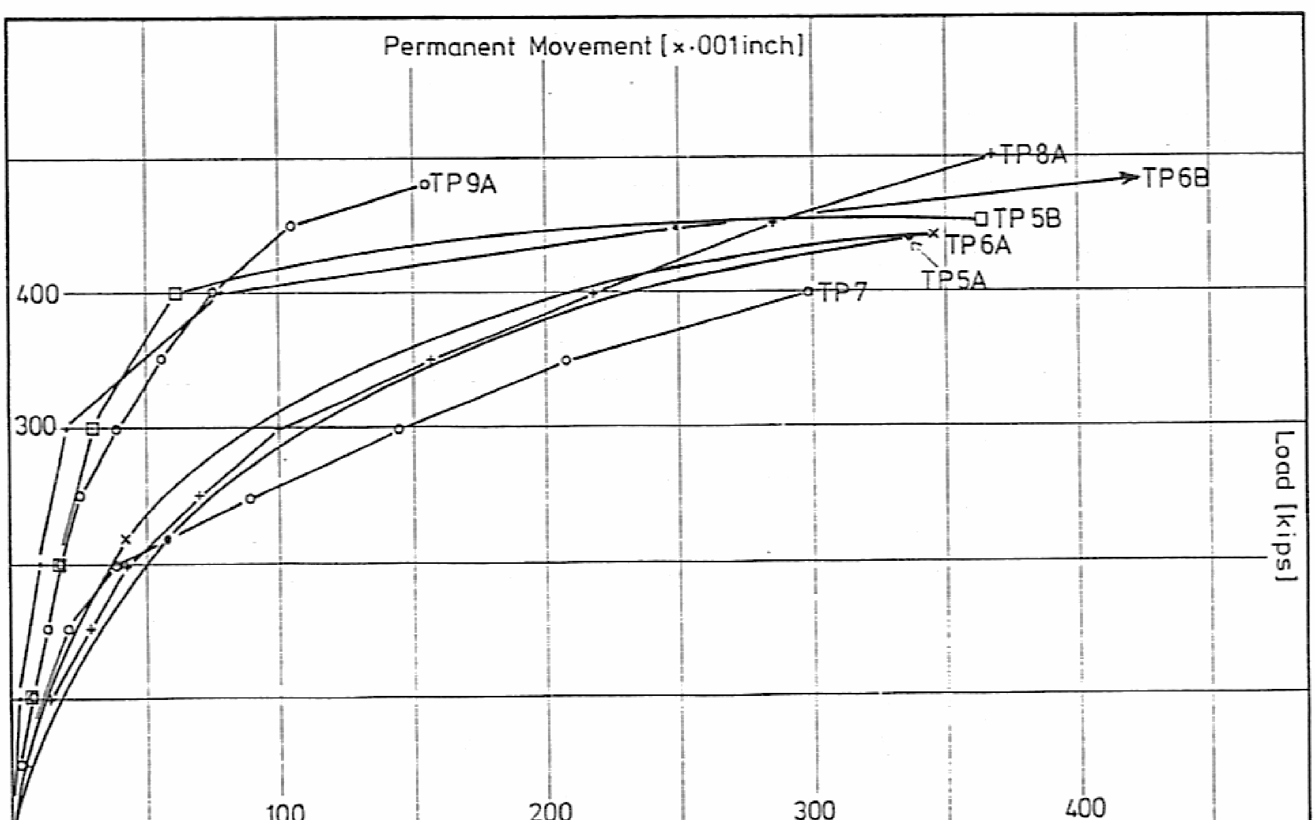


Figure 12. Permanent movements TP5-9.

	Creep at Load Maximum (kips) After 60 Minutes (x 0.001 inches)									
	50	100	150	200	250	300	350	400	450	500
TP1 (A)	Not Recorded									
(B)	5	1	15 (20 mins)	19 (30)	59	130				
TP2	0	4	8 (20 mins)	15	18	22	24	47	14 (5)	53 (10)
TP3	6 (20)	3 (20)	4 (20)	45	97 (40)	445 (40)				
TP4	0	8	0	13	13 (20)	49	49	450		
TP5 (B)	-	5 (10)	-	2 (10)	-	3 (15)	-	43	90 (15)	
TP6 (B)	-	2 (15)	-	6 (15)	-	3 (15)	-	39	100	100 (5)
TP7	0	6 (20)	5 (20)	17 (20)	19 (50)	39 (40)	27 (20)			
TP8	0 (5)	0 (5)	2 (5)	1 (5)	0 (5)	1 (5)	1 (5)	3 (5)	5 (5)	13 (30)
TP9 (A)	0 (5)	1 (5)	2 (5)	7 (5)	12 (10)	23 (15)	9 (50)	20 (30)	34	25

Table 2. Summary of Creep Test Data at Load Maxima
(minutes in parenthesis if shorter than 1 hour)

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	• Failed at 300 kips, second load cycle.
TP2	590	28	Grout-soil	21, apparently, but realistically 13	• Failed at 570 kips, second cycle. • Estimate 40% load resisted permanently in free length.
TP3	300	30	Grout-soil	10	• Possibly poor grout. • Failed on first cycle.
TP4	400	31	Grout-soil	13	• Failed on first cycle.
TP5	450	37	Internal	12 OK	• Failed on first cycle.
TP6	500	37	Internal	14 OK	• Failed on first cycle.
TP7	400	37	Internal	11 OK	• Failed on first cycle.
TP8	550	37	Internal	15 OK	• Failed on first cycle.
TP9	490	37	Grout-soil	13	• Failed on second cycle at 450 kips.

Table 3. Summary of Test Pile Performance

cycles of the same pile increase. It also seems that each pile has its own characteristic rate of increase in ER, although the actual ER value at failure appears to run in a relatively narrow range (around 2.0) for these pile types.

Figures 11 and 12 indicate that for most piles, the permanent movements are relatively small, only increasing beyond 0.100", say, within 25% of the ultimate maximum load. A similar pattern is observed in the creep records, which show that creep amounts typically exceed 0.040" in one hour, only within 15% of that maximum load. Note that since pile deflections were measured only at the head, and no telltales were incorporated into the piles themselves, it is impossible to allocate the cause of the permanent sets and creep movements: it cannot be judged if they arise from grout-soil phenomena, grout-steel phenomena, or any given combination of the two.

Table 2 shows that creep was very small until suddenly increasing close to ultimate load.

Table 3 highlights that there were two modes of pile failure - one, a steady plunge (TP1-4, and 9), interpreted as being along the grout/soil interface, the other or sudden, explosive release of load, consistent with a sudden internal (or structural) failure of the pile (TP5-8).

6.5 Discussion and Analysis: The Theory of Load Transfer in Cased Pinpiles

Table 4 summarizes the ER values and loads at which permanent settlements and creep became significant and failure occurred. It is known that TP1 had an initial free length of 40 feet plus about 2 feet extending above ground: therefore, its initial ER value of 1.42 corresponds to 42' of grouted casing and so 1.42/42 x 10 = 0.33 is the elastic factor. Using this factor one can then calculate the apparent equivalent active lengths at each subsequent load. In general, one can conclude

- permanent sets become significant when the load reaches within 5 or 10' of the bottom of the casing;
- creep becomes significant when the load has reached or exceeded the bottom of the casing;
- failure then occurs soon after (either by overloading the soil-grout interface, or by structural rupture of 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

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 4. Inferred and Estimated ER's and Equivalent Effective Free Lengths at Significant Intervals

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.

By comparing piles TP1 to 4, it appears that the free length composition of TP1 (debonded by outer casing), and TP3 (40' debonded by 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 <u>total</u> load capacities than TP2 and 3, although in reality the average grout-soil bond mobilized <u>in the bond zone</u> was most likely very similar.

This highlights the fact that fully grouted pinpiles 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 pinpiles in general especially when founded through the blocky or permeable materials characteristic of site fill preparations.

The ultimate average grout/soil bonds of 10 - 13 kips/foot were, surprisingly and relatively, low, and it may be speculated that the initial drill fluid used may have had some deleterious effect on either the soil (as a lubricant?) or the grout (as a contaminant?) although there is no firm evidence for either.

The results from the other five test piles are wholly consistent - appreciable load being transferred in upper reaches, progressive and irreversible debonding, and the links between depth of transfer and key failure phenomena. The most interesting information relates to the composition of the bond zone and the mode and magnitude of failure. Incidentally, all these piles were drilled with fluid 'B' (felt to be free from the concerns outlined above, with 'A'), and had no deliberate casing/grout debonding in their free lengths. They were, of course, installed through an average of 30' of clays and silts, about twice the depth present at the first site.

Table 5 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 11). Simplistically, and assuming an effective bond diameter of, say, 10 inches, this grout would provide a resistance of $\pi \times 10^2/4 \times 5$ kips = 392 kips before rupturing in a brittle fashion. This is very close the 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 a failure situation is achieved. Table 4 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.

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 1) 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

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 5. Summary of Performance of Second Group of Piles

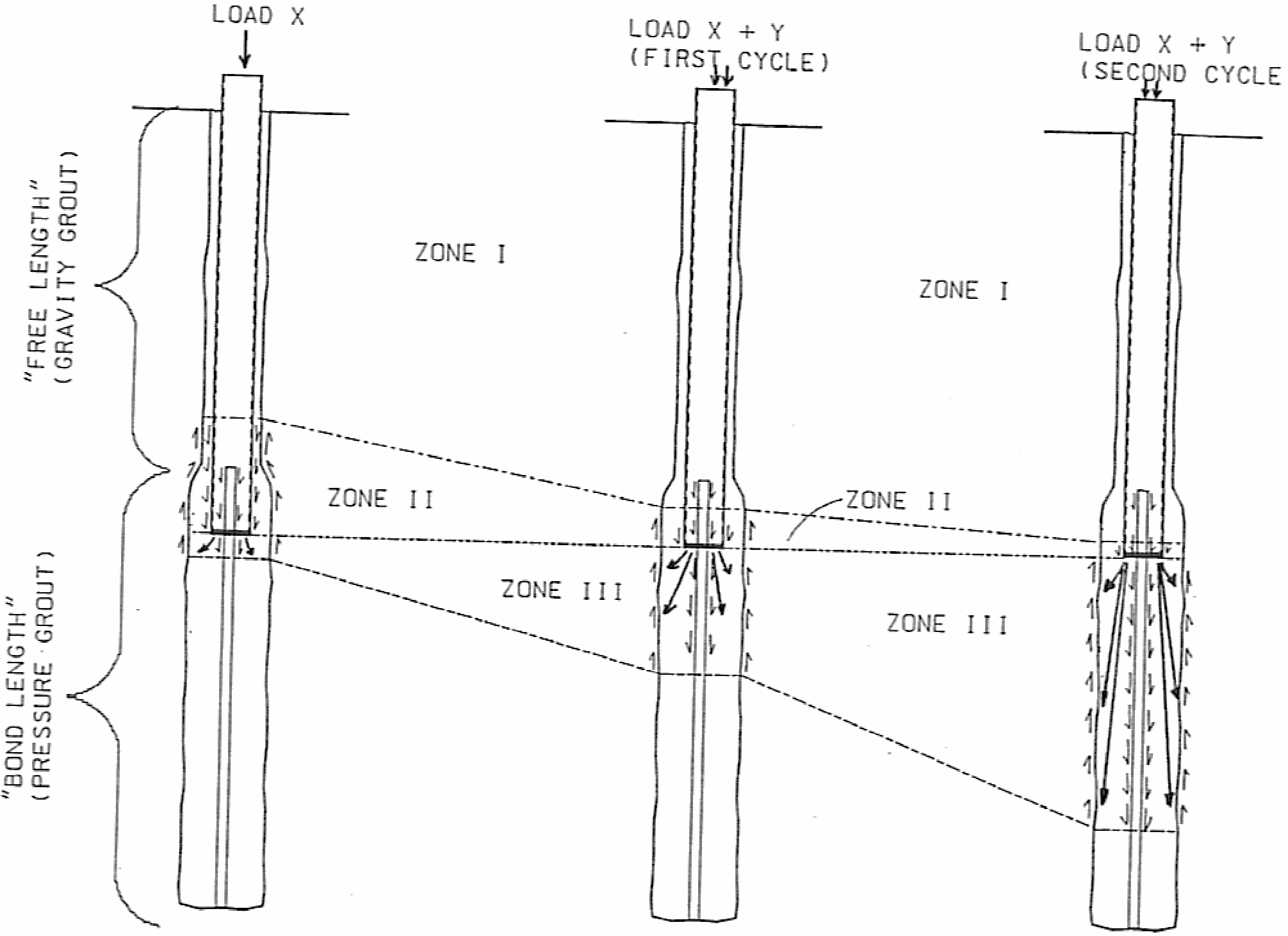
loading, resulting in a true bond zone load of perhaps only 570 x 60%, i.e., 342 kips at grout/soil failure. This would place its peformance far closer to that of its similar piles TP1, TP3, and TP4.

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 13). 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 stabilizes: 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 to the bond zone, a failure will occur. This time of transfer may vary from almost instantaneous (TP7, TP8) to many minutes (TP5, TP6). On a similar project in Port Vancouver, WA, identical phenomena have been observed, with explosive failure being recorded hours into creep test hold periods.

7. EFFECT OF THE UNDERPINNING

Each column was surveyed at weekly intervals throughout the seven months of installation of the pinpiles and the new concrete caps. This confirmed that settlements were occurring at the same rate as had been recorded over the previous 44 month period. This ranged up to 0.178 inches per month. At the end of pile and cap construction, the columns were then connected (by welding) to these new caps at the rate of 20 per month. Since that time, the settlement of every column has been totally arrested.



ZONE I - CASING TOTALLY DEBONDED FROM SURROUNDING GROUND
ZONE II - CASING TRANSFERRING LOAD (BY LATERAL BOND) TO SURROUNDING GROUND
ZONE III - CASING/GROUT COLUMN TRANFERRING LOAD BY END BEARING

Figure 13. Conceptual illustration of load transfer mechanisms at increasing loads, and at repeated load.

8. FINAL REMARKS

This project perfectly illustrates the application, design, construction and performance of Nicholson pinpiles.

In addition, however, the test pile program was enhanced beyond the initial purpose of simply verifying design assumptions for the production piles. This program has answered many of the questions raised against pinpile performance. Major conclusions are:

- in appropriate soil conditions, and with adequate bond zone reinforcment, compressive loads at least as high as 550 kips can be resisted.
- progressive debonding occurs between the casing and the surrounding grout, permitting load to be transferred ever deeper down the pile.
- the ultimate load of the pile strongly reflects the proportion of the load which can be accepted in this free length.
- when debonding reaches the bottom of the casing, load is then increasingly transferred into the bond zone. The subsequent pile capacity then depends on the composition of the bond zone. If insufficiently reinforced, this bond zone will quickly fail explosively. If better reinforced it will sustain more loads, perhaps even high enough to cause failure at the soil-grout interface.
- debonding is irreversible in its effect on load transfer length.
- within 20% or so of the ultimate load, permanent movements and creep rates will become quickly excessive, reflecting the fact that a significantly higher proportion of the total applied load is then having to be resisted by the bond length.

These observations have been noted in parallel series of tests being currently run elsewhere in the country on equally highly loaded pinpiles. These data will be published in due course, together with the laboratory and mathematical simulations.

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