Micropiles for Seismic Retrofit

D.A. Bruce and E.K. Chu
Nicholson Construction Company
P.O. Box 98, Bridgeville, PA 15017 (412) 221-4500

Abstract

The technique of micropiling, more commonly known as pinpiling in the United States, has been used for the static retrofit of existing structures throughout the world for over 40 years. These small diameter (<300 mm), high capacity (>1000 kN) friction piles are ideally suited for solving foundation problems in areas of tight access, low overhead, difficult geology or restrictive environmental impact conditions. More recently, and especially in California, micropiles have proved to be a cost effective underpinning option for seismic retrofit projects. The acceptability of their performance in axial loading conditions was demonstrated at the CALTRANS sponsored test program at I-280, and has been confirmed at a number of production sites throughout California.

This paper firstly provides a synopsis of sections of a recent state-of-practice study, with respect to definitions, classifications and applications. The second part addresses design considerations specific to seismic demands, and the paper concludes with details of two full scale field installations.

1. INTRODUCTION TO MICROPILES

A major state-of-practice study has recently been funded by the Federal Highway Authority (FHWA) and conducted by the authors' company. The subject was defined as drilled and grouted, cast-in-place, reinforced piles of nominal diameter less than 300 mm. Such piles are used for direct structural underpinning, and, where installed closely spaced in groups or networks, may be used for in-situ soil reinforcement. They have become increasingly popular throughout the world since their inception in Italy in 1952 and are widely used in association with urban and industrial development and redevelopment projects.

The study was extremely intense and wide-ranging, being commissioned partly to compliment the experimental research being conducted in France under the national research project FOREVER (Fondations Renforcées Verticalement). The review focused on existing procedures for design, construction, performance and quality assurance/control. However, it has also been able, from its international perspective, to recognize and articulate new, fundamental, generic classifications in these aspects.

The balance of Section 1 of this paper is an introduction to these new generic classifications, while Section 2 addresses detailed design concepts specific to their application for seismic situations. Section 3 provides case history information from work recently conducted in California.

While the term micropile is generically used, reference is also made to the term Pin Pile, which is a high capacity micropile specially developed by the authors' company (Bruce 1994; Bruce et al., 1993).

1.1 Characteristics and Definition of Micropiles

Typical overviews of bearing pile types (e.g. by Fleming et al., 1985) begin by making the distinction between displacement and replacement types (Figure 1). Piles which are driven are termed displacement piles because their installation methods displace laterally the soils through which they are introduced. Conversely, piles that are formed by creating a borehole into which the pile is then cast or placed, are referred to as replacement piles because

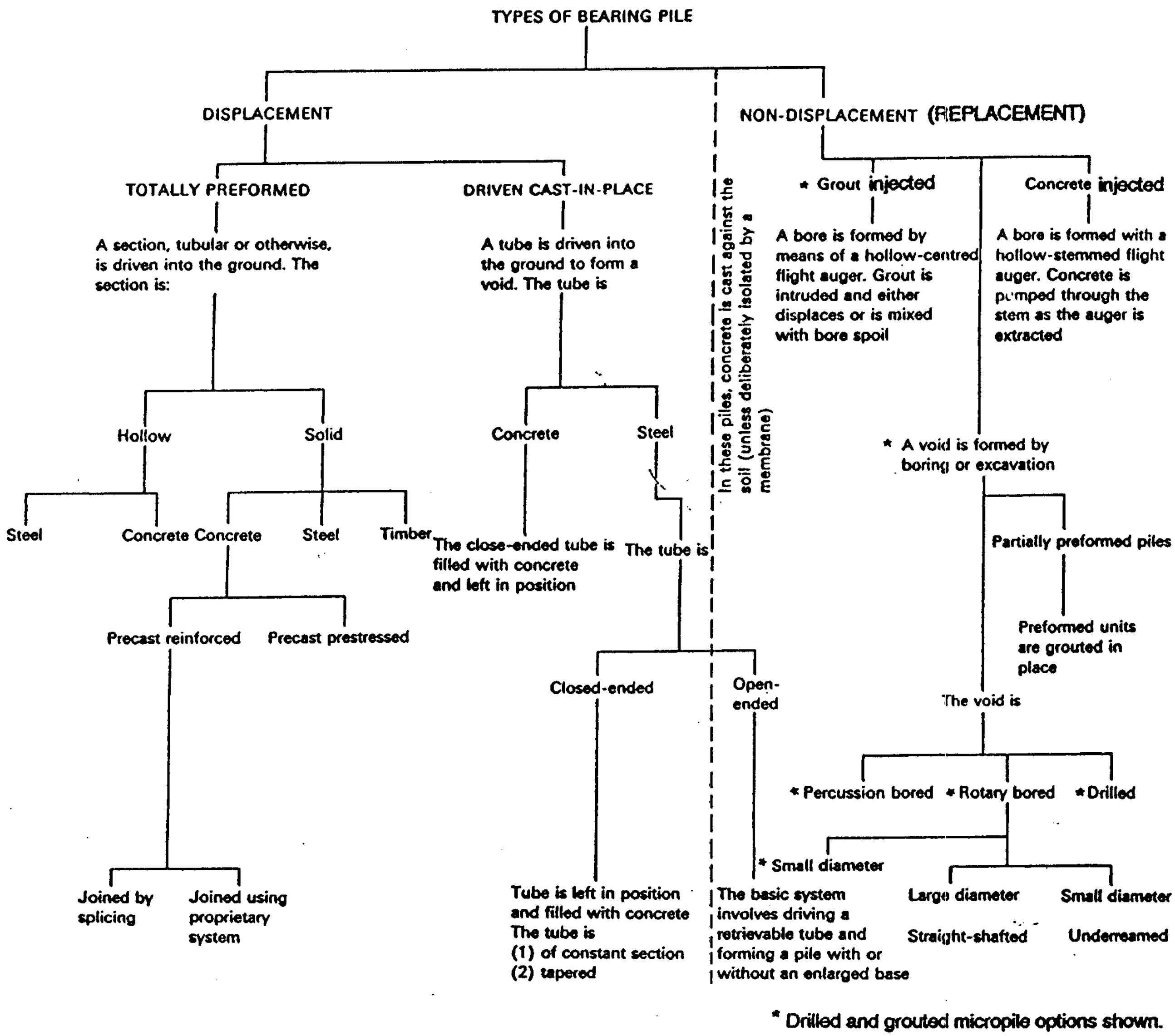


Figure 1. Classification of bearing pile types based on construction and current practice. Note: small diameter taken to be ≤ 600 mm; large diameter taken to be>600 mm. Fleming et al. (1985), after Weltman and Little (1977).

existing material, usually soil, is removed as part of the process. **Micropiles are a small-diameter subset of cast-in-place replacement piles.**

With conventional cast-in-place replacement piles, in which most, and occasionally all, the load is resisted by concrete as opposed to steel, small cross-sectional area is synonymous with low structural capacity. Micropiles, however, are distinguished by not having followed this pattern: innovative and vigorous drilling and grouting methods such as those developed in related geotechnical practice such as ground anchoring, permit high grout/ground bond values to be generated along the micropile's periphery. To exploit this potential benefit, therefore, high capacity steel elements, occupying up to 50 percent of the hole volume, can be used as the principal (or sole) load bearing element, with the surrounding grout serving only to transfer, by friction, the applied load between the soil and the steel. End-bearing is not relied upon, and in any event, is relatively insignificant given the pile geometries involved. Early micropile diameters were around 100 mm, but with the development of more powerful drilling equipment, diameters of up to 300 mm are now practical. Thus micropiles are capable of sustaining surprisingly high loads (compressive loads of over 5000 kN have been recorded), or conversely, can resist lower loads with minimal movement.

The availability of highly specialized drilling equipment and methods also allows micropiles to be drilled through virtually every ground condition, natural and artificial, with minimal vibration, disturbance and noise, and at any angle below horizontal. Micropiles are therefore used widely for underpinning existing structures, and the equipment can be further adapted to operate in locations with low headroom and severely restricted access.

All of these observations of its traditionally recognized characteristics therefore lead to a fuller definition of a micropile: a small diameter (less than 300 mm*), replacement, drilled pile composed of placed or injected grout, and having some form of steel reinforcement to resist a high proportion of the design load. This load is mainly (and initially) accepted by the steel and transferred via the grout to the surrounding rock or soil, by high values of interfacial friction with minimal end bearing component, as is the case for ground anchors (FHWA, 1984) and soil nails. They are constructed by the type of equipment used for ground anchor and grouting projects, although micropiles often must be installed in low headroom and/or difficult access locations. They must be capable of causing minimal damage to structure or foundation material during installation and must be environmentally responsive. The majority of micropiles are between 100 and 250 mm in diameter, 15 to 30 m in length, and 300 to 1000 kN in compressive or tensile service load, although far greater depths and much higher loads are not uncommon in the United States.

1.2 Classification of Micropiles

It has been common to find micropiles sub-classified according to diameter, some constructional process, or by the nature of the reinforcement. However, given the definition of a micropile provided above, the FHWA team concluded that a new, more rigorous classification be adopted based on two criteria:

- The philosophy of behavior, and
- The method of grouting.

*In France, the limit is set as 250 mm.

The former criterion dictates the basis of the overall design concept, and the latter is the principal determinant of grout/ground bond capacity.

1.2.1 Classification based on Philosophy of Behavior

Micropiles are usually designed to transfer structural loads to more competent or stable strata. They therefore act as substitutes or alternatives for other conventional pile systems (Figure 2a). For axially loaded piles, the pile/ground interaction is in the form of side shear and so is restricted to that zone of ground immediately surrounding the pile. For micropiles used as in-situ reinforcements for slope stabilization, recent research by Pearlman et al. (1992) suggests that pile/ground interaction occurs only relatively close to the slide plane, although above this level, the pile group may also provide a certain degree of continuity to the pile/ground composite structure. In both cases, however, the pile (principally the reinforcement) resists directly the applied loads. This is equally true for cases when individual piles or groups of piles are used. In this context, a group is defined as a tight collection of piles, each of which is subjected to direct loading. Depending on prevailing pile group codes, the individual pile design capacity may even have to be reduced in conformity with conventional "reduction ratio" concepts. These concepts were typically developed for driven piles, and so this restriction is almost never enforced for micropiles, given their mode of construction which tends to improve, not damage, the inter-pile soil.

When axially-loaded piles of this type are designed to transfer their load only within a remote founding stratum, pile head movements will occur during loading, in proportion to the length and composition of the pile shaft between structure and the founding stratum. In this instance, the pile can be preloaded (Bruce et al., 1990) to ensure that the structure can be supported without further movements occurring. Equally, if suitably competent ground conditions exist all the way down from below the structure, then the pile can be fully bonded to the soil over its entire length and so movements under load will be smaller than in the previous case.

These directly loaded piles, whether for axial or lateral loading conditions, are referred to as CASE 1 elements. They comprise virtually all North American applications to date, and at least 90 percent of all known international applications.

On the other hand, one may distinguish the small group of CASE 2 structures. Dr. Lizzi introduced the concept of micropiling when he patented the "root pile" (palo radice) in 1952. Soon after, he experimented with networks of such piles to provide reticulated root pile structures. The name alone evokes the concept of support and stabilization by an interlocking, three-dimensional network of reticulated piles similar to the root network of a tree. This concept involves the creation of a laterally confined soil/pile composite structure that can work for underpinning stabilization and earth retention, as illustrated in Figure 2b. In this idea, the piles are not heavily reinforced since they are not individually and directly loaded: they circumscribe a zone of reinforced, composite, confined material that offers resistance with minimal movement. The piles are fully bonded to the soil over their entire length and so for this case to work, the soil, over its entire profile, must have some reasonable degree of competence. Lizzi's research (1982) has suggested that a positive "network effect" (as opposed to a group reduction factor) is achieved in terms of load/movement performance, such is the effectiveness and efficiency of the reticulated pile/ground interaction producing the composite mass.

It is clear, therefore, that the basis of design for a CASE 2 network is radically different from a CASE 1 pile (or group of piles). Notwithstanding this difference, however, there will be occasions where there are applications transitional between these cases. For

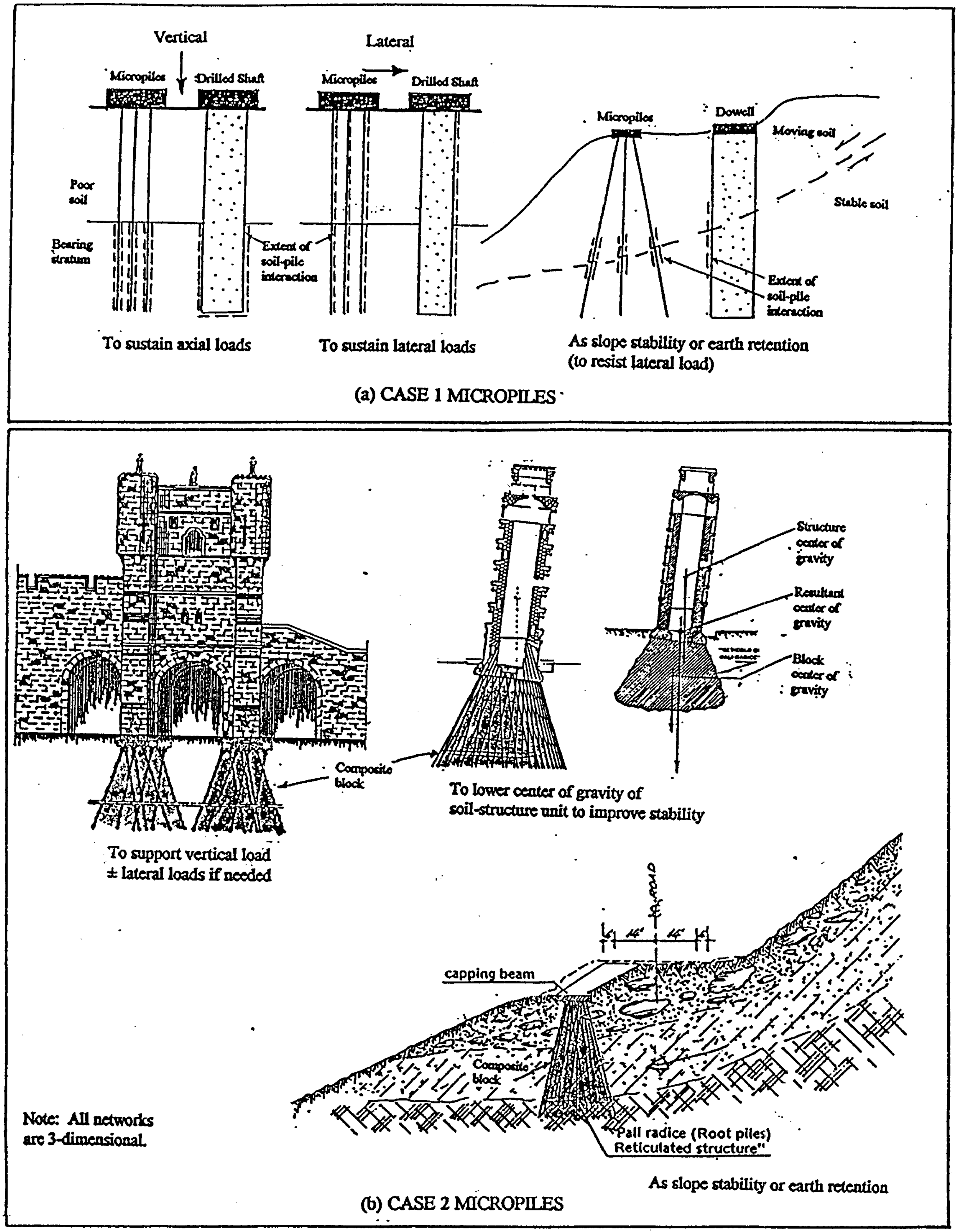


Figure 2. Fundamental classification of micropiles based on their supposed interaction with the soil.

example, it may be possible to achieve a positive group effect in CASE 1 designs (although this attractive possibility is currently, conservatively, ignored for pile groups), while a CASE 2 slope stability structure may have to consider direct pile loading conditions (in bending or shear) across well defined slip planes. By recognizing these two basic design philosophies, even those transitional cases can be designed with appropriate engineering clarity and precision.

This classification also permits us to accept and rationalize the often contradictory opinions made in the past about micropile fundamentals by their respective champions. For example, Lizzi (1982), whose focus is CASE 2 piles, was understandably an opponent of the technique of preloading high capacity micropiles, such as those described by Mascardi (1982) and Bruce (1992): these latter piles are now recognized as being of a different class of performance, in which complete pile/ground contact and interaction is not fundamental to their proper behavior.

1.2.2 Classification based on Method of Grouting

The successive steps in constructing micropiles are, simply:

- Drill, (usually with a temporary steel casing)
- Place reinforcement, and
- Place and, typically, pressurize grout (usually involving extraction of temporary steel drill casing).

There is no question that the *drilling* method and technique will affect the scale of the grout/ground bond which can be mobilized. On the other hand, the act of *placing reinforcement* should not influence this bond development. Overall, however, international practice both in micropiles (e.g. French Norm DTU 13.2, 1992) and ground anchors (e.g. British Code BS 8081, 1989) confirms that the method of *grouting* is generally the most sensitive construction control over grout/ground bond development. The following classification of micropile type, **based primarily on the type and pressure of the grouting** is therefore adopted. It is shown schematically in Figure 3.

- Type A: Grout is placed in the pile under gravity head only. Since the grout column is not pressurized, sand-cement "mortars", as well as neat cement grouts, may be used. The pile drill hole may have an underreamed base (to aid performance in tension), but this is now very rare and not encountered in any other micropile type.

- Type B: Neat cement grout is injected into the drilled hole as the temporary steel drill casing or auger is withdrawn. Pressures are typically in the range of 0.3 to 1 MPa, and are limited by the ability of the soil to maintain a grout tight "seal" around the casing during its withdrawal, and the need to avoid hydrofracture pressures and/or excessive grout consumptions.

- Type C: Neat cement grout is placed in the hole as for Type A. Between 15 and 25 minutes later, and so before hardening of this primary grout, similar grout is injected, once, via a preplaced sleeved grout pipe at a pressure of at least 1 MPa. This type of pile, referred to in France as IGU (Injection Globale et Unitaire), seems to be common practice only in that country.

- Type D: Neat cement grout is placed in the hole as for Type A. Some hours later, when this primary grout has hardened, similar grout is injected via a preplaced sleeved grout pipe. In this case, however, a packer is used inside the sleeved pipe so that specific

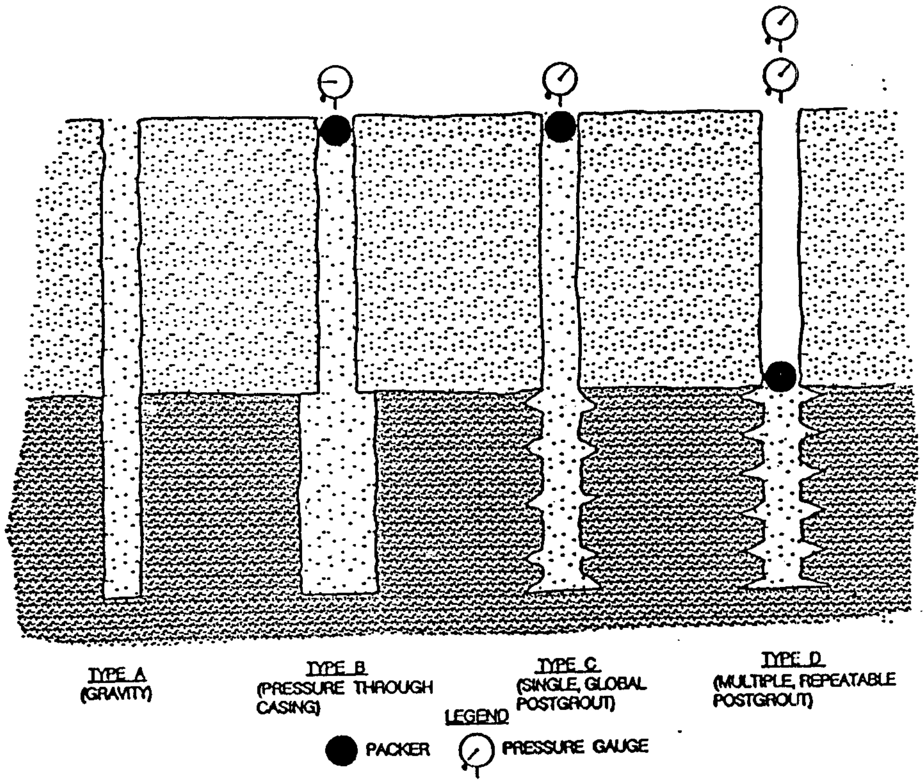


Figure 3. Classification of micropile type based on type of grouting.

horizons can be treated, if necessary, several times, at pressures of 2 to 8 MPa. This is referred to in France as IRS (Injection Répétitive et Sélective), and is a common practice worldwide.

The Pin Pile variant can be a Type A, B or D micropile, as illustrated in Sections 2 and 3 of this paper.

1.2.3 Combined Classification

Micropiles can therefore be allocated a classification number denoting the philosophy of behavior (CASE 1 or CASE 2), which relates fundamentally to the design approach, and a letter denoting the method of grouting (Type A, B, C or D), which reflects the major constructional control over capacity.

For example, a repeatedly post-grouted micropile used for direct structural underpinning is referred to as Type 1D, whereas a gravity grouted micropile used as part of a stabilizing network is Type 2A.

1.3 Classification of Micropile Applications

Micropiles are used in two basic applications: as structural support and as in-situ soil reinforcement (Figure 4). For direct structural support, groups of micropiles are designed on the CASE 1 assumptions, namely that the piles accept directly the applied loads, and so act as substitutes for, or special versions of, more traditional pile types. Such designs often demand substantial individual pile capacities and so piles of construction Types A (in rock or stiff cohesives), B and D (in most soils) are most commonly used.

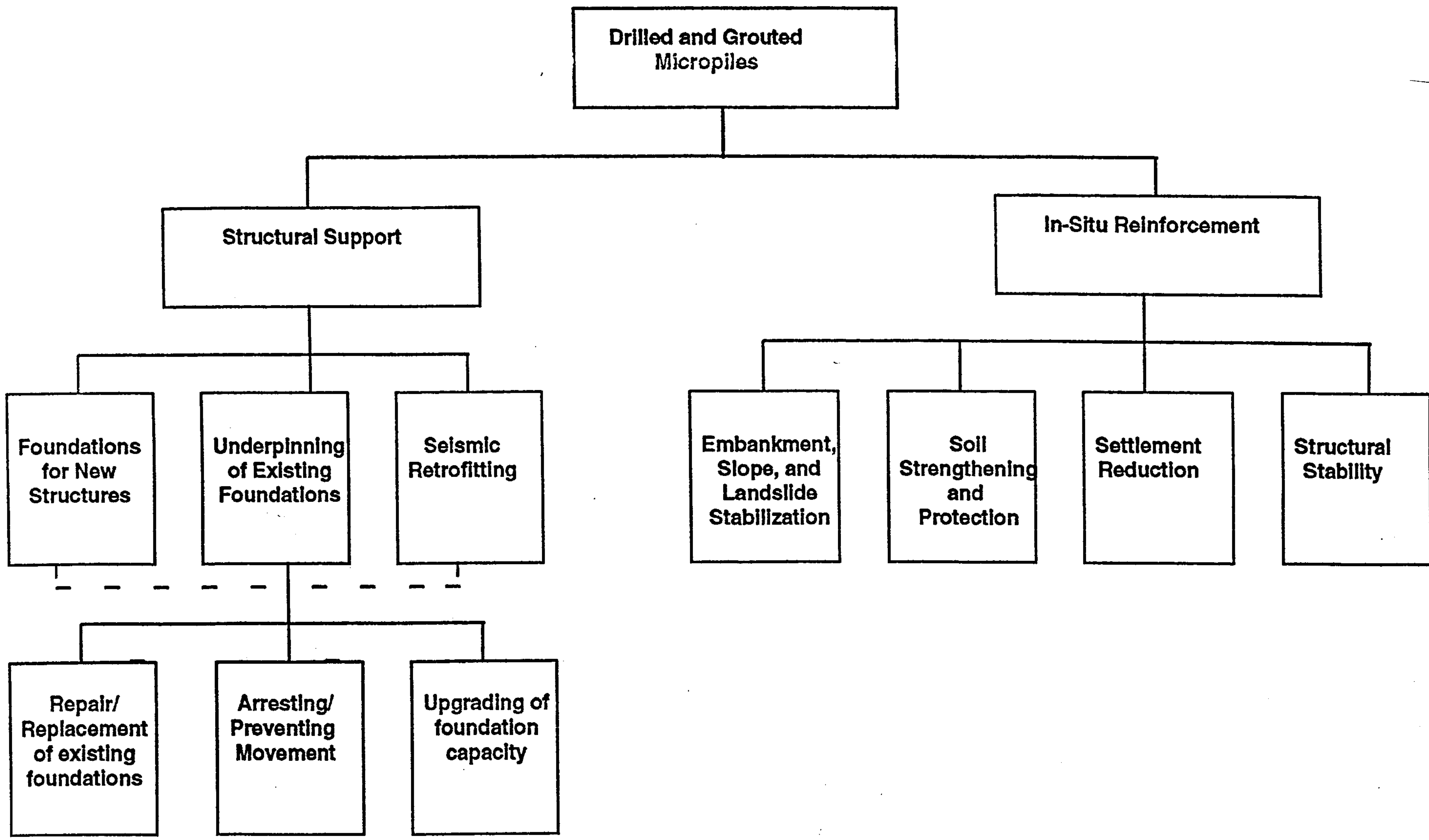


Figure 4. Classification of micropile applications.

For micropiles used as in-situ reinforcement, the original CASE 2 network featured low capacity Type A piles. Research by Pearlman et al. (1992), on groups of piles, suggests that in certain conditions and arrangements, the piles themselves are principally, directly, and locally subjected to bending and shearing forces. This would, by definition, be a CASE 1 design approach. Such piles typically are highly reinforced and of Type A or B only.

Whereas CASE 1 and CASE 2 concepts alone or together can apply to slope stabilization and excavation support, generally only CASE 2 concepts apply to the other major applications of in-situ reinforcement. Little commercial work has been done in these applications (with the exception of improving the structural stability of tall towers, (Figure 2b). However, the potential is real and the subject is being actively pursued in the FOREVER program in France. Table 1 summarizes the link between application, classification, design concept and constructional method. It also provides an indication of how common each application appears to be world-wide.

2. ASPECTS OF DESIGN SPECIFIC TO SEISMIC RETROFIT APPLICATIONS

In seismic conditions, micropiles may be subjected to two types of response, namely the interaction response (superstructure, substructure and soil interaction under earthquake excitation) and free-field earth response (earthquake ground movements), as illustrated in Figure 5.

Understanding the interactive behavior of the superstructure, the micropile system and the surrounding soil is crucial for proper seismic design. These three elements are inseparable under earthquake loading. One cannot determine the seismic loading on the superstructure without considering the effect of the stiffness coupling of the micropiles and the soils on the superstructure. One key to an adequate pile seismic design is the correct determination of the magnitude of the cyclic foundation displacement or rotation associated with soil yield deformations. In most cases, the maximum relative displacements among the superstructure, the pile system and the soils govern the seismic design.

In flexible soil environments where free-field ground motion dominates, large relative lateral displacements may occur along the pile length inducing higher bending moment in the piles than that resulting from the interaction response. Under this condition, the more flexible vertical pile systems, where lateral load is resisted by bending near the pile head, are recommended. Typically, the piles are not designed to exceed the yield stress under seismic loads. However, the rotational yielding of the piles can provide a useful form of energy dissipation. Furthermore, in soils where large changes in soil stiffness may occur over the pile embedment, large curvature could develop at the soil interface between soft and rigid soil layers causing high pile bending moments at that interface. The larger the pile curvature, the stronger in bending the micropile needs to be. Therefore, there may be limitations to the use of micropiles in ground conditions in which large curvatures caused by free-field displacements are likely to occur.

In all cases, the use of flexible ductile piles in severe seismic areas should be specified. Micropiles are high strength pipe piles which offer high axial strength, flexibility and ductility. In low headroom situations where joints in the pipe are inevitable, the joints can be placed below the high bending moment areas of the pile. Alternatively, the newly developed NCC Pin Pile threaded coupler can be used. This coupler ensures a ductility and strength equal to that of the pile casing. The use of this coupler eliminates the need to avoid having joints at critical bending areas.

APPLICATION	STRUCTURAL SUPPORT	IN-SITU EARTH REINFORCEMENT			
Sub-applications	Underpinning of Existing Foundations New Foundations Seismic Retrofitting	Slope Stabilization and Excavation Support	Soil Strengthening	Settlement Reduction	Structural Stability
Design concept	CASE 1	CASE 1 and CASE 2 with transitions	CASE 2 with minor CASE 1	CASE 2	CASE 2
Construction type	Type A (bond zones in rock or stiff clays) Type B and D in soil (Type C only in France)	Type A (CASE 1 and 2) and Type B (CASE 1) in soil	Type A and B in soil	Type A in soil	Type A in soil
Estimate of relative application	Probably 95% of total world applications	0 to 5%	Less than 1%	None known to date	Less than 1%

Table 1. Relationship between micropile application, design concept, and construction type.

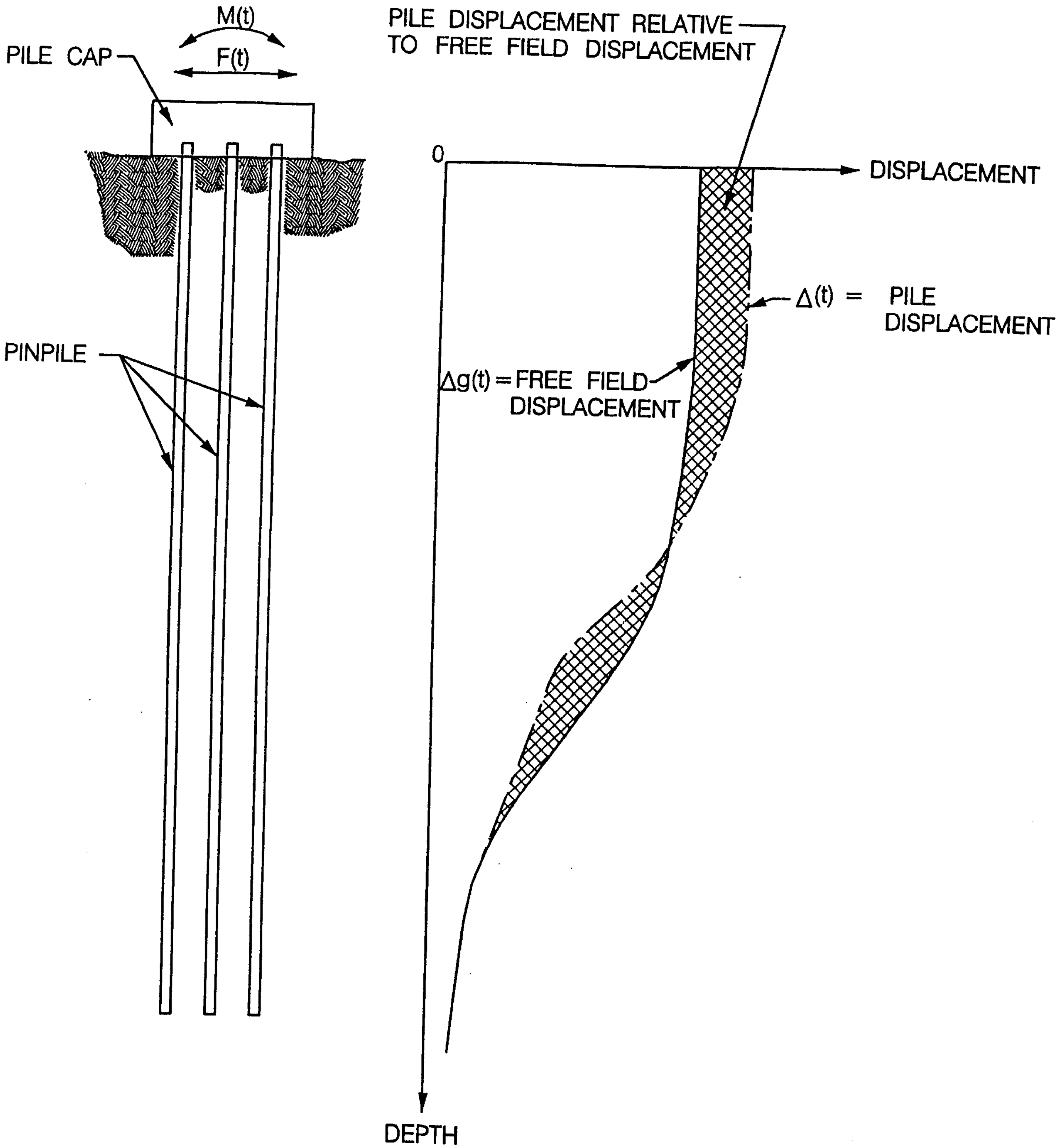


Figure 5. Schematic Soil-Pile interaction response under seismic loading.

Due to the complexities of seismic analysis, pile designers usually simplify the design process by making the assumption that the interaction of the superstructure, the piles and the soils can be uncoupled. Once the system is uncoupled, each of these elements can be tackled separately. One approach is to assume an equivalent elastic spring stiffness for the pile-soil system when analyzing the superstructure. Choosing the equivalent spring stiffness concept can be difficult. It is at best an engineering judgment and could lead to erroneous results at times if the non-linear soil response is ignored. Caution should always be exercised as to the validity of the design assumptions for specific project conditions.

Some of the commonly used seismic retrofit Pin Pile design concepts can be summarized as follows:

- Axially Loaded Vertical Pin Pile (Rocking) - When lateral resistance is derived from the soil passive resistance within the depth of the existing/retrofitted footing, Pin Piles are introduced as reinforcements around and/or within the existing pile cap to help resist uplift or rocking during earthquake loading. Particularly in low headroom conditions and restricted areas where existing footings cannot be extended, their small physical size and their high load capacity make Pin Piles a viable solution. Generally, the required Pin Pile tension capacity is equal to the required compressive Pin Pile capacity. Lateral resistance is not a requirement of the Pin Piles for this concept.

- Laterally and Axially Loaded Vertical Pin Pile (Rocking and Base Shear) - The Pin Piles can be placed around and/or within the existing footing depending on the rocking resistance requirement and superstructure base shear to be resisted (soil passive resistance within the depth of the footing is insufficient to provide the required total lateral resistance). A laterally loaded pile analysis using the COM624 or LPILE Program can be used to determine the pile bending moment under a given lateral load on the pile. The pile is subjected to both bending stress and axial stress simultaneously during earthquake loading. Again, their small physical size makes them relatively flexible under lateral load. In retrofit cases where a significant increase in lateral stiffness of the existing foundation is undesirable for the superstructure, vertical Pin Piles can be a solution which offers lateral resistance and flexibility.

- Laterally and Axially Loaded Inclined Pin Pile (Rocking and Base Shear) - This concept is most suitable for foundations which require significant lateral resistance. It has the benefit of limited flexibility and the economy of axial load resistance to lateral load. The required lateral resistance is primarily derived from the horizontal component of an inclined pile's axial load: the larger the inclination of the Pin Pile, the larger is the lateral resistance. The inclinations are preferably arranged in such a fashion that only axial loads are induced. A laterally loaded inclined pile analysis using COM624 or LPILE can be used to determine the bending moment in the pile under a given lateral load. In retrofit cases where a significant increase in lateral stiffness of the existing foundation is needed for the superstructure, inclined Pin Piles can be a solution which offers larger lateral resistance and the desired added lateral foundation stiffness.

It is apparent that piles in severe earthquake zones should be designed to ensure that they do not exceed the yield stress and that the flexural yielding of the superstructure columns is forced to occur above the pile cap. As pile damages after a major earthquake cannot be easily inspected, many DOT's have employed this design philosophy to minimize pile foundation damages. In short, piles under seismic loading must be tolerant flexible ductile piles due to the uncertainties of ground excitation and interaction response. Micropiles using grout filled steel casings (i.e. Type 1A or 1B) are therefore very attractive options in highly seismic areas where conventional pile systems cannot be employed in retrofit projects.

3. SEISMIC RETROFIT APPLICATIONS

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 (Zelinski, 1992). 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.

3.1 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, the authors' company installed six Pin Piles of three different types at the test site in San Francisco (Mason, 1992).

A simplified stratigraphy of the test site was:

0 to 6 m	Fill
6 to 34 m	Clay (Soft Bay Mud deposits)
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 178 mm diameter steel casing. This tendon was stressed and locked off against the top of the casing prior to the pile test. A 11 m long pressure grouted bulb was formed in the sand, which included a 3 m embedment of the casing, a 1.5 m buffer zone, and a 6 m bond length for the tendon. This was a modified Type 1B micropile.

Pin Pile. A 410 MPa yield strength steel reinforcing bar was installed within and below the 178 mm diameter casing. No prestress loading was applied. The pile had a pressure grouted bulb 9 m long in the sand which included a 3 m embedment of the pipe, and 6 m of reinforcement extending below. This was a conventional Type 1B pile.

NFC-Pile. The 178 mm diameter steel casing was drilled full length into the lower sands and a 410 MPa steel bar placed full length. For these piles, the grout was introduced as drilling in the sand progressed. This was also a modified Type 1B micropile.

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 43 to 47 m. Shallow piles were installed to about 32 m. The piles were tested to a 890 kN design load and then loaded to failure. The NCC pile load test results are shown in Table 2.

3.2 CALTRANS North Connector - I110 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 610 mm diameter cast-in-drilled-hole (CIDH) concrete piles placed around the exiting 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

PILE NO.	DESCR.	PILE CAPACITY (KIPS)		ACTUAL ELASTIC DEFL. @ 200 KIPS		ACTUAL TOTAL DEFL. @ 200 KIPS	
		Tension	Compression	Tension	Compression	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 2. Summary of Test Results for Nicholson Piles in CALTRANS Test Program.
* Pile damaged during tension test loading: No Compression Test Results
NA – Not Applicable
1 kip = 4.45 kN; 1 in = 25.4 mm

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 8.0 m and dense to very dense sands and gravels below. The ground-water table was approximately 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 6m.

The Type 1B micropiles for this project were required to support an ultimate compressive load of 2225 kN with a maximum pile head total movement of less than 15 mm. Each comprised:

- An upper pile length extending to 9 m below the bottom of the existing footing, consisting of a 178 mm o.d., 12.7 mm wall thickness steel casing, reinforced full length with two 35 mm diameter grade 150 threadbars and filled with neat cement grout.

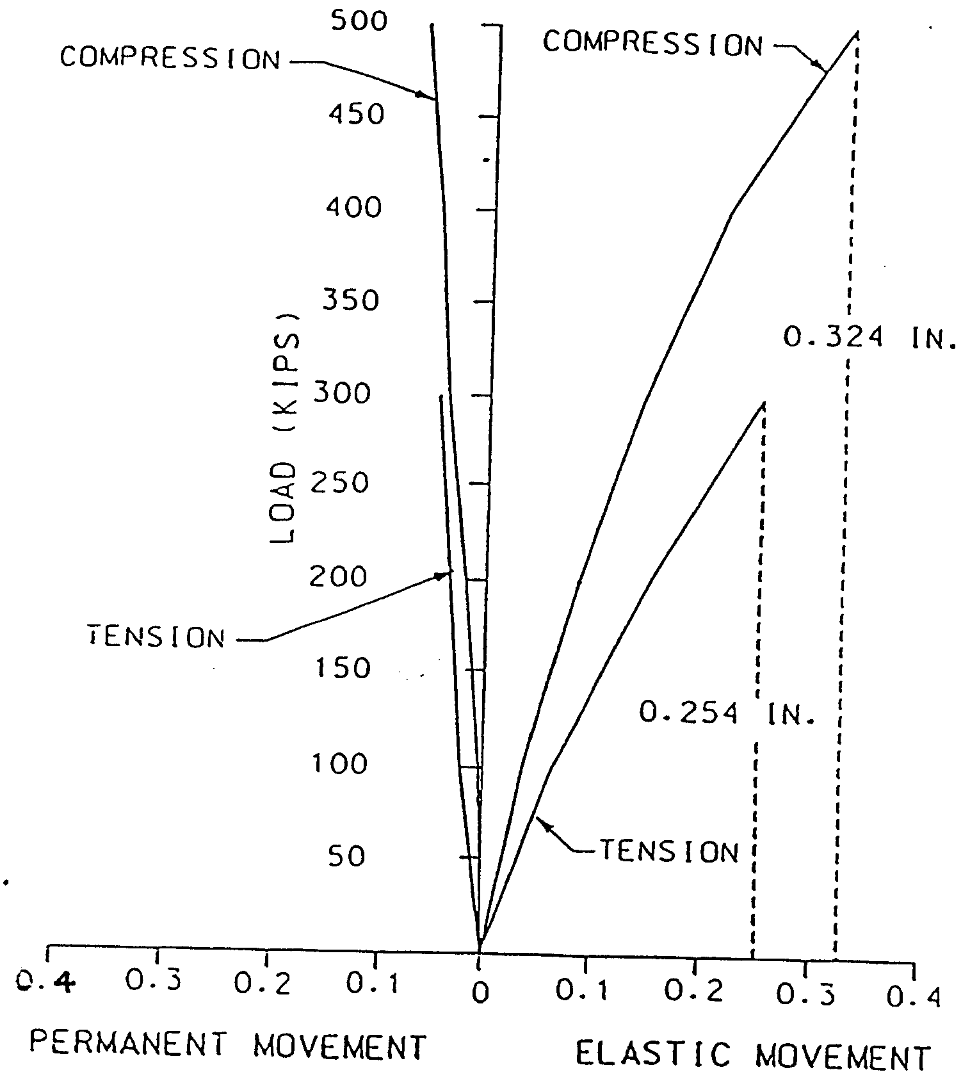
- A pile bond length extending from 9 m to 18 m below the bottom of the existing footing, consisting of a pressure grouted bond zone, reinforced with the two 35 mm diameter threadbars, extending to the pile tip, and the 178 mm diameter steel pipe, extending 1.5 m into the top of the bond length.

- 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 20 m allowing testing to be performed before footing excavation. The casing was placed in 3 m lengths, and the threadbars were placed in 3 m and 6 m coupled lengths, centralized in the pile with plastic spacers. Maximum grout pressure attained during grouting of the pile bond length ranged from 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 1340 kN load and the compression test to the required 2225 kN. The pile was loaded in 445 kN cycles, with the load applied in 89 kN 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. (89-445kN decrements).

<u>Figure 6</u> summarizes the load test data. The pile successfully resisted the required maximum tensile load of 340 kN with a total movement at maximum load of 7.72 mm and permanent movement of 1.27 mm at zero load after loading. Creep movement during the 5 minute hold at 1340 kN was 0.15 mm. The Pin Pile then successfully supported the required maximum compressive load of 2225 kN with a total movement at maximum load of 9.96 mm and a permanent movement of 1.73 mm at zero load after loading. Creep during the 5 minute hold at 2225 kN was 0.18 mm.



<u>Figure 6</u>. Pin Pile Permanent and Elastic Movement Analysis.
North Connector Overcrossing I-110, Los Angeles, CA
1 kip = 4.45 kN
1 in = 25.4mm

4. FINAL COMMENTS

The technique of micropiling, and especially the more highly reinforced variant termed Pin Piling, has true potential in foundation seismic retrofit applications. The international research recently completed has helped to focus attention on critical areas of design and performance which were formerly incompletely understood, leading to imprecision and confusion. This new clarity has been supplemented by excellent field data on both test and production sites in California to the degree that micropiling is being widely regarded as a specialty construction tool of considerable value and reliability.

References

British Standards Institution (BS). (1989). "Ground anchorages." BS8081. London.

Bruce, D.A., S.L. Pearlman and J.H. Clark. (1990). "Foundation rehabilitation of the Pocomoke River Bridge, MD, using high capacity preloaded pinpiles." The International Bridge Conference, June 18-20, Pittsburgh, PA. Paper 1BC-90-42, 9 pp.

Bruce, D.A. (1992). "Recent progress in American pin pile technology." Proceedings ASCE Conference Grouting, Soil Improvement and Geosynthetics. ASC E Geotechnical Special Publication 30. Ed. by R.H. Borden, R.D. Holz and I. Juran. New Orleans, Louisiana. Feb. 25-28. pp. 765-777.

Bruce, D.A., Bjorhovde, R., Kenny, J. (1993). "Fundamental Tests on the Performance of High Capacity Pin Piles." Proc. DFI Annual Meeting, Pittsburgh, PA, October 18-20, 33 pp.

Bruce, D.A. (1994). "Small diameter cast-in-place elements for load bearing and in-situ earth reinforcement." In *Ground Control and Improvement* by P.P. Xanthakos, L.W. Abramson, and D.A. Bruce. John Wiley and Sons. Chapter 6. pp. 406-492.

DTU (1992). (Unified Technical Document) No. 13.2 "Deep foundation work for buildings." Amending supplement No. 2 to the specifications of DTU 13.2 of June 1987.

FHWA (1984). "Permanent ground anchors." By R.S. Cheney. Report No. FHWA-DP-68-1R.

FHWA (1995). "Drilled and Grouted Micropiles - The State of Practice" by D.A. Bruce and I. Juran. 5 Volumes - to be published by December, 1995.

Fleming W.G.K., A.J. Weltman, M.F. Randolph, and W.K. Elson (1985). "Piling Engineering." First Edition. Surrey University Press. Glasgow. 380 pp. (Second Edition 1992).

FOREVER (1992). "Soil Reinforcement with Micropiles: Introductory Report." July 1992. 22 pp.

Lizzi, F. (1982). "Static restoration of monuments." Sagep Publisher. Genoa, Italy. 146 pp.

Mascardi, C.A. (1982). "Design criteria and performance of micropiles." Symposium on Soil and Rock Improvement Techniques including Geotextiles, Reinforced Earth and Modern Piling Methods. December. Bankgkok. Paper D3. 18 pp.

Mason, J.A. (1992). "Tension Pile Test." Proc. 3rd NSF Workshop on Bridge Engineering Research in Progress, La Jolla, CA., November 16, 17, pp. 67-70.

Pearlman, S.L., B.D. Campbell, and J.L. Withiam. (1922). "Slope stabilization using in-situ earth reinforcements." Specialty Conference on Stability and Performance of Slopes and Embankments - II. ASCE Geotechnical Special Publication No. 31 Ed. by R.B. Seed and R.W. Boulanger. June 29-July 1. Berkeley, California. pp. 1333-1348.

Zelinski, R. (1992). "Bridge Foundation Retrofits." University of Wisconsin Short Course on Specialty Geotechnical Construction Techniques, San Francisco, CA, November 17.