Micropiles: the state of practice. Part II: design of single micropiles and groups and networks of micropiles

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This series of papers summarizes the research project initiated in 1993 by the US Federal Highway Administration to review the state of practice, case studies and design methods of micropile group systems. Following a brief description of the recently adopted classification system for micropile design previously outlined by Bruce *et al.* (1997) (part 1), this paper (part 2) presents a summary of recommendations for the design of single micropiles and groups and networks of micropiles for selected engineering applications, including direct structural support and *in situ* soil reinforcement. Preliminary estimates of the ultimate axial and lateral capacity of micropiles, as outlined by different authors, are presented first. Design guidelines which have been developed generally through observations on full-scale testing and field experiences are discussed with regard to cohesionless soils, cohesive soils and rocks. Group and network effects are investigated and preliminary conclusions are presented along with proposed design guidelines for micropile groups. Existing analytical approaches are evaluated through comparisons with experimental data obtained by different investigators on the engineering behaviour of micropile groups and reticulated micropile networks under different loading conditions.

Cette serie d'articles resume les principales conclusions de l'étude entreprise par l'Administration Federale des Routes (FHWA) afin d'evaluer l'etat des connaissances actuelles, les experiences envrae-grandeur, et les méthodes de conception et dimensionnement des micropieux. Après une brève description de la nouvelle classification des micropieux proposée par Bruce *et al.* (1997)—1 ére partie, ce papier (2 éme partie) présente un resumé des recommendations pour le dimensionnement des micropieux seuls, des groupes et des reseaux des micropieux pour diverses applications, et en particulier le support direct et la renforcement in-situ. Ces recommendations concernent l'evaluation preliminaire de la capacité portante, axiale et laterale, des micropieux, ainsi que les méthodes de dimensionnement proposées par different auteurs a partir des experimentations en vraie grandeur et observations sur ouvrages réels pur differents types des sols et des roches. L'effet de group et l'effet de reseau sont aussi presentes et analyses.

Keywords: Mini-piles; pin-piles; soil reinforcement; network systems

Introduction

Micropiles are defined as small-diameter drilled and grouted piles, a subset of cast-in-place replacement piles. With conventional cast-in-place replacement piles, in which most of the load is resisted by concrete as opposed to steel, small cross-sectional area is synonymous with low structural capacity. This is not the case with micropiles, however. Innovative drilling and grouting methods permit high grout/ground bond values to be generated along the micropile's periphery. To exploit this benefit, high-capacity steel elements can be used as the principal 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 on, 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 considered practical. Micropiles are capable of sustaining surprisingly high loads (compressive loads of more than 5000 kN have been recorded) or, conversely, can resists lower loads with minimal movement. Most micropiles are 100–250 mm in diameter, 20–30 m long and 300–1000 kN in compressive or tensile service load, although far greater depths and much higher loads are common in the US.

Micropiles have been subclassified by others according to diameter, construction process or the nature of the reinforce-

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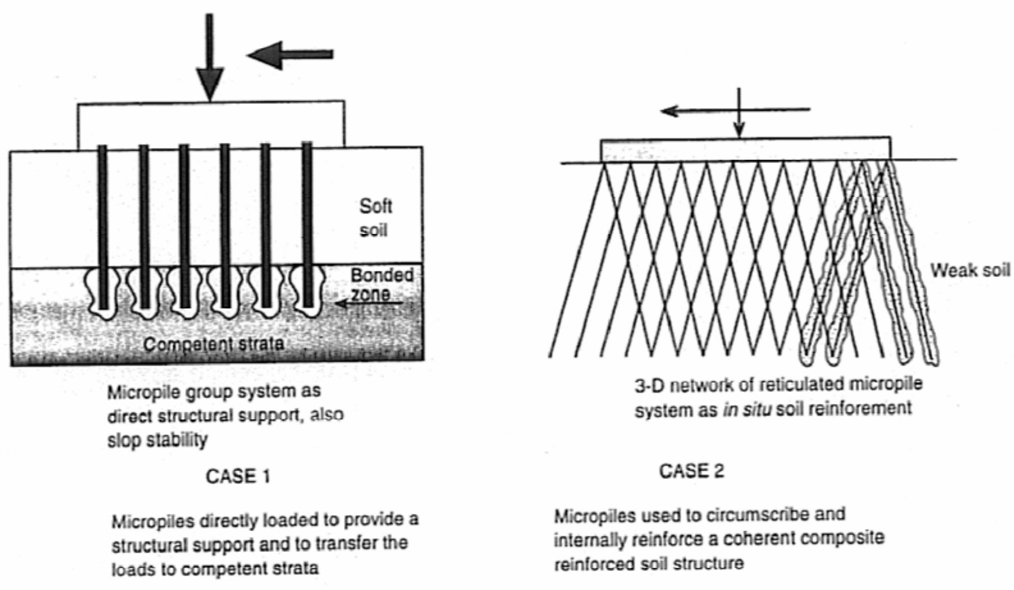
ment. However, in the course of the Federal Highway Administration (FHWA) study (Bruce and Juran, 1997), it has been concluded that a new, rigorous classification system for micropile design should be adopted on the basis of two criteria: the philosophy of behaviour, which dictates the basis of the overall design concept; and the method of grouting, which mainly determines grout/ground capacity. The classification of micropiles is based primarily on the type and pressure of the grouting. As shown in Fig. 1, four types have been defined, as follows.

- Type A (gravity-grouted micropiles): the grout is placed in the pile under gravity head.
- Type B (low-pressure-grouted micropiles): pressures are typically in the range of 0·3–1 MPa, and the neat cement grout is injected into the drilled hole as the temporary steel drill casing or auger is withdrawn.
- Type C (high-pressure-grouted micropiles): the neat cement grout is placed in the hole as for type A but 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.
- Type D (postgrouted micropiles): the neat cement grout is placed in the hole as for Type A. When this primary grout has hardened, similar grout is injected via a preplaced sleeved postgrout pipe. The use of a packer inside allows specific horizons to be treated several times if necessary, at pressures of 2–8 MPa.

On the basis of the philosophy of behaviour, two basically different design concepts, illustrated in Fig. 1, have been developed, as follows.

Case 1 refers to micropiles which are designed to transfer axial and/or lateral structural loads through soft or weak soils to more competent strata. These micropiles are generally used as stuctural support to resist directly the applied loads. As illustrated in Fig. 1, this design concept relies mainly on substituting conventional large-diameter pile



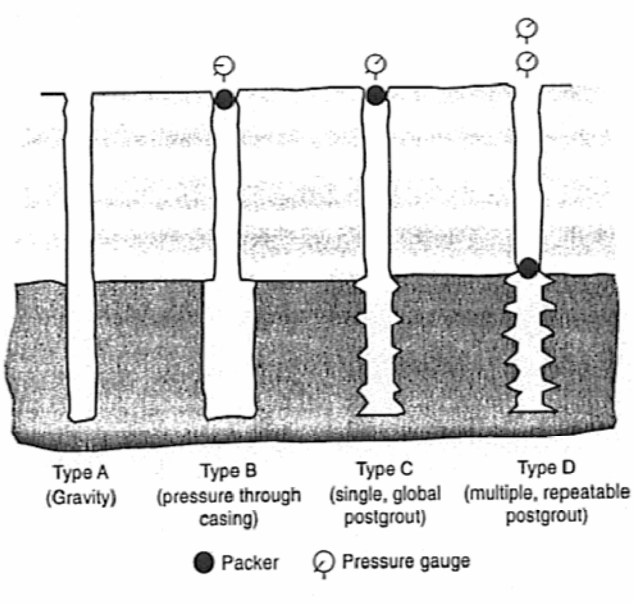


Fig. 1. Design concept of micropile systems: (a) philosophy of behaviour; (b) type and pressure of grouting

types with closely spaced, small-diameter, high-strength piles to arrest settlement and cost-effectively allow for engineering applications such as underpinning and seismic retrofitting that cannot be accomplished with current piling techniques.

Case 2, referring to Lizzi's (1978) original 'root pile' design concept, relies primarily on using a three-dimensional network of reticulated micropiles to create *in situ* a coherent, composite, reinforced soil system. According to this design concept, the piles are not designed individually and directly to support the applied load but rather to circumscribe and internally reinforce the *in situ* soil, forming a composite gravity structure to support the applied loading with minimal movement.

As demonstrated by Lizzi (1982), the engineering behaviour of micropile-reinforced soil is highly dependent on group and network effects that may significantly improve the settlement response characteristics and the overall resistance and shear strength of the composite soil–micropile system. However, the impacts of the group and network effects on the overall response of the micropile system to boundary loading have not yet been sufficiently investigated and are not taken into consideration in current design practice.

These two design concepts result in different resisting forces in the micropiles and lead to a significantly different selection of micropile types and installation techniques. Case 1 designs generally envision the micropiles connected to the rigid loading cap as a structural frame that has to withstand combined loading and bending moments and, therefore, often demand substantial individual capacities. Hence micropiles of Type A (gravity-grouted, bond in rock), Type B (pressure-grouted through the head) and Type D (postgrouted) with high-strength reinforcements are most commonly used. Case 2 designs feature a redundant, monolithic 'gravity' system with low-capacity Type A (gravity-grouted, fully bonded in soil) or Type B (low-pressure-grouted) micropiles.

Furthermore, while according to Lizzi (1982) no preloading should be applied in Case 2 systems, for Case 1 directly loaded micropiles, preloading can be used to eliminate or minimize postconstruction movements by anticipating the elastic shortening of the 'unbonded' micropile length in soft weak/soils under the applied loading (Xanthakos *et al.*, 1994). In some cases, specific applications and/or site conditions may require design schemes which represent intermediate cases between the two basic design concepts outlined above.

This paper presents a summary of recommendations for the design of single micropiles and groups and networks of micropiles for selected engineering applications, including direct structural support and *in situ* soil reinforcement. It presents the main conclusions of the state-of-practice review on drilled and grouted micropiles conducted by the FHWA in conjunction with the five-year French national research project 'FOREVER'. The French FOREVER project, organized under the aegis of the Institute for Applied Research and Experimentation in Civil Engineering (IREX) in partnership with industry and research institutions, recognized specifically two major research issues: (i) the need for recommendations related to long-term corrosion, and (ii) assessment of group and network effects on the performance of a micropile system under static and seismic loading conditions.

This paper presents recommendations for preliminary estimation of the ground and network effects on the ultimate axial and lateral capacity of micropiles, as outlined by different authors. Design guidelines, which have been developed generally through observations on full-scale testing and general field experiences, are discussed with regard to cohesionless soils, cohesive soils and rocks. Preliminary conclusions are presented along with proposed design guidelines for micropile groups. Available analytical approaches are evaluated through comparisons with experimental data obtained by different investigators on the engineering behaviour of micropile groups and networks under different loading conditions.

Fundamentals of single micropile design—Case 1

Introduction

The basic philosophy of micropile design differs little from that required for any other type of pile: the system must be capable of sustaining the anticipated loading requirements within acceptable movement limits, and in such a fashion that the elements of that system are operating at safe stress levels. In detail, attention must be paid analytically to movement, bursting, buckling, cracking and interface considerations, whereas from a practical viewpoint, corrosion protection and compatibility with the existing ground and structure (during construction) must be considered. The system must, of course, also be economically viable.

Whereas the design of a conventional pile is normally governed by the external (i.e. ground-related) carrying capacity, micropile design is frequently controlled by the internal design (i.e. the selection of the pile components). This reflects both the relatively small cross-section available and the high grout/ground bond capacities that can usually be mobilized, as a consequence of the micropile installation methods. This emphasizes the point that micropiles are usually designed to transfer load to the ground through skin friction as opposed to end bearing: a pile 200 mm in diameter with a 5 m long bond zone has a peripheral area 100 times greater than the cross-sectional area. This mode of load transfer directly impacts performance, in that the pile movements needed to mobilize lateral frictional resistance are of the order of 20 to 40 times *less* than those needed to mobilize end bearing.

Occasionally, in the United States, micropiles are designed as simple struts between the structure and a particularly resistant bedrock surface. In such cases, assuming the rock mass has sufficient 'punching' resistance, the internal pile design governs pile capacity.

While the application of micropiles is growing rapidly, the current state of practice for design is still primarily based upon the experience of and research performed on large-diameter drilled-shaft piles and ground anchors. In the United States, in the absence of specific design codes for Type A micropiles, their design commonly requires compliance with specifications which have been established for large-diameter drilled shafts (e.g. AASHTO, 1992; Caltrans, 1994). However, it should be noted that such design practice, specifically with regard to the design of micropiles, requires a careful evaluation of the scale effect and the construction effect on the grout/ground interface parameters. Load testing of micropiles has demonstrated that the use of design codes for large-diameter drilled shafts generally results in a conservative design.

It is also commonly assumed that the load transfer mechanisms for pressure-grouted (Type B) and postgrouted (Type C and D) micropiles are similar to those governing the performance of ground anchors. For example, the British Standard BS 8081 (British Standards Institution, 1989), referring to the work of Littlejohn and Bruce (1977), and the French code (CCTG, 1993), following the field correlations developed by Bustamante and Doix (1985), would apply to micropiles as well as ground anchors.

In the United States, design codes relating to micropile performance under lateral loads have not yet been established. Current design practice commonly requires lateral loading tests following existing codes for drilled shafts (e.g. UBC, 1994; BCNYC, 1991; AASHTO, 1992). For preliminary design, guidelines derived from experience and from research performed by different investigators on laterally loaded piles (Matlock, 1970; Reese *et al.*, 1994) which have been incorporated in pile design codes (e.g. American Petroleum Institute, 1988; Caltrans, 1994; CCTG, 1993) can be considered.

With respect to axial, lateral or combined loading, the design of micropiles consists of two basic aspects:

(*a*) the geotechnical (or external) evaluation of the capacity of the micropile, which requires appropriate determination of the grout/ground interface parameters and the initial state of stress in the ground after micropile installation

(*b*) evaluation of the structural (or internal) resistance of the (composite) micropile section, which is governed by its area, and the strength of the reinforcement provided.

External design considerations describing current methods for estimating the ultimate axial and lateral capacity are summarized in this paper. Internal design considerations, which are generally described with regard to the selection of materials (grout and reinforcing steel), corrosion protection, and resistance to buckling and bursting, will not be addressed in the present paper.

Evaluation of the ultimate load-bearing capacity

In the US, pile design practice is still mainly based on correlations between the unit skin friction $f_s(F/L^2)$ and engineering properties of soils established with commonly used laboratory tests (such as the α and β methods) or standard penetration tests (SPTs). More recently, other *in situ* test techniques such as cone penetration tests (CPTs) and pressuremeter tests (PTs) have been increasingly used, and relevant correlations between f_s values and these test results incorporated in engineering manuals both in the United States (e.g. AASHTO, 1992; FHWA, 1994) and abroad (e.g. CCTG, 1993; British Standards Institution, 1989). Therefore, such empirical correlations are primarily used for preliminary design purposes, while design specifications for production micropiles commonly require site-specific loading tests.

With regard to lateral loading, the main mechanism of soil–pile interaction is the passive soil resistance developed against the pile. In the case of flexible small-diameter micropiles, the relative soil–micropile movement required to mobilize the ultimate lateral earth pressure is sufficiently large, in relation to the diameter of the micropile, to allow for its bending resistance to be mobilized. The lateral capacity of such micropiles is, therefore, primarily dependent on their yield moment. Owing to the slenderness of micropiles and their small cross-sectional area, the calculated lateral loading capacity is usually so small compared with their axial loading capacity that specific measures such as reinforcing the upper section or inclining the micropiles may be necessary.

Table 1 presents a summary of geotechnical design guidelines according to available codes currently used for micropile design. It lists both the design codes that have been adapted for small-diameter drilled shafts and pressure-grouted micropiles and the available codes for large-diameter drilled shafts and ground anchors that are commonly accepted in the absence of specific micropile design codes. For preliminary design purposes, charts have been developed (CCTG, 1993; Bustamante and Doix, 1985; DIN, 1978) providing grout/ground interface parameters as a function of the assumptions made with regard to the soil and the type of micropile to be used. For production micropiles, load tests are commonly conducted. The results of preproduction tests enable the back calculation of the grout/ground interface parameters and therefore the verification and updating of the preliminary design before the production piles are installed. Axial and lateral loading tests and related interpretation methods are discussed by Bruce and Juran (1997) and will be summarized in part 3 of this series.

Table 2 summarizes the available recommendations outlined by different authors for the preliminary estimate of the axial loading capacity of different types of micropiles in cohesionless soils, cohesive soils and rock. However, as emphasized above, the axial loading capacity depends on a variety of factors, which cannot be adequately represented in the proposed simplified empirical correlations. Therefore, while such recommendations can be useful for preliminary design, loading tests are commonly required on every site before the production piles are installed.

In evaluating the empirical design recommendations, it is of interest to compare the design values specified by the available design codes for pressure-grouted micropiles (or ground anchors) with experimental values specified by various investigators and contractors. Table 3 presents a comparison between design values for the ultimate frictional capacity given by Cheney (1984) and Lizzi (1985) for pressure-grouted anchors and Type B micropiles in different types of soil and the range of test results obtained for root piles (i.e. Type A) and for Nicholson Pin-piles (i.e. Type B) (Bruce, 1989, 1992a, b). The values given by Cheney (1984) for clayey silts and stiff to hard clays are significantly smaller than the corresponding experimental values, although it should be noticed that those values were obtained for augered anchors.

Table 4 presents a comparison between design values given by the French CCTG (1993) code for high-pressure postgrouted ground anchors and micropiles (Type C and D), test results given by Soletanche, and the empirical correlations developed by Jorge (1969) and Ostermayer and Sheele (1977). These comparisons indicate that, in spite of the difference in micropile construction techniques used by different contractors and the differences in soil profiles where different load tests were conducted to develop databases for empirical correlations, the ranges of ultimate frictional-capacity values proposed by the various authors for different types of soils agree fairly well and could be used for the preliminary estimate of the axial loading capacity.

It is of interest to compare these empirical correlations with experimental data from micropile field testing. The Internet-based International Knowledge Database for Ground Improvement Technology (IKDGIT) recently developed (Levy *et al.*, 1999) by the Technical Committee TC-17, Ground Improvement, Reinforcing and Grouting, of the International Society of Soil Mechanics and Geotechnical Engineering can be effectively used for this purpose.

*Table 1. Geotechnical design guidelines for single micropiles**

Loading	Axial						Lateral						Seismic
Purpose	Ultimate load			Movement control			Ultimate load			Deflection control			
Pile type	A	B	C, D	A	B	C, D	A	B	C, D	A	B	C, D	
USA (Nicholson)	N/Ap (in rock)	LT[1]		N/Ap	LT		N/Ap	LT		N/Ap	LT 'p–y' LPILE		N/Av
AASHTO (1992) (drilled shafts) (piles)	α method—cohesive TSA β method—granular ESA		N/Ap	Refer to ES, FE	N/Ap	N/Ap	Refer to AS	N/Ap	N/Ap	Refer to ES, FE 'p–y' LPILE	N/Ap	N/Ap	N/Av
MBC (1988) (drilled shafts) (small-diameter piles)	LT	N/Ap	N/Ap	LT	N/Ap	N/Ap	LT	N/Ap	N/Ap	LT	N/Ap	N/Ap	N/Av
American Petroleum Institute (1988) (Drilled-shaft piles)	α method—cohesive TSA β method—granular ESA		N/Ap	N/Av	N/Av	N/Av	SEM	N/Ap	N/Ap	'p–y'	N/Ap	N/Ap	N/Av
Caltrans (1994) (drilled shafts)	α method, cohesive β method, granular LT	N/Ap	N/Ap	't–z' (Reese and O'Neill, 1987)	N/Ap	N/Ap	SEM	N/Ap	N/Ap	'p–y', different soils	N/Ap	N/Ap	ARS curves FE codes (GT Strudl or ADINA)
Post Tensioning Institute (1986) (ground anchors)	LT DC	LT DC	LT DC	LT	LT	LT	N/Ap	N/Ap	N/Ap	N/Ap	N/Ap	N/Ap	N/Av
Germany (DIN 4128) (small-diameter injection piles)	N/Ap	RV (Ostermeyer and Sheele, 1977) LT		N/Av	N/Av	N/Av	N/Av	N/Av	N/Av	N/Av	N/Av	N/Av	N/Av
France (DTU) (CCTG, 1993) (micropiles)	DC (pressuremeter, SPT)			't–z'			'p–y' Pressuremeter			'p–y' Pressuremeter			N/Av
UK (British Standards Institution, 1989)	α method β method SPT	[‡]RV (Ostermeyer and Scheele, 1978)		LT	LT	LT	N/Ap	N/Ap	N/Ap	N/Ap	N/Ap	N/Ap	N/Av
Italy		(Lizzi, 1985)											N/Av

* N/Ap, not applicable; N/Av not available; FE, finite element; ES, elastic solutions; SEM, semi-empirical model for ultimate load values used in 'p–y' curves; TSA, total stress analysis; ESA, effective stress analysis; RV, recommended values specific to micropiles; LT, load testing; DC, design charts; AS, analytical solutions; A, tremie-grouted micropiles; B, low-pressure-grouted micropiles; C, postgrouted (French IGU); D, postgrouted (French IRS).

[1] $f_s = p_g \tan\phi'$, where p_g is the grout pressure and ϕ' is the effective angle of resistance.

[‡] $Q_s = L_b n \tan\phi'$, where Q_s is the ultimate axial load-holding capacity (in kN), L_b is the bond length or fixed anchor length (in m) and n is the factor that takes into account the drilling technique, depth of overburden, pile diameter, grouting pressure in the range 0·03 to 1 MPa, *in situ* stresses and dilation characteristics.

Table 2. Summary of available recommendations for preliminary design of micropiles

Soil type	Micropile type Type A Tremie-grouted	 Type B Pressure-grouted	 Type C, D Post-grouted
Cohensionless	β method $f_s = \beta\sigma'_{vz}$ $\beta = K\tan\phi'$ $K = K_0 = (1 - \sin\phi')\mathrm{OCR}^{\sin\phi'}$ $K = 0{\cdot}7$	$f_s = p_g \tan\phi'$ $f_s = \beta\sigma'_{vz}$ $\beta = K_1 K_2 \tan\phi'$ $K_1 = 1{\cdot}4$ to $1{\cdot}7$ $K_2 = 1{\cdot}2 \text{ to } 4 \longrightarrow \begin{cases} 1{\cdot}2\text{–}1{\cdot}5 \text{ (DS)} \\ 1{\cdot}5\text{–}2{\cdot}0 \text{ (MS)} \\ 3\text{–}4 \text{ (G)} \end{cases}$ $K = 4$ to 7 (Turner, 1995)	Ostermayer and Scheele (1978) CCTG (1993)
Cohesive	α method $f_s = \alpha s_u$ $\alpha = 0{\cdot}6$ to $0{\cdot}8$ (Bruce, 1994)	Similar to type A	Ostermayer (1974), with and without postgrouting CCTG (1993)
Rocks	$f_s = \dfrac{\mathrm{UCS}}{10}$ $f_s = 0{\cdot}007\,\mathrm{N} + 0{\cdot}12$ (MPa) (weathered rocks) $f_s = 0{\cdot}01\,\mathrm{N}$ (MPa) (stiff to hard chalk) Published design values (Barley, 1988; Turner, 1980; Littlejohn and Bruce, 1977)	Similar to type A	Not applicable

* DS, dense sand; MD, medium sand; G, gravel.

Table 3. Ultimate frictional-capacity design values according to Cheney (1984), Lizzi (1982) and Nicholson (1989–1992)

Soil type	Ultimate frictional capacity: kN/m Cheney (1984) Ground anchors Type B	 Lizzi (1985) Type B	 Fondedile (1993) Root piles Type B	 Nicholson (1989–1992) Pin piles (Type B)
Soft soil		16·5		72–120
Clayey silt	22*			150
Stiff to hard clay	30–60*		112·5	123–225
Loose soil		13·5–60		
Silty sand	75–135			300
Soil of average compactness		78–105		225
Sand	105–285		135	630–375
Very stiff soil		132		
Dense sand and gravel	150–300			450

* Values obtained for augered anchors.

Table 4. Ultimate frictional capacity given by the French code CCTG (1993), Soletanche (1992), Jorge (1984) and Ostermayer (1977)

Soil type		Ultimate frictional capacity: kN/m (assumed diameter 200 mm) CCTG (Type C)	 CCTG (Type D)	Ultimate frictional capacity: kN/m Soletanche	 Cheney (1984)	 Ostermayer (diameter 10–15 cm)
	A	<19	—			
Clays, silts	B	44–48	61–88	32–66		
	C	>50	109·5			
	A	<19	<19		51–66	40–70
Sand, gravel	B	47–67	56–101		66–124	70–140
	C	>73	>131		124–168	140–190
Marls	A	88–146	111–161	50–61	73–146	
	B	>161	>175		>146	

Figure 2 displays the data obtained from the IKDGIT for the empirical correlations between the testing/design load transfer rate and the grout pressure for sandy soils. The scatter in the experimental data is due to the range of diameters varying from 12 to 22 cm. The figure illustrates that the IKDGIT data for design loads are lower than the values of ultimate load transfer rate recommended by Jorge (1969) for pressure-grouted anchors and recently adopted by Bruce and Juran (1997) for the design of micropiles.

Figure 3 shows the data obtained for the IKDGIT for the

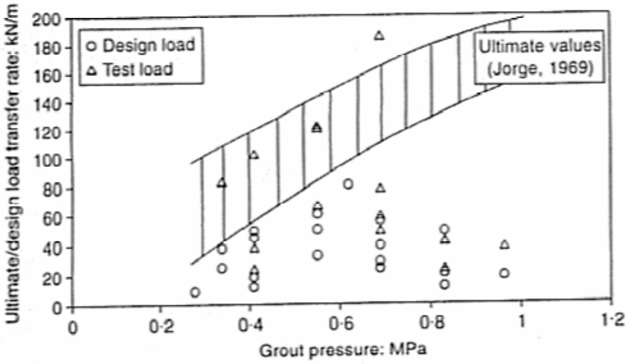


Fig. 2. Load transfer rate versus grout pressure: (a) values from the IKDGIT: (b) Jorge (1969)

empirical correlations between the design load and the bond length in sandy soils. The IKDGIT data are consistent with the range of experimental values proposed by Ostermayer and Sheele (1978) for the design of ground anchors and recently adopted by Bruce and Juran (1997) for the design of micropiles.

Movement estimation

Introduction

Micropile movement under applied loading results from two basic movement components:

(*a*) the compression or elongation of the micropile, which is controlled by its composite elastic modulus and cross-sectional area

(*b*) the relative soil–pile interface shear displacement, which is controlled by the interface properties, the initial state of stress in the ground and the changes that occur with pile installation and time.

Owing to the difficulties involved in simulating soil–micropile interaction during loading, micropile loading tests are commonly required to estimate the movement prior to the installation of production micropiles. However, both elastic solutions and '*t–z*' load transfer models have been used in the design of friction piles.

The elastic solutions derived from the so-called Mindlin's equations were developed by different investigators (Poulos and Mattes, 1969a, b; Poulos and Davis, 1980). These solutions yield the vertical movement at any given point in a semi-infinite elastic and isotropic soil due to a downward force in the interior of the soil. The drawback of the elasticity methods lies in the basic assumptions that must be made. The actual ground conditions rarely satisfy the assumption of uniform and isotropic material. In spite of the highly non-linear stress–strain characteristics of soils, the only soil properties considered in the elasticity methods are based on the Young's modulus E and the Poisson's ratio ν, which clearly leads to an oversimplification. In actual field conditions, the parameter ν may be relatively constant, but the parameter E can vary through several orders of magnitude. Therefore, the practical use of elastic methods in micropile engineering practice is rather limited.

The use of the '*t–z*' method, in spite of the difficulties involved in selecting the appropriate interface parameters, does provide practical analytical tools for a preliminary estimate of micropile movement under the anticipated loading.

Several approaches have been developed or proposed to analyse the load–movement relationship of micropiles for short- and long-term movement estimates. These approaches can be classified within the following four broad categories: (i) analytical load transfer models ('*t–z*' or more complex interface models) commonly used in ground anchor design, (ii) the 'partially bonded' design concept assuming the micropile movement to correspond to its elastic shortening (or elongation) in the unbonded zone in the soft/weak soil, (iii) site-specific pile-loading tests and relevant interpretation methods, and (iv) long-term performance testing. In the following, the partially bonded design concept and the *t–z* method are presented.

'Partially bonded' micropile design concept

The movement response z_0 of a micropile transferring the applied compressive (or tensile) load Q_0 to a competent bearing stratum is assumed to correspond to the elastic shortening Δ_e (or extension) of its portion in the unbonded weak/soft soil layers overlying the competent stratum. Therefore

$$z_0 = \Delta_e = \frac{Q_0 L}{E_p A_p} \quad (1)$$

where Δ_e refers to the elastic shortening.

Bruce *et al.* (1993) have used testing procedures and interpretation methods commonly applied in ground anchor practice to analyse and predict the engineering performance of high-capacity micropiles. In particular, extensive field tests have been conducted in a variety of soil types, with

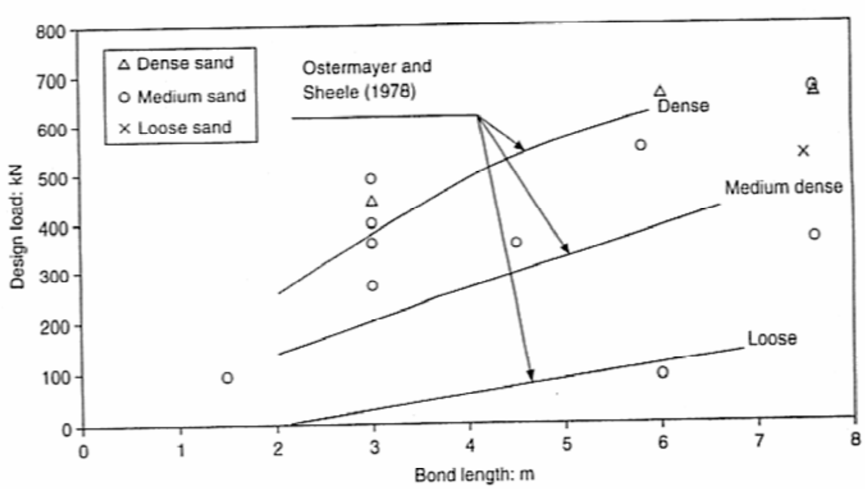


Fig. 3. Design load versus bond length (values from the IKDGIT)

cyclic loading to failure to investigate the progressive interface debonding phenomenon in micropiles.

Figure 4 illustrates the load–elastic-compression (Δ_e) relationship and the load–permanent-compression (Δ_p) relationship of the micropile obtained from these tests. The micropile transfers the applied load to the surrounding bearing stratum through the interface shear resistance along the pressure-grouted bonded zone, and it is commonly assumed that no load transfer is mobilized along the casing in the soft soil. Accordingly, for test interpretation the pile is assumed to be a free column of length L (in the cased 'debonded' zone) and its elastic compression is therefore defined by Equation (2) below. The elastic ratio of the micropile is defined as the following ratio:

$$ER = \frac{\Delta_e}{Q_0} = \frac{1}{E_p A_p} L \qquad (2)$$

Figure 5 shows the variation of ER with increasing load Q obtained from load tests on high-capacity micropiles incorporating 177 mm casing to a depth of approximately 21·33 m from grade, including a minimum of 9·1 m into a very dense gravelly and cobbly bed. The upper portion was reinforced with the casing and the lower-pressure-grouted portion was reinforced by a central reinforcing bar. The variation of ER with Q indicates a progressive debonding down the micropile, which results in an apparent increase of the 'debonded' length L. If debonding were not occurring, ER would be constant, corresponding to the length of the cased 'debonded' portion of the micropile.

The results of the field tests have demonstrated, through comparisons of the recorded and calculated ER values, that significant load transfer occurs in the upper strata of relatively soft soil. This is usually neglected at the design phase. The ER approach to field analysis of micropile testing offers an analytical and predictive tool, especially when combined with creep data. When the extent of apparent casing debonding reaches within a metre of the end of the casing, explosive failure may be expected shortly. At such

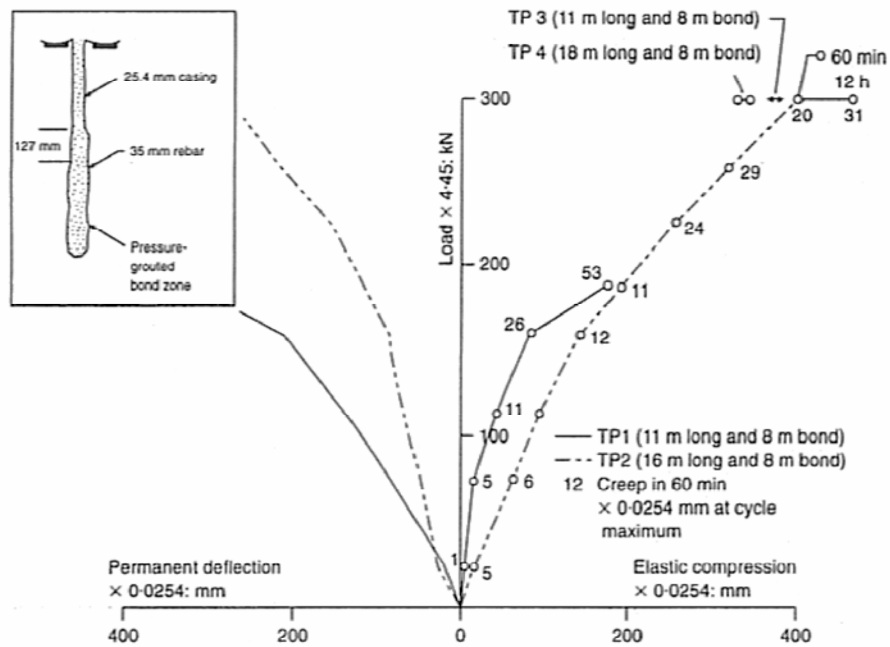


Fig. 4. Elastic/permanent movement performance of test piles TP1 and TP2, Postal Square, Washington, DC (after Bruce, 1992a)

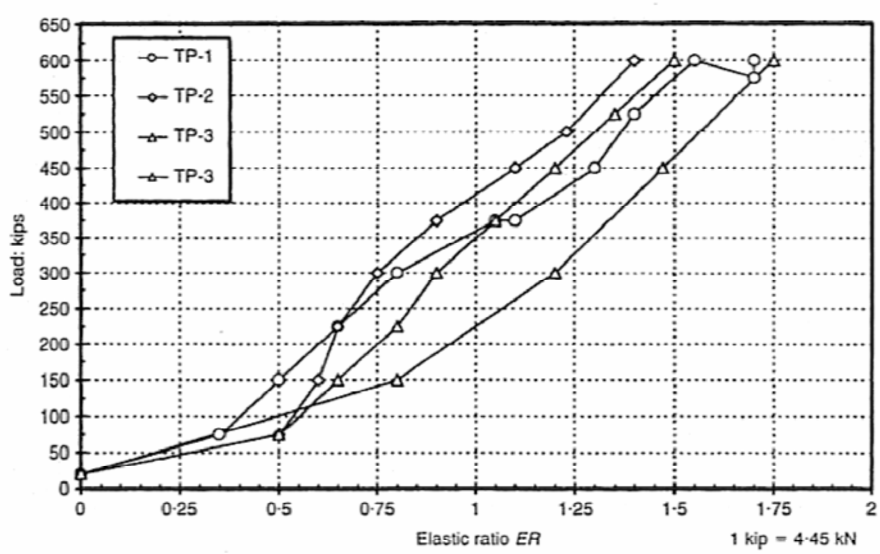


Fig. 5. Elastic ratio comparison, piles TP-1, 2, 3, United Grain Terminal, Vancouver, WA (after Bruce et al., 1993)

times, the creep monitored may be more a result of grout/steel interfacial phenomena rather than grout/soil phenomena, as conventionally assumed. Furthermore, as a result of progressive debonding, less of the casing becomes capable of resisting the load and a higher proportion of the load must be resisted in the bonded zone. This bond zone has a finite capacity (internal and external), and will fail as this capacity is exceeded. The monitored *ER* values that can be directly determined from the experimental Q versus Δ_e curves provide a useful index to assess the extent of debonding under any loading level.

It is worth noting that this interpretation method assumes that no interfacial residual shear stress is mobilized in the upper stratum of soft soils. Furthermore, as the debonding process propagates downwards along the pile, the residual interface shear stress along the debonded zone is assumed implicitly to be zero and, therefore, this 'partially bonded' design concept generally results in an overconservative design and leads to overestimating the movement due to the applied loading. In fact, Lizzi (1982) and Kenny *et al.* (1993) demonstrated that, as micropiles are generally fully bonded, the load transfer in soft/weak soils can significantly reduce the movements under the applied loading.

In practice, similarly to ground anchors, when the loading is directly applied on the micropiles, preloading can be used to reduce postconstruction displacements. Analysis of the axial loading capacity will therefore primarily focus on the load transfer to and within the component bearing stratum and the mechanics of the bond that propagates along the pressure-grouted bonded zone of the micropile in this bearing stratum.

t–z method

Analysis of load test results on instrumented pressure-grouted micropiles or ground anchors (Bustamante and Doix, 1985) has demonstrated that representive '*t–z*' interface curves can be adequately derived from measured load variations with depth along the micropile. The major advantage of such instrumented loading tests is that they provide characteristic shear stress/movement '*t–z*' curves at different depths, which are representative and integrate the effect of all the parameters governing the interface behaviour, including construction techniques, the soil profile and the *in situ* state of stress. As these curves are experimentally derived, they represent actual soil conditions and construction effects on the interface behavior. The applicability of this approach for both micropiles and ground anchors has been recognized and relevant engineering guidelines have been incorporated in design codes (e.g. Reese *et al.*, 1994; CCTG, 1993).

Figure 6(a) shows the results of a pull-out test on an instrumented postgrouted anchor in a plastic clay (Bustamante, 1980). The slope of the tension force distribution along the anchor corresponds to the skin friction mobilized at a specific depth under the applied pull-out force. As shown in Fig. 6(a), the shear stress–upward anchor movement curves obtained for different depths indicate overconsolidation of the subsurface soil layer and illustrate that the anchor movement required fully to mobilize the shear stress is about 5 to 10 mm. The slope of the compression force distribution along the micropile yields, for different depths, interface shear stress–upward shear movement characteristic curves that can be directly used in '*t–z*' models for movement estimates. The variation of skin friction along the micropile (or ground anchor) during compression (or tension) loading is mainly the result of its compressibility (or extensibility) during the loading test. It is primarily dependent upon the relative rigidity (or elastic modulus ratio) of the micropile (or ground anchor), the grout/soil interface characteristics and the soil properties, particularly its density and overconsolidation ratio.

With the acquisition of load transfer '*t–z*' curves from

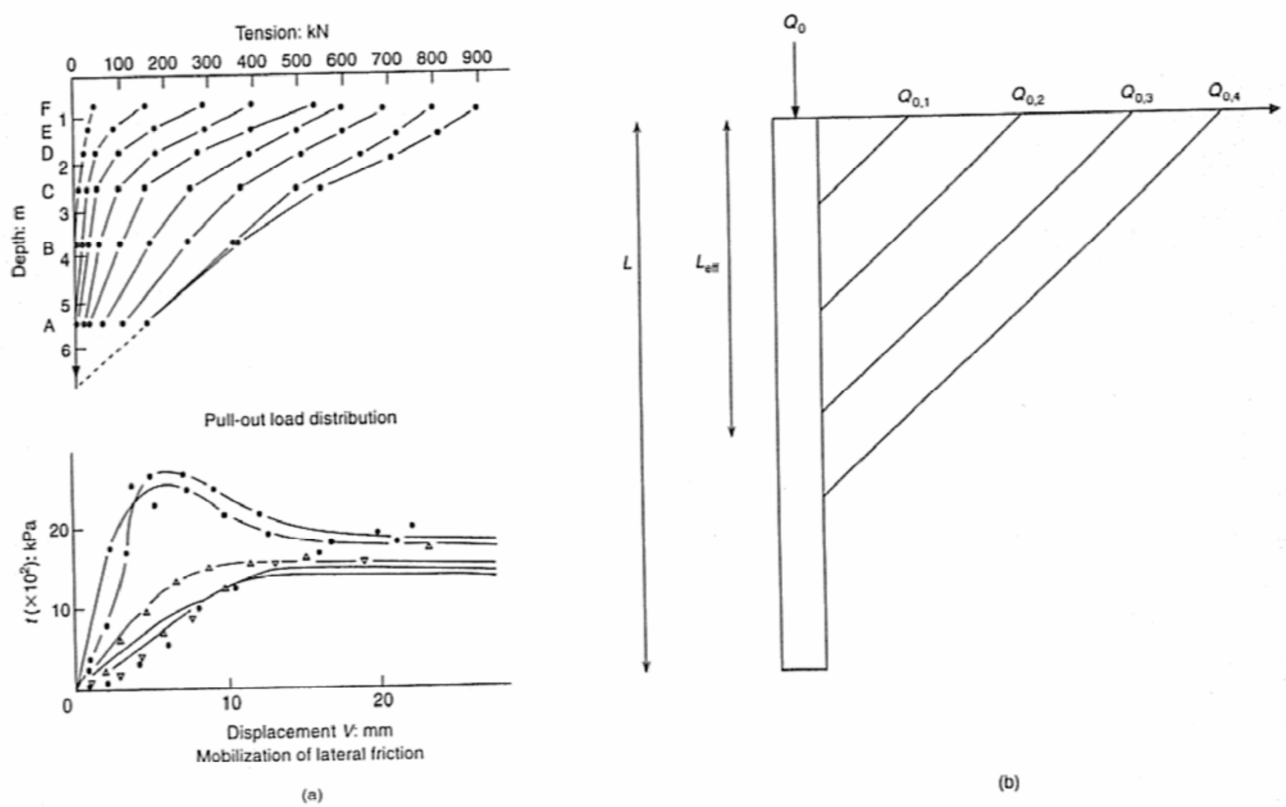


Fig. 6. (a) Mobilization of the lateral friction along an anchor in plastic clay (Bustamante, 1980); (b) simplified load transfer model

loading tests on instrumented micropiles, the problem of the load distribution along the micropile and the determination of the downward movement at any depth can be solved using finite-difference techniques with available computer codes (e.g. LPILE, GROUP). For this purpose, several simplified assumptions have been proposed to establish analytical approximations of experimentally derived 't–z' curves. Assuming a linear 't–z' curve, Juran and Christopher (1989) derived an analytical expression for the relationship between the applied compression force Q_0 and the movement Z_0 of the top of the pile:

$$Z_0 = \frac{1}{2} Z_c \left[1 + \left(\frac{\lambda}{Z_c}\right)^2 \left(\frac{Q_0}{E_p A_p}\right)^2 \right] \tag{3}$$

where Z_0 is the top movement and Z_c is the critical movement for which the ultimate interface shear resistance t_{max} is fully mobilized, i.e.

$$Z_c = \frac{t_{max}}{k} \tag{4}$$

where k is the interface shear modulus, E_p is the pile modulus of elasticity, A_p is the pile cross-section and λ is a characteristic 'axial transfer length' defined as

$$\lambda = \sqrt{\frac{E_p A_p}{kp}} \tag{5}$$

where p is the perimeter of the pile.

Figure 6(b) presents a simplified model assuming that the ultimate interface shear resistance t_{max} is fully mobilized along an effective length of the pile, denoted L_{eff}, which is determined by the equilibrium equation of the loaded pile subjected to the applied load Q_0. This assumption implies a perfectly plastic interface model ($Z_c = 0$), resulting in load distribution curves which are parallel with a slope proportional to t_{max}.

Considering the simplified load transfer model, the equilibrium of the pile subjected to the load Q_0 can be written as

$$Q_0 = L_{eff} t_{max} \pi D \tag{6}$$

Substituting Equations (4)–(6) into Equation (3) yields, for the simplified load transfer model, the following analytical expression:

$$Z_0 = \frac{1}{2}\frac{t_{max}}{k} + \frac{1}{2}\frac{L_{eff}}{E_p A_p} Q_0 \tag{7}$$

or

$$\frac{Z_0}{D} = \frac{1}{2}\frac{Z_c}{D} + \frac{L_{eff}}{2L}\left(\frac{L}{D}\right)\frac{Q_0}{E_p A_p} \tag{8}$$

where L is the total micropile length.

It is of interest to assess this analytical load–settlement relationship through comparison with available experimental data from micropile site testing. The micropile testing data available in the IKDGIT (Levy *et al.*, 1999) yield information regarding the geometry and mechanical properties of the piles, as well as the measured settlement values Z_0 for the applied loads Q_0. It is therefore of interest to assess the non-dimensional load–settlement relationship expressed by Equation (8) through comparisons with empirical correlations derived from the available data. For this purpose, Equation (8) can be written as

$$\frac{Z}{D} = A + B\left(\frac{L}{D}\right)\frac{Q_0}{E_p A_p} \tag{9}$$

The load transfer model parameters to be experimentally determined are

$$A = \frac{1}{2}\frac{t_{max}}{kD} = \frac{1}{2}\frac{Z_c}{D} \tag{10}$$

corresponding to the interface shear characteristics k and t_{max}, and

$$B = \frac{L_{eff}}{2L} \tag{11}$$

corresponding to an apparent safety factor with respect to the frictional load-bearing component of the pile, expressed in terms of the ratio of the total pile length L to the effective length of the pile L_{eff} along which the ultimate interface shear stress t_{max} is fully mobilized under the applied load Q_0.

Figure 7 shows the empirical relationships derived from the IKDGIT data between the normalized settlement (Z/D) and the normalized load ($(L/D)(Q_0/(E_p A_p))$). The data are presented with linear regressions for the different soil types to allow for the experimental determination of the values of A and B.

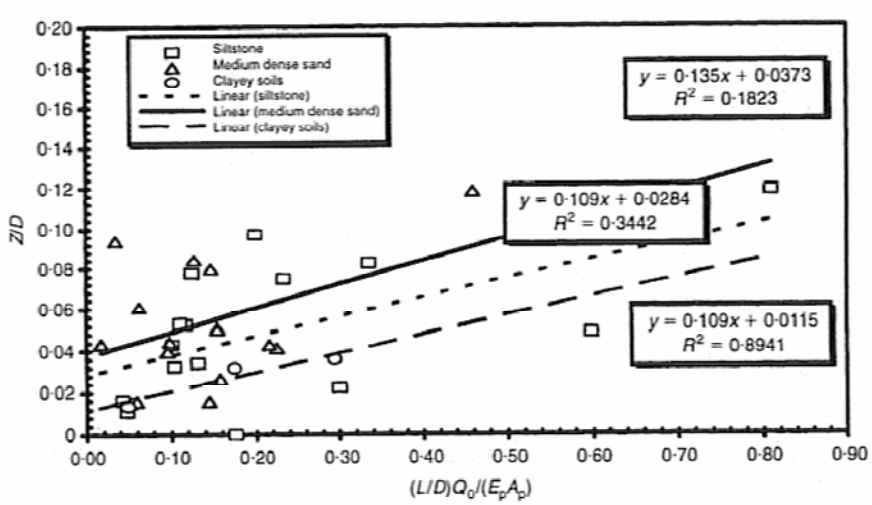


Fig. 7. Normalized settlement Z_0/D versus normalized load $(L/D)Q_0/(E_p A_p)$

The value of A is determined from

$$A = \left(\frac{Z_c}{D}\right)_{Q=0}$$

The value of B is determined by the slope of the linear regression line.

For a typical micropile diameter of $D = 200$ mm, the experimentally derived A values correspond to Z_c values within a range of 4 to 8 mm, which, as illustrated in Fig. 6(a), are consistent with the experimental data reported by Bustamante *et al.* (1989). The experimentally derived B values correspond to apparent safety factor values ranging from 3 to 5, which are commonly used in practice. It should, however, be emphasized that the low values of the coefficient of correlation clearly indicate the great variety of cases under consideration, limiting the predictive conclusions of such statistical analysis to qualitative observations on the current state of practice.

Analysis of the engineering behaviour of micropile groups and networks

Introduction

The design of micropile systems, particularly for underpinning applications, usually dictates the need for groups of closely spaced piles. With conventional piles, there is usually a compromise to be resolved between the desire to select a close micropile spacing, thus minimizing the size and cost of the pile cap and, on the other hand, the need to maintain a certain minimum interpile spacing so as to avoid the 'group effect' necessitating a reduction in the nominal capacity of each pile. Depending on pile spacing the load capacity of a group of piles can be significantly smaller and its movement larger than the loading capacity and movement of a single pile under the same average load per pile in the group. To account for the group effect on the loading capacity and pile movement, different design codes (e.g. AASHTO, 1992; CCTG, 1993; BOCA, 1990) specify a minimum spacing between piles and/or relevant reduction factors (e.g. Naval Facilities Engineering Command, 1982; Canadian Geotechnical Society, 1992; CDF, 1984). Ultimately, when piles are closely spaced, interaction between these piles has to be considered.

In view of the difficulties in evaluating the group and network effects for different types of micropiles, soils and site conditions and in the absence of sufficient field data, no specifications have yet been established to take into account group and network effects; hence, these are commonly neglected in micropile design practice.

In this section, the available analysis approaches are presented and evaluated through comparisons with experimental data obtained by different investigators on the engineering behaviour of micropile groups and micropile networks under different loading conditions. Preliminary conclusions are presented along with proposed design guidelines for micropile groups.

Estimate of ultimate axial loading capacity

Experimental results of laboratory and full-scale experiments reported by various investigators (Lizzi, 1978; Plumelle, 1984; Maleki, 1995) indicate significantly different and apparently contradictory group effect paradigms in micropile systems. Lizzi (1978), through the results of laboratory loading tests on micropile models, has demonstrated the 'knot effect' whereby a 'positive' group effect is achieved under axial loading of the soil–pile system.

Plumelle (1984), through full-scale loading tests on isolated, instrumented (Type A) gravity-grouted micropiles and groups of them, has demonstrated that a negative group effect will develop in a micropile group, with the movement of the micropile group being greater than the movement of a single pile under axial loading equivalent to the axial loading per micropile in the group. Maleki (1995) reported apparently contradictory observations. He analysed results of full-scale pull-out loading tests on isolated, instrumented (Type A) gravity-grouted micropiles and groups of them embedded in chalk and illustrated that, in this case, a positive 'group effect' could develop, reducing the movement of the micropile group as compared with that of a single micropile under the same load as the average load per pile in the group.

Figure 8 shows that the group efficiency values obtained by Lizzi (1978) are in good agreement with those obtained by O'Neill (1983). The efficiency factor η_v is the ratio of the actual loading capacity of the group Q_{gu} to the sum of the loading capacities Q_{iu} of all the piles in the group:

$$\eta_v = Q_{gu} \Big/ \sum_{i=1}^{n} Q_{iu} \tag{12}$$

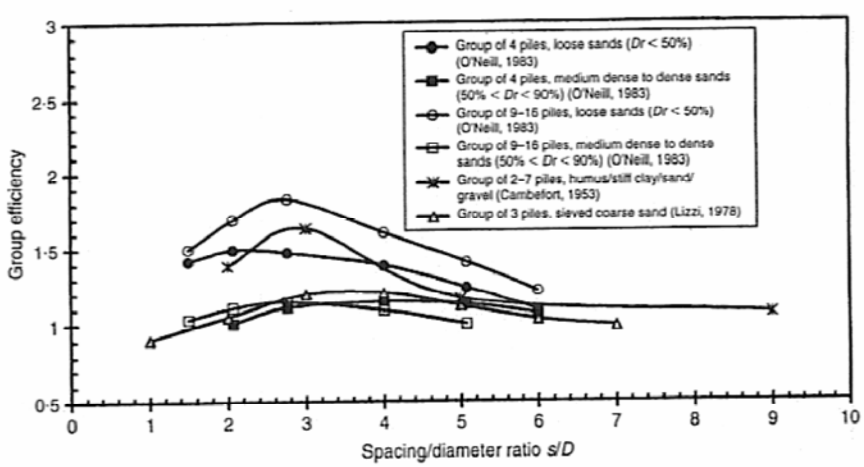


Fig. 8. Group efficiencies from tests of model pile groups in cohesionless soils subjected to vertical loads reported by O'Neill (1983), Lizzi (1978) and Cambefort (1953)

O'Neill (1983) suggested the following conclusions for driven piles.

- In loose cohesionless soils, the group efficiency factor η_v is always greater than 1 and reaches a peak at $s/D \approx 2$. It also seems to increase with the number of piles in the group.
- In dense cohesionless soils with $2 < s/D \approx 4$ (the normal range), η_v is usually slightly greater than 1 so long as the pile is installed without predrilling or jetting. However, either of these construction techniques can significantly reduce the group efficiency.

The results of full-scale loading tests in cohesionless soils (O'Neill, 1983) also suggest η_v values greater than 1, except when predrilling or jetting is used.

It is of particular interest to note that, as illustrated in Fig. 8, the pile-loading tests conducted by Cambefort (1953) on small-diameter driven micropile groups (5 mm in diameter and a slenderness ratio of 50) correspond fairly well to the results reported by Lizzi (1978) and O'Neill (1983).

Results of tests on some model piles, in groups of four and nine, were reported by Vesic (1969). Vesic measured the point load separately from the shaft resistance and, in the light of his measurements, he concluded that when the efficiency of closely spaced piles was greater than unity, this increase was in the shaft rather than the point resistance. The broad conclusion to be drawn from the above data is that, except in very dense sand or when the piles are widely spaced, the overall efficiency for driven piles is likely to be greater than 1. The maximum efficiency is reached at a spacing of 2 to 3 diameters and generally ranges between 1·3 and 2. It is anticipated that pressure-grouted micropiles will result in a similar group effect. The high values of the group efficiency factor η_v in cohesionless soils seem to be primarily due to the radial consolidation that occurs during driving and the resulting increase in lateral stress, which may also be induced by pressure grouting. Less consolidation occurs if predrilling or jetting is used, so η_v is lower for such groups and is likely to be less than 1 for bored or partially jetted piles (O'Neill, 1983).

For conventional piles, available design codes (AASHTO, 1992; CCTG, 1993) specify a minimum spacing between piles and/or relevant reduction factors (Naval Facilities Engineering Command, 1982; Canadian Geotechnical Society, 1992) for the determination of the pile group axial loading capacity. AASHTO (1992), following Terzaghi and Peck (1948), recommends the axial group capacity to be computed as the lesser of (i) the sum of the ultimate capacities of the individual piles in the group and (ii) the axial loading capacity of an equivalent composite pier circumscribing the group, for a block failure of the group, that is, for a rectangular block $B_g \times L_g$,

$$Q_{gu} = B_g L_g c N_c + 2(B_g + L_g) L c_{av} \tag{13}$$

Here Q_{gu} is the ultimate axial loading capacity of the pile group, c the undrained cohesion at the base of the group, L the pile length, N_c the bearing capacity factor corresponding to a depth L, and c_{av} the average cohesion between the surface and depth L.

At present, several design codes, such as the French code (CCTG, 1993) and the AASHTO (1992) Bridge Specifications, still suggest the use of the Converse–Labarre group efficiency equation for friction piles including (in the French code) micropiles in different types of soils. The Converse–Labarre formula assumes the piles to be vertical and identical and is limited to rectangular groups with identifiable values of the numbers of piles in columns n_c and rows n_r. The Converse–Labarre equation can be written as

$$\eta_v = 1 - \frac{\arctan(D/s)}{\pi/2}\left(2 - \frac{1}{n_c} - \frac{1}{n_r}\right) \tag{14}$$

It is noted that the Converse–Labarre formula relies only on assumed relationships between the pile group geometry and the group efficiency factor, with practically no relevant test data available for its justification. In particular, it does not allow for any considerations with regard to parameters such as installation technique effect, slenderness ratio and soil type. The comparison between experimental and predicted values of the group efficiency factor for driven piles in sand and, specifically, for the micropile tests conducted by Lizzi (1978) strongly suggests that the Converse–Labarre formula should not be used in micropile design practice.

In the absence of sufficient field data, the French CCTG (1993) recommendations, as indicated in Table 5, can be adapted for preliminary conservative assessment of the group efficiency factor in micropile systems.

Estimate of micropile group movement

Depending on pile spacing, the movement of a group of piles can be significantly larger than the movement of a single pile under the same average load per pile in the group. Owing to the group effect, a contiguous pile creates increased movement of its neighbors as compared with a single pile under an equal loading. Several approaches have been developed in order to predict the movement of a group of piles, including

(*a*) empirical correlations relating the movement of pile groups to the movement of a single pile (e.g. Skempton, 1953; Vesic, 1969; Meyerhof, 1976; Fleming *et al.*, 1985)
(*b*) continuum elastic methods based on the Mindlin (1936) equations (e.g. Poulos, 1968; Poulos and Davis, 1980; Poulos and Hewitt, 1986; Poulos, 1989; Yamashita *et al.*,

Table 5. Preliminary recommendations for group efficiency factor η_v values (adapted from the French code (CCTG, 1993))

	Cohesive	Cohesionless dense	Cohesionless loose and medium dense	Rock (component strata)
Type A	$\eta_v = 1$ $\quad s > 3D$ $\eta_v = \frac{1}{4}\left(1 + \frac{s}{D}\right)$ $\quad 1 \leq \frac{s}{D} \leq 3$ Check for block failure (Equation (9))	Converse–Labarre equation (Equation (6)) Check for block failure (Equation (5))	Converse–Labarre equation (Equation (6)) Check for block failure (Equation (9))	$\eta_v = 1$
Type B	Same as above	Same as above	$\eta_v = 1$	$\eta_v = 1$
Type C	Same as above	Same as above	$\eta_v = 1$	$\eta_v = 1$
Type D	Same as above	Same as above	$\eta_v = 1$	$\eta_v = 1$

1987; Banerjee, 1978; Butterfield and Banerjee, 1971; Banerjee and Davis, 1978; Randolph and Wroth, 1979)

(c) load transfer models and 'hybrid solutions' (O'Neill *et al.*, 1977; Maleki and Frank, 1994; Chow, 1986; Lee, 1993) combining characteristic load transfer 't–z' curves for each pile with continuum elastic solutions to assess interaction factors for estimating the group effect

(d) a pure shear interface model developed by Randolph and Wroth (1979) assuming that the vertical loading produces pure shear with negligible radial movement.

The group movement can be expressed by the group reduction factor R_g, defined as

$$R_g = \frac{\text{Average group movement}}{\text{Movement of single pile at same total load as the group}} \tag{15}$$

Figure 9 shows the comparisons between empirical correlations of the group settlement reduction factor and group breadth-to-diameter ratio proposed by Skempton (1953) and Fleming *et al.* (1985) and in continuum elastic analysis and the pure shear interface model for various pile groups.

According to Fleming *et al.* (1985),

$$R_g = n^{\alpha - 1} \tag{16}$$

where n is the number of piles. For practical cases, the value of the exponent α lies in the range 0·4 to 0·6. This simplified empirical relationship is recommended by the Hong Kong Department of Transportation for pile group design.

As shown by Benslimane *et al.* (1997) and illustrated in Fig. 9, for the typical case of a spacing-to-pile-diameter ratio of $s/D = 3$, with an exponent α within the range 0·6 to 0·7, the empirical correlations proposed by Fleming *et al.* (1985) yield R_g values which are consistent with those predicted by the Skempton formula and the elastic solution.

Randolph and Wroth's (1979) pure shear interface model has been used by Bruce and Juran (1997) and Benslimane *et al.* (1997) to evaluate the group effect, yielding for axially loaded micropile systems

$$R_g = \frac{R_s}{n} \quad \text{with } R_s = n\frac{\sum_{i=1}^{n-1} \ln(2r_i/D)}{\ln(2fL/D)} \tag{17}$$

where n is the number of piles in the group, L/D is the pile length-to-diameter ratio, r_i is the distance of each pile i from the pile under consideration and $f = 25\,\rho(1-\nu)$, where ρ is the ratio between the soil shear modulus $G_{L/2}$ at depth $z = L/2$ and the soil shear modulus G_L at depth $z = L$, and ν is Poisson's ratio.

As illustrated in Fig. 9, Equation (17) yields slightly more conservative values of the group settlement reduction R_g as compared with the empirical correlations and the elastic solutions.

Network effect

Plumelle (1984) investigated the inclination effect on the performance of driven micropile group systems. The results illustrated in Fig. 10 show that the inclination of the micropile leads to a network effect that may significantly increase the ultimate axial loading capacity and decrease the movement of the micropile group. However, a comparison of the test results obtained for the micropile groups and networks with the load–movement curve obtained for the reference micropile group ($Q_g = 16Q_s$, where Q_s is the load

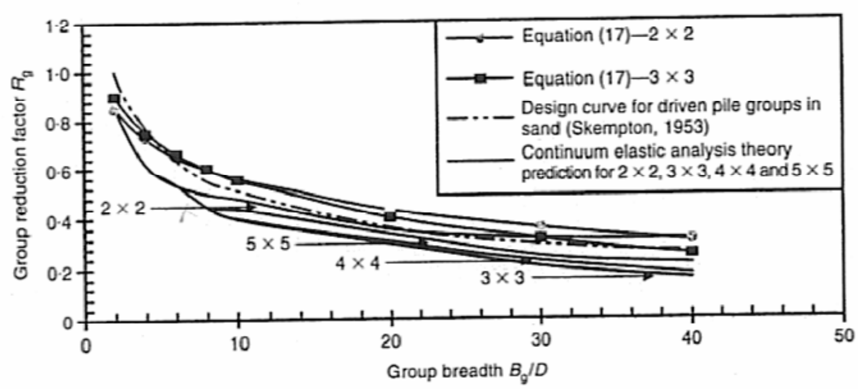


Fig. 9. Comparisons between empirical correlations of group settlement reduction factor versus group breadth-to-diameter ratio proposed by Skempton (1953), Fleming *et al.* (1985) (Equation (16)) pure shear interface model predictions (Equation (17)) and elastic continuum analysis predictions

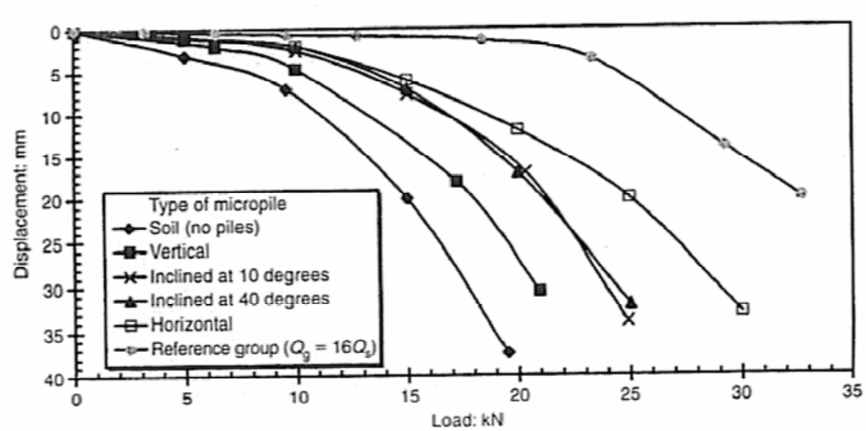


Fig. 10. Effect of micropile inclination on the load–movement curves of micropile group systems

applied to a single micropile) at the same movement as for the reference micropile group indicates that, apparently, a negative group effect develops because of the soil disturbance induced by the hammering of the pile into the soil. This apparent negative group effect results in a significant decrease of the loading capacity of the group and a significant increase of its movement, as compared with that of a single pile under a load identical to the average load per pile in the group.

It is of particular interest to note that for the test results reported by Plumelle ($n = 16$, $s/D = 8$, $L/D = 94$), and for a loading level of 50% of the ultimate loading capacity of the reference group, Equation (2) with an exponent value of 0·7 and Equation (3) yield approximately the same calculated value of $R_s = 7$, which agrees fairly well with the experimental results.

Figure 11 illustrates the comparison between the experimental values of the interaction factor α_I obtained for these tests and the numerical predictions obtained by Maleki and Frank (1994) with the GOUPEG hybrid model and the CESAR finite-element method. The normalized interaction factor $\hat{\alpha}_i$ for a group of n micropiles ($n = 16$ for Plumelle's test) is defined by

$$\hat{\alpha}_i = \frac{R_s - 1}{n - 1} \tag{18}$$

The experimental values of $\hat{\alpha}_i$ are obviously highly dependent upon the loading level. The $\hat{\alpha}_i$ values indicated in Fig. 11 were obtained for a loading level of $Q_{gu} = 16$ kN, corresponding approximately to 50% of the ultimate loading capacity of the reference group. While any quantitative comparison between the experimental results obtained by Plumelle for a group of 16 inclined, driven micropiles and the numerical predictions obtained for a two-inclined micropile group system is highly approximate, both the experimental results and the numerical simulations consistently illustrate that the inclination of the micropiles in the group can significantly minimize the group effect and, thereby, improve the movement response of the soil–micropile system.

Estimate of the ultimate lateral loading capacity

To account for the group effect on the lateral loading capacity and the pile deflections, different design codes (e.g. AASHTO, 1992; CCTG, 1993) specify a minimum spacing between piles and/or relevant reduction factors (e.g. Naval Facilities Engineering Command, 1982; Canadian Geotechnical Society, 1992; CDF, 1984). Ultimately, when piles are closely spaced, interaction between those piles has to be considered.

Group efficiency factors for side-by-side piles and line-by-line piles have been proposed by different investigators, as well as combined factors from side-by-side and line-by-line positions for skewed piles. As indicated by Reese *et al.* (1994), at present insufficient data are available to allow the group efficiency factors to be derived for individual soil types and the values specified below are to be used for any kind of soil.

On the basis of the experimental studies conducted by Prakash (1962), Cox *et al.* (1984) and Lieng (1988), for s/D values greater than 3, which are generally used in micropile design practice, the reduction is negligible.

It is of interest to note that for side-by-side piles, the French code (CCTG, 1993) specifies an efficiency factor of $\eta_h = 1$ independently of the pile spacing. While field data of lateral load tests on micropile groups are presently practically unavailable, Reese *et al.*'s (1994) recommendations are consistent with the French code, indicating that for the spacing-to-diameter ratios generally used in micropile design practice (i.e. $s/D > 3$), the group effect for side-by-side micropiles can be ignored for practical design purposes.

The interaction of piles in the direction of loading is more complicated than that of piles in a row. As indicated by Reese *et al.* (1994), many experiments have concluded that the interaction is not a simple function but depends greatly on the relative positions of the piles. Although experiments were conducted in different soil conditions, the influence of soil properties on group efficiency factors is not possible to quantify at present. Therefore, the group efficiency factors are based only on the relative positions of the piles in the group, and it is necessary to present separate recommendations for leading piles and trailing piles. Dunnavant and O'Neill formalized the data of Cox *et al.* (1984) and recommended reduction factors for leading piles and trailing piles as a function of pile spacing in the direction of loading. A similar approach to that of Dunnavant and O'Neill, based on available data, has been used by Reese *et al.* (1994) to define the group efficiency factor and is outlined below.

For the leading piles in a line, the results show that the load carried by the leading piles is only slightly smaller than for a single pile and the group effect becomes negligible for $s/D > 3$. The group efficiency factors for the trailing piles in a line may be determined by referring to the curve in Fig. 12(b). Referring to the group of three piles in Fig. 12(a), pile 1 is a leading pile, pile 2 is a trailing pile relative to pile 1, and pile 3 is a trailing pile relative to piles 1 and 2. The

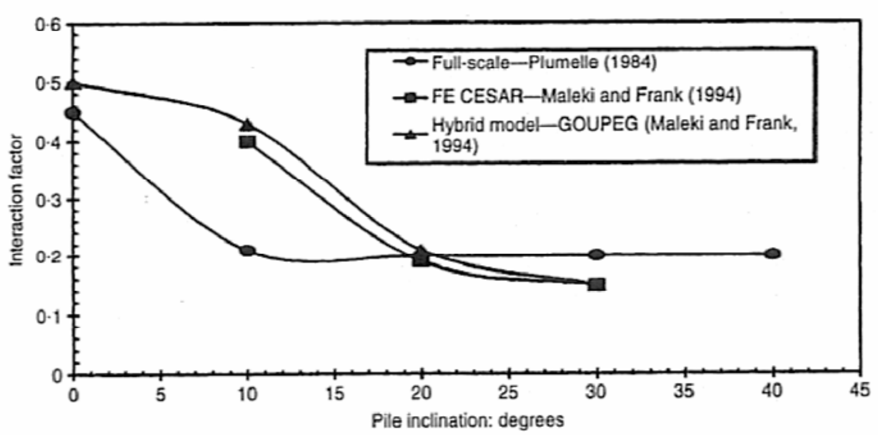


Fig. 11. Comparison between the experimental interaction factor values α_I obtained by Plumelle (1984) and the numerical predictions of the OUPEG hybrid model and the CESAR finite-element method

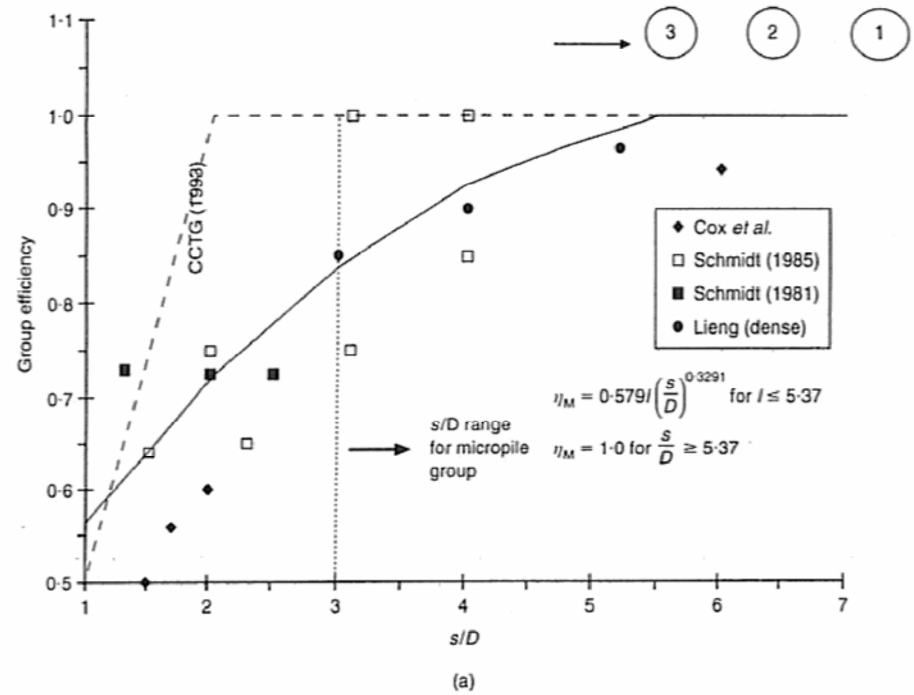


Fig. 12. Group efficiency factors η_{hl} versus s/D for trailing piles in a line (Reese *et al.*, 1994)

study conducted by Prakash (1962) concluded that the trailing-pile reduction can only be ignored if s/D is equal to or greater than 8. Test data from Cox *et al.* (1984), Schmidt (1985) and Lieng (1988) are presented in Fig. 12(b), and the curve they recommended for analysis shows that this reduction can be ignored if s/D is about 6.

It should be noted that while the Reese *et al.* (1994) recommendations were established for conventional piles, the French code (CCTG, 1993), which is currently used in France for micropile design practice, appears to be less conservative. As indicated in Fig. 12, the French recommendations (CCTG, 1993) indicate that in the direction of the lateral loading, no group efficiency factor should be applied for an s/D ratio greater than 2, which is generally the geometry used in micropile groups. However, in the absence of sufficient field data, it is recommended that the French recommendations be used only as long as the horizontal loads are small as compared with the axial load applied to the micropiles (i.e. up to 10% of the allowable axial load).

Design methods and observations on full-scale structures

The purpose of the following section is briefly to outline the principles of the design of micropile systems for structural foundation underpinning and *in situ* reinforcement for slope stabilization and retaining structures with regard to Case 1 (micropile groups) and Case 2 (micropile networks).

Structural foundation underpinning

As pointed out by Lizzi (1982), the most significant feature of micropiles used in underpinning work is the immediate response to any movement, however slight, of the structure. Underpinning does not supersede the existing foundation. The micropile underpinning can be considered practically inactive at the moment of its construction. When the structure has a subsequent, small movement, the piles respond immediately, absorbing part of the load and reducing at the same time the stress on the soil. If, despite this, the structure continues to settle, the piles continue to take the load until, finally, the entire building load is supported by them. Even in the most extreme case, the settlements would be limited to a few millimetres; the factor of safety after underpinning will be a combination of the safety factor of the existing foundation, which depends on the shear resistance of the soil, and the additional safety factor due to piling. The problem is complex and requires consideration of strain compatibility and group effects.

The inclined pile configuration shown in Fig. 13(a) imparts to the wall stability against overturning and lateral translation. This is important if the underpinning system is subjected, besides vertical loads, to the action of lateral loads and bending moments near its top. The behaviour of the micropiles subjected to the action of the vertical force F_z and the lateral force F_x may be analysed with reference to the force equilibrium diagram shown in Fig. 13(b).

In the general case of Type A micropiles, as the movements of a micropile system are extremely small, the use of the system is of great advantage for solving excavation problems and underground construction, where it is essential to avoid the decompression of the soil. The application shown in Fig. 14(a) indicates that some arching is developed over the tunnel and contributes to the overall stability. In this case, as shown in Fig. 14(b), the non-reticulated micropile system and the rigid cap can be assumed to act as a frame with each micropile system functioning as a 'composite beam'.

Bending-moment capacity is required of all three members in order to resist the applied combined loading. The moment and shear resistances required of each 'composite beam' are assumed to be provided by the steel reinforcement, while, unless otherwise specified by the code to be used, the axial compression capacity of the composite beam is assumed to be provided jointly by the steel and the grout in a certain proportion (Xanthakos *et al.*, 1994). The area of the grouted pile and the pile spacing are used to compute

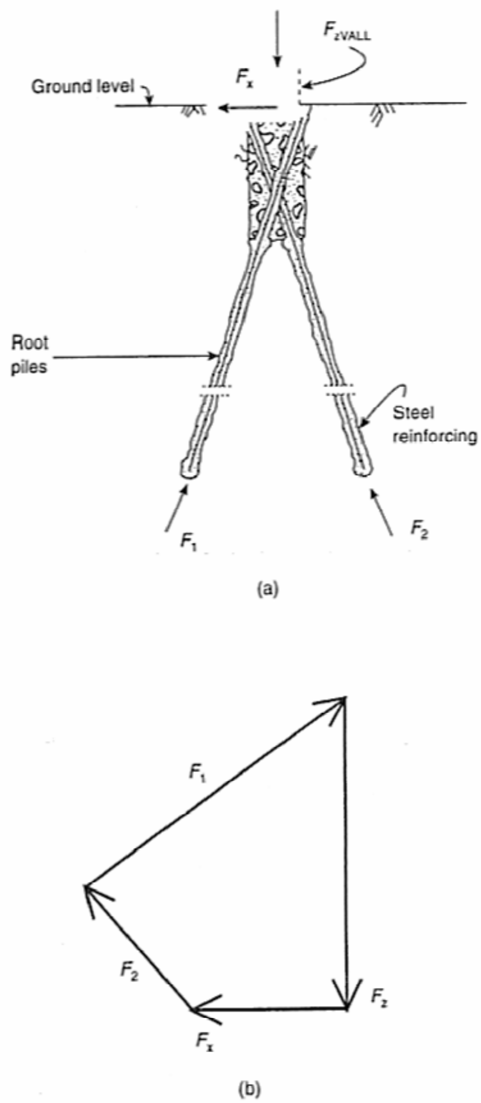


Fig. 13. Typical scheme of a micropile underpinning: (a) vertical cross-section; (b) force equilibrium diagram

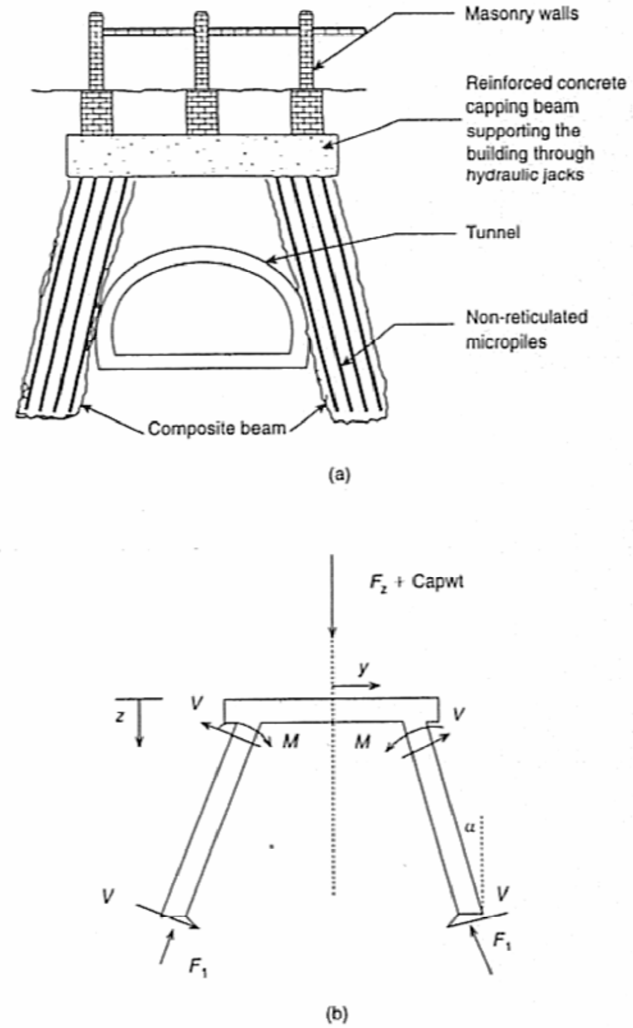


Fig. 14. Non-reticulated micropile system for underpinning building above a subway tunnel: (a) vertical cross-section; (b) structural loads and assumptions

the moment of inertia of the composite beam for the indeterminate structural analysis of the frame.

In situ reinforcement for slope stabilization and retaining structures

Engineering practice for slope stabilization with small-diameter, flexible inclusions such as non-reticulated micropiles or soil nails generally relies on the two design concepts illustrated in Fig. 15, namely,

- the structural-frame concept
- the slope reinforcement concept.

These two basically different design concepts are often associated with different site conditions and design criteria regarding allowable displacements and the required increase of the safety factor with respect to the slope stability. As illustrated by Herbst (1995), a variety of design schemes can be developed to accommodate specific engineering applications and relevant design criteria. Figure 15 illustrates a potential relationship suggested by Herbst (1995) between design schemes, the load-bearing mode of the soil–micropile system and the anticipated displacement. The design schemes presented in this figure can be broadly classified as

(*a*) slope reinforcement
(*b*) structural frame
(*c*) anchored micropile retaining systems.

For each one of these categories, the geometry and the cap will strongly affect the overall stiffness of the micropile retaining system. A high-tensile-capacity micropile can also be used as a ground anchor to restrain potential movements of the retaining system.

The design methods currently used in evaluating the stability of non-creeping reinforced slopes can be classified into two categories:

(*a*) limit stability analysis methods, which generally consider the moment equilibrium of the potentially sliding reinforced soil zone in evaluating the safety factor with respect to its rotational stability (Schlosser, 1983)
(*b*) a displacement method (Cartier and Gigan, 1983), which uses load transfer $p-y$ analysis to assess the resisting forces developed in the reinforcement for a specified admissible soil displacement in evaluating the safety factor with respect to the rotational stability of the reinforced soil mass.

A case study reported by Guilloux and Schlosser (1984) illustrated the use of the limit stability analysis approach for slope reinforcement design practice. This case, shown in

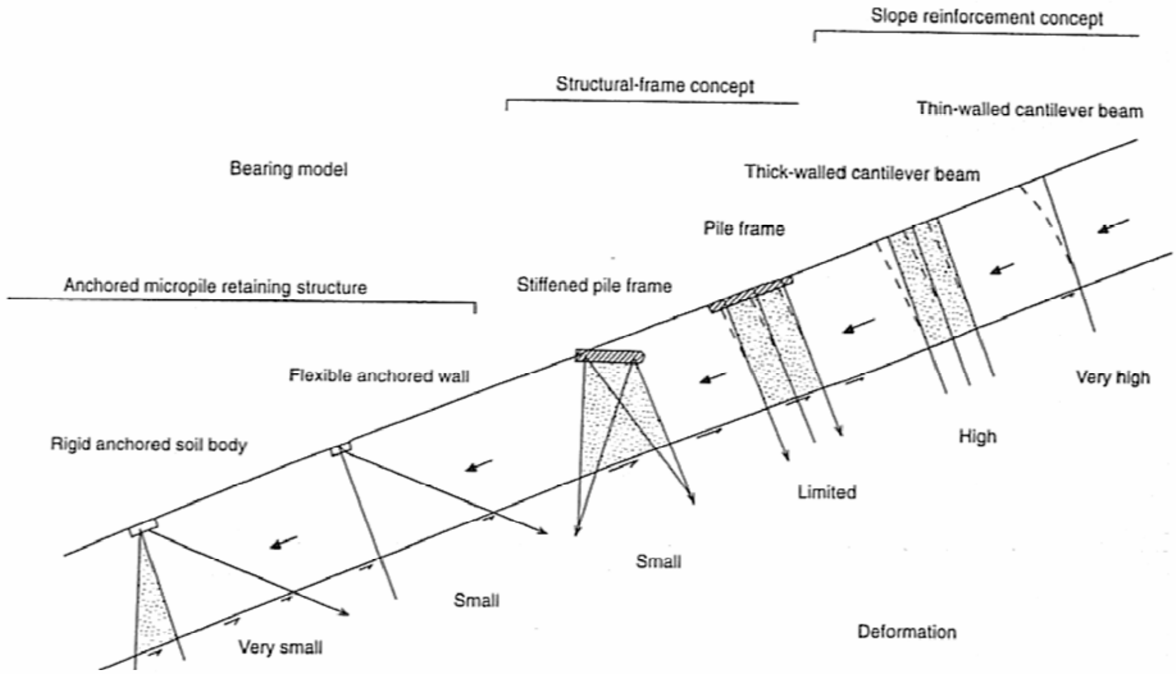


Fig. 15. Slope stabilization with micropiles (Herbst, 1995)

Fig. 16, refers specifically to the stabilization of a railway embankment which experienced significant movements prior to its stabilization with low-pressure-grouted (Type B) vertical micropile groups. Using the limit stability analysis, it was possible to determine that the calculated safety factor of the reinforced slope with the micropiles installed was 1·38. Inclinometer measurements made for nine months after installation of the micropiles showed that the rate of movement of the slope had decreased significantly, and for the last three months of measurements, movement had virtually ceased.

In the United States, micropile slope stabilization systems are called Type A (Bruce and Jewell, 1986, 1987a) walls by one contractor because, as illustrated in Fig. 17, the pile arrangements result in a distinctive Type-A-like cross-sectional shape. Their use has become increasingly popular in a wide range of applications for slope and excavation stability associated with deep foundations, tunnelling and highway construction. Within the past few years, intensive research has been conducted by Pearlman *et al.* (1992), indicating that the movements associated with a reticulated micropile group retaining system seem to be confined to a relatively thin and localized zone along the slide plane, and additional slope movements occur after construction.

A procedure was developed for preliminary design better to model the behaviour of this relatively flexible slope stabilization system. This design procedure consists of evaluating, primarily, the potential for structural bending

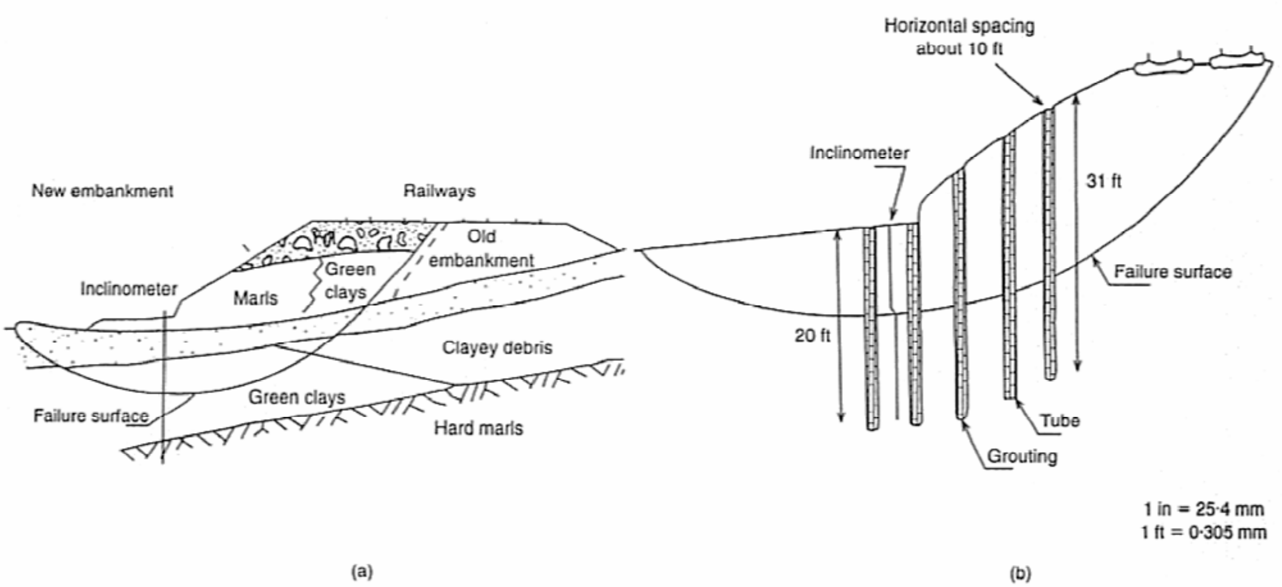


Fig. 16. Case study illustrating the use of limit stability analysis approach for slope reinforcement design practice: (a) geotechnical cross-section of a railway embankment; (b) sliding-slope stabilization by micropiles

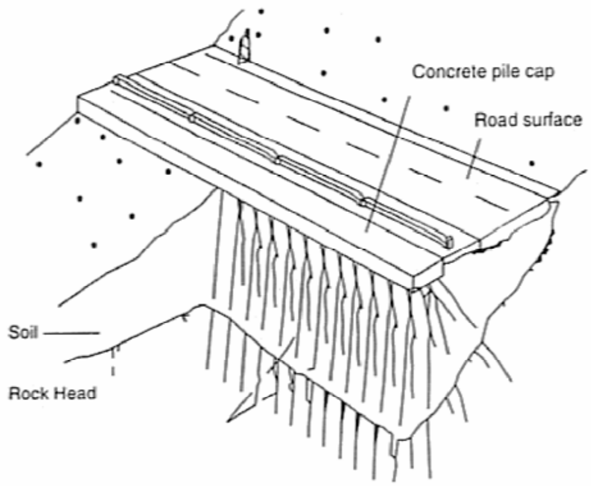


Fig. 17. *In situ* soil reinforcement by reticulated micropile system (Bruce and Jewell, 1986, 1987a)

failure of the piles due to loading from the moving soil mass. Preliminary design charts were developed by Fukuoka (1977) to evaluate the bending moments that develop in a pile orientated perpendicular to the slip plane, assuming a uniform velocity distribution of the soil above the slip plane. These charts were adapted by Pearlman *et al.* (1992) for the design of *in situ* micropile slope reinforcement. Furthermore, Pearlman *et al.* (1992) recommended the solution proposed by Ito and Matsui (1975) for the evaluation of the potential for plastic flow of the soil around the piles and proposed preliminary design charts for various pile spacings and soil characteristics.

The proposed design procedure has been evaluated by comparison with back analyses of instrumented walls. Brown and Chancellor (1997) reported results of an instrumented INSERT wall in Littleville, AL. The wall was instrumented with inclinometers for measuring the slope movement, with piezometers and with strain gauges to measure axial and bending stresses developed in the piles and the ground anchors. A typical cross-section and the measured axial forces along the downhill and uphill micropiles are illustrated in Fig. 18. The measured values of axial and shear forces suggest a ratio of mobilized axial to shear forces of 16:1 in the uphill pile and 14:1 in the downhill pile, which is considerably greater than the original design values. According to the authors, these results implied that the stabilizing effect of the piles is more closely related to axial forces and the batter angle than was expected in the original design procedure following Pearlman *et al.*'s recommendations.

The micropile network design concept developed by Lizzi (1982), illustrated in Fig. 19, consists of 'a three dimensional lattice structure built into the soil according to a pre-established scheme depending on the purpose that the structure has to carry out'. The purpose of the micropile network is twofold: first, to encompass the soil portion above the critical surface, and second, to 'nail' this surface, thereby supplying additional shear forces to increase the shear resistance of the natural soil. The monolithic action of the different structural components (steel, grout and soil) is significantly dependent upon the horizontal compaction caused by the injection pressure of the grout.

The design anticipates a highly redundant system in which no tension is applied on any of the piles. This system is therefore subjected to compression and shear, and the reinforced piles provide confinement to the *in situ* soil, thereby improving its deformation modulus and increasing its shear resistance. The design presents an analogy to reinforced concrete design considering a homogenized section of a 'composite beam'. The extreme fibre stresses are kept compressive in the heel of the 'wall' by the proper choice of design parameters. In order to resist the overturning moment and maintain a compressive stress in the soil, the design should verify that the resultant of the earth pressure and dead-load forces acts in the middle third of the foundation. The horizontal component of the resultant force acting on the base of the 'structure' is resisted by the

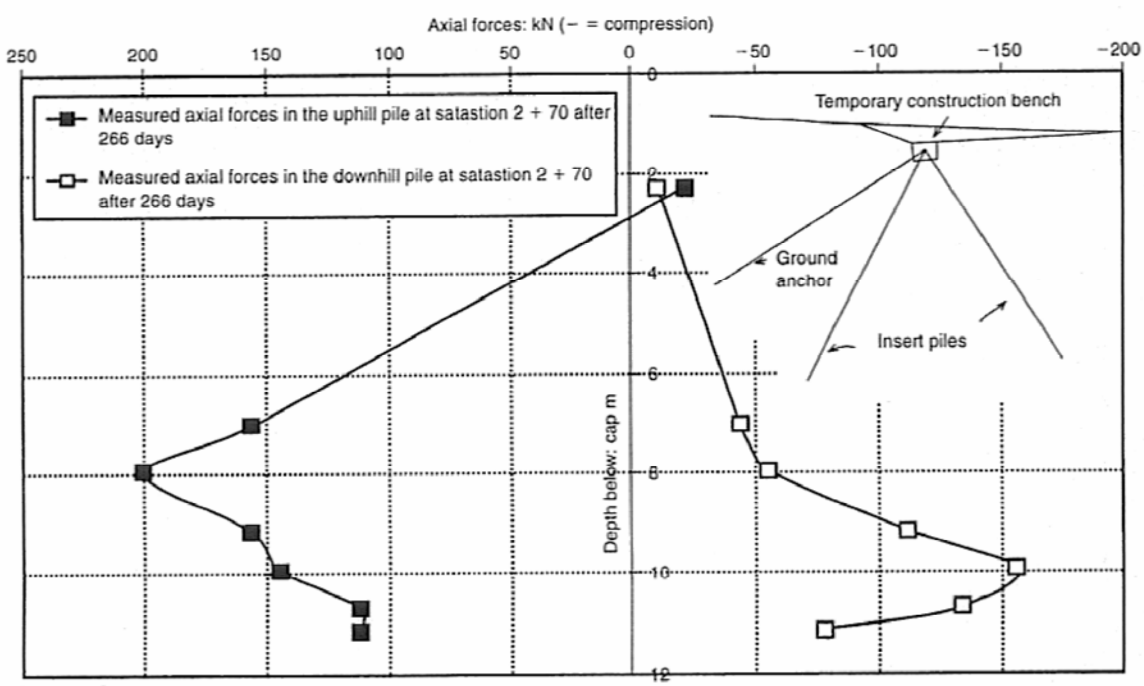


Fig. 18. Measured axial forces in the uphill and downhill piles of the INSERT wall in Littleville, Alabama (Brown and Chancellor, 1997)

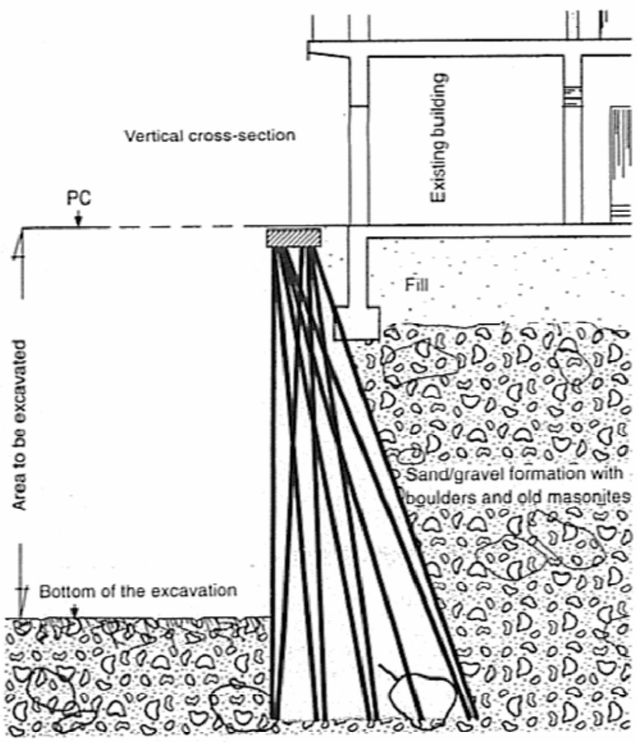


Fig. 19. Typical scheme for a reticulated network for the protection of a building during a deep excavation in close proximity (Lizzi, 1982)

combined shear resistance of the soil and the shear resistance of the piles acting as dowels. It is recommended that the piles be extended into rock if possible and should always be extended below the zone in which failure is suspected.

The detailed design procedure for slope reinforcement with a reticulated micropile network has been illustrated by Cantoni *et al.* (1989) with reference to the slope stabilization along the Milan–Rome motorway shown in Fig. 20(a). The stability of the structure is generally analysed with respect to the following failure mechanisms: (i) plastic deformation of the soil between adjacent micropiles, (ii) sliding of the reinforced block on the firm soil and (iii) structural failure of the composite cross-section of the block. The condition derived from mechanism (i) allows the determination of the spacing of the micropiles transverse to the movement, while the conditions from (ii) and (iii) establish the total numbers of micropiles and the spacing between the rows.

A Case 2 micropile system was used in Mendicino, California (Palmerton, 1984) for a highway slope repair. A typical section of the pile network, which was connected to a 1·00 m thick reinforced concrete cap beam, is shown in Fig. 20(b).

The piles were installed at inclinations ranging from vertical to about 16 degrees from vertical. A total of 28 piles with a length of 3·60 m was required to construct each repetitive unit of the wall. The centre-to-centre spacing between adjacent piles at the cap beam ranged between 0·45 and 0·9 m. The performance of the piles during and after construction was monitored by the US Army Corps of Engineers, Waterways Experiment Station (Palmerton, 1984), using strain gauges bonded to reinforcement bars. The results of strain gauging of the steel reinforcement indicated that, with some exceptions, the steel was loaded in compression with calculated stresses ranging from 5 to 52 MPa. Measured tension strains were generally limited to an area in the vicinity of the cap beam or near the bottom of the piles below the presumed shear surface. Strains in the reinforcing steel developed rapidly during the first and second months following construction but stabilized thereafter. The recorded postconstruction strains in the rebars were too small to establish apparent trends. Apparently, the slope (at least in the area of instrumentation) had stabilized upon construction, indicating that the design was too conservative.

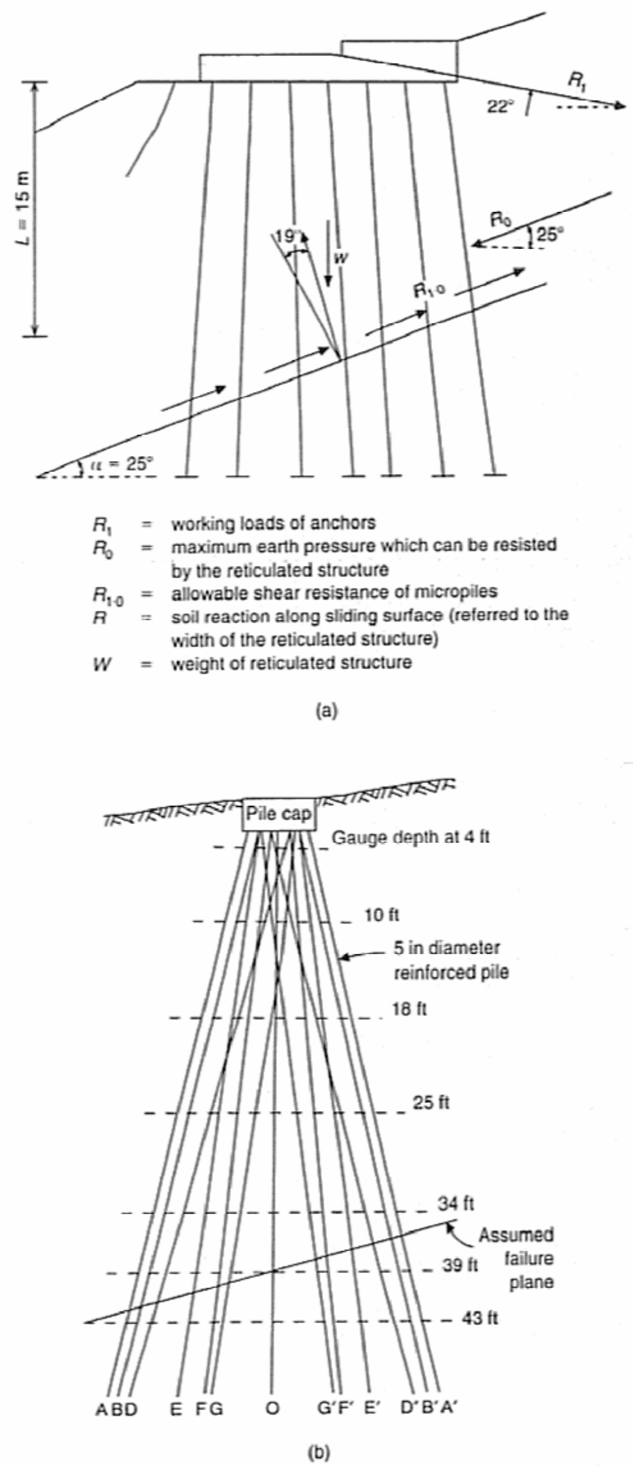


Fig. 20. (a) Milan–Rome motorway slope stabilization project (Cantoni et al., 1989); (b) typical section of the reticulated network micropile system (Case 2) used in Mendicino National Forest, California (Palmerton, 1984)

These observations support Lizzi's (1982) conclusion that monitored structures have demonstrated that reticulated network systems effectively satisfy the design criteria with no significant forces in the micropiles. Therefore, current design approaches appear to be conservative as they do not take into consideration the interaction developed between the soil and the micropile.

Conclusion

The broad conclusion to be drawn from this study is that micropile construction techniques greatly affect the axial loading capacity, and so raise significant limitations with regard to the use of empirical design rules. Therefore, the design of micropile systems relies essentially upon field loading tests, which are of paramount importance for on-site evaluation and optimization of the design and construction of the micropile systems and for establishing the actual factors of safety. A significant database is needed to assess the validity of the empirical design rules currently proposed for the different ground conditions encountered in micropile design practice.

The group efficiency factor in micropile systems is highly dependent on a variety of factors, in particular the pile inclination and installation techniques. The group effect in gravity-grouted micropile systems can significantly increase pile movement, while pile inclination will significantly reduce the group effect on pile movement. The experimental results are consistent with empirical pile design correlations proposed by Fleming *et al.* (1985) and Skempton (1953), as well as with the load transfer hybrid models. A pure shear interface model is proposed for evaluating the group effect in micropile design practice. Both the model test results reported by Lizzi (1978) and the full-scale test results reported by Plumelle (1984) demonstrate that the inclination of the micropile results in a network effect that increases the axial loading capacity and significantly decreases the movement of the soil–micropile group. However, no specifications have yet been established to take into account this network effect in micropile design practice.

For slope stabilization, current design methods provide a sound and conservative analytical framework for the stability evaluation of a micropile-reinforced slope provided appropriate micropile–soil interaction parameters can be determined for the specific site conditions and engineering applications. However, the difficulties involved in the *in situ* determination of the appropriate interaction parameters for different micropile types and installation techniques raise significant needs for further research. Furthermore, the engineering behaviour of the composite reinforced soil system is not yet clearly understood and no design method has yet been established to assess the concept of ground reinforcement by a three-dimensional network of reticulated, small-diameter piles. The absence of such engineering design methods raises significant limitations with regard to the market penetration of this cost-effective technique in North America. It is therefore essential that both field studies and analytical simulations be conducted in order to develop and experimentally evaluate reliable design guidelines for the engineering use of micropiles groups and reticulated network systems.

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