Strengthening of foundations through peripheral confinement

Renforcement de fondations au moyen d'un confinement périphérique

J.M.Rodriguez Ortiz – *Polytechnical University of Madrid*

ABSTRACT: In some singular cases underpinning of foundations cannot be performed by the customary methods of reinforcing elements installed through the footings. The paper discuss, from a theoretical point of view, the expected increase in bearing capacity according to the stiffness of the peripheral element. The limited Poisson effect also results in smaller settlements under additional loading of the foundations. Several cases are dealt with in the paper where the foundations have been strengthened by peripheral installation of vertical micropiles or jet-grouted columns. In the first case, the ground around the footings of a bridge had to be removed in order to gain room for water passage. In the second case the foundations of a Gothic cathedral, in very bad condition, were improved through said method.

RÉSUMÉ: Dans certains cas la reprise en sous-oeuvre des fondations ne peut être achevée par la méthode usuelle de placer éléments de renforcement traversant les semelles. On commente, du point de vue théorique le gain en capacité portante en fonction de la rigidité et résistance des elements placés au contour de la fondation. En limitant l'effet Poisson on reduissent au même temps les tassements associés à une augmentation de la charge. On passe en revue quelques cas réels où le renforcement périphérique a été réalisé au moyen de micropieux ou colonnes de jet-grouting. Dans le premier cas le terrain autour la fondation d'un pont devait être excavé pour augmenter le tirant d'eau de la rivière. Le second cas décrit le renforcement des fondations des piliers d'une cathedrale Gothique, dans très mauvais état, vis-à-vis d'une rédistribution de charges ou dans l'hypothèse de la dégradation du terrain d'appui.

1 INTRODUCTION

As it could be expected, the introduction of high strength vertical dowels around a foundation creates a confined nucleus with its lateral spreading prevented. This results in increased bearing capacity and reduced settlements.

Continuous confinement has been used since a number of years forming the so-called a skirted foundation (Kurian & Devi, 1997).

In this paper discontinuous skirting is considered, but assuming that the dowels are close enough to prevent the escaping of soil through the gaps and able to develop a quasi-continuous screen effect.

2 THEORETICAL APPROACH

As it is well known the dowels are subjected to bending and shear. If the soil is rigid enough shear work predominates. In a simplified way this can be viewed as shear force close to the failure surface, although it should be expected that the inclusion of the dowels changes the shape of this surface. For a pure cohesive soil the strength of the dowel is superimposed on the usual failure mechanism (fig. 1) thus giving:

$$\sigma_f = \sigma_0 + (\pi + 2)\, c_u + 2\sqrt{2}\ \underline{S} / B \qquad (1)$$

$\underline{S}$ being the mobilized shear strength of the dowels per meter length, for a given settlement of the footing or deflection of the inclusion, and B is the foundation width.

In the case of the usual mechanisms for frictional materials the strength of the dowel acts tangentially to a logarithmic spiral and the resultant moment with respect to the edge of the foundation directly adds to the bearing capacity expression (fig. 2). If one takes the Buisman formula, for instance, the increase in the limit stress is

$$\Delta\sigma_f = \frac{2S}{B} N_d = \frac{2S}{B} \frac{e^{\left(\frac{\pi}{4}-\frac{\varphi}{2}\right)\tan\varphi}}{\tan\left(\frac{\pi}{4}-\frac{\varphi}{2}\right)} \qquad (2)$$

Similar expressions can be obtained with the bearing capacity formulae of Terzaghi, Brinch Hansen, etc.

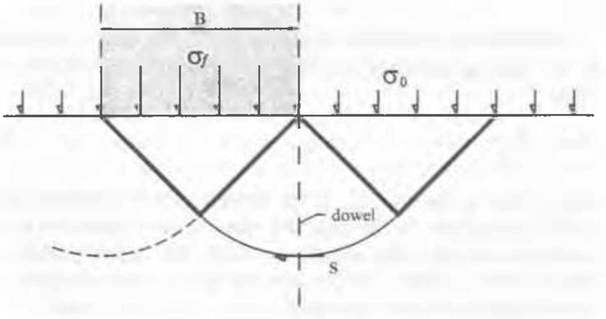

Figure 1. Dowel in cohesive soil

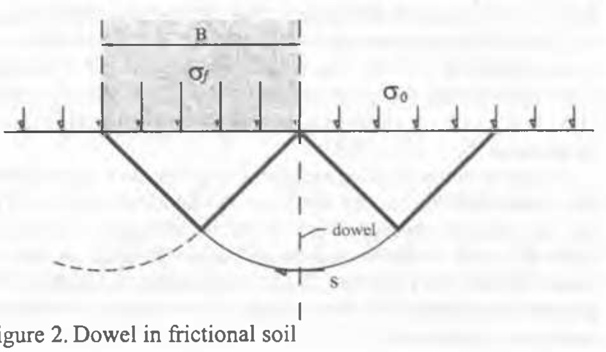

Figure 2. Dowel in frictional soil

The values of N_d range between 1.78 and 3.09 for friction angles of 20° to 40°.

The introduction of the dowels influences the shape of the failure mechanism. If the dowels are properly connected to the footing and they have a good length into the ground below the failure surface (> 2 –3 B), the block of soil below the footing dilates by Poisson effect against the dowels and the outside ground, putting the dowels in bending and mobilizing a passive wedge. Fig. 3 illustrates in a simplified way the possible failure mechanism. The passive wedge do not corresponds exactly to the Rankine hypothesis as the displacements at the vertical boundary follow a parabolic law. Furthermore it is necessary to check whether the displacements for reaching the passive state exceed the limit deflection for the dowels failing in bending.

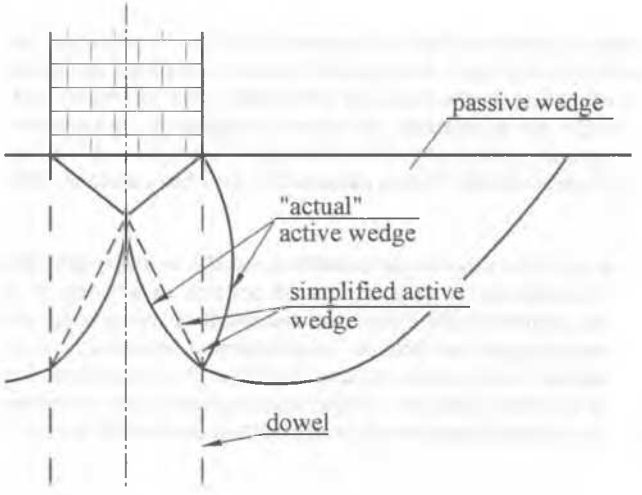

Figure 3. Dowels working in bending

In the case of shear work, neglecting compressive or tensile stresses, le limit shear force can be assessed by

$$S = f_{ye} \cdot A / \sqrt{3} \tag{3}$$

f_{ye} being the elastic limit of the steel inclusion and A its section area.

Some authors introduce this strength in the term of cohesion of the bearing capacity equation as an improved cohesion of value:

$$\bar{c} = \frac{S}{s_x s_y} \tag{4}$$

being s_x and s_y the spacing of the dowels in the tangent plane to the failure surface. In the case of a single row of dowels it is difficult to assess the length (said s_y) along the failure surface affected by the inclusion. Moreover not all the shear strength can be mobilized as the soil can locally fail around the dowel.

When in bending, the horizontal pressure yielded by the dowels, before failure, depends on their deflection, as in the case of a spring. Said pressure is also highly dependent on the fixity conditions at the ends of the dowels. The allowable deflection δ_h is thus related to the allowable settlement s_v. A stiffness ratio can be established as $R = BE_s / E_d I_d$, the subscripts *s* and *d* referring to the soil and the dowel respectively. For low R ratios (close to 1) $\delta_h = 0{,}1$ to $0{,}15\ s_v$, whereas for higher values (> 300) it can be assumed $\delta_h = 0{,}2$ to $0{,}3\ s_v$.

As refers to the bending moment it depends on the position of the lower fixed end of the dowel or the bending height *h*. This can be obtained through a p-y analysis, although *h* usually is close to 1.5 B (cohesive soil in undrained loading) or lies between 2B and 3B (frictional soil). Neglecting the weight of the ground in the block below the footing, this results in a stabilizing horizontal pressure of

$$q = \frac{187\,EI}{h^4}\delta_h \le \frac{14.2\,M_d}{h^2} \tag{5}$$

EI being the dowel stiffness and M_d the design bending moment of the dowels.

The corresponding increase in bearing capacity is approximately given by

$$\Delta\sigma_f = q\,h^2 / 2 \tag{6}$$

Simple footings (B = 2 m) have been computed by the finite element method in order to check the results above. The working bearing pressure (lower than the ultimate) q_w, corresponds to a settlement of 1" (2.54 cm).

The dowels, installed at the edges of the footing, can be modeled as individual vertical beams provided that their spacing does not allow the escaping of soil and the work as a equivalent continuous diaphragm is assured.

Fig. 4 shows the results for several sections of steel reinforcement as compared with the case without reinforcement in cohesive soils. The same comparison is made in fig. 5 for frictional soils.

Quite good agreement is obtained by using expressions (5) and (6).

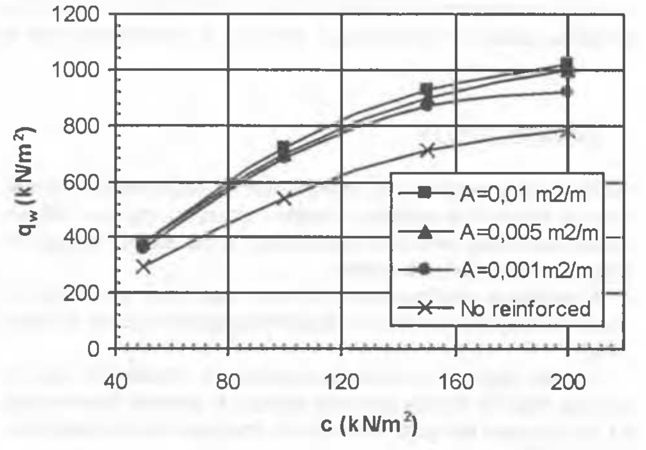

Figure 4. Bearing capacity of reinforced cohesive soil

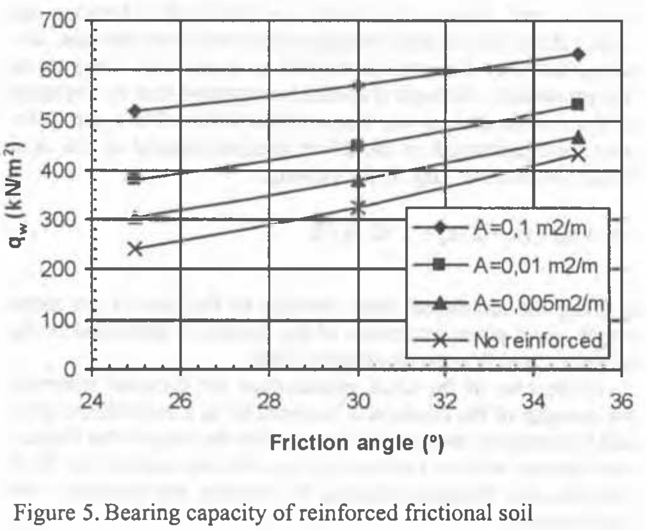

Figure 5. Bearing capacity of reinforced frictional soil

3. CASE HISTORY 1. EXPOSED CONFINEMENT

The peripheral confinement was used in 1992 as strengthening of the foundations of a viaduct in a small river which had to be deepened 1 to 3 m for the channelization of that section.

The footings, of about 3x3 m in dimensions carried working pressures of 400 kN/m^2. The ground was a claystone of Cretacic age, slightly weathered.

The underpinning consisted of an enclosure of vertical micropiles, at 30 cm spacing, embedded about 3 m below the final bottom profile. The head of the micropiles was temporarily connected by a steel beam until finishing the excavation or trimming of the riverbed rock around the footing.

In order to protect the micropiles against corrosion as well as the soft rock between, the block exposed was encapsulated by a reinforced concrete lining, with an upper capping slab receiving the head of the micropiles and shear connected, by steel dowels, to the upper face of the footing and the base of the pier above (fig. 6).

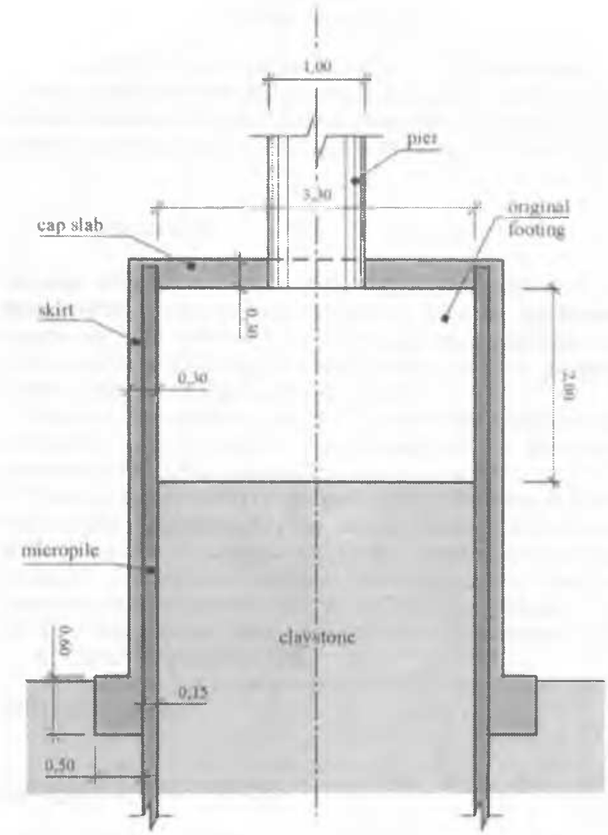

Figure. 6 Footing on reinforced block of claystone

The effect of these operations was studied by means of a finite element code, with an axisymetrical model. The computed settlements are shown in the table below:

Free excavation	Settlements (mm)		Horiz. Deflection (mm)	
	No dowels	Dowels	No dowels	Dowels
3 m	4,4	2,0	0,40	0,23
4 m	6,6	2,4	10,25	0,24
5 m	24,0	2,5	94,20	0,24

The maximum measured settlement amounted to 1.5 mm in a footing excavated 3.5 meters.

The maximum computed vertical stresses at the footing axis were:

Free excavation	Stresses (kN/m^2)	
	No dowels	Dowels
3 m	410	216
4 m	521	264
5 m	785	270

The relieving of load achieved through the micropiles is due to their underpinning effect, not always needed nor desired. This can be avoided connecting the head of the micropiles as a sliding hinge.

4. CASE HISTORY 2. BURIED CONFINEMENT

The method of peripheral confinement has been applied to the strengthening of the piers of the Gothic Cathedral of Tarazona . This monument was severely damaged by tilting and settlement of the main piers at the crossing between transept and nave. Softening of the ground due to water seepage from a nearby channel was assumed as the cause of the distress, but some structural interventions, as the careless repair of the flying buttresses may have also contributed to the problem.

As a part of the recent repair works the strengthening of the foundation was envisaged, in order to prevent further movements as a result of force redistribution following the anticipated structural interventions.

The foundations were constructed as pits filled with an old lean concrete composed of big sized stones and lime-cement mortar. In some places remains of Roman constructions appeared incorporated to the foundations.

The ground consists of an upper layer of made fill and historic debris, ancient pavements, etc., followed by a colluvium of silty clay with some gravel and boulders. The substratum is the original formation of overconsolidated claystones of Miocene age. The base of the foundations was laid in the colluvium close to the water table, without reaching the competent substratum below.

As the piers could not be unloaded and being evident that the ground surrounding the foundation pits was collaborating in resisting the overturning moments at the base of the piers, any excavation for underpinning was disregarded.

For the same reasons, any intervention below the foundations, as the installation of micropiles, was deemed very risky, as the drilling would probably induce additional settlements, not compatible with the precarious condition of the cathedral.

The monitoring of the piers showed that the movements were practically stabilised. Thus, only a reduced increased in bearing capacity was required against the anticipated new actions.

The solution finally selected consisted of encircling the piers with jet-grouted columns, penetrating deep enough below the foundations in order to form a deep seated foundation pit with some penetration in the substratum. In order to increase the bending strength of the grouted columns a reinforcement bar was introduced in their axis after removing the drilling and jetting rods.

The bars were connected at their upper end by means of a capping beam surrounding the original foundation (fig. 7). In this case some underpinning effect was deemed advisable.

The settlements measured during the construction of the jetted columns amounted less than 1 mm.

Two hypotheses were considered:

i) A 20% increase of the vertical loading due to new actions with some excentricity

ii) Softening of the ground due to wetting. This was modelled by reducing the deformation modulus to half its value and putting the cohesion equal to zero.

The results of the calculus were as follows:

Hypothesis	Settlement (mm)	
	No reinforcement	With reinforcement
Loading increase	8,9	2,3
Foundation softening	6,3	2,1

As these hypotheses involve limit unfavourable cases, a direct comparison with the actual values is not possible nor desirable. Current measurements, however, with lower load increases, show values within the expected range.

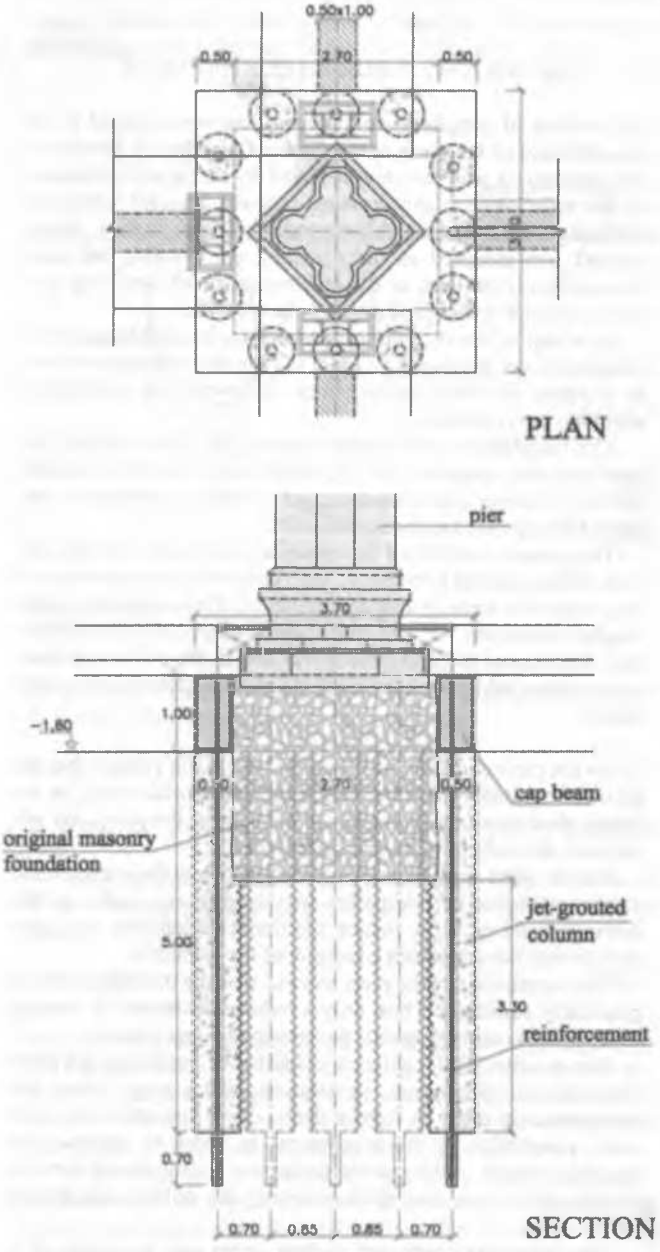

Figure 7. Strengthening through peripheral jet-grouted columns

5 CONCLUSIONS

The installation of vertical dowels around existing foundations result in a marked increase in bearing capacity and in reduction of settlements under new loadings.

This procedure do not require any intervention below loaded foundations, thus the effects of the installation of the reinforcement can be minimised.

A simplified method is proposed in order to evaluate the gain in bearing capacity achieved by dowels of a given section and stiffness.

6 REFERENCES

Azuar, J.J. 1977 Stabilisation des massifs rocheux fissurés par barres d'acier scellées. Raport de Recherche LPC n° 73. Paris.

Ho,C.L., Coyne,A.G. and Canou,J. 1997. Model tests of micropile networks applied to slope stabilization. *Proc. 14th Int. Conf. SM&FE*. Hamburg, 2, 1223-1226.

Juran,I., Baudrand,G., Farrag,K. and Elias,V. 1990. Kinematic Limit Analysis for Design of Soil-Nailed Structures. *J. of Geot. Eng.* ASCE, 116, Jan. 54-72.

Juran,I. and Elias,V. 1991. Ground anchors and soil nails in retaining structures. *Foundation Eng. Handbook*. 868-905.

Kurian,N.P. and Devi, N.C.N., 1997. Performance of skirted footings in sand. *Proc. 14th Int. Conf. SM&FE*. Hamburg, 2, 827-830.

Recommandations Clouterre 1991. Presses Ecole Nationale des Ponts et Chaussées. Paris. 268 p.