Driven, bored and auger piles in calcareous soils

Pieus battus, pieus fores et pieus hélice dans calcarie sol

O.I.Gomez & D.J.Compagnucci – *South University, Bahía Blanca, Buenos Aires, Argentina*

ABSTRACT: Calcareous soils exist in many places of the world where arid o semiarid conditions prevail. In this soils the low grade of interparticulate cementation do almost impossible to attain undisturbed samples. Therefore in situ tests are necessary for to apprehend the behavior of such soils. With the purpose of identification and classification were carry out penetration tests (SPT, CPT) chemical and cross hole tests. Drive, bored and auger piles, with diameters between 400 mm to 800 mm in medium to dense silica sands with calcareous fines were tested being the proof load 2 factor times the working load. In contrast to the behavior in clean sandy soils, the tests indicate that bored piles offer substantial load transfer.

RÉSUMÉ: Les sols calcaires se trouvent où prévalent les conditions de climat arides ou demi-arides. Dans ces sols, les bas degrees de cimentation entre ses particules font presque imposible d'obtenir des échantillons sans trouble. C'est pour cela qu'il faut faire des épreuves in situ pour connaître le caractère distinctif de ces sols. Avec le désir de les reconnaître et de les classer on a fait des épreuves SPT, CPT, Cross hole et chimistes. Pieus battus, pieus fores et pieus helices, d'un diamètre de 400 mm. a 800 mm. sur le sable siliceux, de peu denses à denses, avec des calcaires très fins, ont fait essayés avec des charges d'épreuve de 1,5 à 2 fois la charge de service. Au contraire des caractéristiques des sables propres, les essais ont indiqué que les pieus fores produissent une considérable transférence de charge.

1 INTRODUCTION

This paper presents the geo technical study of sandy deposits with a structure originated in a slight cementation and/or fabric such as small silty layers, inhomogeneities, oriented particles, etc.

Their behavior under load differs from that of normally consolidated clean quartzitic sands whose preconsolidation degree is assigned a value of 1 (OCR=1).

From a qualitative viewpoint the effects of the structure on the mechanical behavior of a soil can be similar to its stress history.

Cementation is a process occurring in many sand deposits. It originates from contact points between particles and cementing agents which generally do not exceed 7% of the total sample weight and which are hydrated carbonates and silicates. The cementation degree is weak to moderate (20 to 40 Kpa). The prevailing mineral is silicon. They belong to the SM or SP-SM group of the Unified Classification (USSC).

Generally a slight cementation cannot be detected when sampling is made by conventional procedures as a destructuration of the sample is produced (Mitchell 1993). However, the results of research work show the importance of including the effects of any level of cementation when shear parameters are evaluated.

2 IDENTIFICATION BY ON SITE ANALYSIS

The identification of structured sands involves important practices and procedures given the fact if cementation is disregarded, no matter how slight it may be, this shall result in a conservative foundation design, among other consequences.

Both penetration resistance and the Go shear module for small strains are two different functions which depend on the same variables, namely:

1- Density and vertical/horizontal pressure levels.
2- Compressibility.
3- Structure (cementation and fabric).

For the above reasons it is possible to identify structured sands by means of CPT or SPT tests together with Cross Hole tests which measure shear wave speed and enable to calculate Go, the shear module for small strains, provided the results are expressed as adimensional parameters.

Baldi's (1986), Accar's (1986), Jamiolkoski and Robertson's (1988), Fioravante's (1991), Eslaamizaadad S. And Robertson's (1996) research works made in laboratory calibration chambers on quartzitic sands of different degree of mechanical preconsolidation, allowed to define different relationships between the standardized Go/qc shear module and the standardized CPT conic resistance with effective vertical pressure to eliminate the influence of depth on the measured results.

In fact, in such chambers it is not possible to duplicate the natural creep (aging) effects.

3 ON SITE TESTS

Boreholes were made using rotary drilling method and samples taken for laboratory soil identification (ASTMD 1586/84).
Dynamic (SPT) and static penetration test (CPT) also were made. The SPT test were made with 60% effective energy by using a cathead to pull a rope attached to the hammer and by an automatic drop system.

The CPT tests were made with a cone 60° apex angle and 10 cm^2 base area. The penetration rate is 2 cm/sec and the friction sleeve has an area of 150 cm^2.

For the design of foundations subject to dynamical loads, Cross Hole tests were made by following the methodology proposed ASTM D 4428 Standard, using three boreholes in line spaced 3 meters apart center to center on the ground surface with 100 mm inside diameter pvc pipe. The casing was grouted in place by inserting a 38 mm pvc pipe through the center.

In figure 1 are detailed the results of laboratory and in situ tests.

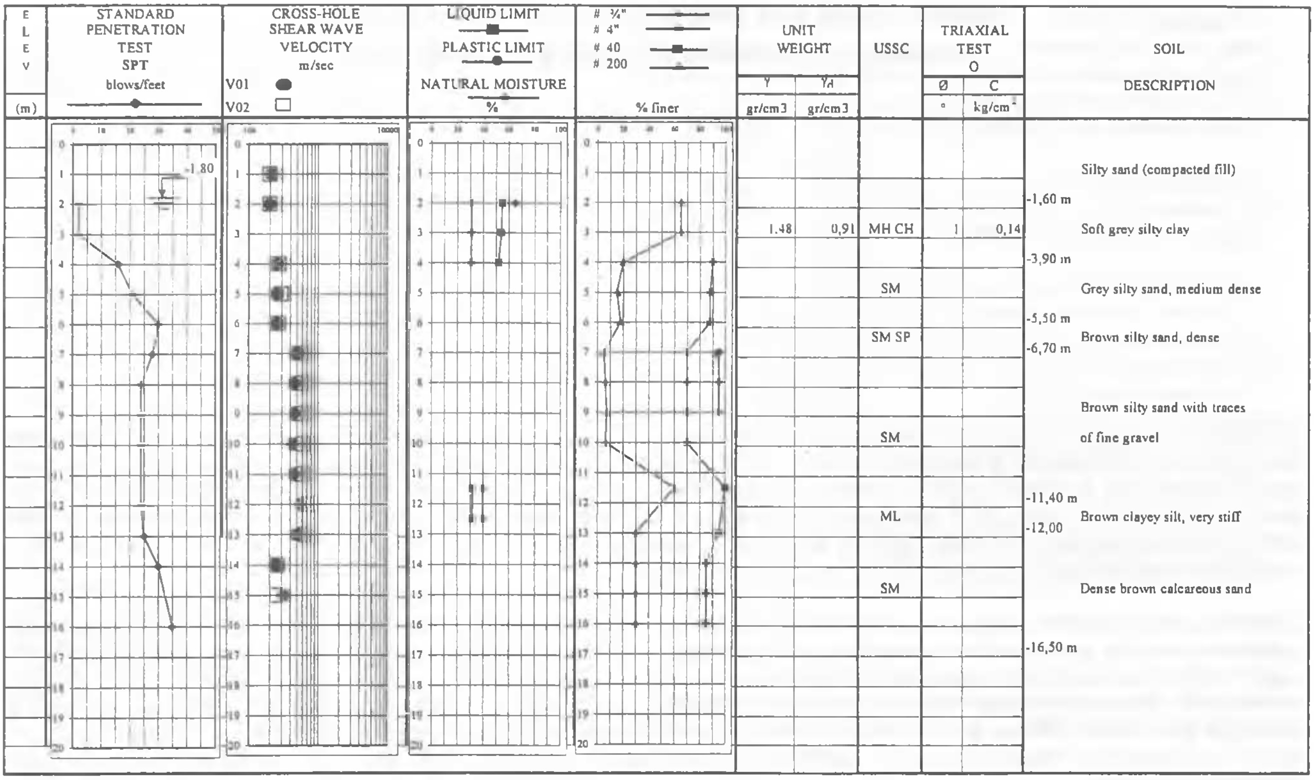

Figure 1 SPT, Cross Hole and laboratory tests.

3.1 *SPT - CPT correlations*

In one of the industrial development areas CPT tests were not made. N values resulting from SPT resistance to dynamic penetration tests were correlated in order to obtain CPT values. The on site correlation thus obtained closely confirms Roberstson and Campanella´s research work conclusions (1983) for effective D_{50} diameters in the range of 0,2 to 0,5 mm. (Fig. 2). qc/N ratio was 5 (D50= 0,2 mm) to 7 (D50= 0,5 mm). If the N values are corrected to an energy level of 55% the qc/N ratio agrees well with the historical average.

4 RESULTS OBTAINED

Table 1 shows the results of field tests and the calculate values of G_o.

Go shear module was calculated on the basis of the Cross Hole test shear wave speed Vs: Go $=\rho V_S^2$ were ρ is the soil mass density.

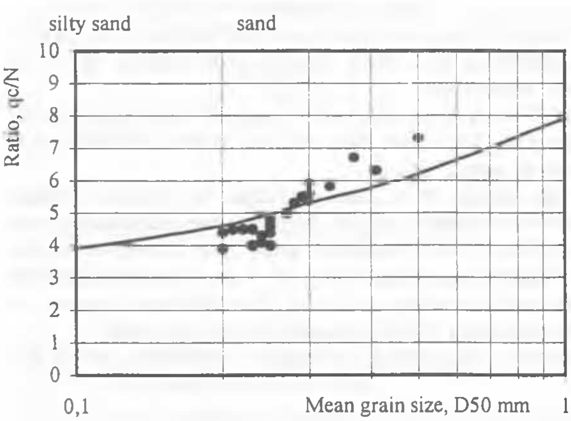

Figure 2. Relationship qc/N vs D50

The effective vertical pressure σo at the test depth was calculated.

Go/qc and $qc.(\sigma o)^{-0.5}$ relationships were calculated.

Coordinates were plotted in a log-log graphic.

The chart shown in figure 3 can be used to identify potentially cemented sands.

The following zones were defined in the chart:

A-Zone of non-cemented sands:

Two sub-zones are distinguished: one of low compressibility with the values of adimensional parameters Go/qc < 10 and $qc.(\sigma o)^{-0.5} > 1000$ expressed in kpa.

Another high compressibility zone where soft sands values were taken as reference with 30% CO_3Ca. The values that set the boundaries of this areas have been taken from the literature.

B-Zone of slightly cemented sands:

The effect of cementation on qc resistance values has been confirmed by several research workers such as Rad and Tumay (1986) Puppala et al (1993-1995).

Acar´s paper (1986) shows a growth of the Go shear module with low levels of cementation with the growth of the S rigidity coefficient. With resistance q_u values of cemented samples that ranged between 0,10 kg/cm^2 and 0,80 kg/cm^2 shear wave speed increased between 18% and 60%.

According to this simultaneous growth variation cementation growth shall be shown in the graph in an oblique direction as indicated by the arrow.

In the chart were the values obtained from sands values obtained from dredged sands during the 1980´s a fast cementation growing with time is observed. These are sands with calcareous fines and some calcareous granule gravel.

C-Zone of holocenic sands:

Jamiolkowski y Robertson (1988) consider that though traditionally it has been considered that sands are normally consolidated there exists evidence that sands of over 3000 years of age behave as preconsolidated for most loads.

In this paper the results of load tests made on bored and driven piles are presented, which show a high degree of rigidity, well over the one indicated by STP values for usual sands. See Figure 3.

Table 1. Shear module. Correlation G0/qc vs qc/ $(\sigma_v)^{0.5}$

Boring	Depth	N (SPT)	Vs	Vs^2 X 10^{-4}	γ	γ/g	G_0	qc	G_0/qc	σ_v	$(\sigma_v)^{0.5}$	qc/$(\sigma_v)^{0.5}$	Notes
	m		m/sec	m2/sec2	kg/m3		kg/cm2	kg/cm2		kPa	kPa		
S6	01,60	6	200	4,00	1.800	184	734	25	29,36	27	5,20	481	Dredged fill
	04,00 - 06,00	18	300	9,00	1.670	170	1.530	75	20,40	85	9,22	813	Sand medium dense
	07,00 - 13,00	23	550	30,00	1.800	184	5.490	95	57,79	180	13,42	708	Sand with calcareous
	14,00 - 20,00	35	800	64,00	1.580	161	10.304	150	68,69	289	17,00	882	Calcareous silty sand
S3 (P5)	03,00	27	230	6,76	1.670	170	900	116	7,76	27	5,20	2232	Dredged calcareous fill
	04,50	16	185	3,40	1.580	161	580	63	9,21	42	6,48	972	Sand with shells.
	07,50	13	140	1,96	1.670	170	333	52	6,40	70	8,37	622	Sand medium dense
	09,00	14	20	4,00	1.670	170	652	58	11,24	83	9,11	637	Sand medium dense
	12,00	44	260	6,76	1.600	163	1.100	196	5,61	105	10,25	1913	Calcareous silt
S4 (P4)	03,00	6	230	5,30	1.800	184	900	32	28,13	53	7,28	440	Loose sand
	04,00	28	270	7,30	1.670	170	1.240	166	7,47	76	8,72	1904	Dredged calcareous fill
	05,00	27	200	4,00	1.670	170	680	163	4,17	84	9,17	1778	Dredged sand
	06,00	20	230	5,30	1.670	170	900	113	7,96	93	9,64	1172	Dredged sand
	07,00	2	200	4,00	1.600	163	380	8	47,50	103	10,15	79	Soft clay
	08,00	2	130	1,70	1.670	170	290	8	36,25	111	10,54	76	Soft clay
	09,00	10	170	2,90	1.670	170	490	85	5,76	120	10,95	776	Sand clay
	10,00	10	170	2,90	1.600	163	490	67	7,31	129	11,36	590	Silty sand
	11,00		115	1,30	1.600	163	220	8	27,50	138	11,75	68	Soft clay
	12,00		115	1,30	1.600	163	220	6	36,67	147	12,12	49	Soft clay
	13,00	19	185	3,42	1.670	170	581	85	6,84	156	12,49	681	Sand with calcareous
	14,00	32	600	36,00	1.670	170	6.100	157	38,85	165	12,85	1222	Calcareous silty sand

5 RESULTS OF LOAD TESTS ON PILES

In order to assess the design pressure in some cases and to obtain the safety coefficient in others, compression load tests and in some cases tensile load tests were made.

Bored piles show a good performance with high frictional resistance values for areas of slightly cemented sands in the results obtained from tensile load tests.

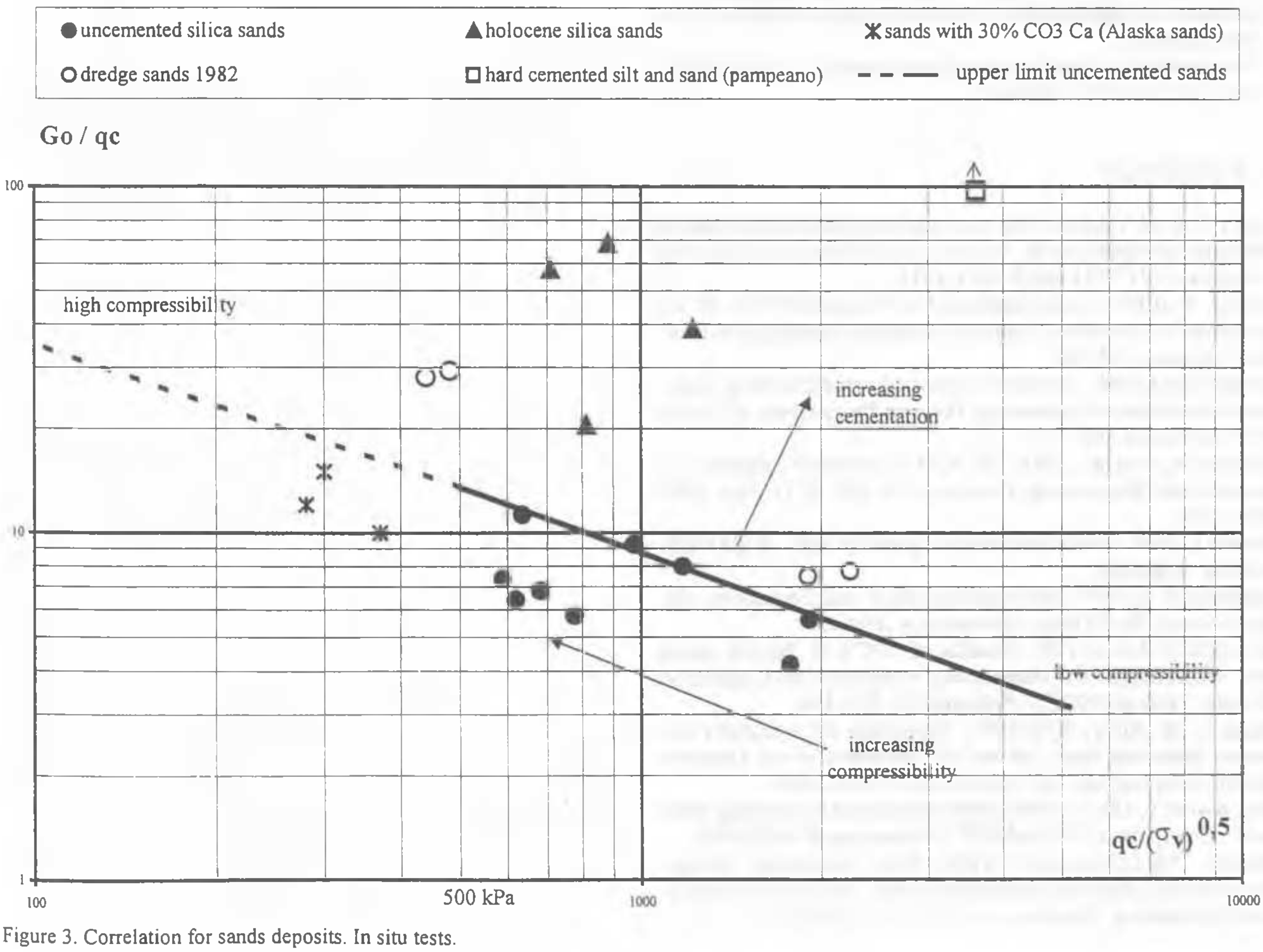

Figure 3. Correlation for sands deposits. In situ tests.

Table 2. Tests of individual piles

Date	Pile			Compression load			Tensile load		
	Type	Diameter	Length	P_{MAX}	$\delta^{(1)}$	$_{MAX}/A^{(2)}$	T_{MAX}	δ	$\tau^{(3)}$
--	--	cm	Cm	t	mm	kg/cm^2	t	mm	t/m^2
97-11	bored	40	600	76,8	2,6	61	25,6	0,97	3,4
97-11	bored	40	600	--	--	--	51,2	1,72	6,8
97-11	bored	60	600	149,7	3,35	53,4	43,3	1,14	3,8
97-2	driven	49	700	142,5	7,15	75,8			
97-3	driven	53	750	227	18,9	103,6			
86-7	bored	40	800	110	2,2	68,7	52	5,7	5,4
99-5	CFA$^{(4)}$	40	700	67,5	2,3	53,7			
99-5	CFA	50	670	82,5	10,0	41,2			

(1) δ: max strain
(2) A: butt pile area
(3) τ: max shear stress
(4) CFA: continuos flyght auger

For compression tests loads that duplicate the service loads, strains are not over 4 mm for pressures of 53 kg/cm^2 to 61 kg/cm^2 applied on the piles head.

Friction values for strains between 1,14 mm to 1,72 mm were in the range of 3,9 t/m^2 to 6,8 t/m^2. See Table 2.

6 CONCLUSIONS

In cohesionless soils reliable soil characterization is difficult using conventional sampling.

In this technical work the objective in first place has been to identify silica sands with calcareous fines, with lower content of carbonates based on cross hole tests and penetration tests.

In second place, to provide information on the uplift and compression capacity of bored, driven and auger piles in such soils field tests were carry out. The piles were all short piles located above or slightly below the critical depth defined by 15 to 20 pile diameters.

The results shows an important load transfer for bored piles. In this piles bentonita was used.

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