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DEFORMATION CHARACTERISTICS OF COHESIONLESS SOILS FROM IN SITU TESTS

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ABSTRACT

The evaluation of the deformation parameters from in situ test results is of great practical interest in cohesionless soils.
This paper is a summary of seven years experience gained by the National Electricity Board of Italy (ENEL), ISMES and Technological University of Turin in assessing these parameters using self-boring pressuremeter, static cone penetrometer and Marchetti's flat dilatometer tests. The tests have been performed in calibration chamber and at two sites, where general geotechnical information, cross-hole measurement and standard penetration tests were also available.

INTRODUCTION

The present work briefly summarizes the seven years experience gained by the National Electricity Board of Italy (ENEL), ISMES and Technological University of Turin (TUT) in assessing the deformation parameters of cohesionless deposits, mainly sands, on the basis of in situ tests results.
The data presented here represent the results of:

a. Research performed in the calibration chambers (CC) on two medium fine sands named Ticino sand (TS) and Hokksund sand (HS) using a standard electrical cone penetrometer (CPT), Marchetti's flat dilatometer (DMT) and the self-boring pressuremeter (SBPT) Camkometer MK VIII.
b. Extensive geotechnical investigations performed by ENEL at two sites proposed for the possible construction of 2000 MW nuclear power plants. The first site is located in the central part of the Po river valley, close to the ancient town of Mantova, where the subsurface conditions are characterized by the presence of thick strata of medium to coarse slightly silty sand. The second site is located in the upper part of the Po river valley, close to the town of Trino Vercellese. At this site the soil is mainly composed of gravel and sand with small percentages of fines.

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BACKGROUND

The evaluation of deformation parameters from in situ test results is of great practical interest in soil deposits where undisturbed sampling is still impossible or unreliable. This is the case with cohesionless soils. However the interpretation of the in situ tests suffers from many limitations [18] which render the assessed parameters, and especially those describing the deformation characteristics, very difficult to link to the relevant stress or strain level and drainage conditions of the specific project.
In this respect it is worthwhile grouping the available in situ techniques in the following three categories:

a. The solution of a more or less complex boundary value problem can lead to the determination of stress-strain and strength characteristics. All the soil elements strained during the test follow very similar effective stress paths.
   Therefore, with appropriate assumptions about the drainage conditions during the test and the stress-strain relationship of the tested soil, it is possible to evaluate deformation and strength characteristics.
   This category of tests includes the pressuremeter test and especially the SBPT.
b. The strained soil elements follow different stress paths depending on the geometry of the problem and on the magnitude of the applied load. In this case a rational interpretation of the test is very difficult. Even with an appropriate assumption concerning the drainage conditions and the soil model, the solution of a complex boundary value problem leads to something like the "average" soil characteristics. The comparisons between these average values and the behaviour of a typical soil element tested in the laboratory or their use in the specific design calculation are far from straightforward. A typical example of such in situ test is the plate load test.
c. The in situ tests results are empirically correlated to selected soil properties. Typical examples are the widely used and well known correlations based on penetration resistances measured in Cone Penetration Test (CPT) and Standard Penetration Test (SPT). Because of the purely empirical nature of these correlations, they are subjected to many limitations which are not always fully recognized by potential users. In addition, it should be recognized that these correlations are formulated for either fully undrained or fully drained conditions.
   Their correct use requires one of these two limit conditions to be satisfied during the test. This may be done with the aid of the Piezocone test (CPTU) which helps identify whether the CPT or DMT are penetrating the soil under either of the two conditions mentioned above.
   Within this category a possible improvement is represented by the recently developed DMT [3], [24] which loads the soil in the horizontal direction.
   In-situ tests belonging to this category have the disadvantage that the penetration induces unknown changes in the effective stress field and large straining of the surrounding soil which can erase all the features related to the stress and strain history of the tested soil. However the stress-strain characteristics of the soil are strongly dependent on stress-strain history. The stress-strain history of the soil deposits are not known a priori [2], therefore it is difficult to use the correlations obtained for deformation moduli for different stress-strain histories.

CALIBRATION CHAMBER TEST

The calibration chamber (CC) tests have been performed using the facilities of ENEL and ISMES which are described in detail by Bellotti et al. [4].
The characteristics of the tested sands are summarized also in [4]. All the tests have been performed on pluvially deposited dry [4], [17] and saturated specimens, for a sample size of 1.2 m in diameter and 1.5 m in height. The test in the CC consists of two distinct stages:

1st: The specimen is subjected to a desired stress or/and strain history. During this stage, the soil deformation properties are measured.

2nd: The specimen with the assigned stress-strain history is subjected to the in situ test under strictly controlled conditions imposed as boundary stresses or/and strains as shown in Fig.1.

Testing in the 2nd stage involved CPT and DMT penetrations and SBPT expansion. An "ideal installation" of the probe was simulated by placing the probe in the CC before preparing the specimen.
In order to obtain a comprehensive geotechnical characterization of the two sands a series of laboratory tests have been performed on pluvially deposited specimens. The results of these tests, in terms of deformation moduli are summarized in Table 1. More details are given by Baldi et al. [2].
The laboratory moduli has been linked to the relative density (D_R) and to the mean effective consolidation stress σ'_o using the following equation [39]:

$$\text{Modulus} = B_o\, p_a \exp\,(B_1\, D_R)\left(\frac{\sigma'_o}{p_a}\right)^{B_2} \qquad [FL^{-2}]$$

where:

p_a = reference stress taken equal to 1 kg/cm^2

B_o, B_1, B_2 = empirical coefficients

BC1 $\sigma_v = \text{const}$ $\sigma_h = \text{const}$

BC2 $\Delta\epsilon_v = \Delta\epsilon_h = 0$

BC3 $\sigma_v = \text{const}$ $\Delta\epsilon_h = 0$

BC4 $\Delta\epsilon_v = 0$ $\sigma_h = \text{const}$

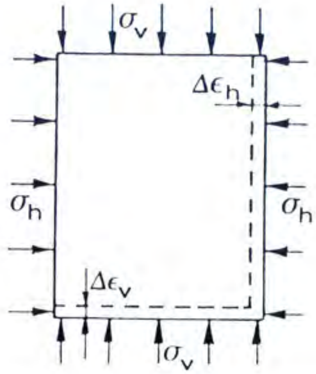


FIGURE 1. CC TEST, BOUNDARY CONDITIONS THAT CAN BE APPLIED DURING IN SITU TEST

TABLE 1
DEFORMATION CHARACTERISTICS OF TEST SANDS FROM LABORATORY TESTS

Sand	Modulus	B_o	B_1	B_2	R^2	N	D_R	Test
TS	G_o	399.2	1.39	0.43	0.95	34	0.50-1.0	RCT
TS	E_{25}^{NC}	48.6	2.78	0.44	0.90	29	0.48-0.94	TX-CK$_o$-D-CL
TS	E_{25}^{OC}	894.8	1.03	0.75	0.85	63	0.39-0.96	TX-CK$_o$D-CL
TS	E_{uR}^{NC}	1118.8	0.62	0.45	0.87	16	0.31-0.92	TX-CK$_o$D-CL
TS	M_o^{NC}	145.7	1.47	0.37	0.94	76	0.17-0.95	Oedometer
HS	E_{25}^{OC}	943.0	0.73	0.45	0.87	36	0.31-0.92	TX-CK$_o$D-CL
HS	M_o^{NC}	131.7	1.36	0.43	0.95	20	0.31-0.92	Oedometer

G_o = maximum shear modulus from resonant column tests (RCT) run on isotropically consolidated specimens

E_{25} = drained secant Young modulus from triaxial compression in loading tests, run on specimens consolidated in K_o - conditions (TX-CK$_o$D-CL) evaluated at deviatoric stress level equal to 25% of the failure stress

E_{uR} = drained secant Young modulus from TX-CK$_o$D-CL evaluated from the unload-reload loop carried out during the application of deviatoric stress

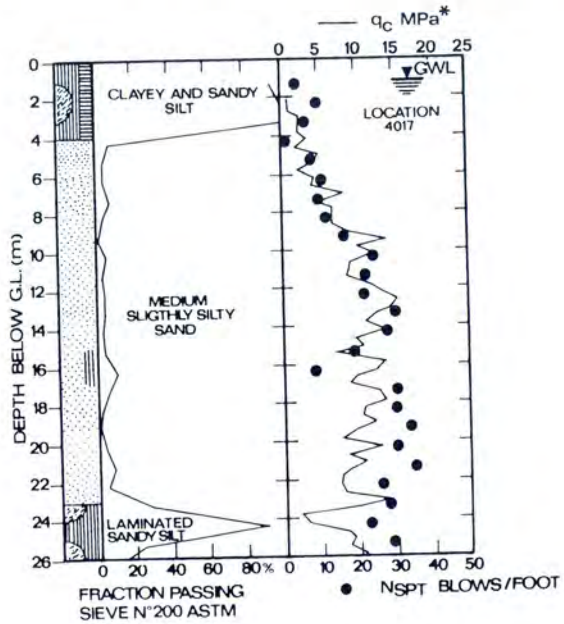
M_o = constrained modulus from 1-D compression run on the CC specimens during 1st stage of the test

NC = normally consolidated

OC = overconsolidated

SANDS AT TWO TEST SITES

The relevant soil properties of the two sites considered, from which field data are available, are shown in Fig.2 and Fig.3.



(*) ELECTRICAL CPT DATA FILTERED ACCORDING VIVATRAT [42]

FIGURE 2. SOIL CONDITIONS AT MANTOVA SITE

At the Mantova site, the following in situ techniques were used: SBPT, CPTU, CPT, DMT, SPT and v_s-CH, where v_s-CH = shear wave velocity determined using the cross-hole technique.

The tests were performed under strictly controlled conditions within the limits of quality assurance. The SPT's were carried out in 75 mm OD holes supported by casing filled with bentonite mud. The SPT spoon was driven using the trip hammer, for which the measured rated energy at the top of the rod was 65% of the theoretical maximum.

The CPTU penetration at the site indicated that the sand strata present between 4 and 25 m below the G.L. were penetrated under fully drained conditions.

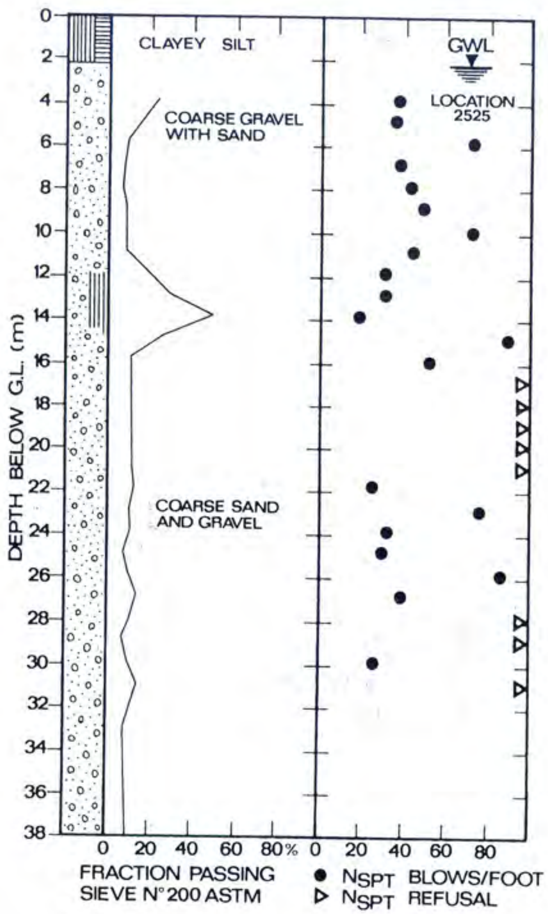


FIGURE 3. SOIL CONDITIONS AT TRINO VERCELLESE SITE

The stress history of the deposit has been assessed on the basis of the geological information and laboratory tests run on undisturbed specimens taken from the cohesive strata and lenses embedded in the sand deposit. The data leads to the conclusion that the deposit under consideration is slightly OC, partially due to aging and partially due to small erosion and limited G.W.L. oscillations [5]. On the basis of this information and of the total horizontal stress measured during the SBPT's it was assumed that the coefficient of

earth pressure at rest K_o, within the sand strata encountered in the first 25 meters, is approximately equal to 0.55. This is thought to depend on the slight mechanical overconsolidation (erosion and G.W.L. oscillations) [18].
At the Trino Vercellese site (Fig.3) coarse gravel and sand deposits predominate: as a result, the in situ tests discussed here are limited to v_s-CH and SPT. The latter being performed in the same manner as for the Mantova site. The stress history of this site is even more difficult to assess than at the Mantova site. Based mainly on geological information, it can be argued that the soil strata within the depth of 5 to 30 m below G.L. has not been subjected to any appreciable erosion. However the soil may be lightly overconsolidated (OCR < 1.5) due to environmental factors and GWL oscillations.

SELF-BORING PRESSUREMETER TEST (SBPT)

If a long pressuremeter probe is expanding in an elastic soil, the surrounding soil is only subjected to shear. Thus it should be realized that the test only measures the shear modulus of soil G [43], [46].
Three different shear moduli may be computed from the SBP drained expansion curve as shown in Fig.4: initial tangent

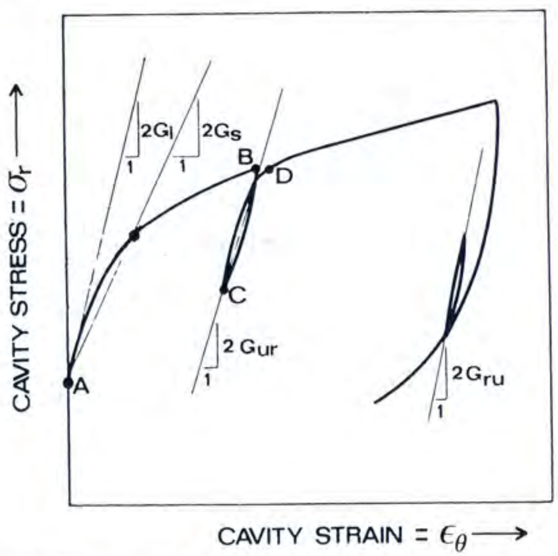


FIGURE 4. ILLUSTRATION OF THE DETERMINATION OF THE THREE SHEAR MODULI FROM THE SBP TEST

modulus G_i, secant modulus G_s, unload reload modulus G_{uR} or reload-unload modulus G_{Ru}.

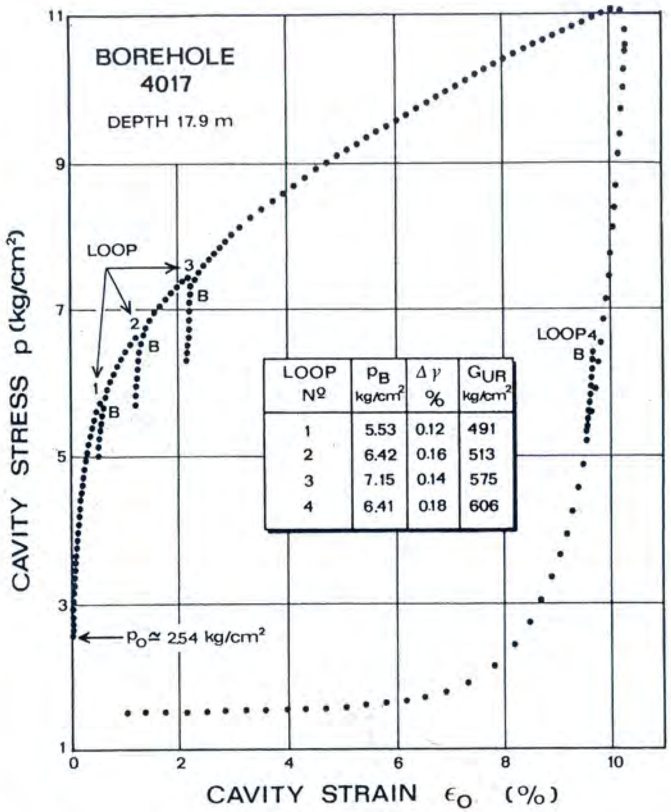


FIGURE 5. A TYPICAL EXPANSION CURVE

Fig.5 shows a typical expansion curve from Camkometer test in Po River sand at the Mantova site. Three unload-reload loops (1 to 3) and one reload-unload loop have been performed. The slope of these loops correspond approximately to twice the G_{uR} and G_{Ru} of the tested sand. In a similar manner the slope of the secant to any point on the expansion curve is conventionally considered [1] to be twice the secant shear modulus G_s.

Tables 2 to 3 present values of G_{uR} and G_{Ru} obtained from the Camkometr and PAF-76 tests run in Po River sand at location 4017. Table 4 gives similar data resulting from Camkometer tests run in CC.

TABLE 2

PO RIVER SAND - BOREHOLE N.4017 - CAMKOMETER PROBE

n=0.80; γ=1.85 t/m^2; Depth to G.W.L.=1.1 m; (*) Reload-unload loop

	1st Loop				2nd Loop			
Z (m)	p_B (kg/cm^2)	G_{uR} (kg/cm^2)	K_G (-)	G^c_{uR} (kg/cm^2)	p_B (kg/cm^2)	G_{uR} (kg/cm^2)	K_G (-)	G^c_{uR} (kg/cm^2)
7.4	2.71	255.0	214.2	104.3	3.18	275.0	201.2	97.9
8.9	3.18	361.0	269.9	149.2	3.76	353	227.9	125.9
10.4	3.69	285.0	190.6	117.6	4.17	308	184.8	114.0
11.9	4.41	337.0	195.6	132.9	4.89	366.0	193.8	131.7
13.4	7.20	573.0	219.0	162.1	8.27	554.0	187.9	139.1
14.8	6.39	416.0	178.2	141.9	7.28	465.0	177.6	141.4
15.9	5.79	493.0	232.6	195.4	6.63	502.0	209.7	176.1
17.9	5.53	491.0	245.9	225.5	6.42	513.0	223.9	205.3
19.4	5.87	489.0	234.7	228.5	6.61	490.0	210.6	205.1
20.9	6.63	600.0	260.0	267.8	7.47	619.0	240.3	247.5
22.8	7.19	612.0	249.3	274.2	7.63	658.0	253.6	278.9
24.9	6.04	499.0	244.3	287.2	6.45	495.0	227.2	267.2
	3rd Loop				4th Loop			
7.4	3.07(*)	274.0	206.6	100.5	2.55(*)	240.0	212.7	103.5
8.9	4.38	374.0	211.6	116.9	3.67(*)	340.0	224.2	123.9
10.4	3.52(*)	303.0	211.4	130.4	-	-	-	-
11.9	5.33	392.0	192.5	130.8	4.73(*)	396.0	216.0	146.8
13.4	7.09(*)	574.0	222.3	164.6	-	-	-	-
14.8	8.01	445.0	156.4	124.5	-	-	-	-
15.9	7.22	509.0	197.2	165.6	-	-	-	-
17.9	7.15	575.0	227.7	208.8	6.41(*)	606.0	264.9	242.9
19.4	6.69(*)	589.0	250.4	243.8	-	-	-	-
20.9	8.29	641.0	226.3	233.1	7.55(*)	715.0	274.9	283.1
22.8	8.05	606.0	222.3	244.5	7.58(*)	670.0	259.8	285.8
24.9	6.87	457.0	197.3	232.0	6.46(*)	536.0	245.6	288.8

(1 kg/cm^2 = 98.1 kPa)

At present it is well recognized that if the amplitude of the loops is properly limited, i.e. so that the surrounding soil does not fail in extension [44], the values obtained for G_{uR} and G_{Ru} represent a reliable measure of the so called "elastic" shear stiffness of the tested sand. This statement applies to the drained expansion test, for which it may be postulated, within the framework of the theory of elasto-plasticity, that any unload of the expanding cavity wall will bring the surrounding soil below the currently expanded yield surface in the domain where strains are small and to a large extent reversible [15], [32].

TABLE 3

PO RIVER SAND - BOREHOLE N.4017 - PAFSOR PROBE

$n=0.81$; $\gamma=1.85$ t/m^3; Depth to G.W.L.=1.1 m;

	1st Loop				2nd Loop			
Z (m)	p_B (kg/cm^2)	G_{uR} (kg/cm^2)	K_G (-)	G^c_{uR} (kg/cm^2)	p_B (kg/cm^2)	G_{uR} (kg/cm^2)	K_G (-)	G^c_{uR} (kg/cm^2)
7.50	2.15	168.4	173.9	84.7	-	-	-	-
9.00	5.01	394.0	197.2	109.0	-	-	-	-
10.5	3.61	290.9	197.7	122.1	-	-	-	-
12.0	5.25	393.6	194.3	132.2	5.25	393.6	194.3	132.2
13.5	4.82	317.6	171.8	127.4	-	-	-	-
15.0	4.71	347.1	194.8	156.2	6.18	412.6	180.5	144.8
16.5	6.64	423.9	175.7	151.4	-	-	-	-
18.0	7.54	356.6	133.1	122.4	-	-	-	-
19.5	5.99	418.4	195.3	190.9	7.66	511.2	189.9	185.6
21.0	7.24	438.8	173.4	179.3	-	-	-	-
22.5	6.56	467.5	205.1	223.5	8.39	550.8	191.7	208.9
24.0	7.02	451.4	187.6	214.9	-	-	-	-

(1 kg/cm^2 = 98.1 kPa)

Despite this qualification of the G_{uR} and G_{Ru}, there is still the problem of how to use these moduli in design. This requires assessment of the average stress and shear strain levels relevant to the moduli [32]. As for all boundary value problems this is difficult to assess and requires a number of simplifying hypotheses.

Concerning the relevant stress level, the existing practice is to refer to the same kind of average stress existing around the expanding pressuremeter probe. This average stress may be either the octahedral effective stress [32] or the mean values of the plane strain effective stress [14]. In the present paper the latter stress will be adopted.

When a value for reference stress is selected, the following tentative procedure can relate the measured G_{uR} and G_{Ru} values to the level of effective stress:

a. Consider the value of G_{uR} corresponding to a given value of the shear strain amplitude of the loop $\Delta\gamma$ and to the cavity stress p_B from which the loop starts (see Fig.4).

b. Compute the weighted average of the current stress around the probe (σ'_{av}) with an appropriate constitutive law. For perfect elasto-plasticity, this is approximately the initial horizontal effective stress (σ'_{ho}) plus 0.4 to 0.5 times the net effective cavity stress increase.

TABLE 4

CALIBRATION CHAMBER TESTS - CAMKOMETER PROBE

NC TS: n = 0.65 OC HS : n = 0.75 (assumed)

OC TS: n = 0.75 (*) Reload-unload loop

Sand Type	DR (%)	OCR (-)	σ'_{vo} (kg/cm^2)	K_o (-)	p_B (kg/cm^2)	G_{uR} (kg/cm^2)	K_G (-)	G^c_{ur} (kg/cm^2)
					1st Loop			
TS	43.2	1	1.11	0.40	1.22	256.5	289.1	170.5
TS	49.2	1	1.14	0.44	1.39	351.4	364.4	232.7
TS	53.3	1	5.01	0.48	5.47	772.2	318.2	555.1
TS	67.4	1	5.02	0.47	5.05	738.2	315.1	550.6
TS	64.6	2.9	1.09	0.75	2.78	489.2	315.0	270.8
TS	47.5	2.2	1.11	0.74	2.68	488.4	320.9	276.9
TS	42.4	1	1.10	0.48	1.42	329.3	335.0	221.2
TS	92.3	1	5.04	0.44	5.60	955.1	393.8	660.8
TS	46.3	7.7	0.60	0.93	2.11	417.5	324.1	246.2
TS	65.4	7.7	0.70	0.99	2.64	467.8	318.9	242.3
TS	65.9	5.4	1.15	0.90	3.55	628.1	337.1	345.9
TS	47.2	1	3.19	0.47	3.35	524.4	294.9	383.8
TS	44.6	2.9	1.11	0.76	2.70	465.8	303.3	267.0
TS	46.2	5.5	1.14	0.86	3.12	548.9	320.4	315.6
TS	74.6	5.4	1.16	0.82	3.23	623.0	358.3	345.1
TS	74.6	5.5	1.15	0.79	3.25	495.8	286.3	266.5
HS	67.0	2.8	1.11	0.66	2.64	486.0	328.4	260.1
HS	43.9	3.3	1.08	0.59	2.56	368.8	259.4	185.0
					2nd Loop			
TS	43.2	1	1.11	0.40	1.51	278.6	282.9	166.9
TS	49.2	1	1.14	0.44	1.73	376.5	350.6	223.9
TS	53.3	1	5.01	0.48	6.63	808.6	303.4	536.7
TS	67.4	1	5.02	0.47	6.66	805.7	302.7	528.8
TS	64.6	2.9	1.09	0.75	3.74	515.8	278.1	239.1
TS	47.5	2.2	1.11	0.74	3.55	496.0	276.0	237.8
TS	42.4	1	1.10	0.48	1.90	353.6	311.6	206.2
TS	92.3	1	5.04	0.44	7.36	957.2	345.8	580.7
TS	46.3	7.7	0.60	0.93	2.59	428.5	304.9	196.9
TS	65.4	7.7	0.70	0.99	3.19	469.7	285.9	216.3
TS	65.9	5.4	1.15	0.90	4.44	625.7	293.8	302.6
TS	47.2	1	3.19	0.47	3.96	518.8	270.1	351.5
TS	44.6	2.9	1.11	0.76	3.23	449.8	263.8	232.2
TS	46.2	5.5	1.14	0.86	4.03	526.1	264.2	260.3
TS	74.6	5.4	1.16	0.82	4.20	636.8	313.2	301.7
TS	74.6	5.5	1.15	0.79	3.49	533.7	295.5	275.3
HS	67.0	2.8	1.11	0.66	4.30	517.1	258.9	204.5
HS	43.9	3.3	1.08	0.59	3.12	369.1	230.0	164.1

continues

(1 kg/cm^2=98.1 kPa)

continued table 4

Sand Type	DR (%)	OCR (-)	σ'_{vo} (kg/cm^2)	K_o (-)	p_B (kg/cm^2)	G_{uR} (kg/cm^2)	K_G (-)	G^c_{ur} (kg/cm^2)
					3rd Loop			
TS	43.2	1	1.11	0.40	1.31(*)	278.3	303.1	178.8
TS	49.2	1	1.14	0.44	2.28	416.3	336.1	214.2
TS	53.3	1	5.01	0.48	7.77	809.4	281.2	497.3
TS	67.4	1	5.02	0.47	8.27	831.4	280.7	490.4
TS	64.6	2.9	1.09	0.75	4.88	533.1	243.0	209.4
TS	47.5	2.2	1.11	0.74	4.15	524.5	265.0	228.4
TS	42.4	1	1.10	0.48	2.40	375.5	293.0	193.9
TS	92.3	1	5.04	0.44	9.01	1079.3	351.6	590.5
TS	46.3	7.7	0.60	0.93	3.10	416.8	265.0	171.1
TS	65.4	7.7	0.70	0.99	3.75	480.7	264.3	200.1
TS	65.9	5.4	1.15	0.90	5.48	662.9	273.2	281.4
TS	47.2	1	3.19	0.47	4.64	552.2	266.4	346.6
TS	44.6	2.9	1.11	0.76	3.79	495.7	264.1	231.8
TS	46.2	5.5	1.14	0.86	4.78	526.2	238.0	234.4
TS	74.6	5.4	1.16	0.82	4.99	598.1	264.4	254.4
TS	74.6	5.5	1.15	0.79	4.36	553.1	267.4	249.2
HS	67.0	2.8	1.11	0.66	5.94(*)	625.2	253.3	200.7
HS	43.9	3.3	1.08	0.59	3.12(*)	448.1	279.1	199.7
					4th Loop			
TS	43.2	1	1.11	0.40	–	–	–	–
TS	49.2	1	1.14	0.44	1.55(*)	393.1	386.9	246.5
TS	53.3	1	5.01	0.48	6.84(*)	901.7	333.5	589.1
TS	67.4	1	5.02	0.47	13.13(*)	1028.1	271.8	474.9
TS	64.6	2.9	1.09	0.75	6.65(*)	598.7	228.8	192.0
TS	47.5	2.2	1.11	0.74	5.67(*)	591.8	244.8	210.9
TS	42.4	1	1.10	0.48	3.88(*)	485.1	290.2	192.0
TS	92.3	1	5.04	0.44	16.11(*)	1243.9	295.0	494.9
TS	46.3	7.7	0.60	0.93	4.09(*)	480.6	255.3	165.2
TS	65.4	7.7	0.70	0.99	5.17(*)	561.6	250.8	189.9
TS	65.9	5.4	1.15	0.90	6.32(*)	799.9	301.1	310.1
TS	47.2	1	3.19	0.47	4.48(*)	619.8	304.1	395.8
TS	44.6	2.9	1.11	0.76	3.42(*)	512.1	290.5	254.9
TS	46.2	5.5	1.14	0.86	5.45(*)	640.8	213.5	210.3
TS	74.6	5.4	1.16	0.82	7.67(*)	795.7	266.0	256.0
TS	74.6	5.5	1.15	0.79	–	–	–	–
HS	67.0	2.8	1.11	0.66	–	–	–	–
HS	43.9	3.3	1.08	0.59	–	–	–	–

($1\ kg/cm^2 = 98.1\ kPa$)

c. Compute the modulus K_G in the Janbu [20] empirical formula using this value of average stress and measured value for the modulus:

$$G = K_G \cdot p_a \left(\frac{\sigma'_{av}}{p_a} \right)^n$$

where:

K_G = modulus number [-]

n = modulus exponent [-]

p_a = reference stress $[FL^{-2}]$

σ'_{av} = average effective stress around the probe $[FL^{-2}]$

d. Evaluate the shear modulus G for the desired level of average stress.

The procedure outlined above requires a value of modulus exponent n. Conventionally for cohesionless soils n is taken equal to 0.4 to 0.6 [5], [32] without taking into account the strain level dependency of n as postulated by Wroth et al., 1979 [46]. In order to avoid this uncertainty the exponent n used in the present paper has been obtained in the following manner:

a. For the Po River sand the values of log G_{uR} have been plotted vs log σ'_{av} as shown in the example in Fig.6,

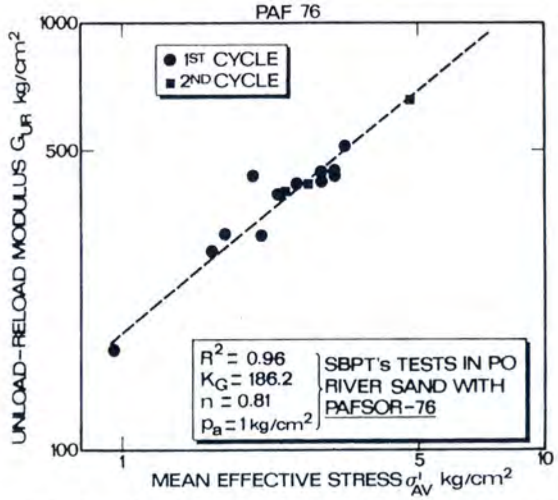


FIGURE 6. MODULUS EXPONENT FOR PO-RIVER SAND

giving n ≈0.80 and n=0.81 for Camkometer and PAFSOR tests respectively.

b. For the SBPT's run in CC on TS the value of n has been determined by the same procedure but separating the availabe tests according to their relative density and OCR.
The values of n are given in Table 5.

TABLE 5
MODULUS EXPONENT OF TICINO AND PO - RIVER SANDS

Sand	Probe	n	R^2	N	Notes
NC Ticino Sand	Camkometer	0.65	0.95	14	CC Tests; D_R≈45%
OC Ticino Sand	Camkometer	0.75	0.78	8	CC Tests; D_R=44±3%; $2\leq OCR\leq 8$
OC Ticino Sand	Camkometer	0.75	0.78	10	CC Tests; D_R≈68±5%; $2\leq OCR\leq 8$
Po-River Sand	Camkometer	0.80	0.80	22	BH-4017; D_R≈68±12%
Po-River Sand	PAFSOR-76	0.81	0.96	15	BH-4017; D_R≈68±12%;

Following the previously outlined procedure the value of K_G has been computed for all values of G_{uR} and G_{Ru}, and thereafter the "corrected" G^c_{Ru} values relevant to the values of σ'_{ho} have been evaluated. The results obtained are shown in Tables 2 to 4.
Plots of G^c_{uR} obtained from the SBPT's at the Mantova site for the Po River sand are given in Fig.7 and compared to G_o resulting from the v_s-CH measurements.
The available data indicate the following:

a. Both the Camkometer and Pafsor probes provided comparable and consistent values of G_{uR} and G_{Ru} at the Mantova site for the imposed $\Delta\gamma$. This occurs despite the fact that for the latter device all the expansion curves had lift off pressure p_o very close to the hydrostatic pore pressure u_o. The value of p_o measured during the Camkometer tests was appreciably higher leading to the average value of K_o≈0.56 [5]. This indicates the relative insensitivity of the mesaured G_{uR} and G_{Ru} to the initial disturbance

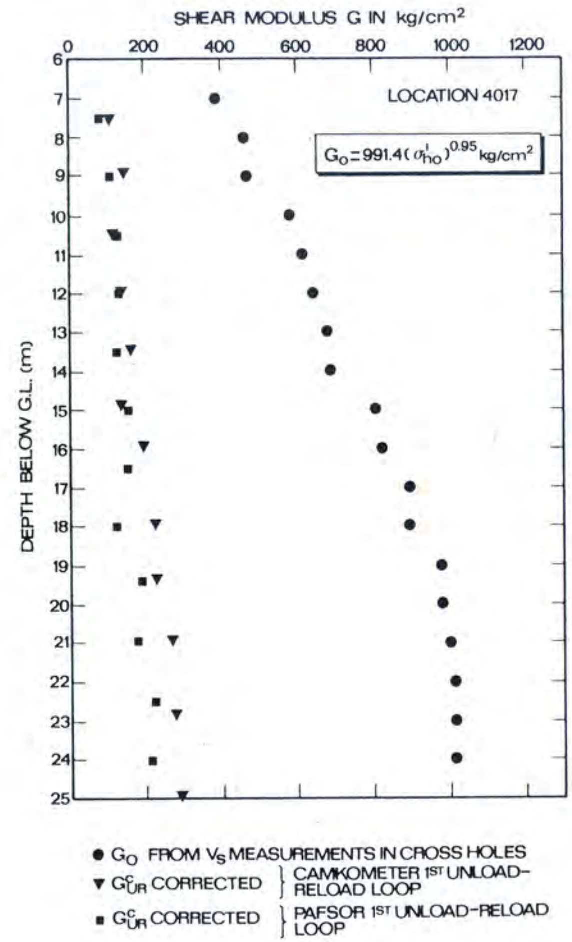


FIGURE 7. SHEAR MODULUS OF PO RIVER SAND

caused by insertion of the probe, whilst on the contrary G_s values are very sensitive to sample disturbance [44], [45], [23], [13], [34].

b. G^c_{uR} and G^c_{Ru} values obtained from the CC tests were independent of OCR. This confirms once more that they should be considered as elastic shear moduli representative for stress field inside the area limited by the yield and large strain strength envelopes.

c. Due to the scatter of the available data it is difficult to establish a relation between K_G and D_R.

d. The G_o vs σ'_{ho} trend can be fitted by:

$$G_o = 991.4\ p_a\ (\sigma'_{ho}/p_a)^{0.95} \quad \text{with } R^2 = 0.93$$

The initial shear modulus G_o relates to the elastic shear moduli G^c_{uR} obtained from the 1st unload-reload loop of the SBPT's:

$$3.6 \leq \frac{G_o\ (\gamma \approx 10^{-5}\%)}{G^c_{uR}\ (\gamma \approx 1.7\ .\ 10^{-1}\%)} \leq 4.8, \text{ for the Camkometer tests}$$

$$4.6 \leq \frac{G_o\ (\gamma \approx 10^{-5}\%)}{G^c_{uR}\ (\gamma \approx 4.7\ .\ 10^{-1}\%)} \leq 5.9, \text{ for the Pafsor tests}$$

This is similar to the findings of Robertson and Hughes [34].
The ratio of G_o obtained from resonant column tests and G^c_{uR} for the 1st loop from Camkometer tests run in the CC in NC TS is:

$$2.5 \leq \frac{G_o\ (\gamma \approx 10^{-4}\%)}{G^c_{uR}\ (\gamma \approx 1.5\ .\ 10^{-1}\%)} \leq 3.6$$

STATIC CONE PENETRATION

Since 1940's [12] engineers have attempted to assess the deformation moduli of sands from the measured cone resistance q_c.
Most of these correlations suffer from a large scatter of the inferred moduli, and of the assumption of the constrained modulus as a reference, which is seldom directly relevant in the design problems.
Lambrechts and Leonards first demonstrated that the stress and strain history has a marked influence on the deformation moduli of the sand, but much less on the cone resistance. These findings have been confirmed by means of the special tests performed in the large CC [19], an example of which is shown in Fig.8. The pluvially deposited specimen of TS has been subjected to 1-D straining up to point A along the K_o-line. At this point the cone was penetrated to the depth of 70 cm. Subsequently the specimen was subjected to 10 load-

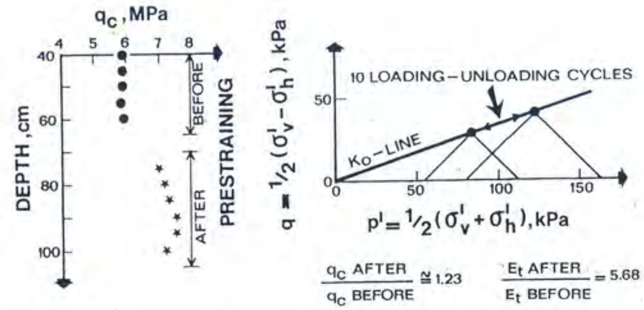


FIGURE 8. EFFECT ON CONE RESISTANCE OF PRESTRAINING ALONG K_o-LINE, TS, TEST No.71, D_R=36%, BC-1

unload cycles between points A and B along the K_o-line. At the end of this prestraining process, whose state of the stress within the specimen corresponds to point A, the cone penetration was completed, reaching the depth of 120 cm.
The experimental evidence yielded by this kind of test showed that q_c increases only by a small amount as a result of the applied strain history while the measured tangent Young modulus (E_t) of the sand increases appreciably.
In light of these findings the correlations between E_{25} and q_c from approximately 170 CC tests run on TS and HS and summarized in Table 6 [2], [18] must be considered. In fact the large differences in the E_{25}/q_c as obtained respectively for NC and OC sands must be attributed to the already mentioned lack of sensitivity of q_c to the strain and stress history of the sand. Generally E_{25}/q_c decreases with D_R increasing.

TABLE 6
E_{25} vs q_c FROM CC TESTS ON TS AND HS AT DIFFERENT D_R VALUES

Sand	OCR	D_R (%)	E_{25}/q_c
TS	1	58 to 82	2.3 to 1.8
TS	2 to 8	54 to 92	20 to 9
HS	1	43 to 91	2.3 to 1.3
HS	2 to 8	39 to 91	19 to 7

The values for NC sands in Table 6 agree with those suggested by Schmertmann [38], [40]. For OC sands, the ratios of E_{25}/q_c are much higher than those previously suggested by other researchers. The above data are not corrected as regards the chamber size effect [2], [30] which is negligible in loose sands but leads to an underestimate of q_c in dense and very dense sand.

The CC tests yield the values of M_o to q_c ratio shown in Table 7. For NC sand, M_o corresponds to the tangent constrained modulus of the last load increment applied to the CC specimen before the cone penetration. In OC sand, M_o corresponds to the secant constrained modulus of the whole unload stage performed before the cone penetration. The M_o/q_c ratio exhibits the same overall trend already mentioned in the case of E_{25}/q_c.

TABLE 7
M_o vs q_c FROM CC TESTS ON TS AT DIFFERENT D_R VALUES

Sand	OCR	D_R (%)	M_o/q_c
TS	1	40 to 90	7.5 to 4.5
TS	1.5 to 15	40 to 90	28 to 10.5

As regards the field correlations between q_c and the deformation moduli, the data available for the Po River sand from Mantova site allow the relations of q_c vs $G_o(v_s\text{-CH})$ and vs $G_{uR}(SBP)$ to be explored, where the latter was obtained from the drained pressuremeter expansion test performed using the Camkometer probe:

$$\frac{G_o\ (v_s\text{-CH})}{q_c} = 6.9 \pm 1.7; \qquad \frac{G_{uR}(SBP)}{q_c} = 1.7 \pm 0.6$$

where:

G_{uR} = corrected modulus from 1st unload-reload loop having the shear strain amplitude $\Delta\gamma \approx 1.7 \cdot 10^{-1}\%$

q_c = cone resistance "filtered" according Vivatrat [42]

The relationship between G_o and q_c for the Po River sand is similar to the empirical relationship proposed by Robertson [32].

FLAT DILATOMETER

The evaluation of sand moduli, on the basis of the flat dilatometer test results [25], [26], is covered by the companion paper [3] presented to this symposium. In virtue of this only the relevant conclusions based on the results of CC tests performed in TS and HS are here recalled:

a. The assessment of the constrained modulus M_D, on the basis of DMT results following Marchetti's [25] procedure, when compared to the M_o measured in the CC, leads to:
   - a moderate underestimate ($\approx$ 30%) of M_o for NC sands;
   - a pronounced underestimate ($\approx$ 200 to 400%) of M_o for OC sands.

b. This reflects a reduced sensitivity of the dilatometer parameters [3] K_D, E_D and consequently of M_D to the strain history imposed on the CC specimen, which, on the contrary, has a paramount influence on the measured M_o. This fact is evident by the results of the special CC tests with prestraining along K_o-line (analogous to the one showed in Fig.8) and shown in Fig.9.

c. The dilatometer modulus E_D has been empirically correlated to other deformation moduli of sand [18], [35], [3]. The direct comparison between E_D and E_{25}, as obtained from CK_oD triaxial compression tests run on pluvially deposited specimens of TS and HS, indicated that:
   - $E_{25}/E_D \approx 0.9 \pm 0.3$ for NC sands;
   - $E_{25}/E_D \approx 3.5 \pm 1.2$ for OC sands.

   The ratio E_{25}/E_D, close to the one for NC sands, confirms similar findings presented by Campanella et al. [8].

d. Researchers [3] [18] have also tried to find the direct correlation of E_D against G_o. This has led to the following tentative results:
   - Making reference to G_o as obtained from RCT's, run on pluvially deposited specimens of NC Ticino sand, the value obtained for G_o/E_D is 3 ± 0.5.
   - This ratio is similar to the one obtained for the Po river sand at the Mantova site where $G_o(v_s\text{-CH})/E_D$ results = 2.2 ± 0.7.

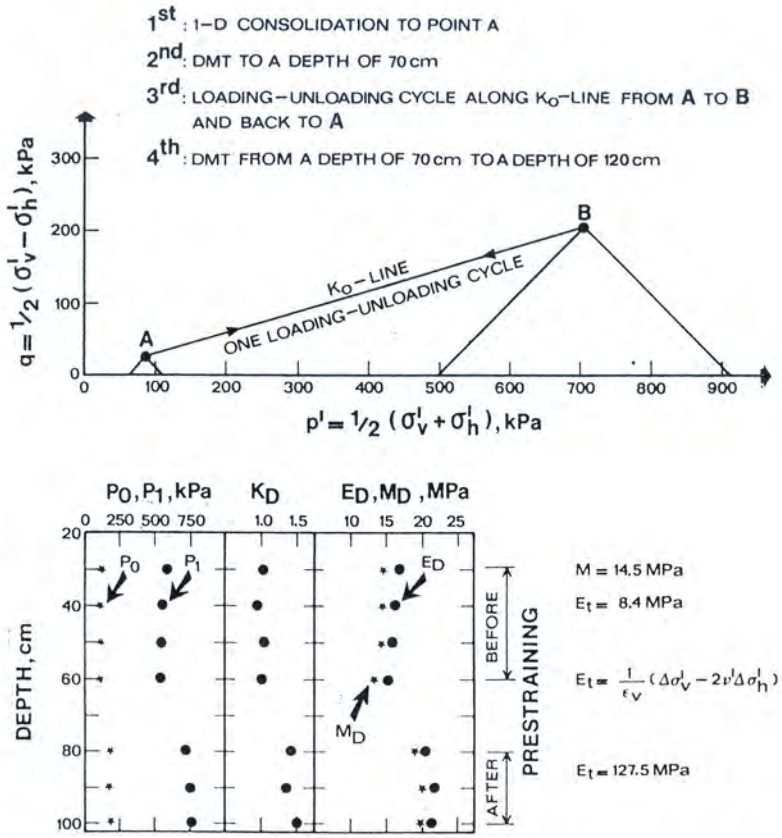


FIGURE 9. EFFECT ON DMT RESULTS OF PRESTRAINING ALONG K_o-LINE, TS, TEST No.139, D_R=40%, BC-1

STANDARD PENETRATION TEST

The oldest in situ technique used by engineers to evaluate the deformation moduli or directly the settlements of footings in cohesionless deposits is the SPT [41], [31], [6], [7].
It has the same disadvantages as other penetration devices already mentioned when discussing CPT and DMT. This is reflected in the different ratio of $[E/(1-\nu^2)]/N_{SPT}$ indicated in [10], [11] for NC and OC sands. Further confirmation of the modest sensitivity of SPT to the stress and strain history of sands has been demonstrated by a series of CC tests performed recently at the University of

Surrey in UK [9].
In these circumstances it is better to refer to the correlations between some average values of the penetration resistance and the settlement of foundations rather than use direct links between a single value of N_{SPT} and the moduli of cohesionless soils. A notable contribution in this respect is the recent work by Burland and Burbridge [7] which presents a comprehensive method for evaluating the settlements of foundations on sands and gravels from a statistical analysis of a large number of the well documented case records.
Possible exceptions to that stated above are the number of empirical methods developed in Japan and in USA [37] linking N_{SPT} values to the maximum shear modulus G_o. Among these methods the one which takes into account the influence of the age of deposit, soil grain size distribution and depth is suggested by Ohta and Goto [29]. The availability of both v_s-CH measurements and of carefully (3" holes supported by bentonite mud, tests performed every 1.5 m using a trip hammer which delivers a rated energy equal to 60-65% of the theoretical maximum value) performed SPT's at the sites of Mantova and Trino Vercellese offer an excellent opportunity to evaluate the above mentioned empirical correlation for these two cohesionless deposits.
The relationship between G_o and N_{SPT} suggested by Ohta and Goto [29] is the following:

$$G_o = \left[54.33 \cdot N_{SPT}^{0.173} \; \alpha \; \beta \left(\frac{z}{0.303}\right)^{0.193}\right]^2 \cdot \frac{\gamma}{98.1} \qquad [\mathrm{kg/cm^2}]$$

where:
α = coefficient depending on the age of the deposit;
- α = 1, recent Holocene deposits
- α = 1.3, Pleistocene and Tertiary deposits

β = coefficient depending on the grain size distribution;
- β = 1, silts and clays
- β = 1.09, sands
- β = 1.19, sands and gravels.

The comparison between v_c-CH measured at the sites of Mantova and Trino Vercellese and those assessed as function of N_{SPT}, using Ohta and Goto [29] formula , leads to the following results:

a. For the Po-River sand at Mantova site see Fig.10:
$$\frac{v_s\text{-CH}}{v_s\text{-}N_{SPT}} = 0.93 \pm 0.07; \text{ which gives: } \frac{G_o(v_s\text{-CH})}{G_o(N_{SPT})} = 0.86 \pm 0.14$$

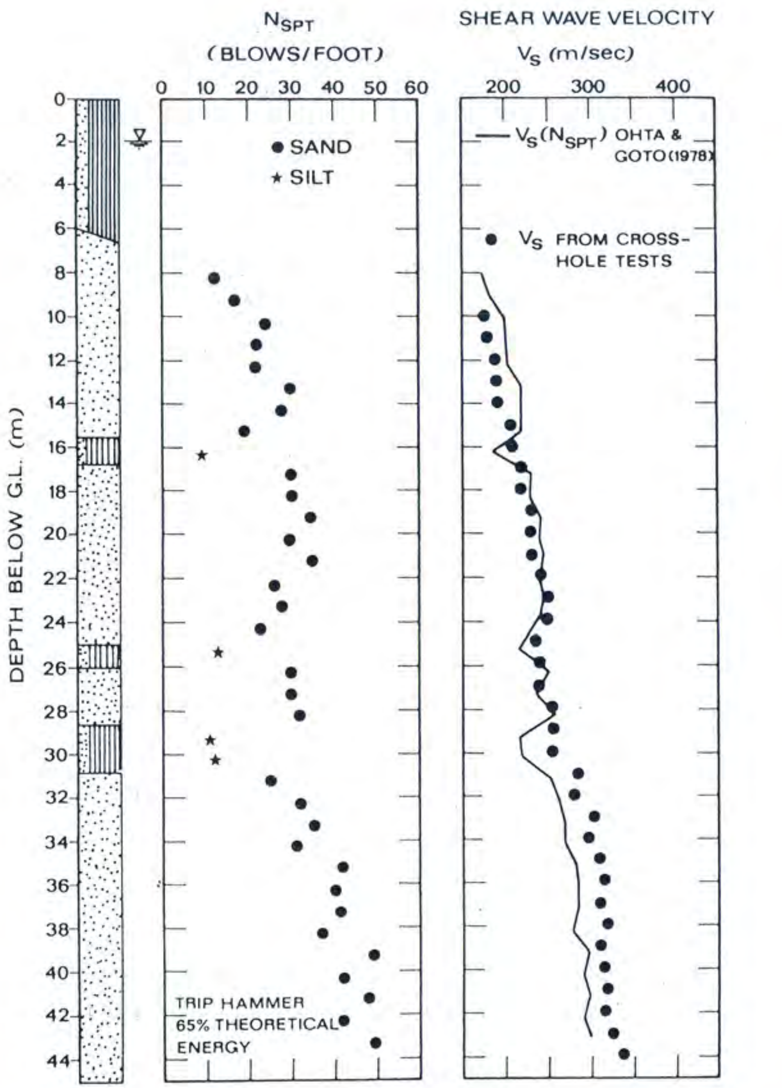


FIGURE 10. SHEAR WAVE VELOCITY IN PO RIVER SAND - CROSS-HOLE VS N_{SPT} DATA

b. For the Po-River sand and gravel at Trino Vercellese site:

$$\frac{v_s\text{-CH}}{v_s\text{-}N_{SPT}} = 1.39\pm0.17; \text{ which gives: } \frac{G_o(v_s\text{-CH})}{G_o(N_{SPT})} = 1.94\pm0.48$$

The above data indicates that in recent Po River sand the SPT's allow a reasonable estimate of both v_s and G_o. At the Trino Vercellese site the v_s and G_o assessed as function of SPT appear less reliable but on the safe side. This disagreement might be attributed to the older age and to the gravelly nature of the considered deposit.

CONCLUDING REMARKS

The experience gained by the National Electricity Board of Italy (ENEL), ISMES and Technological University of Turin (TUT) allow the following remarks to be made as concerns the assessment of deformation parameters from in situ tests:

(a) The unloading-reloading modulus G_{uR} obtained from SBPT's is highly repeatable and little influenced by possible initial disturbance due to probe installation. G_{uR} has a clear significance within the frame of an elasto-plastic theory. Before using the G_{uR} in practice the influence of the average mean effective stress, the magnitude of shear strain and the soil anisotropy must be clearly taken into consideration.

(b) The cone penetration resistance depends on the existing state of effective stress and does not reflect the strain history of the deposit. However, the deformation modulus is strongly influenced by the experienced plastic strain, so that no unique relationship can exist between the modulus and the q_c, but this ratio is highly dependent on the deposits strain and stress-history. In OC sands, this ratio depends also on the relative density.

(c) The DMT parameters are only moderately sensitive to the strain history. This sensitivity increases as the relative density decreases. However this is far from being able to compensate for the increase of the sand deformation modulus.

(d) A rather surprising agreement has been found, at the two sites investigated, between the dynamic shear modulus G_o as obtained from cross-hole tests and the values derived by N_{SPT} using the empirical correlation by Ohta and Goto [29].

(e) When the shear strain level is taken into account, the G_o and the G_{uR} values can be linked in a unique consistent framework.

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