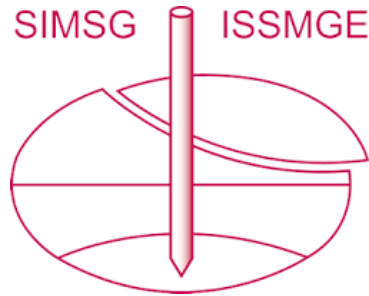


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Cone Resistance of a Dry Medium Sand

Résistance au Pénétromètre d'un Sable Moyen Sec

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SUMMARY.

The paper presents a comparison between static cone resistance q_c of dry dense and very dense medium sand measured during tests in a large calibration chamber with those computed on the basis of the theories proposed by Durgunoglu and Mitchell (1975) and by Vesic (1975, 1977). In the evaluation of cone resistance from the above mentioned approaches, the stress-strain-strength properties of sand determined in triaxial laboratory tests were used.

1. INTRODUCTION.

The paper presents some results of the research undertaken by ENEL-CRIS (Milano) and POLITECNICO di Torino with the aim to calibrate, under very carefully controlled conditions, the Electrical Fugro-type static penetration tip in sand.

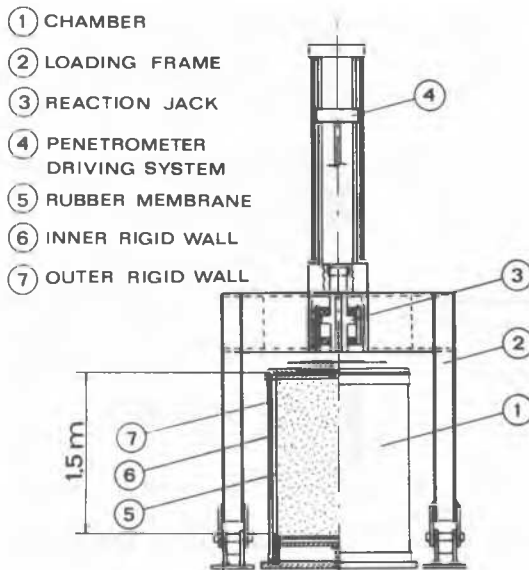


FIG. 1 - Scheme of the calibration chamber.

For this purpose a large calibration chamber has been developed: it houses samples 1.2 m wide and 1.5 m high and allows the performance of cone penetration tests (CPT) under selected boundary conditions.

A scheme of the calibration chamber and the boundary conditions are shown in fig. 1 and 2 respectively. A detailed description of the apparatus used and of the stages of the test are given in Bellotti et al. (1979-a, 1979-b).

Alongside with the calibration of CPT tip, triaxial tests (TX) were carried out on the same sand used in the calibration chamber and prepared in the same manner, in order to determine its stress-strain-strength characteristics.

In this way it was possible to make a comparison, for N.C. sand, between cone resistance measured in the

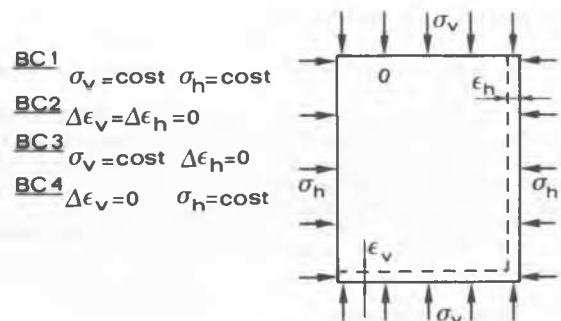


FIG. 2 - Boundary conditions during cone penetration test in calibration chamber.

calibration chamber and that evaluated by means of some theoretical approaches [Vesic (1975, 1977), Durgunoglu and Mitchell (1973, 1975)], in which strength and deformability properties, determined by triaxial tests, were introduced.

2. SAND CHARACTERISTICS.

The sand used in the tests is described in fig. 3.

3. SPECIMENS PREPARATION.

The method of pluvial deposition was adopted to prepare specimens both for the calibration chamber and for triaxial tests. This method, exhaustively discussed by Jacobsen (1976) and Battaglio et al. (1979) and others, allows one to obtain specimens of very uniform density, with relative density (D_R) varying between 35% and 100%; moreover it leads for a given time of deposition, to specimens with well repeatable dry bulk density γ_d .

The description of the sand spreader used to prepare specimens for the calibration chamber is given in Bellotti et al. (1979-a); the TX specimens were manufactured using small laboratory sand spreaders developed by N.G.I., Battaglio et al. (1979) and ISMES; the device used is shown in fig. 4.

In the present paper two classes of sand density are considered, namely:

	Calibration Chamber		Triaxial tests	
	$\gamma_{d,AV}$ (KN/m ³)	$D_{R,AV}$ (%)	$\gamma_{d,AV}$ (KN/m ³)	$D_{R,AV}$ (%)
Dense sand	15.50	70.1	15.43	69.4
Very dense sand	16.03	91.5	15.97	83.9

4. CALIBRATION CHAMBER TESTS.

Seventeen calibration chamber tests are considered here. The tests were performed with N.C. sand specimens, using the boundary conditions BC1 and BC3, which hopefully cover the real field situation.

The results are shown in table 1: the cone point resistance " q_c " and local skin friction resistance " f_s " values given in table 1 were obtained at 75 cm penetration depth, corresponding to the midheight of the specimen, at which a well defined plateau has almost always been observed.

Table 1 reports some other relevant information obtained during the one-dimensional compression phase which precedes the penetration phase.

5. TRIAXIAL TESTS.

Triaxial tests were performed on pluviably deposited cylindrical specimens 3.82 cm in diameter and 7.64 cm in height using the stress path controlled triaxial cell [Bishop and Wesley (1975)] with the performance feed back system show in fig. 5 [see also Menzies et al. (1979)].

Isotropically consolidated and drained compression tests [TX-CID] were performed and the results can be summarized as follows:

5.1. Strength envelope.

For both classes of relative density the strength envelope is not linear (see fig. 6) and can be well

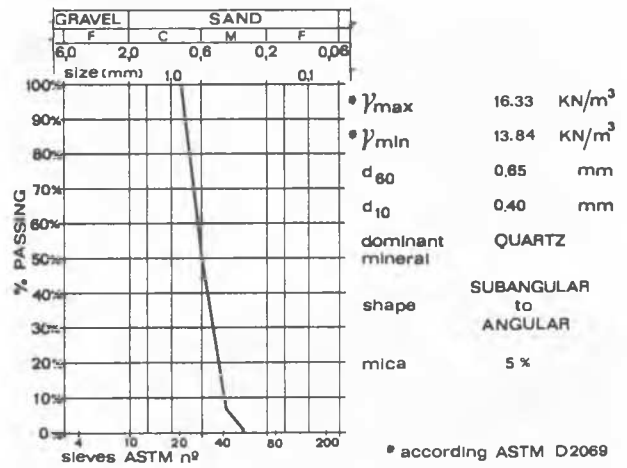


FIG. 3 - Characteristics of tested sand.

approximated by the following function, proposed by Baligh (1975, 1976):

$$\tau_{ff} = c + \sigma'_{ff} \left[\tan \phi_0 + \tan \alpha \left(\frac{1}{2.3} - \log_{10} \frac{\sigma'_{ff}}{\sigma_0} \right) \right] \quad \dots (1)$$

where:

- c = cohesion intercept
- ϕ_0 = angle of friction at the reference normal stress σ_0 ($\sigma_0 = 1 \text{ kg/cm}^2$, say)
- α = angle describing the curvature of the envelope; when α equals zero the envelope is straight

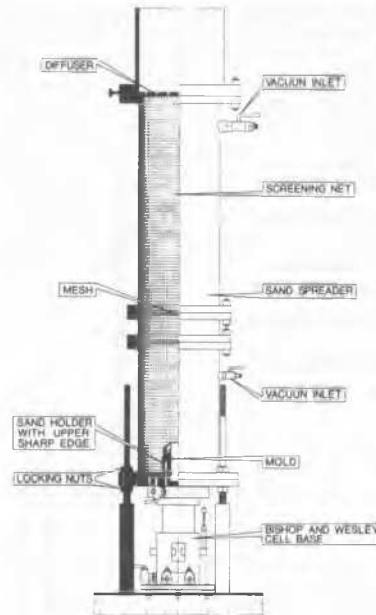


FIG. 4 - Scheme of sand spreader used to prepare specimens for TX tests.

TABLE 1
Calibration chamber test results

Class of Sand	Test No	D _{Ri} (%)	D _{RC} (%)	K _{oc} (-)	K _{op} (-)	σ' _{vc} (KN/m ²)	M _o (KN/m ² · 10 ²)	q _c (KN/m ² · 10 ²)	FR (%)	BC	φ _s ^S / φ _s ^C (°) / (°)	I _R ^S / I _R ^C (-) / (-)	z _{eq} (m)
VERY DENSE SAND D _{Ri} = 91.5 ± 2.8%	19	90.1	92.9	0.423	0.512	515.0	1467	464.5	0.70	3	39.7/41.4	105/160	32.0
	20	91.8	93.6	0.409	0.560	313.9	1231	381.6	0.73	3	40.3/41.9	118/179	19.5
	21	92.9	93.6	0.390	0.704	115.8	851	239.2	0.63	3	41.5/42.8	149/225	7.2
	28	91.8	93.6	0.405	0.530	312.9	1261	361.8	0.54	3	40.3/41.9	118/179	19.4
	34	86.3	87.6	0.421	0.820	65.7	755	184.4	0.59	3	42.0/43.0	168/255	4.1
	61	93.6	96.6	0.427	0.422	512.1	1444	437.0	0.92	1	39.7/41.4	105/160	32.0
	62	95.5	96.5	0.404	0.412	121.6	874	209.3	0.58	1	41.4/42.8	147/222	7.6
	76	89.8	90.5	0.373	0.373	68.7	766	119.6	0.53	1	42.0/43.0	166/253	4.3
DENSE SAND D _{Ri} = 70.1 ± 3.5%	22	69.0	71.2	0.423	0.487	311.0	1499	261.1	0.61	3	33.1/35.0	134/189	20.1
	23	68.2	69.4	0.416	0.531	113.8	761	156.5	0.65	3	34.7/36.2	164/230	7.4
	24	68.2	71.2	0.436	0.485	514.0	1365	344.3	0.56	3	32.2/34.2	127/171	33.2
	25	67.5	71.6	0.442	0.475	716.1	1508	407.1	0.50	3	31.6/33.6	111/153	46.4
	35	66.7	67.5	0.407	0.600	65.7	690	108.6	0.62	3	35.5/36.6	183/257	4.3
	50	68.2	69.3	0.412	0.343	115.7	779	135.6	0.69	1	34.7/36.2	164/230	7.5
	63	72.8	74.0	0.413	0.356	114.8	806	120.6	0.62	1	34.7/36.2	164/230	7.4
	65	72.8	75.2	0.432	0.422	313.9	1148	221.3	0.60	1	33.1/35.0	134/189	20.1
	70	77.2	80.6	0.445	0.441	509.1	1358	316.5	0.77	1	32.3/34.2	127/172	32.6

<p>D_{Ri} = initial relative density</p> <p>D_{RC} = relative density after consolidation</p> <p>K_{oc} = ratio $\frac{\sigma'_h}{\sigma'_v}$ at the end of consolidation</p> <p>K_{op} = ratio $\frac{\sigma'_h}{\sigma'_v}$ during the penetration</p> <p>σ'_{vc} = vertical consolidation stress</p> <p>M_o = constrained modulus at σ'_{vc}</p> <p>q_c = cone resistance</p>	<p>BC = boundary conditions during penetration test</p> <p>I_R^S, I_R^C = triaxial rigidity index</p> <p>φ_s^S, φ_s^C = φ_sⁱ values obtained from expanding cavity theory with non-linear strength envelope</p> <p>SUFFIX: C = Cylindrical; S = Spherical</p> <p>z_{eq} = equivalent depth, obtained from the ratio $\frac{\sigma'_{vc}}{\gamma_{di}}$ (γ_{di} = initial dry density)</p> <p>FR = friction ratio</p>
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τ_{ff} = shear stress on the failure surface at failure
 σ_{ff} = normal stress on the failure surface at failure

The relevant experimental parameters governing equation ... (1) are given below.

Dense sand: φ_o = 36° 12'; α = 7° 08'

Very dense sand: φ_o = 42° 90'; α = 6° 59'

5.2. Young's Modulus.

As far as Young's moduli (E') are concerned, they have been evaluated at stress level half the stress at failure (E'₅₀); from the experimental results we obtained the following relationships between E'₅₀ and σ'_c (effective consolidation stress).

Dense sand:
 $E'_{50} = 41155 \left(\frac{\sigma'_c}{\sigma'_o}\right)^{0.7304} \text{ kN/m}^2 \dots (2)$

Very dense sand:
 $E'_{50} = 46840 \left(\frac{\sigma'_c}{\sigma'_o}\right)^{0.7215} \text{ kN/m}^2 \dots (3)$
 σ'_o = reference stress = 100 kN/m²

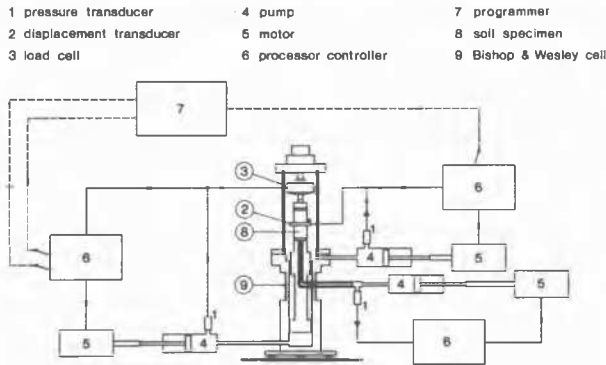


FIG. 5 - Automatic Programmable Triaxial Test system control loop.

5.3. Volumetric strain (ϵ_v).

Another relevant relationship derived from the triaxial test was ϵ_v vs. σ'_c , which can be expressed as follows:

Dense sand:

$$\epsilon_v = 0.0108 \left(\frac{\sigma'_c}{\sigma'_o}\right)^{0.5354} - 0.0200 \left(\frac{\sigma'_c}{\sigma'_o}\right)^{-0.6514} \quad \dots (4)$$

Very dense sand:

$$\epsilon_v = -0.0343 + 0.0037 \frac{\sigma'_c}{\sigma'_o} + 0.0100 \left(\frac{\sigma'_c}{\sigma'_o}\right)^{0.517756} \quad \dots (5)$$

Some uncertainties are connected with the correct evaluation of ϵ_v because of the rubber membrane penetration which can lead to a moderate overestimate of volumetric strain when equations ... (4) and ... (5) are used.

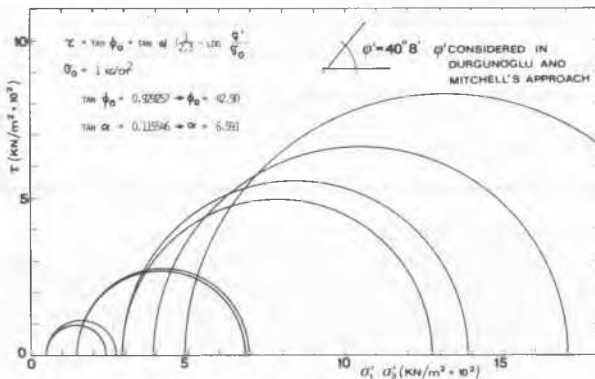


FIG. 6 - Triaxial Test results for very dense sand.

6. THEORETICAL q_c .

To evaluate q_c one has to refer to available computation procedures which are based on the classical theory of the plasticity in a rigid-plastic body or on the theory of expanding cavities of an elastic-perfectly plastic material; this latter allows one to take into account, in an approximate way, soil deformability in both elastic and plastic zones [see Cassan (1969), Vesic (1975), Al Awkati (1975) and others].

Among the numerous computation procedures available, on the basis of the preliminary calculation made by Manassero (1980) we decided to use at this stage:

- Durgunoglu and Mitchell's procedure (1973, 1975), as an example of classical bearing capacity theory.
- Vesic's approaches (1975, 1977), related to cylindrical and spherical expanding cavities.

Durgunoglu and Mitchell (1975) proposed the following expressions for the evaluation of cone resistance in sand:

$$q_c = \rho g B N_{\gamma q} \zeta_{\gamma q} \quad \dots (6)$$

where:

ρ = mass density

g = acceleration of gravity

B = width of penetrometer tip

$N_{\gamma q}$ = bearing capacity factor (on the basis of eq. (8) given in Durgunoglu and Mitchell's paper (1975))

$\zeta_{\gamma q}$ = shape factor (eq. (16), Durgunoglu and Mitchell)

Vesic's approach based on the theory of the cylindrical expanding cavity leads to the following approximate formula for q_c , when considering cohesionless material having curved strength envelopes:

$$q_c = p_c^U \lambda \left[1 + \tan \left(\frac{\pi}{4} + \frac{\phi'_s}{2} \right) \tan \phi'_s \right] \exp \left(\frac{\pi}{2} - \phi'_s \right) \quad \dots (7)$$

where:

p_c^U = ultimate pressure of the expanding cylindrical cavity in an elasto-plastic infinite medium

λ = empirical shape factor = $1 + \tan \phi'_s$ (Vesic, 1974)

ϕ'_s = secant angle of friction, related to the average effective stress in failure zone at failure

The corresponding equation for spherical expanding cavity is (Vesic, 1977):

$$q_c = \frac{p_u^S}{1 + \sin \phi'_s} \tan^2 \left(\frac{\pi}{4} + \frac{\phi'_s}{2} \right) \exp \left[\left(\frac{\pi}{2} - \phi'_s \right) \tan \phi'_s \right] \quad \dots (8)$$

where:

p_u^S = ultimate pressure of the expanding spherical cavity in an elasto-plastic infinite medium

p_c^U and p_u^S were evaluated eqs. (7) and (8) using the theory proposed by Baligh (a non linear strength envelope is considered). Computations were carried out by means of the computer program EXPAND developed at the Civil Eng. Dept of M.I.T., (see Baligh (1975)). Because of inherent difficulties in the assessment of the ϕ'_s values to be used in the formulae, as a first approximation, it was assumed ϕ'_s to be close to the average mobilized ϕ'_s within the plastic zone existing at failure around an expanded cavity. This was computed evaluating average shear (τ_{ff}) and normal ($\sigma_{\tau\tau}$) stresses on the failure plane at failure for each of the soil elements the plastic zone was subdivided

into by code EXPAND, obtaining therefore:

$$\phi'_s = \arctan \left(\frac{\tau_{ff}}{\sigma'_{ff}} \right)_{\text{average}}$$

7. MEASURED (q_c^M) vs. COMPUTED (q_c^C) CONE RESISTANCE.

Figs. 7 and 8 show the q_c values measured in the calibration chamber compared with those computed on the basis of the theoretical approaches mentioned in the previous paragraph, in which consistent and reliable soil parameters have been introduced.

The results allow the following remarks:

- a) For very dense sand (fig.7) both the formulae proposed by Vesic (1975, 1977) (with p_u evaluated considering a non-linear strength envelope) fit the experimental results reasonably well; this fit appears to be a little better for the cylindrical rather than for the spherical cavity approach.
- For dense sand (fig. 8) the comparison between q_c^M and q_c^C is seen to be less satisfactory with respect to the results obtained for very dense sand (the experimental parameters describing the non-linear strength envelope of dense sand are thought to be less reliable than those determined for very dense sand); however Vesic's formulae, combined with appropriate input parameters are able to predict the range of q_c with reasonable accuracy.
- b) As far as Durgunoglu and Mitchell's approach is concerned, used here in connection with the angle of shearing resistance obtained linearizing (see, e.g. fig.6) the strength envelope, fig.7 (very dense sand) shows that it underestimates to some extent q_c

at shallow depth (+) and largely overestimates q_c at depths below 20 ÷ 25 metres. In fig.8 (dense sand) Durgunoglu and Mitchell's approach was utilized with two different angles of shearing resistance obtained linearizing two available strength envelopes obtained from CID-TX tests and CK_D-TX tests; in this case the agreement seems to be better, but one gets the impression, as far as this method is concerned, that its use with constant ϕ' , i.e. neglecting the curvature of the strength envelope, makes it impossible to provide a reasonable assessment of q_c . This fact raises a very practical question: is it reason-

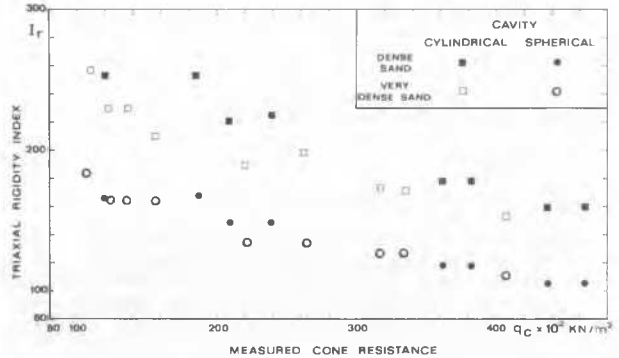
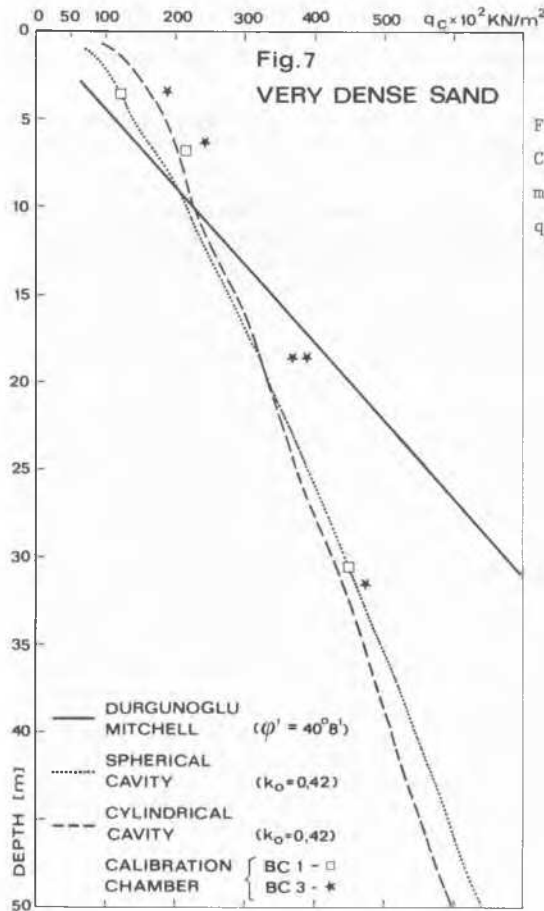
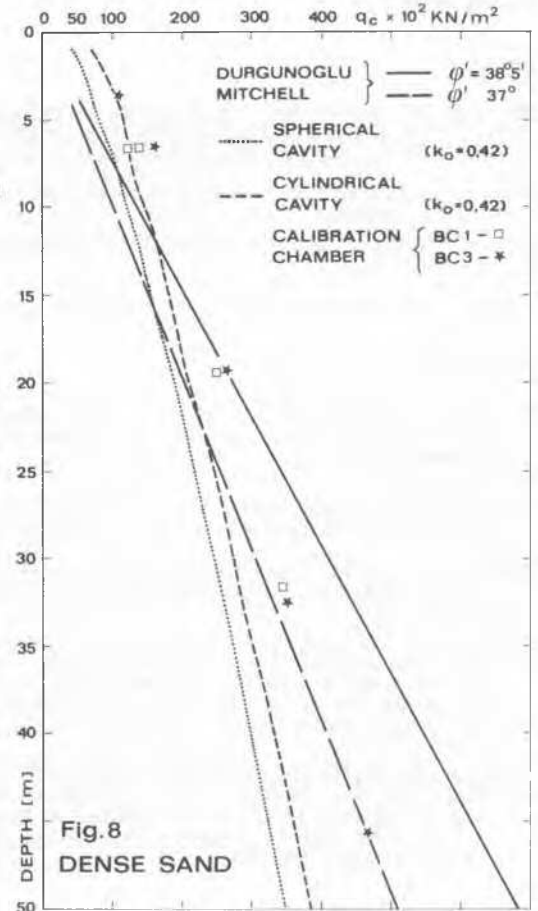


FIG. 9 - Measured cone resistance vs triaxial rigidity index.

(+) but always well beyond the critical depth as defined by Durgunoglu and Mitchell, 1975).



FIGS. 7 and 8 Comparisons between measured and calculated q_c values



nable to utilize this or similar theories to evaluate ϕ' from q_c , considering that many types of sand have a non-linear strength envelope?

c) The use of formulae ... (7) and ... (8) to compute base resistance of driven piles in sand is subjected to large uncertainties as far as the selection of the rigidity index I_r is concerned. In consideration of this it was decided to show a qualitative trend of q_c^M vs I_r as evaluated by EXPAND program with input data obtained by triaxial tests (see table 1 and fig. 9).

Other tentative rules which allow very rough estimates of I_r are given by Al Awkati (1975) and Vesic (1977):

- For Dutch CPT tip:

$$I_r \approx \frac{300}{FR} \quad \dots(9)$$

- For cylindrical electrical tip:

$$I_r \approx \frac{170}{FR} \quad \dots(10)$$

where $FR = \frac{f_s}{q_c}$ = friction ratio (%).

Examining the I_r and FR values given in table 1, eq. (10), for the type of sand used in these tests, tends to overestimate I_r (rigidity index for spherical cavity) and to give the upper range of the experimental values of I_r (rigidity index for cylindrical cavity).

ACKNOWLEDGEMENTS.

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NOTES:

- ENEL CRIS = Hydraulic and Structural Research Center of the Study and Research Department of the Italian National Electricity Board - Milano.
- ISMES = Istituto Sperimentale Modelli e Strutture - Bergamo.
- MIT = Massachusetts Institute of Technology - Cambridge.
- NGI = Norwegian Geotechnical Institute - Oslo.

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