Rate Effect on Cone Penetration Test in Sand

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ABSTRACT: A literature survey has provided quite variable results with respect to rate effect on cone penetration test(CPT) in sand. Most of the available data refer to the penetration rate in the range 2 mm/s – 20 mm/s, and show some rate effect. The analysis of the data shows that the factors controlling rate effect on CPT in sand are excess pore pressure generation (in the case of loose silty sands and loose fine sands) and grain crushing (especially in the case of dense sands). Excess pore pressure generation produces a reduction in cone resistance when the rate is increased from 2 mm/s to 20 mm/s, while the opposite occurs in the case of grain crushing. Since the stress level influences grain crushing, the higher the stress level the higher the rate effect. Moreover, the higher the crushability of the sand grains, the higher the rate effect. There is scarcity of tests at high rates. Few data available from tests with variable high rates indicate a significant rate effect.

1. INTRODUCTION

A significant number of researches have analysed the rate effect on cone penetration test (CPT) in clay, and most of them have been included in Danziger and Lunne (2012). However, a number of researches have also been conducted in sand. The purpose of this paper is to summarize and analyze the most important available data on this subject. This paper is based on a NGI report (Danziger and Lunne, 2012), which is an updating of an unpublished report by Danziger and Lunne (1997), part of it included in Lunne et al. (1997).

For the standard rate of 20mm/s and 10 cm^2 penetrometers, penetration has traditionally been considered undrained in the case of clays and drained in the case of sands, whilst in the case of silts and clayey silts drainage conditions are not well defined (Campanella and Robertson, 1981, Campanella et al., 1982, 1983) and partial drainage may occur. Jones et al. (1981) have proposed a simplified method (based on Blight, 1968, for vane tests) to estimate a penetration rate for drained conditions. Jamiolkowski et al. (1985) mentioned that in clays and uniform silts tests performed with the standard rate occur virtually in undrained conditions, while in clean sands (<10% passing ASTM sieve No. 200) the penetration is virtually a drained process. The assumptions above will be argued based on the analyzed tests.

2. DATA COLLECTED

The data collected regarding rate effectsin sands is summarized in Table 1. It comprises both in situ and laboratory testing. Mechanical and electrical CPT, as well as piezocone (CPTU) penetrometers have been used in the tests. Although most tests have been performed with 10cm^2 penetrometers, other sizes have been used, ranging from 0.5 cm^2 to 15.9 cm^2.

Not only tests in clean sands have been included in the table, but also silty sands, aiming at a comparison with clean sand. The mineralogy of the clean sands has been provided only in few cases. Both saturated and dry conditions have been evaluated.

Rate penetration values are in a great range, from 0.03mm/s to 810mm/s. However, most of the data cover the range 2 mm/s – 20 mm/s.

It must be pointed out that some authors included in table 1 performed tests in other materials, not only in sand, but these have not been included in the table.

Table 1 Summary of data related to the influence of rate of penetration on CPT/CPTU behavior in sand

Author	Penetrometer	Measured Quantity	In situ/ laboratory	Type of soil	Penetration rates (mm/s)	Main conclusions
Kérisel (1961)	mechanical, 15.9 cm^2	q_c, total lateral load	laboratory, special conditions	coarse Loire dense sand, dry	0.06- 0.25 to 0.37 - 2.5 to 3.3	Cone: rate effect is small, about 4% per log cycle. Total lateral load has also shown rate effect, higher rate roughly provided 30% more load than those from the 2 smaller rates.
Jézéquel (1969), Amar, (1974), Amar et al., (1975)	mechanical and electrical, 10 cm^2	q_c	in situ	loose sand, saturated and dry	2-20	q_{c2}/q_{c20} (mean values): elec. cone: 0.93 for loose dry sands, 1.28 for loose saturated sands.
Kok (1974)	mechanical and electrical, 10cm^2	q_c	in situ	medium dense fine sand, saturated	5-20	$q_c$was roughly the same for both rates. Electrical cones give in average 30% higher q_c than the Delft mechanical cone.
Malyshev and Lavisin (1974)	mechanical, 12.6 cm^2	q_c	laboratory	coarse grained homogeneous air-dry quartz sand	0.31-0.7-1.78-8.1	The relationship between the penetration force and penetration rate is of a parabolic nature and reaches its minimum at 4.3 mm/s, independently of the soil density.

Dayal and Allen (1975)	electrical, 10cm^2	q_c, f_s	laboratory	silica-70 sand, medium to fine, dense and loose, dry	1.3-12.8-139.0-811.4	Rate effect on q_c and f_s were insignificant.
Ponte (1977), as mentioned by De Ruiter (1981)	electrical, 10cm^2	q_c, f_s	in situ	dense sand, fine to medium, saturated	0.2-1-10-20-100	Both q_c and f_s increase with penetration rate, and f_s is more influenced by the penetration rate than q_c.
Chapman (1979)	electrical, 10 cm^2	q_c	laboratory, calibration chamber	medium to fine, uniform quartz sand, dry	6.0-20.4	q_c before and after a penetration reduction (from 20.4 to 6.0 mm/s) were the same.
Te Kamp (1982)	electrical, 10cm^2 (in situ), 0.5 cm^2 (lab.)	q_c, f_s	in situ(onshore and offshore), laboratory	dense fine sands, saturated	0.2-1-10-20-100 (on land)/ 2-20 (at sea)/ 0.033-16 or 17 (at sea)/ 0.2-2-20 (lab.)	In situ: there is a tendency for decreasing penetration rate to be associated with decreasing q_c and f_s; $q_{c0.2}$≈(0.8-0.9)q_{c20}, possibly due to the effect of dilatancy. No obvious differences between onshore and offshore tests. Laboratory: possible effect of penetration rate was not proven.
Rocha Filho (1982)	piezocone, 10 cm^2	q_c, u_{tip}	laboratory, calibration chamber (thick wall)	medium to fine silty sand, saturated	20, static test	$q_{cstatic}/q_{c20}$ increases as q_{c20} increases, being ≈ 1 for q_{c20}≈5.5MPa. Mobilization of viscous resistance predominates on loose to medium deposits, while reduction of effective stresses is likely to occur in medium to dense soils.
Juran and Tumay (1989)	piezocone, 15 cm^2	q_c, u_1	in situ	Dunkerquesand, saturated	2-100	Penetration rate was found to have no appreciable effect on q_c, effect noted on u_1; $u_{1,2}$ approaches u_o, whereas $u_{1,100}$ reaches 4u_o.
Takesue et al. (1996)	piezocone, 10 cm^2	q_t, f_s, u_2	in situ	silty sand (Shirasu), saturated	2-5-20	Penetration rate has little effect on q_t and f_s, Δu≈0.
Lo Presti et al. (2010)	piezocone, 10 cm^2	q_t, f_s, u_2	in situ	2italian sites, sand and gravel, saturated	10-20	Differences are due to local lithological heterogeneities.
Sacchetto and Trevisan (2010)	piezocone, 10 cm^2	q_c, f_s, u_2	in situ	Padana valley,saturated sand	7.4-15-20	$q_{c7.4}/q_{c20}$=0.831; q_{c15}/q_{c20}=0.892; f_s values did not allow any comparison

f_s = sleeve friction; q_c = measured cone resistance; q_{cv} = cone resistance corresponding to a rate equal to v mm/s; q_t = corrected cone resistance;u_o = in situ pore pressure; u_{tip} = pore pressure measured at the cone tip; u_1 = pore pressure measured on the cone; $u_{1,v}$ = pore pressure measured on the cone at a rate of v mm/s; u_2 = pore pressure measured behind cone; Δu = excess pore pressure

3. ANALYSIS AND DISCUSSION

The conclusions reached by the different researches may be quite variable (and surprising), and care must be taken before drawing general conclusions from most of the tests.In the following the data will be grouped according to their trend. As mentioned before, most of the data were obtained in the interval 2 mm/s – 20mm/s, thus the trends will mostly refer to this interval.

3.1 Reduction of q_c with rate increase

Jézéquel (1969) performed tests in loose fine sands, in both saturated and dry conditions. For the dry sand Jézéquel (1969) mentioned that there is no apparent influence of the penetration rate, although finding a mean value of q_{c2}/q_{c20} (where q_{cv} is the cone resistance corresponding to a rate equal to vmm/s) equal to 0.93. For the saturated sand Jézéquel (1969) found q_{c2}/q_{c20} = 1.28 and 0.91, in different places, but mentioned that the general trend is that the cone resistance decreases when the rate of penetration increases. This general trend and also a marked influence of penetration rate at 9 to 11 m depth can be seen in Figure 1. Jézéquel (1969) attributed this kind of behaviour to the development of excess pore pressure in the region below the cone.

Jézéquel's (1969) results seem to indicate that with the rate of 2 mm/s drained behaviour occurred while undrained or partly drained behaviour occurred with the rate of 20mm/s at least for 9-11 m depth. Probably partial liquefaction has occurred in this interval which caused the ratio qc2/qc20 to increase to a value as high as 2.5 around 10 m depth. Since Jézéquel (1969) has mentioned that the material tested was a hydraulic fill fine loose sand, the hypothesis of partial liquefaction seems justified.

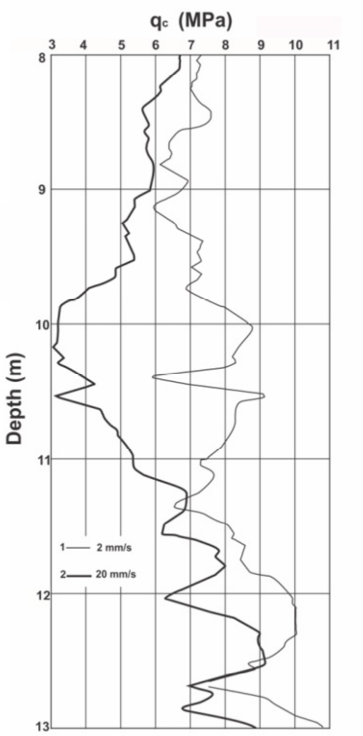

Figure 1 Cone resistance versus depth for different rates in saturated fine loose sand (adapted from Jézéquel, 1969)

Although testing model plates, rather than cone penetrometers, the drained – partly drained – undrainedbehaviour was particularly addressed by Finnie and Randolph (1994) who performed centrifuge tests in calcareous siltysand (with 32% silt and 8% clay) and silt, but just the siltysand tests are reported herein. Those authors have suggested the use of a normalized velocity, vD/c_v, where D is the diameter of the foundation and c_v is the coefficient of consolidation. Finnie and Randolph (1994) have adopted $c_v = 1 \times 10^{-3}$ m^2/s for normalization without discussion. The appropriate c_v value to be used in the normalization of the rate was later discussed by a number of authors (e.g., Lehane et al., 2009), but it is outside the scope of the present paper. The normalized (or non-dimensional) rate was later named V (Randolph and House, 2001). Figure 2 presents the results obtained by Finnie and Randolph (1994), where the vertical axis indicates the non-dimensional bearing modulus $M=q/\gamma'z$, q = the average applied stress, γ' = the effective unit weight of the soil and z = penetration depth. Finnie and Randolph (1994) suggested that the transition from drained to partially drained conditions occurred at a non-dimensional velocity of 0.01, while the undrained limit is reached at a non-dimensional velocity of about 30. The data presented by Finnie and Randolph (1994) are a good illustration of the influence of excess pore pressure generation in silty sands and fine sands.

Partial conclusions: The use of the standard penetration rate of 20mm/s when testing 10 cm^2 penetrometers does not necessarily mean that a drained behaviour will occur, at least for fine and/or silty sands. The pore pressure generation can be responsible for a decrease of the strength of the sand and under certain circumstances (like the one shown by Jézéquel, 1969, see Figure 1) the decrease can be significant.The authorsof the present paper recommend to always measure u and f_s.

3.2 Increase of q_c with rate increase

Most of the results showed an increase of q_c with rate increase, whichKérisel (1961) has shown first. In fact, to the authors´ knowledge, the first series of tests aiming at evaluating the rate effect on CPT in sand have been conducted by Kérisel (1961). The tests reported by this author are quite interesting. The facility developed by Kérisel (1961), although aiming at the study of deep foundations, may be considered one of the initial rigid wall calibration chamber, maybe the biggest ever built (Figure 3). A concrete structure, 6.40 m in diameter and 10.25 m in depth, was filled with dynamic compacted dry coarse Loire sand. A very dense state was reached with the compaction procedure used.

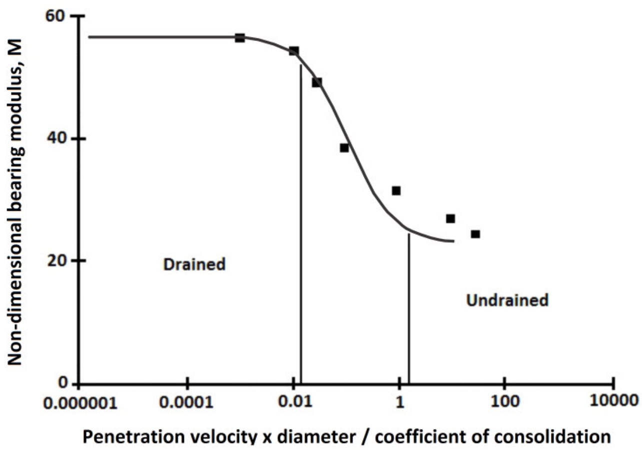

Figure 2 Effect of loading rate on non-dimensional bearing modulus in silty sand (adapted from Finnie and Randolph, 1994)

Figure 3 Facility developed at Iraba to test models of deep foundations and penetrometers (Kérisel, 1961)

Kérisel (1961) used a 45 mm in diameter hydraulic Parez type penetrometer. Three rates have been used, all of them smaller than the standard rate of 20mm/s: 0.06, 0.25 to 0.37 and 2.5 to 3.3 mm/s. The smaller rates were used until 3 m depth, while the test with higher rate reached 9 m. The measured cone resistances are shown in Figure 4.In addition the values of the dry density $\gamma_d$are shown in the figure for different depths. Kérisel (1961) mentioned that the rate effect was small and can be represented by equation (1), which corresponds to a 4 % increase in cone resistance per rate log cycle.

$$\frac{q_c - q_{co}}{q_c} = 0.04 \log \frac{v}{v_o} \quad (1)$$

where q_{co} = cone resistance corresponding to a reference rate v_o

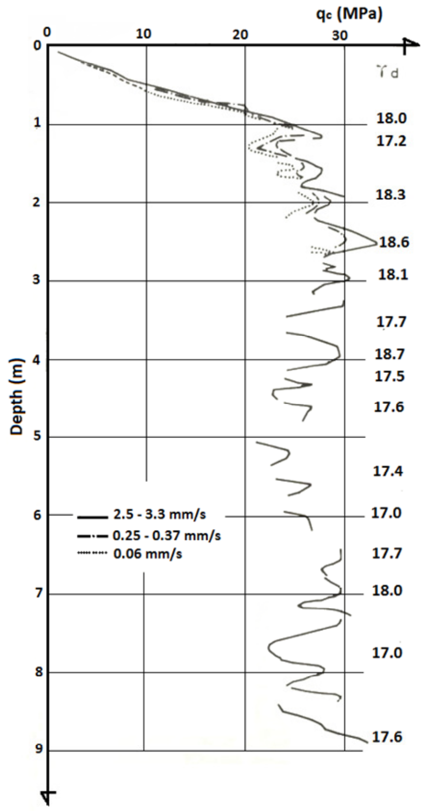

Figure 4 Cone resistance versus depth for 3 rates in dense Loire sand (adapted from Kérisel, 1961)

Although not making a detailed analysis of the test results, Kérisel (1961) also measured the total friction (Figure 5). Apparently rate effect on the measured total friction is always larger than for the cone resistance. In fact, the total friction for the highest rate (2.5 to 3.3 mm/s) is greater than the values for the other 2 rates for depths greater than 1 m. A quite similar result was obtained for the 2 slower rates and the difference with respect to the highest rate is more significant than the one corresponding to the cone resistance. The average difference is around 30 %.

Ponte (1977), according to De Ruiter (1981), has performed tests in a dense sand, fine to medium, and obtained the values shown in Table 2.

Data from Table 2 show that both qc and fs have increased with the increase of penetration rate, but fs is more sensitive to the penetration rate. This is in accordance with Kérisel´s (1961) results, and also with results in clays and silts, as shown by e.g. Danziger and Lunne (2012).

Te Kamp (1982) performed tests both onshore and offshore in dense to very dense saturated fine sands. The penetration rates varied from 0.033 mm/s to 100 mm/s, but in the case of offshore tests less rates were used. A summary of test results is shown in Figure 6. It seems that the data named in the Figure 6a as Leidschendam tests, Ponte, 1976, are the same as in Ponte (1977) and included in Table 2.

The general trend found by Te Kamp (1982), both for onshore and offshore tests was that qcv/qc20 increases with the increase of rate of penetration. Te Kamp (1982) attributed this behaviour to the possible effect of dilatancy.

The generation of negative excess pore pressures for greater rates of penetration could be an explanation for the increase in cone resistance. However, this means that for 20 mm/s most of the data showed by Te Kamp (1982) should correspond to an undrained or a partially drained condition. Unfortunately, Te Kamp (1982) did not measure the pore pressures in order to verify the assumption above. It must be remembered that at least cavitation probably did not occur (see Seed and Lee, 1967), as both onshore and offshore tests showed the same general trend. This was also observed in laboratory cone penetration tests where onshore and offshore conditions were simulated by Te Kamp (1982).

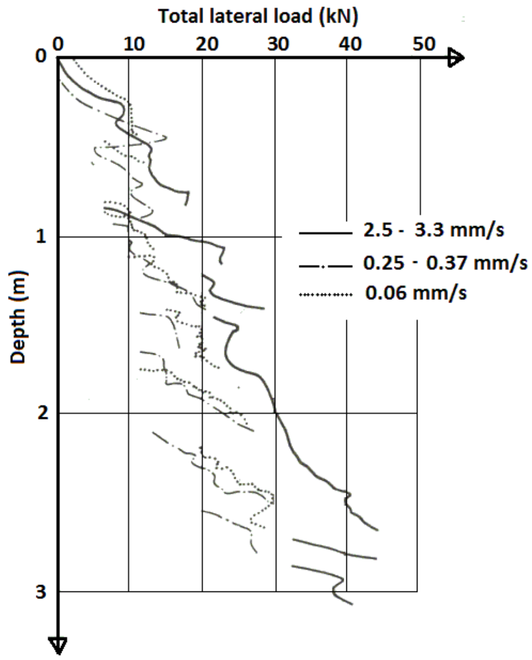

Figure 5 Total friction versus depth for 3 rates in dense Loire sand (adapted from Kérisel, 1961)

Table 2 Influence of rate of penetration on cone resistance and sleeve friction (Ponte, 1977, according to De Ruiter, 1981)

Rate of penetration (mm/s)	0.2	1	10	20	100
q_c	-10.1%	-4.8%	-1.6%	0	5.3%
f_s	-29.0%	-12.1%	5.0%	0	12.8%

Moreover, as the cone region is dominated by the normal stresses, only positive excess pore pressures would have been expected, i.e., lower values of qc in undrained or partially drained conditions. Positive excess pore pressures have in fact been measured by e.g. Rocha Filho (1982), at the cone tip of a 10 cm2 penetrometer, and at the cone face of a 15 cm2 penetrometer by Juran and Tumay (1989).The explanation that seems to fit with Te Kamp´s (1982) test results is related to grain crushing. Lee et al. (1969) performed triaxial compression tests on dry sands at high confining stresses. The tests showed that an increase in strain rate produced an increase in strength, an increase in the initial tangent modulus and a decrease in the strain at failure with the greatest changes in all three properties being observed for the dense sands and for the higher confining stresses. The increase of strength in the case of dense sands at high confining stresses was mainly attributed by Lee et al. (1969) to the effect of strain rate on the energy required for particle crushing. The time-dependency of this phenomenon was illustrated by a series of transient load tests on saturated undrained samples also performed by Lee et al. (1969). In these tests the amount of transient load increase was not sufficient to cause an immediate failure, however pore pressures continued to increase and eventually failure occurred.

Joustra and De Gijt (1982) performed compression tests in hydrostatic conditions with high pressures on 5 sands and found that the plastic volume changes are to a large content generated by crushing of the grains and that the plastic strain rate is time dependent.

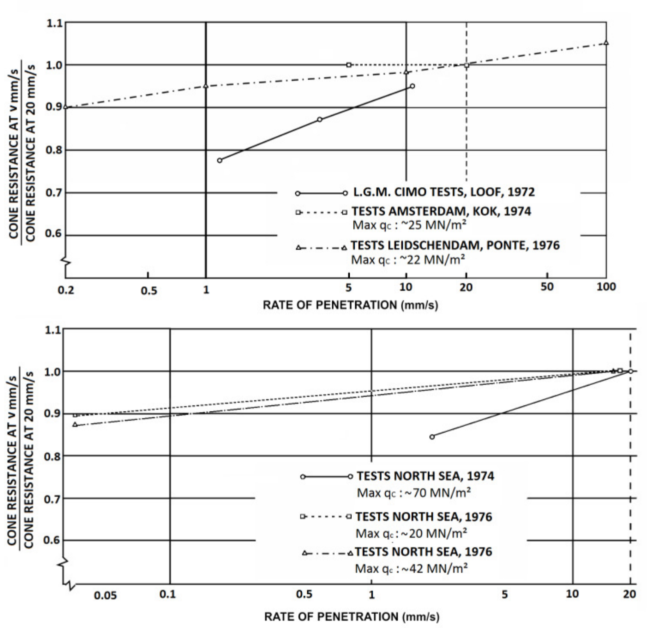

Figure 6 Normalized cone resistance versus rate of penetration for tests (a) on land; (b) offshore (Te Kamp, 1982)

Recent laboratory tests performed by Karimpour and Lade (2010) have also shown that particle crushing is a time dependent phenomenon. They also observed that the amount of crushing relates to the amount of energy input to the soil. These authors have attributed the time dependency of the observed behaviour to static fatigue or delayed fracture of the sand grains. According to Lade and Karimpour (2010), static fatigue is a phenomenon that leads to fracture and crushing of individual soil particles.Brittle fracture of materials such as quartz, feldspar, concrete and rock occur due to time-dependent crack propagation and with negligible deformation prior to fracture.

A comprehensive explanation of the static fatigue phenomenon was provided by Lade and Karimpour (2010). They mentioned that both internal microscopic cracks and surface cracks play a role on the phenomenon. The fracture process in soil particle occurs in three stages, according to Van Mier (2009), quoted by Lade and Karimpour(2010), as indicated in Figure 7: the growth of the microscopic cracks, the coalescence of microcracks and the complete fracture.

The speed with which fractures propagate, and thus the progression of static fatigue is influenced by mechanical and environmental factors. In fact, a quite interesting experiment was undertaken by Lade and Karimpour (2010), where water was introduced in a triaxial testing performed in a previously oven-dried sand under high confining stresses (Figure 8). The introduction of the water did produce a change on the previous trend of the stress-strain curve, indicating the influence of water on the particle breakage.

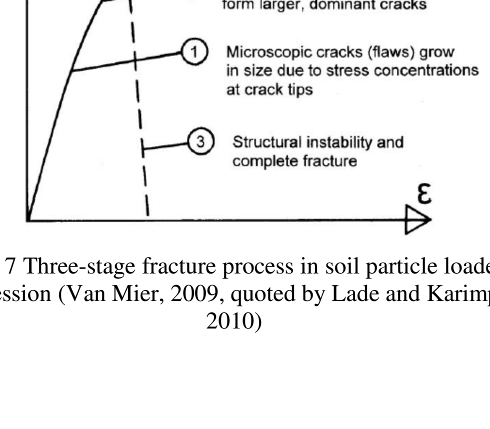

Figure 7 Three-stage fracture process in soil particle loaded in compression (Van Mier, 2009, quoted by Lade and Karimpour, 2010)

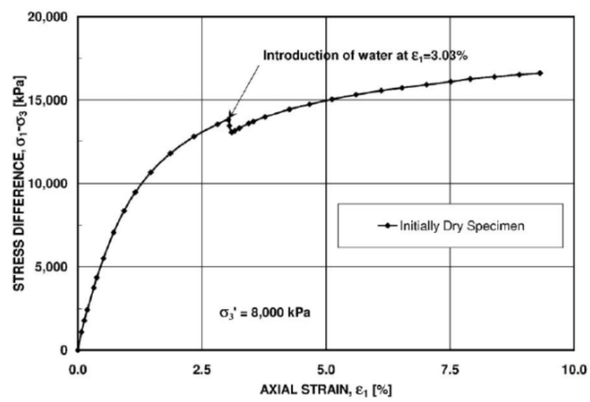

Figure 8 Stress-strain curve from triaxial compression test on dry sand with introduction of water (Lade and Karimpour, 2010)

Michalowski and Nadukuru (2012) made a distinction between the fracture of the grains, which for those authors occur at the mesoscale, and the fracture or cracking of the intergranular contacts, which they call stress corrosion cracking, or static fatigue at the contacts.

Michalowski and Nadukuru (2012) illustrated the time-dependency of the phenomenon showing the images of a silica grain asperity before loading, after 15 minutes of loading and 1 week after loading. It was clear the progression of the fracturing mechanism with time. They also argued that the discrete fracture events do not occur simultaneously at all contacts, but when integrated at the macroscopic scale, they have an appearance of a rate effect.

Those authors also argue that the fracture of asperities brings grains together, which consequence is an increase in contact stiffness, resulting in an increase in the elastic moduli at the macroscopic scale. According to Michalowski and Nadukuru (2012) the adjustments resulting from the phenomenon are of the order of the asperity size, and they are likely to have small consequences on the fabric of the sand, which governs the strength of the material. This argument was used to justify laboratory testing where stiffness was clearly influenced by the time, but not the strength.

In a very simplified manner it might be said that the phenomenon described by Michalowski and Nadukuru (2012), where fractures of the asperities occur is followed by grain crushing (when it occurs), as described by Karimpour and Lade (2010). Alternatively, one might name grain crushing as a single phenomenon, involving both the fracture of the asperities and of the grains themselves.

Whether the CPT would be able to record the cases where only fractures of the asperities occur would be merely speculative, thus in which follows grain crushing will be used as a general term, involving both the fracture of the asperities and of the grains.

If one now brings the discussion above to the CPT case, it might be hypothesized that the CPT increases both the normal stresses and shear stresses. This means that the higher the rate of the test the smaller the time to crack propagation and the static fatigue to occur, therefore more grains will crush and the cone resistance and sleeve friction values will correspondingly increase, in a similar manner than observed in triaxial testing as discussed before.

Now, if a closer look is taken to Figure 6, it can be observed that very significant rate effect was observed in the case of L.G.M. Cimo onshore tests and North Sea Tests 1974, where an increase in q_c of roughly 15-18% per log cycle was obtained. For Cimo tests the q_c values were not provided, but for 1974 offshore tests q_c values were as high as 70 MPa. Probably, very high stress levels were present in the tests.

Takesue et al. (1996) obtained a relatively small influence on rate effect not only on q_c and f_s, but also on u_2, when performing CPTU in a silty sand (Shirasu), which is an indication that the tests were performed in almost drained conditions. Since the authors mentioned that the material tested has a crushable nature, one should expect a higher rate effect when the rate was increased from 2 mm/s to 20mm/s. In fact, taking the values from the Takesue et al. (1996) paper, one obtain the ratios q_{c2}/q_{c20} = 0.97 and q_{c5}/q_{c20} = 0.99. The reason for such relatively small rate effect might be obtained analysing the original data. The average q_c value is 10 MPa, from a depth of roughly 30 m, which would roughly correspond to a relative density D_r = 40% when high compressibility is considered (see Lunne et al., 1997), i.e. the soil could be classified as loose to medium dense. A conclusion that rises from the analysis is that even in compressible sand, rate effect is not significant when the material is on a loose to medium dense state.

Sacchetto and Trevisan (2010) have made comparisons between tests performed with the standard rate and two smaller rates (7.4 mm/s and 15 mm/s), aiming at a proper understanding of the results of tests performed when a wireline rotary drill-rig is used, rather than a pushing rig. This test, named CPTWD (cone penetration test while drilling) has been used by those authors when CPT is not possible. However, in this method a smaller rate is generally achieved.

The results of Sacchetto and Trevisan (2010) show a significant rate effect (average values of $q_{c7.4}/q_{c20}$ and q_{c15}/q_{c20} of 0.83 and 0.89, respectively). However, no detailed information is given by those authors about the sands tested, except that they are from 3 sites at the Padana valley, thus assumed to be saturated. These authors also mentioned that measured pore pressures did not show a significant variation due to the rate changes, thus it might be assumed that the tests were performed in drained conditions.

Partial conclusions: several authors have reported an increase of q_c when the rate was increased from 2 mm/s to 20 mm/s, which might be attributed to grain crushing, which in turn was attributed to static fatigue or delayed fracture. This behavior was especially observed for dense sands and high stress levels, i.e. it is expected that the higher the relative density, the stress levels, and the grain crushability the higher the rate effect. The data from Takesue et al. (1996) indicated that even for crushable materials grain crushing (and consequently rate effect) might be not significant if the material is on a loose to medium dense condition.

3.3 No rate effect observed

The studies showing no rate effect in the case of clean sands are Kok´s (1974), Dayal and Allen´s (1975), Chapman´s (1979),Te Kamp´s (1982), in this last research just the laboratory tests and Juran and Tumay (1989).

All these researches refer to a relatively small rate range, with the exception of the Dayal and Allen´s (1975), which covers the range 1.3 – 811.4 mm/s.

Kok (1974) performed in situ tests in Amsterdam, and compared tests performed with the rate of 5 mm/s with tests performed with the standard rate of 20mm/s. A statistical analysis was conducted, and it was concluded that no rate effect was present. The Kok´s (1974) data are included in Figure 6a. The sand tested was a fine sand and the values of q_c varied typically between 5 and 10 MPa. If the depth of the tests performed and the corresponding effective stresses are taken into account, a medium dense sand is obtained.

Dayal and Allen (1975) performed laboratory tests in a uniformly graded, medium to fine, dry (silica – 70) sand. Both loose and dense sand conditions were tested, and no rate effect was found in both cases. Maximum value of $q_c$for the dense sandwas 7 MPa(average 3.5 MPa), which means that even for the dense condition q_c value was not very high, indicating that the stress level was not very high.

Just one test was performed by Chapman (1979) to evaluate rate effect on q_c, where in part of penetration the rate of 20mm/s was used, then the test was stopped then continued with the rate of 6 mm/s. It was verified no difference between the end of the first part of the test and the test with smaller rate. Although the analysis made by Chapman (1979) could be argued, since the previous trend of the test with the standard rate did present a different behavior, the point is that q_c value in the test was roughly 4 MPa. Besides, the sand used by Chapman (1979) was a medium to fine, uniform, quartz sand, i.e. it gathers the conditions of being less susceptible to grain crushing.

In contrast with the results from the in situ tests (see section 3.2), Te Kamp (1982) found no rate effect for laboratory tests performedin fine to medium very dense (D_r=95%) sands using a miniature cone penetrometer (7.95 mm in diameter). Onshore and offshore conditions were simulated by applying low and high pore water pressures on the samples. Normally consolidated as well as overconsolidated samples (OCR=8.4) were tested with penetration rates of 0.2, 2 and 20mm/s. This kind of behaviour, which represents a different trend from the one found in in situ tests, was attributed by Te Kamp (1982) to the applied scale model.

As Te Kamp (1982) used a miniature cone and penetration rates used were lower than or equal to 20mm/s, there seems to be no doubt that the tests were performed in drained conditions. Therefore one possible explanation for the different behaviour found from the laboratory tests is indeed the scale effect, which is influenced by grain crushing, as mentioned by e.g. Vesic (1967). Besides, the ratio between the penetrometer diameter and the size of the grains must play a role on the phenomenom.

Another possible explanation (or contributing factor) is related to a small susceptibility of grain crushing in the case of the sand tested in the laboratory. Joustra and De Gijt (1982) tested 7 different granular materials and found different susceptibility of crushing for the materials, e.g. quartz sands are less affected by crushing than other sands. Joustra and De Gijt (1982) found that crushing also depends on the dimensions of the grains, and coarse materials show more crushing than fine materials. Although only testing calcareous sands, Datta et al. (1979) found that crushing increases with (i) increasing confinement, (ii) application of shear stress, (iii) increasing abundance of intraparticle voids and plate-like shell fragments, (iv) increasing angularity of particles and (v) increasing size of particles.

Te Kamp (1982) has used in the laboratory tests sand from the Frigg Field and, from data found at NGI concerning this sand, it can be seen that it is a fine, uniform, subrounded silica sand, i.e. the sand from the Frigg Field gather most of the conditions to have a small susceptibility to grain crushing.

Juran and Tumay (1989) performed tests inDunkerque saturated sand, using the penetration rates of 2 mm/s and 100mm/s and measured the cone resistance and the pore pressure at the cone face (u_1) of a 15 cm^2 cone penetrometer. Although there are important localized differences on the q_c values, it is not possible to visualize on the chart q_c versus depth (without any treatment of the data) any special trend. However, in the case of the pore pressure it is always higher for the greatest rate than for the lowest one. Juran and Tumay (1989) mentioned that at the penetration rate of 2 mm/s the measured pore pressures approach the hydrostatic pressure u_o, whereas the pore pressures generated with the penetration rate of 100mm/s reach 4 times u_o. This is an indication that drained behaviour occurred for 2 mm/s while undrained or partly drained behaviour occurred for 100mm/s. In fact, it can be observed from Juran and Tumay´s (1989) data that even for 2 mm/s there is some excess pore pressure generated, indicating that the sand is fine and/or silty.

No information was provided about the characteristics of the sand tested. However, it can also be observed that at least 2 sandy layers are present, the first one, roughly from 3 m to 12 m, a dense fine and/or siltysand, and the second one, from 12 m to 16 m, a medium dense material. Even the first dense layer is not subjected to very high stress levels, therefore the Juran and Tumay (1989)´s data may be included in the same qualitative behavior as for the other sands in this section.

A recent comparison was undertaken by Lo Presti et al. (2010), regarding 2 italian sites, with sand and gravel, where the rates of 10mm/s and 20mm/s were used. However, no conclusions could be drawn, and the differences were attributed to local lithological heterogeneities.

Partial conclusions: the tests where no rate effect was verified do confirm the conclusions from the tests where rate effect was verified, showing that rate effect is due to (i) excess pore pressure generation in the case of loose saturated fine and silty sands; (ii) grain crushing when dense sand and high stress levels are present.

3.4 Surprising behaviour

Quite intriguing conclusions have been obtained in a research conducted by Malyshev and Lavisin (1974). Those authors performed laboratory tests with a mechanical cone with an apex angle of 30^o and a diameter of 4 cm in a homogeneous coarse grained quartz air-dry sand in different densities. The authors have used 4 penetration rates, 0.31, 0.7, 1.78 and 8.1 mm/s and found that the relationship between the penetration force P and the penetration rate v is of a parabolic nature and reaches its minimum at a the rate of 4.3 mm/s. The penetration rate corresponding to the minimum penetration force was found to be independent of the soil density. Malyshev and Lavisin (1974) have assumed that equation (2) below is valid

$$P(v) = K\ (v)\ P_o \qquad (2)$$

where P(v) is the penetration force corresponding to a rate v, P_o is the penetration force when v tends to 0, and

$$K(v) = 1 - 0.0165\ v + 0.0194\ v^2 \qquad (3)$$

with v in mm/s. However, the minimum of equation (3) above occurs for v=0.43 mm/s, and not 4.3 mm/s, as mentioned by the authors. Since no figures were presented in the paper, there is a doubt whether there is a mistake in the equation (3) or in the text. It seems probable that the mistake has happened in the text, since 4.3 mm/s is in the region where minimum values for soft clays were found (Danziger and Lunne, 2012).

A closer look at the results is obtained when the corresponding data are plotted (Figure 9). It can be observed that the minimum of the function cannot be easily distinguished, unless an expanded scale is used. Moreover, a quite significant increase of the measured force is obtained for the higher rates (more than twice the smaller values).Despite of the surprising behaviour, the paper is important, since it was the only one that has recorded a behaviour of the same type as found for saturated clays and silts in air-dry silica sands for cone penetrometers. However, this is a total unexpected behaviour which deserves more research. In fact, Chapman (1979) argued that Malyshev and Lavisin (1974) have used a rigid wall box with a diameter only 12.5 times the penetrometer diameter, which might have influenced the penetration resistance significantly. Parkin and Lunne (1982) showed that for dense sands the diameter ratio should be larger than 50 to avoid boundary effects.

However, similar surprising behaviour was obtained by Vesic et al. (1965) for model plate tests in dense Chatahoochee sand, in both dry and submerged conditions, as shown in Figure 10, where the bearing capacity factor (ultimate capacity qult normalized by 0.5 γ'B, where γ' is the effective unit weight and B is plate diameter) is plotted against the loading rate.

The explanation provided by Vesic at al. (1965) for the reduction and subsequent increase on the bearing capacity with rate is related to the time the sand particles need to adjust their position to new load increments. In the case of submerged sands, they argued that the more pronounced increase of bearing capacity at high rates is due to negative excess pore pressures. It is outside the scope of the present paper to discuss the Vesic et al.´s (1965) arguments, and definitively the kinematic constraints in their case are significantly different than in the case of cone penetrometers. However, no mention was made to the influence of crushing of the sand particles on the obtained behaviour.

One might argue that an explanation for the difference in behaviour between the dry and the submerged sand is the influence of the water on the crushability of sand grains, as shown by Lade and Karimpour (2010) and illustrated in section 3.2.

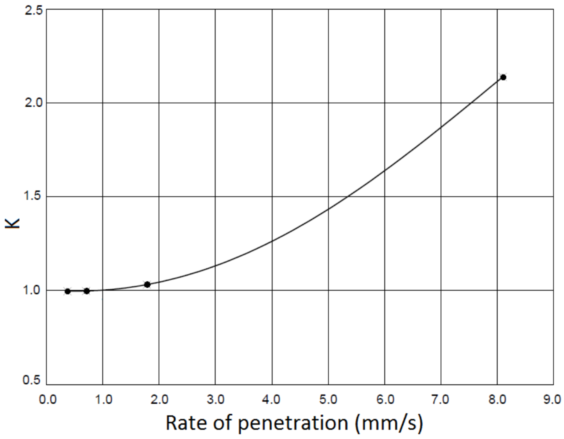

Figure 9 K versus rate of penetration (equation 3, suggested by Malyshev and Lavisin, 1974)

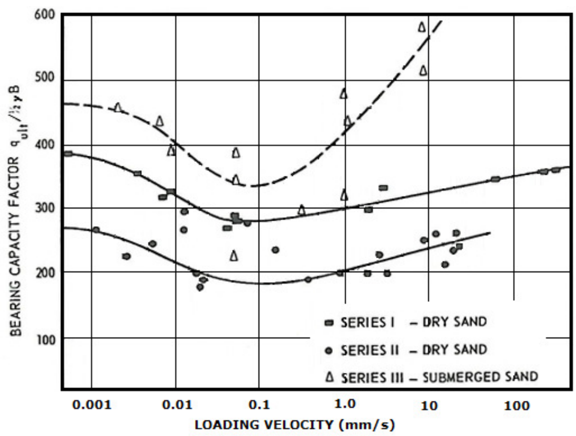

Figure 10 Bearing capacity factor versus rate for model plates in dense sand (adapted from Vesic et al., 1965)

3.5 High rates

The maximum rate used in the tests included in this paper is 811.4 mm/s (Dayal and Allen, 1975). Besides these data, the only available data areTe Kamp´s (1982) onshore data and Juran and Tumay´s (1989) data, corresponding to 2 tests with 100mm/s. Thus, there is scarcity of data on high rates in CPT in sand. Data from other penetrometers are shown below. It must be emphasized that the following data refer to tests where the rate was not maintained constant, as in the previous tests.

Grasshoff (1965) mentioned that in 1944 tests were performed in which projectiles were catapulted into sandy soil with rates between 4000mm/s and 20000mm/s. Grasshof´s (1953) picture of a bullet-shaped 7 cm in diameter projectile with a thin-rod extension protruding from the soil penetrating into dry sand is shown in Figure 11. Excavation of the projectile showed the punching effect and the quasi-liquid flow in the vicinity of the penetrated object (Grasshoff, 1965).

Grasshoff (1965) also compared his previous data with Vesic et al.´s (1965) data and other data in dry sand. His data were plotted as the bearing capacity factor (same as in Figure 10) against rate. The data were plotted in log-log scale due to high differences between the smaller and the higher values. Figure 12 presents the same data, plotted as the bearing capacity factor normalized to the bearing capacity factor at the rate of 20 mm/s, just as a reference. As mentioned by Grasshoff (1965), all data plot in a same general trend. It is worthwhile noting that the maximum ratio is roughly 200.

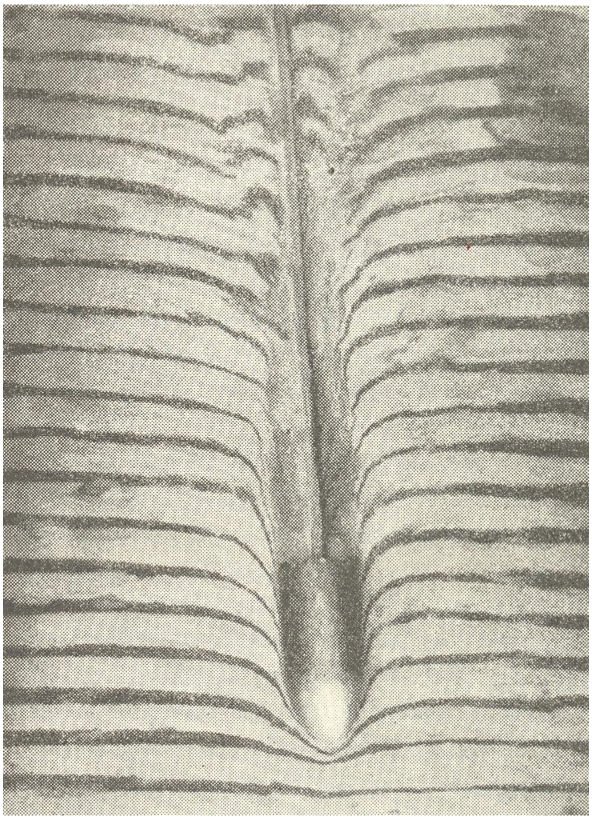

Figure 11 Bullet-shaped penetrometer 7 cm in diameter penetrating into dry sand (Grasshoff, 1953)

3.6 Gathering all data

3.6.1 Cone resistance

If the available q_c data are now plotted together, Figure 13 is obtained. It can be observed that except for the Jézéquel´s (1969) data on loose saturated sand and Malyshev and Lavisin´s (1974) surprising data, all other values show either constant q_c values in the range $q_{c2} - q_{c20}$ or present an increase in q_c when the rate increases from q_{c2} to q_{c20}. It is therefore considered plausible that there is in most cases some grain crushing when performing CPT in sand with the standard rate and 10 cm^2 penetrometers, and especially when dense sand and high stress levelsare present, grain crushing might be quite significant.

3.6.2 Sleeve friction

Few data are available with respect to the rate effect on sleeve friction in sand. However, these data indicate (Kérisel, 1961, Ponte, 1977) that the sleeve friction is more rate effect sensitive than the cone resistance. One of test sites used by Lo Presti et al. (2010), Paganico, seems to indicate a trend for higher f_s values, which could not be noted for the q_c values. Although measured, the Saccheto and Trevisan (2010) data did not allow any comparison.

4. ADDITIONAL CONSIDERATIONS

Among the practical applications from the observations above the following can be highlighted :

1) When studying rate effect on piles in sand, there seems to be important to separate the contributions of point bearing capacity and skin friction.
2) The direct application of CPT data to evaluate the bearing capacity of piles in dense sands and high stress levels may be not straightforward, once grain crushing will probably be affected by size effect. A research on rate effect associated with size effect would provide more data on the subject.
3) The calibration of in situ CPT data against laboratory testing in samples reconstituted to the same relative density must be conducted with the same stress level existing in the field in order to try to properly simulate grain crushing.
4) There seems to be a potential for the use of different penetration rates in CPT to evaluate liquefaction potential of sands.

5. CONCLUSIONS

From the analysis of the tests performed by a number of authors, rate effect on cone resistance in sand is governed by mainly two factors: excess pore pressure generation and grain crushing. Excess pore pressure dominates on loose saturated fine sands and silty sands while grain crushing prevails on dense sands, especially when high stress levels are present.

In summary:

. Loose saturated sands

The use of the penetration rate of 20 mm/s does not necessarily mean that a drained behaviour will occur, at least for fine and/or silty sands. The excess pore pressure generation can be responsible for the decrease of the strength of the sand and under certain circumstances (like the one shown by Jézéquel, 1969, see Figure 1) the decrease can be significant. Therefore it seems important to always record the pore pressure to evaluate drainage conditions. Viscous effects, if there are any, are very small and restricted to silty sands.

. Dense to very dense sands, both saturated and dry

Cone resistance can be influenced by rate of penetration, and q_{cv}/q_{c20} for v<20mm/s can vary from 0.8 to 1. This rate effect is due to grain crushing and thus the greater the susceptibility to grain crushing the smaller the ratio q_{cv}/q_{c20}. The stress level plays an important role on grain crushing, thus the higher the stress level the higher the grain crushing. Since the grain crushing is the main factor governing rate effect, it occurs on both saturated and dry conditions.

. Loose dry sands

Cone resistance (and also sleeve friction, see Dayal and Allen, 1975) are not influenced by the rate of penetration.

Some data available indicate that the sleeve friction is more rate effect sensitive than the cone resistance. There is scarcity of tests at high rates. Few data available from tests with variable high rates indicate a significant rate effect.

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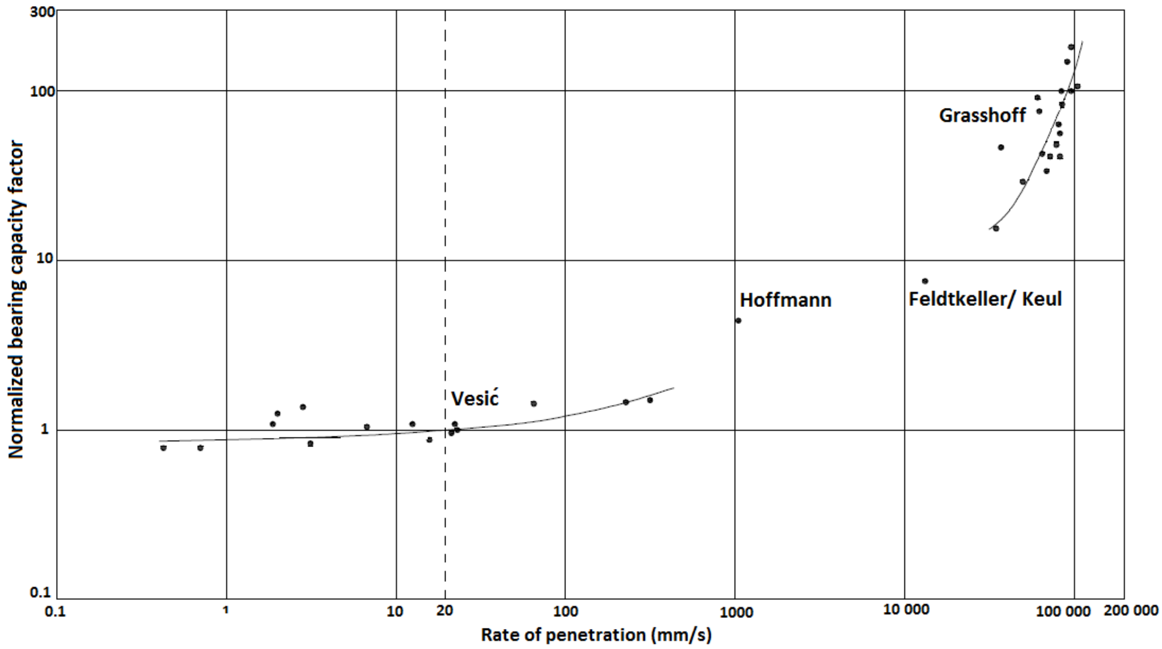

Figure 12 Data from different penetrometers in dry sand (adapted from Grasshoff, 1965)

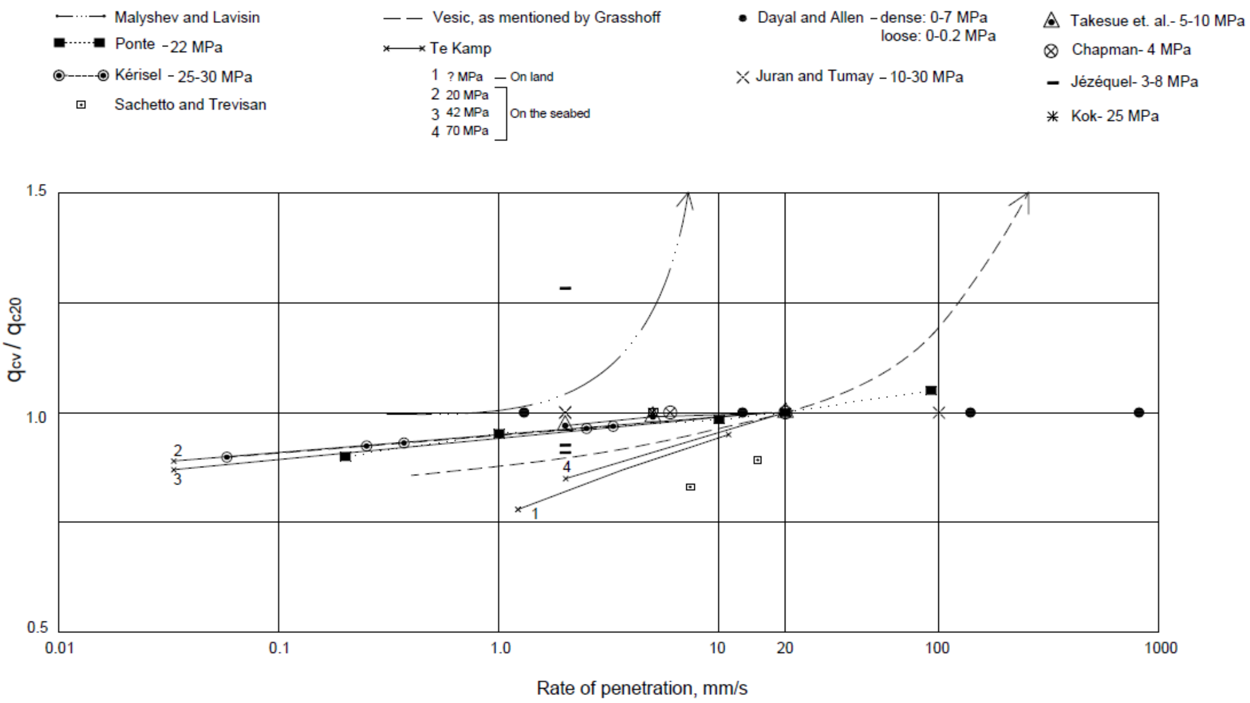

Figure 13 Normalized cone resistances against rate for most of the data

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