SAND COMPACTION CONTROL BY SHALLOW CPT's

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ABSTRACT

Compaction control of sand by cone penetration tests has been applied in The Netherlands for many years. During the last decennium in particular the use of the hand-pushed, shallow cone penetration test with small diameter cones increased considerably; it was recognised to be a simple, fast and cheap mean to check the quality of extensive compacted sand layers.
However the interpretation of the sounding results, measured by these hand-pushed penetration tests appeared to be problematic. In several series of full-scale experiments in small and large laboratory set-ups shallow CPT's were carried out in saturated, partly saturated and dry sands in order to get a clear insight into the parameters governing the CPT-diagram obtained by small size equipment. Apart from the soil parameters and the penetration and registration method in particular the dimensions of the cone turned out to have a significant influence on the results. Simultaneously shallow CPT's were carried out in test sections on different sites. With results of both laboratory and field experiments a new Dutch specification on compaction control in sand subbases will be established.

KEY WORDS

DENSITY CONTROL, CONE PENETRATION TEST, SAND EMBANKMENTS, SMALL DIAMETER CONES

1. INTRODUCTION

In The Netherlands the traditional density control of compacted sand layers in fills, subbases for roads and below shallow foundations is a time consuming, relatively complicated process of proctor tests in the laboratory and density measurements in situ, followed by an interpretation and check against official requirements which are sometimes considered to be debatable.
Moreover the essential problem of this approach is the fact that the degree of compaction or relative density, derived with the aid of this system only represents an indirect indication of the mechanical quality of the compacted sand.
The approach is fully based on the general correlation of the compaction degree or relative density and the mechanical quality in terms of stiffness (modulus of elasticity) and strength (angle of internal friction). Application of plate bearing tests, directly producing stiffness parameters, usually is avoided in The Netherlands because of the relatively long duration and high costs of the test. Due to the minor influence depth and the edge effects of the plunger CBR-tests in sands are not recommendable.
In view of these considerations density control of compacted sand layers with the aid of cone penetration tests was always considered to be an acceptable substitution, although the method was not recognised officially by the government.
In the past relatively primitive penetration methods were applied to test the firmness of a compacted sand layer. Sowers (1979) mentioned the characterisation of the firmness

(expressed in relative density) of superficial sand layers by a primitive dynamic cone penetration test, driving a 1/2-inch steel reinforcing rod in the soil with the aid of a 2.3 kg rammer; see table 1.

Table 1 - Guideline for simple determination of relative density by penetrating a reinforcing rod

state	relative density in %	ϕ 12 mm penetration rod
loose	0 - 50	easily penetrable by hand
stiff	50 - 70	easily driven by 2.3 kg rammer
hard	70 - 90	over 1 ft penetrable by 2.3 kg rammer
very hard	90 - 100	over several inches penetrable by 2.3 kg rammer

Even nowadays a similar technique of density control is applied by elder surveyors in earthwork engineering: they push a foot rule into the compacted sand and 'feel' the resistance, thus approving or rejecting the compaction result.
Although the Dutch cone penetration test originally was meant to survey the deeper soil layers and to determine the length of foundation piles, also shallow cone penetration tests found numerous applications. The quality of top layers of farming land, meadows and sports fields can be characterized satisfactory with the aid of shallow cone penetration tests. The same holds for the mechanical quality of compacted sand subbasses.

2. SHALLOW AND DEEP PENETRATION

The transition of shallow to deep penetration in cone penetration tests is marked by the so-called 'critical depth'. The general cone resistance diagram above and below this critical depth can be explained as follows; see figure 1.

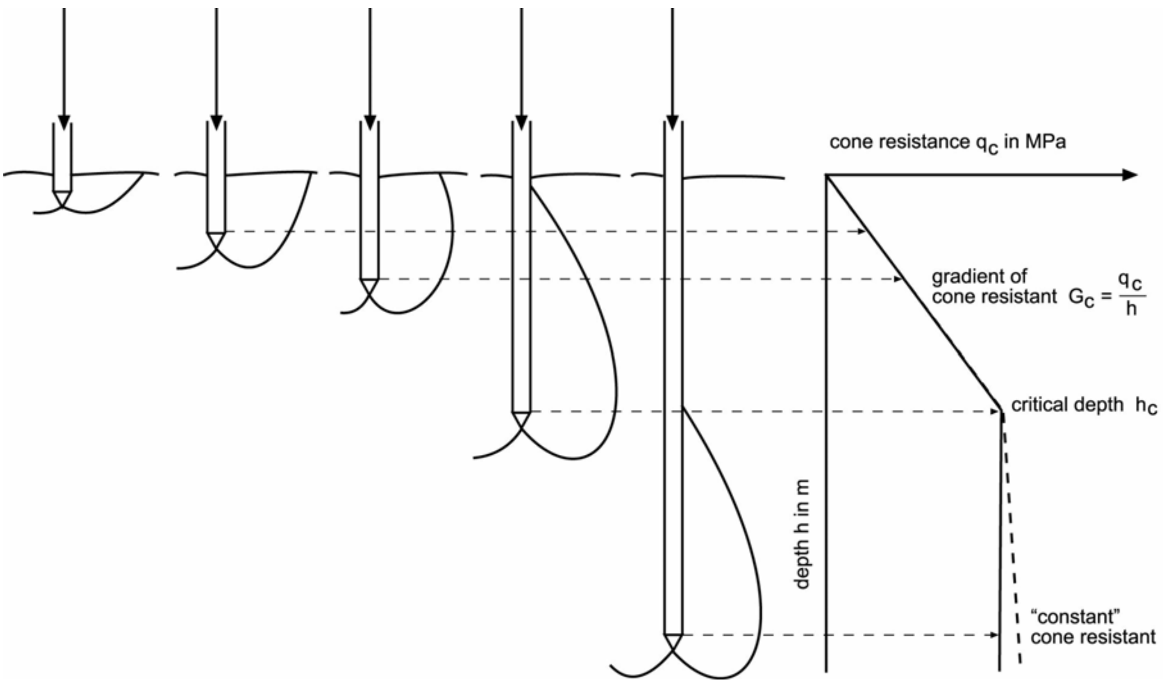

Figure 1 - Sliding planes around a superficially penetrating cone with cone resistance diagram

The critical sliding planes which will develop in the subsoil at the moment of soil failure around a pile tip or a cone tip are continuously transforming during the penetration from ground level. As penetration increases the sliding planes which originally move from the tip in an outside direction to reach the ground level, tend to bend backwards to shaft of the pile or the penetration rod. When the tip has reached a certain depth (the critical depth h_c) the sliding planes departing from the tip don't reach any more the ground level; the ongoing penetration produces a stationary pattern of sliding planes which doesn't deform any more as long as the subsoil below the tip can be considered as homogeneous.

The stationary pattern is governed by the angle of internal friction of the sand and the dimensions of the pile or cone.

According to many researchers the critical depth is not only depending on the pile or cone diameter, but also on the foundation quality of the sand (in terms of cone resistance, compaction degree or density). In loose sand critical depth indications are given of (5 - 10) d, in dense sand of (20 - 25) d, where d represents the pile or cone diameter.

The cone resistance diagram shows in the area above the critical depth a slightly concave or convex curve, which in general can be taken as a linear increase with depth. This increase continues till critical depth; below that level the cone resistance is determined by the stationary sliding pattern. As a consequence the cone resistance remains almost constant and only grows slightly with depth because of the increasing effective stresses.

The part of the resistance curve above the critical depth is represented by the average slope of the cone resistance between ground level and critical depth. This slope is indicated in figure 1 as the gradient of the cone resistance $G_c = \Delta q_c/\Delta h$; where Δq_c = cone resistance interval and Δh = depth interval belonging to Δq_c. If the cone resistance at the critial depth h_c is defined as $q_{c\,c}$ and the cone resistance at ground level is zero, follows:

$$G_c = \frac{\Delta q_c}{\Delta h} = \frac{q_{c\,c}}{h_c}\,.$$

The 'constant' cone resistance below the critical depth can be derived from the well-known Prandtl-formula, which was modified for deeper foundation levels by several authors. The ultimate bearing capacity p_u or cone resistance q_c for a circular foundation at the critical depth can be calculated:

$p_u = q_c = N_q^{**}.\gamma'.h_c + cN_c^{**} + N_\gamma^{**}.\gamma'.d$

where:

N_q^{**}, N_c^{**} and N_γ^{**}	= modified bearing capacity factors, depending on the angle of internal friction and the shape of the foundation
h_c	= critical depth
c	= cohesion
γ'	= effective unit weight
d	= diameter of the foundation

In cohesionless sand (c = 0) and small foundations (d $\approx$ 0) the formula becomes:

$p_u = q_c = N_q^{**}.\gamma'.h_c$. So can be written: $N_q^{**} = \dfrac{G_c}{\gamma}$.

The bearing capacity factor N_q^{**} is linearly related to the gradient G_c. Improving of the foundation quality (for example by compaction) means a higher G_c-value, but also an increase of the critical depth h_c; obviously the 'constant' cone resistance will also be higher.

If the pile or cone diameter is smaller the critical depth will be more shallow. That means that for a certain subsoil quality the gradient of the cone resistance, measured by a cone with a reduced diameter will be larger than the gradient from a normal diameter; see figure 2.

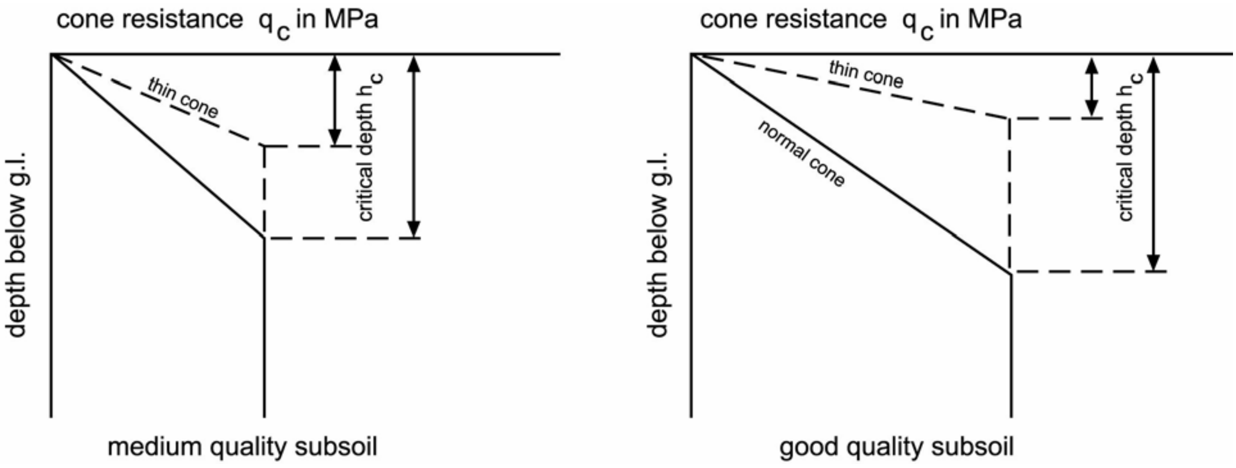

Figure 2 - Cone resistances measured by normal diameter and reduced diameter cones in medium and good subsoil

As the bearing capacity factor N_q^{**} is governed by the angle of internal friction and the angle of internal friction of sand is directly related to the relative density of the sand the gradient of the cone resistance is a function of relative density.

3. INFLUENCE OF MOISTURE ABOVE THE PHREATIC LEVEL

In practice the sand above the critical depth never will be complete dry, apart from uncovered bare top layers in summer.

As a frost-free foundation level should be at least 0.8 m below ground level and the depth of the phreatic line during compaction should be at least 0.5 m deep, the sand above critical depth will be always above the phreatic level. This sand usually contains moisture which rises capillary in the fine pores. As contrasted with the water below the phreatic level (where hydrostatic pressure governs) the water above the phreatic level possesses negative pore water pressures (-u), causing suction in the sand. In the zones above the capillary zone, the funicular zone (where only a few very narrow pores are in contact with the groundwater) and the pendular zone (where only water is present in the contact points of the grains) similar negative water pressures (-u) produce suction. Although the negative pore water pressures in the last mentioned zones are higher than in the capillary zone, they act, due to the lower saturation degree, over a smaller area. The suction s in the three zones can be defined as $s = S_r(-u)$ causing an increase of the effective soil stresses

$\Delta\sigma = s$; herein S_r represents the degree of saturation. According to most researchers the formula $s = S_r(-u)$ is only valid if the saturation degree is larger than 0.5.

In sand the resistance against shearing is usually defined according to Coulomb's Law:

$\tau = \sigma_n' \text{ tg } \varphi$, wherein σ_n' represent the effective soil pressure perpendicular to the shearing plane. In consequence of the acting suction the shearing resistance becomes:

$\tau = (\sigma_n' + s) \text{ tg } \varphi = \sigma_n' \text{ tg } \varphi + s \text{ tg } \varphi$.

This formula shows the same shape as Coulomb's formula for cohesive material:

$\tau = \sigma_n' \text{ tg } \varphi + c$, which means that in partly saturated sand a cohesion c_s is present, amounting to $c_s = s \text{ tg } \varphi$. This cohesion, which disappears if the sand becomes fully dry or saturated is called apparent cohesion.

The negative pore water pressures above the groundwater level can be simulated in a retention test; see figure 3. From the retention curve the increase of the effective stress and also of the apparent cohesion can be derived.

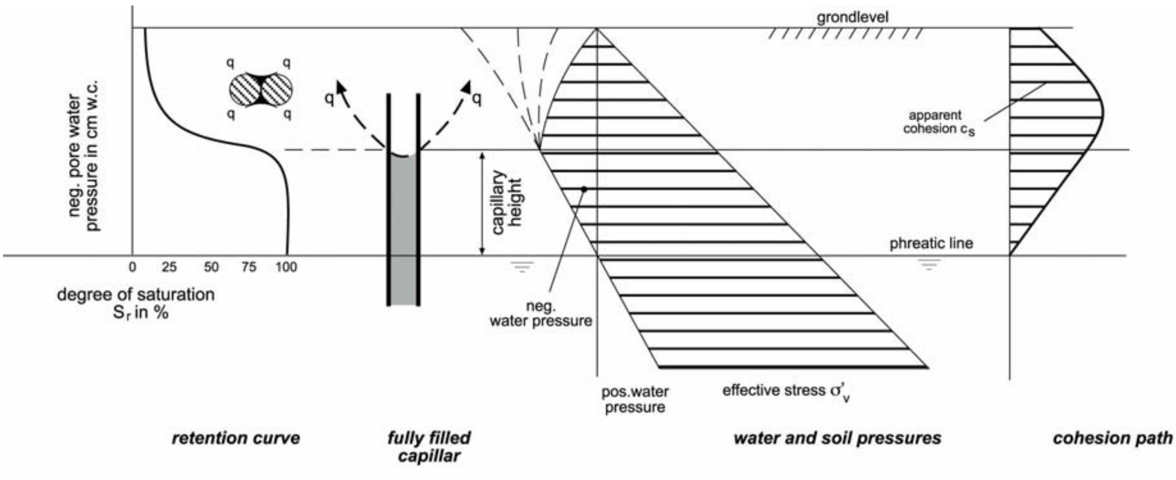

Figure 3 - Moisture situation above ground water level with effective stress and cohesion curves

According to Prandtl's theory the contribution of the cohesion term ($c.N_c$) to the ultimate bearing capacity is considerable. Also the apparent cohesion in the area above ground water level will increase the cone resistance. However it is not clear to what extent the apparent cohesion will contribute, as the penetration of the cone causes around the cone shaft a pattern of radial cracks which may decrease the cohesion values. Therefore it is not easy to get an exact indication of the bearing capacity or the cone resistance in moist sand due to the term $c.N_c$.

4. EXPERIENCES IN PRACTICE

In the course of time several test series of shallow cone penetration tests were carried out with cone types, differing in diameter and shape. The tests were performed in situ in partly saturated sand for highway embankments and foundation beds and also in large scale laboratory set-ups where dry, partly saturated and fully saturated sands were applied under more controled boundary conditions.
Four of five different cone diameters were used in the large scale laboratory tests, but in the in situ tests only the ϕ 36 mm standard cone (area = 10 cm^2) and the ϕ 11.3 mm cone (area = 1 cm^2) were employed. The ϕ 11.3 mm cone is extensively used as a handoperated tool; one person with a weight of approx. 750 N is usually capable to push this cone deep enough into a compacted sand bed to properly registrate the gradient of the cone resistance till the desired depth.

4.1. Fundamental experiences with different cone diameters

Tests by Pal (1997) and later by Meave Silva (1999) and Mugambe (1999) in the large scale test set-up in Delft Geotechnics with the aid of four different cone diameters in saturated and moist sand showed that:
- the gradient of the cone resistance increases with decreasing cone diameter

- the presence of apparent cohesion in moist sand provides significantly higher gradients of the cone resistance than in saturated sand.

The tests were carried out with the aid of hydraulically pushed, electric cones with diameters of 36 mm, 25 mm, 11.3 mm and 7 mm Moreover a ϕ 16mm hydraulically pushed (Penetrologger) cone with a ϕ 10mm shaft was applied; see figure 4.

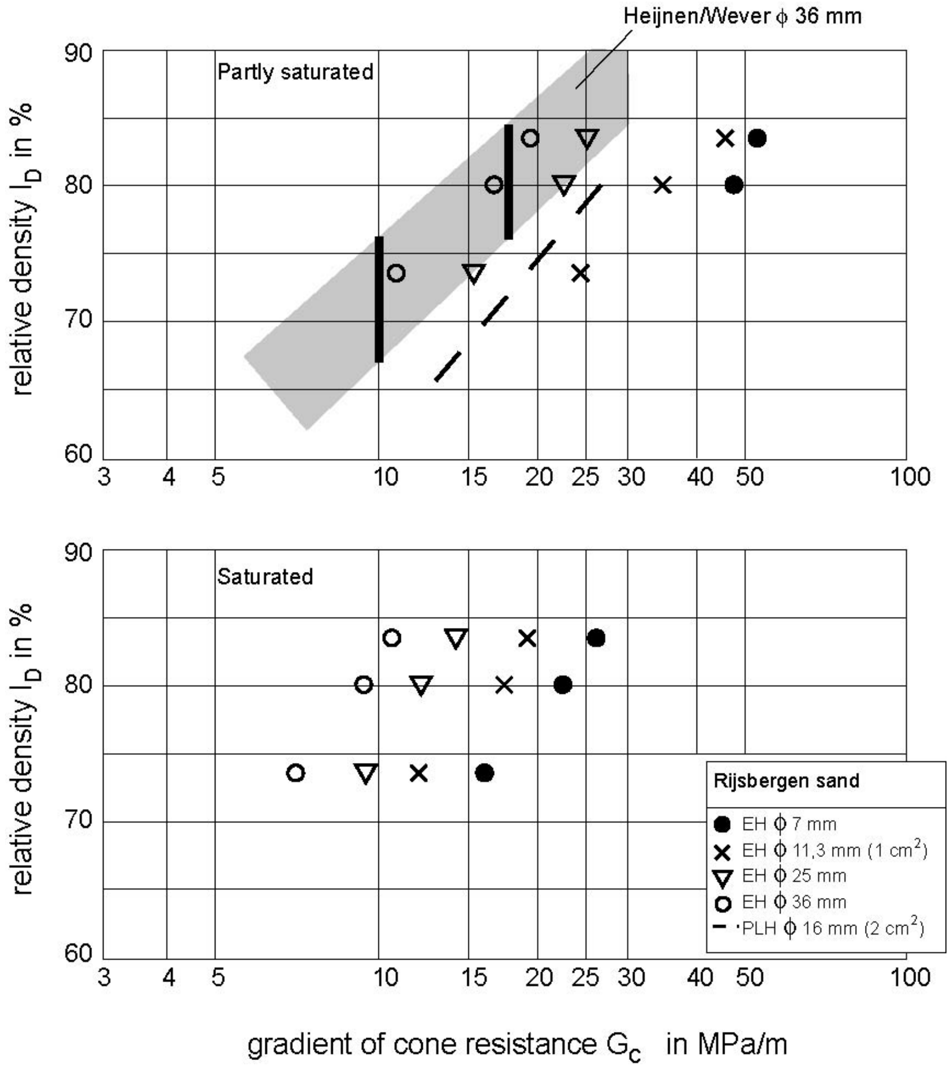

Figure 4 - Test results of different cone diameters in saturated and moist sand

The influence of the cohesion was confirmed by Banados (2001), who performed ϕ11.3 mm cone penetration tests in saturated and partly saturated sands in large rings. The results from tests on five different Dutch sand showed that similar cone resistance values were measured on saturated sands of equal relative density. As the Dutch sands on average consist of uniform, relativity fine and saturated material, this outcomes could be expected. He also proved that the contribution of apparent cohesion to cone resistance is almost constant, provided that the phreatic level lies at a depth of 0.5 - 0.8 m below ground level as a minimum; the apparent cohesion amounted to approx 3-4 kPa. This implies that there is a constant difference in gradient of cone resistance between moist and saturated or dry sand.

4.2. Practical experiences with cones ϕ 36 mm

Many years ago the application of the ϕ 36 mm standard cone for density control of shallow sand layers was started in practice.
Based on numerous shallow cone penetration tests in sand, compacted in embankments and subbases for highways Heijnen en Wever [1959] published a guideline for approval/rejection of sand compacted in earthworks. In this guideline the gradient of the cone resistance was applied already; the guideline was valid for moist, typical Dutch sand (relatively uniform, fine and subrounded) at a level of at least 0.8m above the phreatic line. The criterion for subbase sand is a cone resistance of 10 MPa at a depth of 0.6m below ground level (G_c=16.7) and for embankment sand a cone resistance of 6 MPa at a depth of 0.6m below ground level (G_c=10). These specifications corresponded with the official requirements for subbase (100% maximum proctor density or 80% relative density) and for embankment (98% maximum proctor density or 70% relative density)
Teferra [1975] also derived a relation between cone resistance q_c and relative density I_d for shallow penetration: $I_D = -26 + 34 \log \frac{q_c}{\sigma_v^{'}}$

Where:
I_D = relative density in %
q_c = cone resistance in kPa
$\sigma_v^{'}$ = effective overburden pressure in kPa

For effective soil pressures up to a shallow depth of 0.9m below ground level the cone resistance can be plotted against relative density. By subsequent replacement of the cone resistance by the gradient of cone resistance G_c follows: $I_D[\%] = 32 + 32 \log G_c [MPa/m]$
This coincides with the results of Heijnen en Wever [1959], which can be modified to: $I_D[\%] = 33 + 38.5 \log G_c [MPa/m]$
Hergarden (1990) tested the density of compacted sand in trenches by shallow cone penetration tests. The compaction degree varied between 94% and 98% maximum Proctor density and he found the relation: $I_D[\%] = 34 + 36 \log G_c [MPa/m]$
All these practical results corresponded well with the results of Mugambe (1999) for ϕ 36 mm cones in moist sand; see also the range of the Heijnen and Wever criterion in figure 4.

4.3. Experiences with ϕ 11.3 mm cones

Hergarden [1990] also performed some tests on compacted sand in trenches with the aid of the ϕ 11.3mm cone. He found the relation $I_D[\%] = 17 + 36 \log G_c [MPa/m]$ which means almost 3 times higher values than with the ϕ 36mm cone.
Lateron many tests were carried out with the aid of the ϕ 11.3 mm cone in large scale laboratory set-ups by Mugambe (1999), by Karki and Shanker (2000) and by Banados (2001) in moist sands. The ϕ 11.3 mm cone equipment showed differences in:

- way of penetration (by hand, by machine)
- way of registration (continuously, discontinuously)
- type of registration (electrically, mechanically)
- cone apex angle (30°, 60°).

Nevertheless the cone resistances demonstrated no appreciable differences. From the test results could be derived that the gradient of the cone resistance for the ϕ 11.3 mm cone is approximately 2.5 times higher than the gradient for the ϕ 36 mm cone.

5. RECOMMENDATIONS FOR SPECIFICATIONS

Based on the results of earlier experiences with ϕ 36 mm cones and recent research with smaller cones tentative recommendations can be given for compaction control in shallow sand layers.
As for the ϕ 36 mm cone the criteria from Heijnen en Wever [1959] have proven to be a reliable tool for density control in moist, shallow sand layers in The Netherlands. Moreover the criteria link up with the official density requirements for embankment and subbase sand; according to these requirements the average percentage proctor density should be 98 respectively 100. Although different smaller cone diameters are available the ϕ 11.3mm ($1 cm^2$) cone is mostly used in handoperated equipment, because this cone can be pushed deep enough into the densified subsoil to determine the gradient of the cone resistance. For this cone diameter cone resistance gradients of 25 MPa/m respectively 40 MPa/m are recommended to reach the required degree of compaction for embankment and subbase sand.

Table 2 - Summary of present and recommended specifications

requirement	sand	sand for subbase		sand for embankment	
		average	range	average	range
officially	% max. Proctor	100	99 - 101	98	97 - 99
	% rel. density	80	76 - 84	71.5	67 - 76
cone ϕ 36 mm	G_c in MPa/m	16.7	13 - 20	10	7.5 - 12.5
cone ϕ 11.3 mm	G_c in MPa/m	40	35 - 45	25	20 - 30

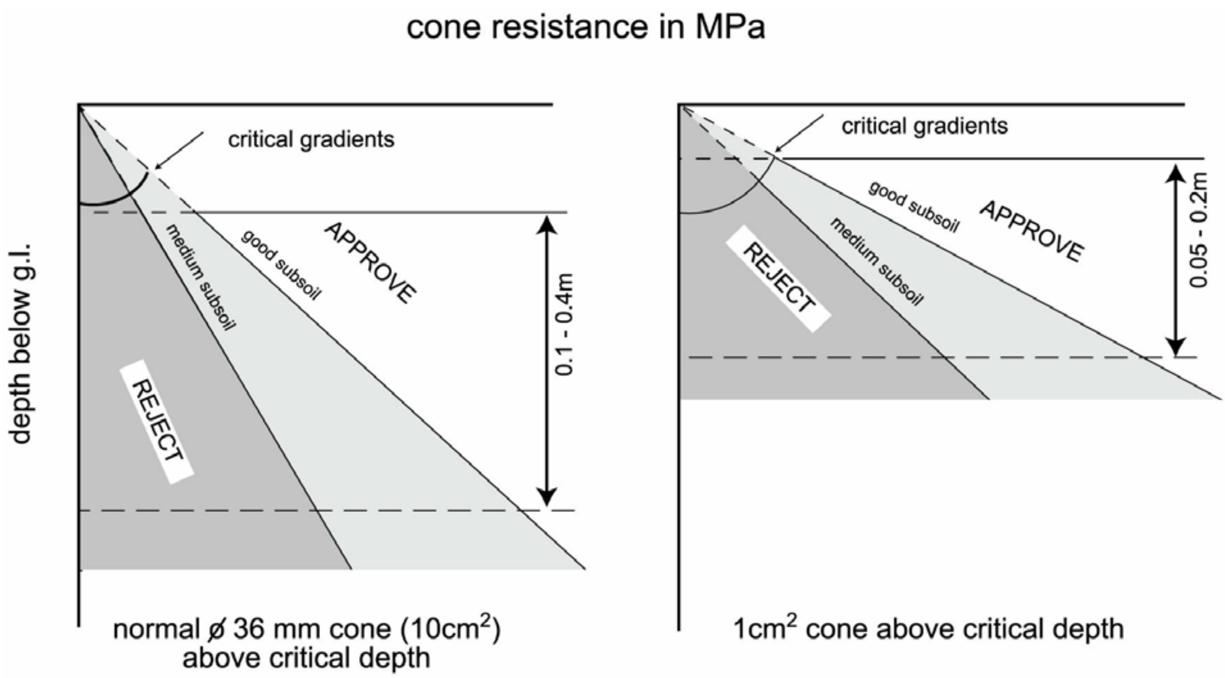

Figure 5 – Recommended specifications in terms of (gradient of) the cone resistance

The depth range over which the gradient of the cone resistance must be determined is also depending on the cone diameter. For the ϕ 36 mm cone a range of 0.1m – 0.4m below ground level is recommended, for the ϕ 11.3mm cone a smaller range of 0.05m – 0.2m below ground level.
In table 2 all recommended data are summarised; they are visualised in figure 5

The given requirements are possibly exigent. They are derived from tests on relatively uniform, fine, subrounded sands. Well-graded, more angular sand may produce higher angles of internal friction and thus higher N_q^{**} - values for the same relative density. Moreover, if these sands also contain more fines higher values of apparent cohesion are likely to occur. Both influences will lead to higher gradients of the cone resistance.
Where realistic specifications on density control are required for new sand types (other than the typical Dutch sands) it is recommended to establish new criteria based on measurements in representative test locations of different compaction quality. There the results of cone penetration test can be compared with the outcomes of traditional tests like density measurements or preferably, plate bearing tests or other stiffness measurements.

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