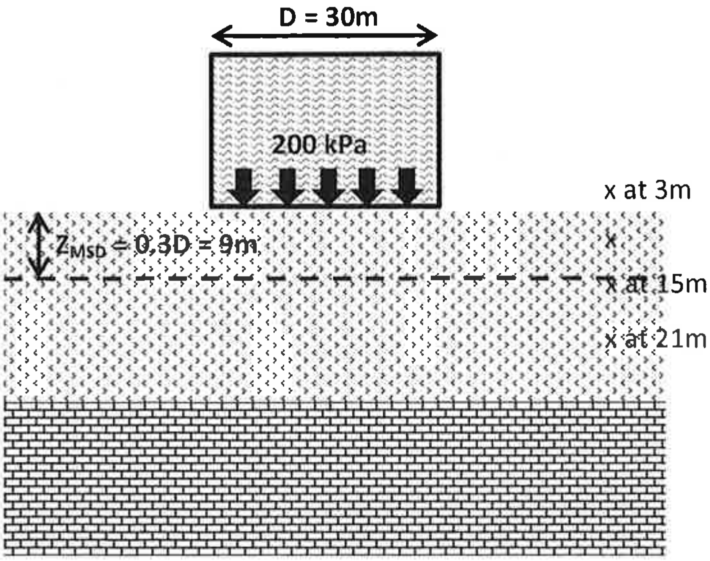
1



(a) With I_p = 0.55, Databook offers: $\left(\frac{s_u}{\sigma'_v}\right)_{nc} \approx 0.11 + 0.37\, I_p = 0.31$

After overconsolidation by factor n, Databook gives: $\frac{s_u}{\sigma'_v} \approx \left(\frac{s_u}{\sigma'_v}\right)_{nc} n^{\Lambda}$

Now assume that the water table is at the clay surface, since it is a coastal site.

At any depth z: $\sigma'_v = (21 - 10)z = 11z$ kPa and $\sigma'_{v,max} = (11z + 800)$ kPa

$\therefore n = (11z + 800)/11z = (1 + 72.7/z)$

And $\gamma_{M=2} = 0.004\, n^{0.68}$

z	σ'_v	$s_{u,nc}$	n	s_u	$\gamma_{M=2}$
3	33	10	25.2	135	0.036
9	99	31	9.1	181	0.018
15	165	51	5.8	210	0.013
21	231	72	4.5	240	0.011

[7]

(b) Assume that the bed of sand is placed at the same time as the tank, which is filled immediately afterwards. Take the bulk density of the sand fill above the water table to be 20 kN/m^3. Then the additional surcharge will be 0.5 x 20 = 10 kPa.

So the total surcharge with the full tank is q = 210 kPa.

The outline of the MSD solution for monolithic undrained settlement following Osman & Bolton (2005) is given in the Databook section 4.4.

The average mobilised shear strength in the assumed deformation mechanism:

$\tau_{mob} = q / 5.9 = 36$ kPa

At the operational depth z = 0.3D = 9 m, shear strength s_u = 181 kPa

Then the operational mobilisation ratio will be $\tau_{mob} / s_u = 0.2$

We can invoke the undrained soil response: $\frac{\tau_{mob}}{s_u} \approx 0.5 \left(\frac{\gamma}{\gamma_{M=2}} \right)^b$ with b = 0.6

Furthermore, at 9 m depth we saw that $\gamma_{M=2} = 0.018$

$\therefore \gamma_{mob} = 0.018 \times (0.4)^{1/0.6} = 3.9 \times 10^{-3}$

And we can use: $w_u/D = \gamma_{mob} / 1.35 = 2.9 \times 10^{-3}$

$\therefore$ <u>w_u = 87 mm</u>

This assumes that the tank base is sufficiently stiff to settle as though it were rigid.

In estimating the settlement w_d after consolidation, we will invoke a secant Poisson's ratio in a drained test, $\nu_d \approx 0.3$, which may be appropriate at these relatively small degrees of mobilisation. Then accepting an undrained Poisson's ratio $\nu_u \approx 0.5$, we might infer from the Databook Section 4.3.1 solution to the settlement of a rigid punch on a linear elastic bed of great depth that:

$$w_d / w_u = (1 - \nu_d) / (1 - \nu_u) = 0.7 / 0.5 = 1.4$$

$\therefore$ <u>w_d = 1.4 x 87 = 122 mm</u> [7]

(c) If the compacted sand, below the water table, weights the same as the clay it replaces, the only difference between (b) and (c) is that the operational depth in the settlement calculation moves down to 12 m.

z	σ'_v	$s_{u,nc}$	n	s_u	$\gamma_{M=2}$
12	132	41	7.1	197	0.015

Then $\tau_{mob} / s_u = 36 / 197 = 0.18$

If we continue with the power curve even though 0.18 < 0.2, we find:

$\gamma_{mob} = 0.015 \times (0.36)^{1/0.6} = 2.7 \times 10^{-3}$

Then we would get settlements <u>w_u = 60 mm</u>; <u>w_d = 84 mm</u> [3]

(d) If a second tank is placed too close to the first it will cause additional stresses under that first tank, and vice versa. This will tend to lead to differential settlement which, with stiff bases, would lead to increased bending moments as well as some tilting of the tanks. An approximate solution could be obtained using Fadum's chart (databook section 4.2), treating the tank bases as flexible. The magnitudes of local settlement could be calculated using confined modulus E_0 derived from G and ν, and a solution for a rigid tank base on a linear elastic bed could be calibrated against the MSD solution. [3]

2 (a) Most foundations are designed simply with SPT data, comprising N_{60} blow-counts together with disturbed samples recovered by the split spoon sampler. This is only sufficient to identify clays from sands, and to estimate the penetration resistance. In sands this can be correlated with the soil's relative density, from which peak friction angle ϕ_{max} can be estimated. In clays, an estimate of s_u can be derived. However, soil stiffness does not correlate very well with strength; it is also strain-dependent. In practice, foundations are indeed designed for strength, usually with a safety factor of 2.5 to 3 on ultimate bearing capacity. Since few foundations give rise to differential settlement problems it might be surmised that the use of a large safety factor generally protects against worst-case differential settlements of the most sensitive structures founded on compliant soils. This implies that many foundation designs, for less sensitive structures on stiffer soils, must be over-conservative.

The most critical foundations will be those supporting the external framed walls. The critical performance criterion will be that limiting differential settlement to that which would induce incipient tensile cracking in the masonry. If there are 10m spans and a 5m storey height, a differential settlement of 10mm might cause an average shear strain of 10^{-3}, and the associated tensile strain would therefore be about 0.5×10^{-3} which roughly corresponds to the onset of visible cracking in brick panels. The foundations for the external columns might therefore be designed to give a total settlement of 10mm, while foundations for internal columns might be designed for greater bearing pressures, since they could accept significantly larger differential settlements – perhaps 30mm – without compromising the steel frame.

16 pertinent comments = [8]

(b) The advantage of LSD is that a framework of considerations is clearly set out, as described below. LSD requires check calculations both for safety (ULS) and serviceability (SLS). In ULS checks, EC7 requires that characteristic soil strength values should be "cautious estimates" of the values expected to govern in the field. Then, specified partial factors on soil strength are given, to distance the design from states of failure, presumably in response to parameter variability. Depending on the case, load factors, especially for variable loads, are also applied. In SLS checks further calculations are requested with the intention of checking settlements, for example, in relation to limiting settlement criteria. It is clearly stated that there are to be no partial factors in SLS calculations.

The chief drawbacks are as follows:

- It is potentially dangerous to link safety concerns with soil failure, since spread foundations can settle sufficiently to compromise the safety of the superstructure while the soil remains some way from failure.
- It is illogical to demand the same partial safety factors irrespective of how rigorous the ground investigation has been, and demeaning to the designer who should always have the best understanding of soil variability at the site in question.

- The words “cautious estimate” fail to provide a clear definition, and must lead to stochastic variability between practitioners.
- Settlement calculations should have been specified in relation to published databases of soil deformability, and to published calculation methods, at least two of which – Atkinson’s equivalent stiffness approach, and MSD – have been verified for simple cases.
- Since there is no guidance of settlement calculations, and no mention of the influence of soil variability, soil non-linearity, or of the influence of load variations, EC7 fails to set proper serviceability requirements.
- No clear advice is given on suitable settlement criteria, presumably because the Eurocode committees were told not to make references to externally published works, such as databases of structural damage in relation to settlement.

A short definition of LSD by EC7 followed by six pertinent bullet-points of criticism = [6]

(c) A partial factor γ_m on soil strength and a mobilisation factor M on soil strength give arithmetically identical results. But the former is a value specified in the Code, and the latter is selected by the designer from the presumed representative soil stress-strain data in order to limit strains and deformations. So the designer is free to satisfy an appropriate settlement criterion, and to associate it with a permissible soil strength or bearing pressure. It is also possible to incorporate a minimum value of M, e.g. 1.25, which would serve as a partial safety factor, to distance the design from potential softening beyond peak strength at strain concentrations.

Consider clays, and apply Databook values from MSD.

If $\tau_{mob} = q / N_c = q / 6$; and $\gamma_{mob} = 1.35\, w/D$ for a footing width D

And if $\frac{\tau_{mob}}{c_u} = 0.5\left(\frac{\gamma_{mob}}{\gamma_{M=2}}\right)^b$ for 1.25 < M < 5

Then $\frac{q}{3c_u} = \left(\frac{1.35}{\gamma_{M=2}}\frac{w}{D}\right)^b$

Putting $q = \frac{V}{D^2}$ and choosing b = 0.5:

We infer $w \approx \frac{\gamma_{M=2}}{12}\left(\frac{V}{c_u}\right)^2 D^{-3}$ for 1.25 < M < 5

We can then proportion footings in relation to the loads they carry: $V^2 \propto D^3$.

Other variations will lead to differential settlements. An assessment of variations in $\gamma_{M=2}$, by conducting tests on a number of samples taken from trial pits, would be helpful in estimating bounds, and therefore selecting differentials.

Six salient points = [6]

3. a) As a pile is driven into sand, the soil is displaced by the pile tip leading to increases in stress. Considering a given soil element initially on the axis of the pile:

As the pile advances, the vertical and horizontal total stresses increase due to the approaching pile.
As the pile tip reaches the location, the soil is forced aside, leading to very large horizontal stresses.
As the pile tip passes the location, the vertical stresses will relax, but the horizontal stresses will remain elevated.
Cyclic driving of the pile will cause axial compression of the pile, leading to cyclic up and down movement of the pile surface relative to the soil. This will cause the sand to densify leading to temporarily increased pore-pressures which when dissipated will lead to decreasing horizontal effective stresses as the soil moves away from the pile.
After the last driving stroke, elastic extension of the pile results in downwards shear stress on the pile shaft and a locked-in vertical compressive stress beneath the pile base.

b) The API design procedure calculates shaft resistance based on a Ktan delta factor multiplying the vertical effective stress prior to pile installation. Changes in the stress state during pile installation, especially friction fatigue around the pile shaft make this problematic. Pile shaft capacity should increase as the square of depth, but friction fatigue limits this increase. The API code accounts for this by limiting the maximum frictional stress that can act on the pile shaft to a fixed value, indirectly accounting for the fall in resistance due to diminishing lateral stresses.
c) Set-up is a phenomenon by which pile capacity increases with time. When piles are driven in soft clays, positive pore-pressures are generated around the pile shaft. As these pore-pressures dissipate, effective stresses on the pile shaft rise. This is counteracted by the decrease in total horizontal stress due to soil consolidating away from the pile. In soft clays, overall this leads to an increase in effective stress and hence capacity
d) Displacement piles are stiffer than bored piles due to locked in stresses, but their installation is more disruptive, leading to the popularity of bored piles in built-up areas. Driven piles cause significant noise during installation and also ground-borne vibrations which can be damaging to neighbouring structures. Pile jacking can overcome these disadvantages while still installing a stiff pile.

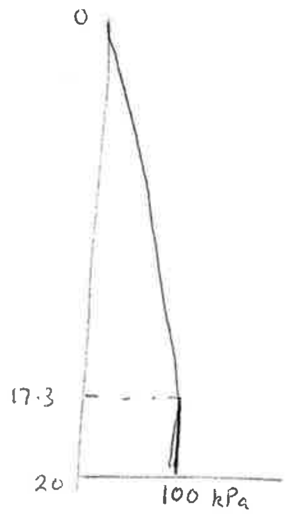
Dense sand $\delta = 30^\circ$

$\tau_{s,lim} = 100 \text{ kPa}$

$K = 1$

$\tau_{sf} = K \sigma'_{vo} \tan \delta \leq \tau_{s,lim}$

$= 1 \times 10z \tan 30 \leq 100$



~~Fr~~ τ_{lim} limits below 17.3 m.

$Q_s = \pi D \left(\frac{17.3 \times 100}{2} + 2.7 \times 100 \right)$

$= \underline{\underline{1783 \text{ kN}}}$

~~q~~$_b$ = 40 × 200 = 8 MPa < (9.6 MPa) no limit

$Q_b = 8 \text{ MPa} \times \frac{\pi D^2}{4} = \underline{1571 \text{ kN}}$

$Q_{total} = \underline{\underline{3354 \text{ kN}}}$

b) Load = 1118 kN

After installation, shaft will unload rapidly whereas base will unload slowly, resulting in locked-in stresses.

Shaft ~ 6x stiffer than base

So we end up with ⇒ Q_b, Q_s

$Q_s \approx 0$ $Q_b = \frac{5}{6} Q_{bo}$ = FOS ~ 2.4

So at F.O.S of 3, shaft resistance is slightly negative + all load plus a bit is carried by base.

After installation, as base + shaft capacity are very similar, shaft loads will be highly negative with significant locked-in base load.

c) Databook p. 17

$$\lambda = \frac{E_p}{G_L} = 667$$

$$\rho = \frac{150}{250} = 0.6$$

$$\frac{L}{D} = 40$$

$$\eta = 1$$

$$\xi = 1$$

$$\zeta = \ln\{3 \times 0.8 \times 40\} = 4.56$$

$$\mu = \frac{\sqrt{8/4.56 \times 667}}{0.5} = 0.103$$

$$\frac{V}{w_{head} D G_L} = 16.6$$

$$w_{head} = \frac{800}{16.6 \times 0.5 \times 250 \times 150} = 2.6\,mm.$$

d) Maintained load test.

Vertical loads applied to pile using rams or dead weight and pile allowed to equilibrate before further loading.

CRP test

Pile pushed into ground at a constant rate using rams

Statnamic test.

Rocket used to apply low duration static load to pile using a small reaction mass. Pile load + settlement monitored.