a) $D = 0.3\,m$ $L = 15\,m$ $\nu = 0.2$ $G = 0.6z$

$$\frac{V}{w_h D G_L} = \frac{2}{1-\nu}\,\frac{\overset{1}{\eta}}{\underset{1}{\xi}} + \frac{2\pi}{\zeta}\,\overset{1/2}{\rho}\,\frac{L}{D}$$

$\zeta = \ln\left(\frac{12}{0.15}\right) = 4.38$

$$= \frac{2}{0.8} + \frac{\pi}{4.38} \times \frac{15}{0.3} = 38.36$$

$$w_h = \frac{V}{D G_L \times 38.36} = \frac{500{,}000}{0.3 \times 0.6 \times 15 \times 10^6 \times 38.36} = \underline{\underline{4.8\,mm}}$$

b) In an elastic soil, $\tau = G\gamma$

moving outward from pile, $\gamma = \dfrac{\partial w}{\partial r}$ settlement

equilibrium gives that $r\tau$ is constant.

$\therefore$ $\tau \propto \frac{1}{r}$

$\gamma \propto \frac{1}{r}$

$w \propto \ln(r)$

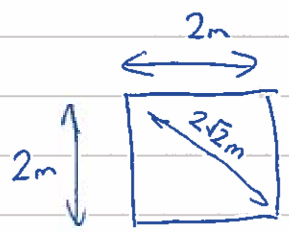
At $r = 0.15\,m$ $w = \cancel{4.2}\ 4.8\,mm$

$r = 12\,m$ $w = 0$

$$\therefore\ w = -1.095\ln\left(\frac{r}{12}\right)$$

@ $r = 2\,m$ $w = 1.97\,m$

$r = 2\sqrt{2}\,m$ $w = 1.58\,m$



c) Total load = 2 MN
load per pile = 500 kN (as in b)

Each pile is influenced by the other three as soil is dragged downwards

Total settlement = sum of pile-soil settlement + movement of soil.

$$\rho = 4.8 + 2 \times 1.97 + 1.58$$
$$= \underline{\underline{10.32\,mm}}$$

d) If pile cap also carries load, loads carried by each pile will be reduced.

Could do a shallow foundation calculation to find stiffness of pile cap alone and then calculate load shares to equate settlements of piles and cap.

Additional complexity comes from extra downwards movement of soil within influence zone of cap, which varies with depth making calculation by hand difficult. Could be dealt with by numerical method such as F.E.

Q1 Pile settlement

19 attempts, Average mark 14.33/20 (72%), Maximum 19, Minimum 6.

Not a very popular question, but in general well answered by most students. Most students coped well with superposing settlement troughs of the four piles, but did not comment on the settlement trough for the raft in part (d).

a) Sand initially at K_0 conditions
As pile tip approaches, soil below pile tip is subjected to very large vertical stresses, inducing large horizontal stresses.
Soil is then heavily sheared and pushed outwards around pile tip inducing large horizontal stresses but reduced vertical stresses.
Cyclic movement of pile during driving causes friction fatigue. Soil compresses due to cyclic shear, allowing horizontal stresses to reduce as soil moves away from pile tip.
Final hammer blow leaves behind locked-in soil stresses due to axial extension of pile after impact. Top part of pile carries downward shear in unstressed state.
API code deals with increased H stresses due

b) to soil being pushed outwards with K factor being higher for closed piles.
Friction fatigue dealt with indirectly through limiting τ_s value which reduces predicted capacity of very long piles.
Contraction of loose sands is assumed to reduce friction angle δ rather than K. Erroneous but works as ($K \tan \delta$) is essentially a single term in the design code.

$D = 0.5$ $\quad t = 50mm$ $\quad$ medium dense sand

$V = 2MN$

$\delta = 25°$ $\quad \tau_{s,lim} = 85\ kPa$

$N_q = 20$ $\quad q_{b,lim} = 4.8\ MPa$

$\sigma'_{v0} = 9z$

$K = 1$

$\tau_{sf} = 9z \times 1 \times \tan 25 < 85kPa$

$4.2z < 85$

limit at $\sim 20m$ depth

$q_b = 20 \times 9z = 180z < 4800\ kPa$

limit at $\sim 26.7m$ depth

If neither limit is reached:

$V_{ult} = (\pi \times \cancel{0.25}\ 0.5\ L) \times 2.1L$ — shaft

$+ (\pi \times 0.25^2) \times 180L$

$= \pi \left[1.05L^2 + 11.25L \right] = 2000$

$3.30L^2 + 35.34L - 2000 = 0$

$$L = \frac{-35.34 + \sqrt{35.34^2 + 8000 \times 3.3}}{6.6}$$

$= \underline{\underline{19.84m}}$ $\quad$ limits ok!

c) $M_p = \left(\frac{d_o^3 - d_i^3}{6}\right)\sigma_y$

$= \frac{0.5^3 - 0.4^3}{6} \times 300 = 3.05$ MNm

$K_p = \frac{1+\sin(35)}{1-\sin(35)} = 3.65$ $K_p^2 = 13.33$

$n = \underline{120}$ kPa/m

Short pile: $L = 25$ $D = 0.5$ $L/D = 50$

extrapolating $\frac{H_{ult}}{nD^3} \sim 200$

Long pile: $\frac{M_p}{nD^4} = \frac{3050}{120 \times 0.5^4} = 407$

$H_{ult}/nD^3 \simeq 40 < 200$ so critical

$\Rightarrow H_{ult} = 40 \times 120 \times 0.5^3$

$= \underline{600 \text{ kN}}$

Failure by bending of the pile (long failure mechanism)

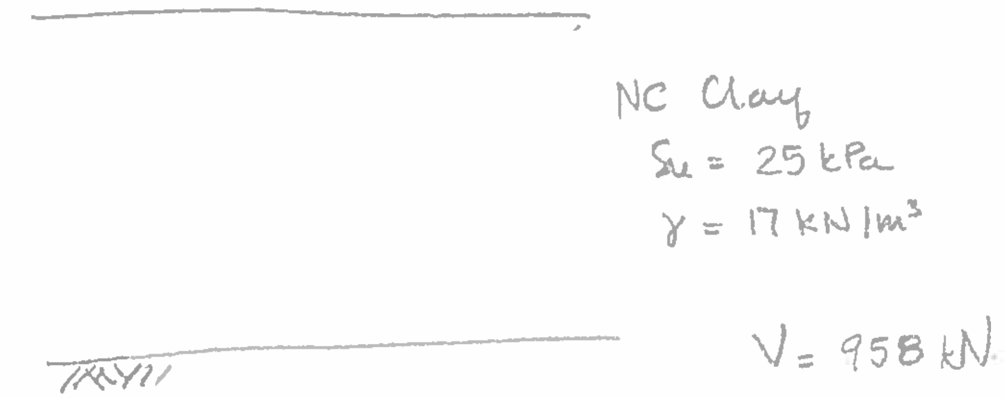
d) Lateral loading mobilises most strength of surficial soil, whereas vertical loading uses deep soil strength. Little interaction between the two loading regimes so ~~a~~ high proportions of both failure loads can be carried simultaneously.

Q2 Pile design

29 attempts, Average mark 13.93/20 (70%), Maximum 18, Minimum 6.

A very popular question, being answered by almost all students. The calculations in parts b & c were well tackled with only minor arithmetic errors. The descriptive parts (a) and (d) were of variable quality with some very complete answers and some lacking the detail needed for the marks on offer. That said, even the shorter answers tended to be along the right lines.

PROBLEM 3



(a) (1) Design approach 1, combination 1

$Y_F = 1.35$

$q_{ult} = S_c d_c N_c S_u + \gamma h$

$S_c = 1 + 0.2B/L = 1.2$ for square foundation

$d_c = 1.0$ for no embedment $(h \approx 0)$

$N_c = 2 + \pi = 5.14$

$q_{ult} = (1.2)(1.0)(5.14)(25 \text{ kPa}) = 154.2 \text{ kPa}$

$Y_F V \leq q_{ult} B^2$

$B > \sqrt{\frac{Y_F V}{q_{ult}}} = \sqrt{\frac{(1.35)(958 \text{ kN})}{(154.2 \text{ kPa})}} = 2.9 \text{ m}$

(2) Design approach 1, combination 2

$\gamma_M = 1.4$

$S_{ud} = \frac{25 \text{ kPa}}{1.4} = 17.9 \text{ kPa}$

$q_{all} = S_c d_c N_c S_{ud} + \gamma h$

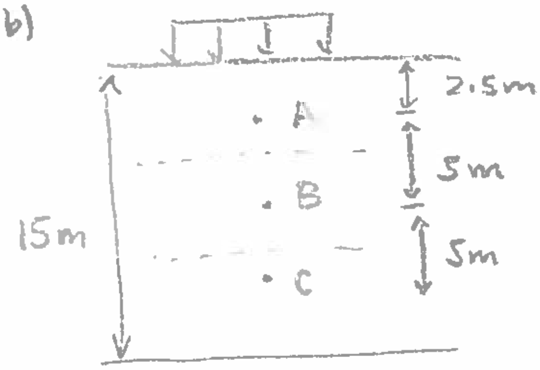
$q_{all} = (1.2)(1.0)(5.14)(17.9\,kPa) = 110.4\,kPa$

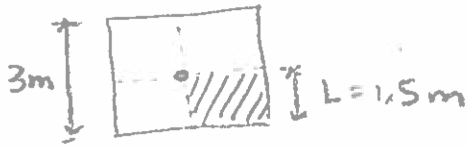
$V \leq q_{all} B^2$

$B \geq \sqrt{\frac{V}{q_{all}}} = \sqrt{\frac{958\,kN}{110.4\,kPa}} = 2.95\,m$

$\Rightarrow \boxed{B \approx 3m}$

(b)





$\Delta\sigma = 4 I_r q$

Use Fadum's chart from data book

$q = \frac{V}{B^2} = \frac{958\,kN}{9\,m^2} = 106.4\,kPa$

@A $\quad m = n = \frac{L}{z} = \frac{1.5m}{2.5m} = 0.6$

$I_r \approx 0.14$, $\quad \Delta\sigma = (4)(0.14)(106\,kPa) = 59.4\,kPa$

@B $\quad m = n = \frac{1.5m}{7.5m} = 0.2$

$I_r \approx 0.023 \quad \Delta\sigma = (4)(0.023)(106\,kPa) = 9.75\,kPa$

@C $\quad m = n = \frac{1.5m}{12.5m} = 0.12$

$I_r \approx 0.01 \quad \Delta\sigma = (4)(0.01)(106\,kPa) = 4.24\,kPa$

(c) $w = 56\%$ $\qquad e_o = (2.7)(0.56) = 1.512$

$$\rho = \frac{\Delta e}{1+e_o} H_o = \frac{H_o \lambda}{1+e_o} \ln\left(\frac{\sigma'_{vf}}{\sigma'_{vo}}\right)$$

The clay is NC.

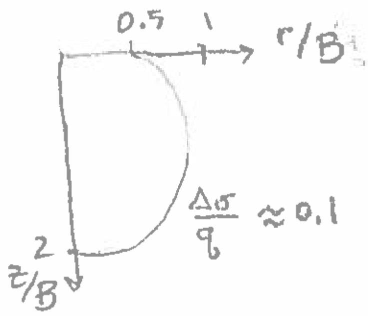
	e_o	H_o	σ'_{vo} (kPa)	$\Delta\sigma$ (kPa)	σ'_{vf} (kPa)	ρ (cm)
A	1.512	5m	18	59.4	77.4	95.8
B	1.512	5m	56.3	9.75	66.1	10.5
C	1.512	5m	90	4.24	94.24	3.0

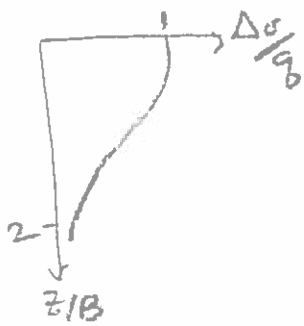
$\boxed{\rho_{TOT} = 109.3 \text{ cm}}$

The prediction of consolidation settlements can be improved by:

- obtaining properties (water content and λ) at more locations
- Subdivide the clay layer into more sub-layers, especially at the top where the variation in stress is changing more rapidly.

Most of the stress increase will be concentrated in the top half of the clay layer.





(d) One of the main issues with shallow footings is the larger potential for differential settlements. Shallow footings tend to result in larger settlements than deep foundations because the increase in stress due to the surface load can be large.

The footings for this building need to have a size of 3m × 3m. The column spacing is 7m o.c., which means that footings will cover almost all the footprint of the building. In addition, the long-term consolidation settlement is estimated (roughly) to be over 1 m, which is very large. The potential for large differential settlements leading to unacceptable distortion are very high.

Therefore, shallow footings are not an acceptable foundation system for this building in this configuration

Potential improvements, before considering deep foundations, may be:

- a raft foundation: the contact pressure would decrease, leading to smaller consolidation settlements. The raft also needs to be sufficiently stiff to prevent differential settlements. Cost may be similar to that of deep foundations

- Increasing embedment will increase capacity

but it will also lead to higher contact pressure
and, therefore, settlements _

Q3 Square footing design

25 attempts, Average mark 13.12/20 (65.6%), Maximum 16.5, Minimum 6.5.

This was a popular question. Only 3 students recognised that the design factors should be applied either to load or strength properties, not to both. Most students were able to use Fadum's chart correctly to estimate load increase in the soil, but a number of them used the factored load, instead of the unfactored one. About half of the students could not calculate consolidation settlements correctly.

PROBLEM 4

Sand

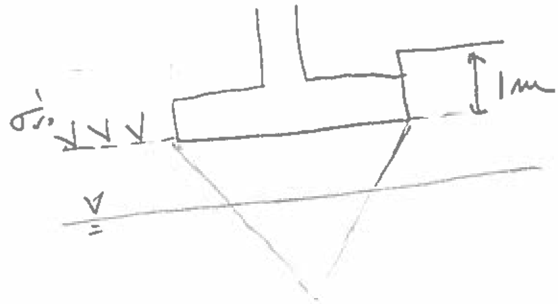
$\phi' = 32^\circ$

$\gamma_{sat} = 20\,kN/m^3$ $\gamma_d = 16.4\,kN/m^3$

$G_s = 2.65$

(a) $V = 820\,kN$

$h = 1\,m$



Design approach 1, combination 1:

$\gamma_F = 1.35$

$$q_{ult} = s_q N_q \sigma'_{v0} + s_\gamma N_\gamma \frac{\gamma' B}{2}$$

$$N_q = \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) e^{\pi \tan\phi} = (3.25)(7.12) = 23.1$$

$$s_q = 1 + \frac{B \sin\phi}{L} = 1 + \sin\phi = 1.53$$

$$\sigma'_{v0} = \gamma_d h = (16.4\,kN/m^3)(1\,m) = 16.4\,kPa$$

$$N_\gamma = 2(N_q - 1)\tan\phi = (2)(23.1-1)\tan 32^\circ = 27.6$$

$$s_\gamma = 1 - 0.3\,B/L = 0.7$$

Since the water table is below the foundation, but still withing the zone that is affected by the failure mechanism we should consider which γ should be used in the bearing capacity equation. Ideally, we could calculate an effective value taking into account what portions of the failure mechanism lie below the water table. However, using γ' is conservative for bearing capacity calculations. It is also safer if the water table were to rise.

Assume $B \rightarrow$ calculate $q_{ult} \rightarrow$ compare with $\frac{\gamma_F V}{Area} = q$

$$q_{ult} = (1.53)(23.1)(10\,kPa) + \frac{(0.7)(27.6)(10\,kN/m^3)(2m)}{2} =$$

$$= 353 + 96.6B = 353 + 193 = 546\,kPa$$

$$q = \frac{\gamma_F V}{\left(\frac{\pi B^2}{4}\right)} \leq q_{ult}$$

B (m)	q_{ult} (kPa)	A (m^2)	q (kPa)
2	546	3.14	352 ✓
1	450	0.79	1401 ×
1.5	498	1.77	625 ×
1.7	517	2.27	488 ✓

$\boxed{B = 1.70\,m}$

Anything more refined would be difficult to construct. In fact, the more realistic specification would be

$B = 1.7\,m$

(b) $V = 820\,kN$ $B = 2\,m$

$M_x = 123\,kN$ $L = 3\,m$

$M_y = 574\,kN$

Use Meyerhof's approach and find effective foundation sizes:

$e_x = \frac{M_x}{V} = \frac{123}{820} = 0.15\,m$ $B' = B - 2e_x = 1.7\,m$

$e_y = \frac{M_y}{V} = \frac{574}{820} = 0.7\,m$ $L' = L - 2e_y = 1.6\,m$

* Note B is usually taken as smaller than L in calculations

$q_{ult} = s_q N_q \sigma'_{vo} + s_\gamma N_\gamma \frac{\gamma' B}{2}$

Combination 1

$\gamma_F = 1.35$

$N_q = 23.1$

$N_\gamma = 27.6$

$s_q = 1 + \frac{1.6}{1.7} \sin 32° = 1.5$

$s_\gamma = 0.72$

$q_{ult} = (1.5)(23.1)(10) + \frac{(0.72)(27.6)(10)(1.6)}{2} =$

$= 346 + 159 = 505\,kPa$

$\frac{\gamma_F V}{B'L'} \leq q_{ult}$

$\frac{(1.35)(820\,kN)}{(1.6\,m)(1.7\,m)} = 407\,kPa < 693\,kPa$

✓ OK

Combination 2

$\gamma_M = 1.25$

$\phi'_d = \tan^{-1}\left(\frac{\tan\phi'}{\gamma_M}\right) = 26°$

$N_q = \tan^2\left(\frac{\pi}{4} + \frac{\phi'_d}{2}\right) e^{\pi \tan\phi'_d}$

$= (2.56)(4.63) = 11.8$

$s_q = 1 + \frac{1.6}{1.7} \sin 26 = 1.41$

$\sigma'_{vo} = 16.4\,kPa$

$N_\gamma = 2(N_q - 1)\tan\phi'_d = 10.5$

$s_\gamma = 1 - (0.3)\left(\frac{1.6}{1.7}\right) = 0.72$

$q_{ult} = (1.41)(11.8)(10\,kPa) + \frac{(0.72)(10.5)(10\,kN/m^2)(1.6}{2}$

$= 166 + 60 = 226\,kPa$

$$\frac{V}{B'L'} \leq q_{all}$$

$$\frac{(820\ kPa)}{(1.6m)(1.7m)} = 301\ kPa > 226\ kPa$$

X NO!

(c) $D = 1.5m$

$$q = \frac{820\ kPa}{\frac{\pi (1.5m)^2}{4}} = 463\ kPa$$

Assumption: the footing is acting as a rigid punch

$$w = \frac{\pi (1-\nu)}{4G} q_{avg} \frac{D}{2} = \frac{\pi (1-0.3)}{4\ (20MPa)} (463\ kPa) \left(\frac{1.5m}{2}\right) = 9.5\ mm$$

The settlement is low and the potential for eccessive differential settlement is equally low →

No concern for serviceability limit state -

(d) If the footing is also subjected to horizontal loads the full V-H-M solution should be used. In general a combination of loads will result in capacities that are lower for each term than the ultimate value for each load case taken independently

Q4 M-V loading on shallow foundation

18 attempts, Average mark 11.83/20 (59.17%), Maximum 17, Minimum 6.

As in problem 3, students applied both design factors at once. A few applied one of the factors, and chose either to decrease strength or increase load, but only one checked both cases. Students seemed confused by the fact that the proposed sizing was not adequate. Most students correctly identified the procedure to handle M-V loading.