General Briefing on Code of Practice for Foundations 2017

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Contents

- An overview of the 2017 Foundation Code of Hong Kong;
- To highlight revisions / additions in the 2017 Code as compared with the 2004 Code with discussions on the rationale behind;
- Discussions on some difficult or argumentative aspects.

Overview

- The 2017 Code is an updated version of the 2004 Foundation Code which was promulgated in 2004 by the Buildings Department;

- As a code of practice, the Code contains provisions for design, site investigation, construction practice and testing;

- In parallel, the Building (Construction) Regulations (the most updated version promulgated in 2012) does contain provisions for foundation work which have been incorporated in the Foundation Codes. Of course the Foundation Codes contain more details than the Regulations. It is an intention of the Government that detail provisions of the engineering requirements be placed in the Codes of Practice while leaving the B(C)R to contain only principles;

- The Foundation Codes tend to be local codes embracing recognized local practices;

Overview

- Unlike the Concrete Code and the Steel Code which are largely based on the British Standards BS8110 and BS5950, the Foundation Code draws lesser materials from BS8004;

- References, especially structural element design are also made to other local design codes such as the Concrete Code and the Steel Code;

- Comparing with the 2004 Code, the 2017 Code also explicitly allows design of structural steel sections for foundation elements to be based on the limit state design. However, in order to avoid overstressing steel beyond yield strength during loading tests (which requires at least twice the allowable working load), some further limitations on section design have to be imposed.

Allowable Bearing Pressure

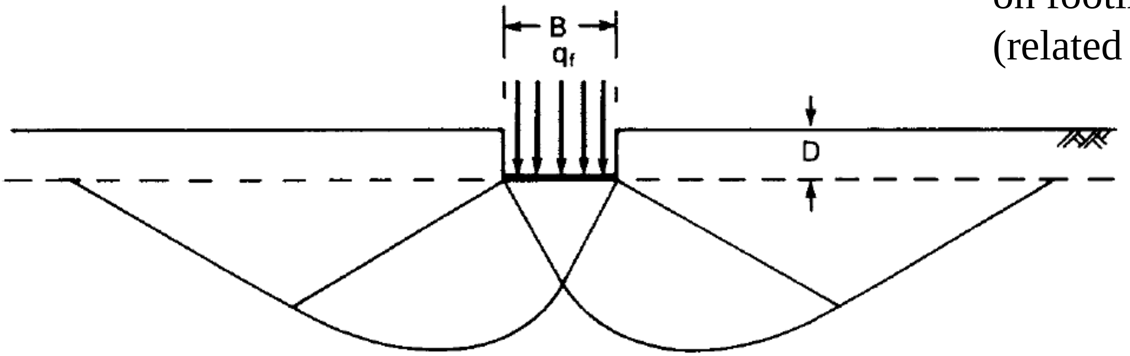
- An important content in Chapter 2 is the provision on the “allowable bearing pressure” of ground to the foundation both in the 2004 Code and 2017 Code;
- Basically, the 2004 Code has expanded the table “safe bearing capacities of ground under foundation” of the old Building (Construction) Regulation – 1985 in constructing its Table 2.1. (The current B(C)R has removed this table). The 2017 Code has further expanded the table by adding the item for sedimentary rock;
- The allowable bearing pressure is used to check against working load (or characteristic load);
- The allowable bearing pressure can be taken as the ultimate bearing capacity divided by a factor of safety of 3;
- The allowable bearing pressure can however, be based on “Presumed Values” listed in Table 2.1 which depends on the strength / quality of the soil / rock only;

Allowable Bearing Pressure

- However, it should be noted that the allowable bearing capacity of a building structure should depends also the dimensions and adjacent surcharge of the foundation which affects the ultimate bearing capacity involving failure of shear surface. The foundation dimensions also directly affects settlement;
- So strictly speaking it would be a trial and error process for foundation design – trying the foundation dimensions against allowable bearing pressure which varies with foundation dimensions. The process is tedious;
- Thus, like BS8004:1986, the concept of “Presumed bearing value” is used in the Code depending on quality and strength of the soil / rock only which are now universally acceptable and are normally conservative values.

$$q_f = cN_c + \gamma D N_q + \frac{1}{2}\gamma B N_\gamma$$

Ultimate bearing capacity depends also on footing plan width and depth (related to surcharge)



Settlement again depends also on footing plan width

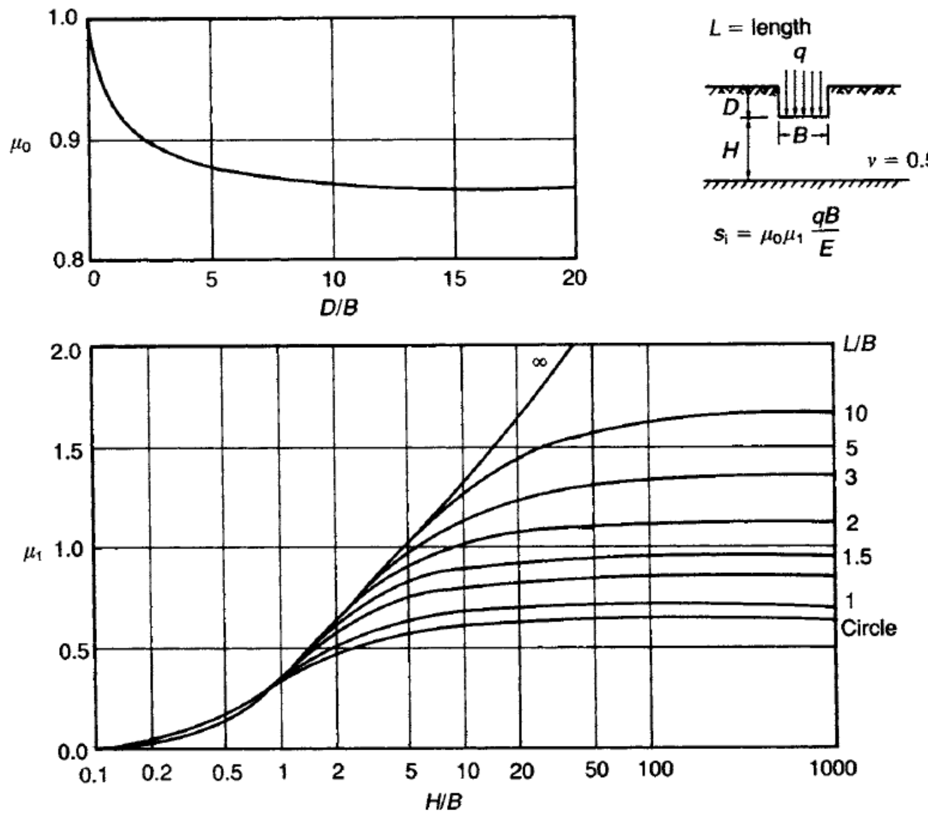


Figure 5.15 Coefficients for vertical displacement.

2.2.2.2 *Allowable bearing pressure and presumed allowable bearing value*

2.2.2.2.1 *General.* Universally applicable values of allowable bearing pressure cannot be given. Factors affecting bearing capacity have been discussed in **2.1.2** and they show that, for any important structure, the allowable bearing pressure cannot be assessed without taking into account the effect of settlement. However, foundation design is at present possible only by trial and error methods, so that it is desirable to have some basis for preliminary design assumptions. Therefore, the concept of presumed bearing values is used in this code. It is emphasized that the presumed bearing value should be used by the designer only for preliminary foundation design purposes and, in all cases, he should then review and, if necessary, amend his first design. This will frequently entail an estimate of settlements.

Extract from BS8004:1986

5.4.4 Combined bearing and settlement check using prescriptive measures

*COMMENTARY ON **5.4.4***

The design of many simple foundations has traditionally been checked against "allowable bearing pressures" which are normally very conservative estimates of the ultimate bearing resistance of the ground, selected on the basis of soil and rock descriptions. The settlement of a spread foundation that has been designed using allowable bearing pressures is commonly assumed to be acceptable.

In BS EN 1997-1, "allowable bearing pressures" are now called "presumed bearing resistance" and this method of design is termed a "prescriptive method".

Extract from BS8004:2015

Allowable Bearing Pressure

- BS8004:1986 also has a table of presumed allowable pressure as similar to our Foundation Code but emphasized that the values are for preliminary design. The designer should then review his design afterwards;
- But in the updated version of BS8004:2015, the requirement of review is not included and reference made to BS EN 1997-1 for the presumed values;
- Nevertheless, both the 2004 and 2017 Codes retain the provision of "Rational Design Method" for deriving the allowable bearing pressure if presumed values are not to be used;
- In addition, in the 2017 Code, the bearing capacity equations are added for the user's estimation of bearing capacity for shallow foundations (depth $\leq$ 3m) which are dependent on the foundation dimensions and basic parameters of the soil, though the depth restriction is not found in other Standards such as BS8004:2015 (6.4.1.2) or the Eurocode 7 (6.5.2.2 and Annex D).

Table 1 — Presumed allowable bearing values under static loading (see **1.2.3** and **1.2.4**)

NOTE These values are for preliminary design purposes only, and may need alteration upwards or downwards. No addition has been made for the depth of embedment of the foundation (see **2.1.2.3.2** and **2.1.2.3.3**).

Category	Types of rocks and soils	Presumed allowable bearing value		Remarks
		kN/m^2 [a]	kgf/cm^2 [a] $tonf/ft^2$	
Rocks	Strong igneous and gneissic rocks in sound condition	10 000	100	These values are based on the assumption that the foundations are taken down to unweathered rock. For weak, weathered and broken rock, see **2.2.2.3.1.12**
	Strong limestones and strong sandstones	4 000	40	
	Schists and slates	3 000	30	
	Strong shales, strong mudstones and strong siltstones	2 000	20	
Non-cohesive soils	Dense gravel, or dense sand and gravel	> 600	> 6	Width of foundation not less than 1 m. Groundwater level assumed to be a depth not less than below the base of the foundation. For effect of relative density and groundwater level, see **2.2.2.3.2**
	Medium dense gravel, or medium dense sand and gravel	< 200 to 600	< 2 to 6	
	Loose gravel, or loose sand and gravel	< 200	< 2	
	Compact sand	> 300	> 3	
	Medium dense sand	100 to 300	1 to 3	
	Loose sand	< 100 Value depending on degree of looseness	< 1	
Cohesive soils	Very stiff boulder clays and hard clays	300 to 600	3 to 6	Group 3 is susceptible to long-term consolidation settlement (see **2.1.2.3.3**). For consistencies of clays, see Table 5
	Stiff clays	150 to 300	1.5 to 3	
	Firm clays	75 to 150	0.75 to 1.5	
	Soft clays and silts	<75	<0.75	
	Very soft clays and silts	Not applicable		
Peat and organic soils		Not applicable		See **2.2.2.3.4**
Made ground or fill		Not applicable		See **2.2.2.3.5**

[a] 107.25 kN/m^2 = 1.094 kgf/cm^2 = 1 $tonf/ft^2$.

2.2.2 – Presumed Values of Allowable Bearing Pressure

- Comparing with the 2004 Code, the 2017 Code contains following revisions / additions:

- 2.2.2(5)
  - Presumed values of 100kPa (if dry) or 50 kPa (if submerged) be used for minor temporary structures on horizontal ground. Presumably SI in accordance Section 3 by 2.2.2(1)(a) can be exempted. The Code gives examples : fencing and hoarding.

  - This is a new sub-clause but is more or less a current practice;
  - By "temporary", permanent structures such as planter boxes, small manholes or even light covered walkway have been excluded;
  - By 4.2.2(2), plate load tests can be exempted even bearing capacity equations are used to determine allowable bearing pressure;

2.2.2 – Presumed Values of Allowable Bearing Pressure

- Category 2 in Table 2.1

- The category specifies the presumed allowable bearing pressure for meta-sedimentary rock which is an addition to the previous code.

- Under 85% TCR, the presumed allowable bearing pressure of meta-sedimentary rock is 3000kPa which is also a ceiling value, being equivalent that of volcanic rock of 50%TCR;

- So the clause is useful for structure founding on meta-sedimentary rock.

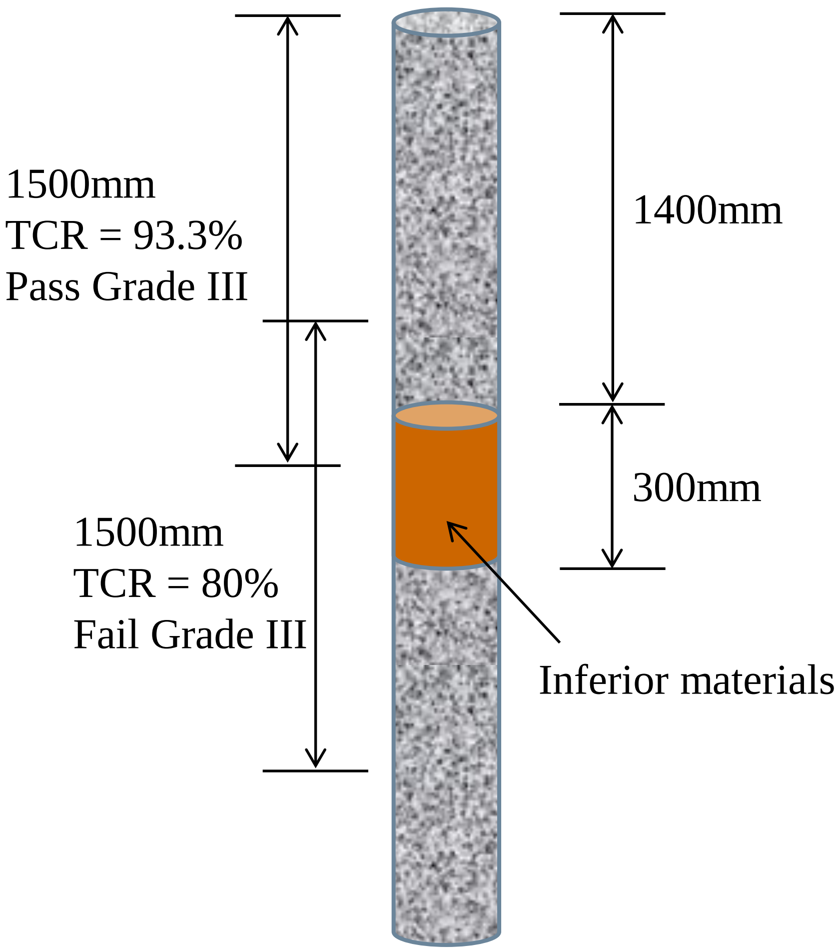
2.2.2 – Presumed Values of Allowable Bearing Pressure

- Note (5) under Table 2.1

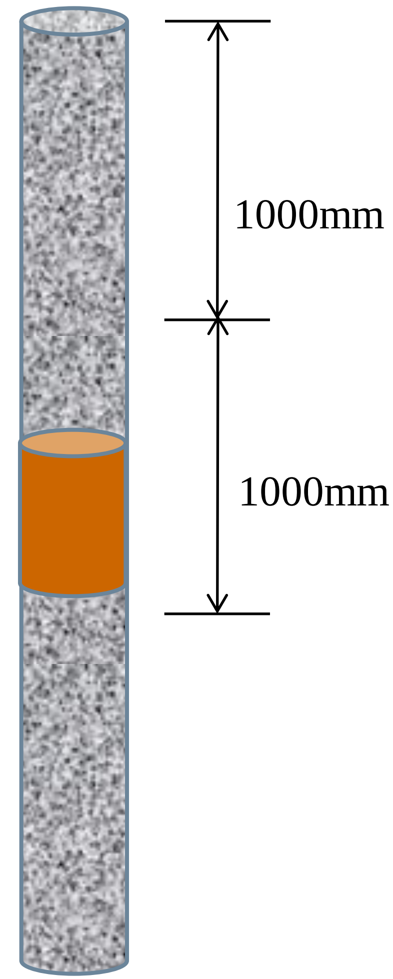
- The note re-defines TCR :
  (1) Length of each core-run becomes 1m in 2017 Code vs 1.5m in 2004 Code;
  (2) TCR to be calculated on consecutive core-runs. There is ambiguity in the 2004 Code that TCR can be based on consecutive core-runs or the "worst 1.5m run".

- Under 85% TCR, the presumed allowable bearing pressure of meta-sedimentary rock is 3000kPa which is also a ceiling value, being equivalent that of volcanic rock of 50%TCR;
- So the clause is useful for structure founding on meta-sedimentary rock.

2004 Code if taking worst 1.5m core-run or consecutive 1.5m core-runs



2017 : Shorter core-run, generally higher risk of failure because (i) more core-runs, though may be offset by shorter core length and (ii) smaller denominator for TCR calculations



2.2.4 – Bearing Capacity Equation Method

- This is a new clause in the 2017 Code quoting the bearing capacity equation method for determination of "ultimate bearing capacity of shallow foundation" (depth of bottom level ≤ 3m)

- The equations were developed by Vesic based on theoretical approach which are listed in Geoguide 1 – $q_u = cN_c + qN_q + 0.5B\gamma N_\gamma$

- By the approach, generally higher bearing capacities are resulted and the bearing capacity increases with foundation plan dimensions (by $0.5B\gamma N_\gamma$), as in contrast with the presumed values dependent only on quality of rock or soil.

- The bearing capacity factor qN_q, q can be very large for deeply seated footing, thus contributing substantially to q_u. However, the Code limits to the use of the equation to 3m depth foundation.

- The Code also limits the q_u to 3000kPa or roughly $q_{allowable}$ = 1000kPa

Derivation of the *N* coefficients for the Ultimate Bearing Capacity of Footing

Reference is made to the assumed failure mechanism diagram shown in Figure A-1.

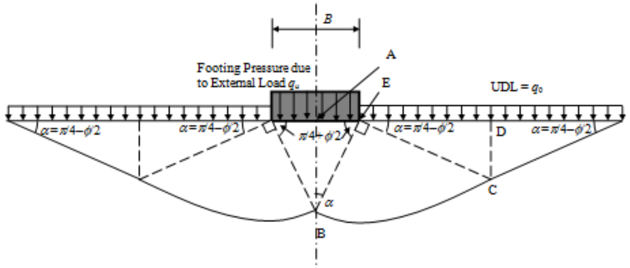


Figure A-1 – Assumed Failure Mechanism for Ultimate Bearing Capacity of Footing of Infinite Length

EB is the base line of the radial zone taken as $r=\dfrac{B}{2\csc\alpha}$.

By geometry, $\alpha=\dfrac{\pi}{4}-\dfrac{\phi}{2}$

Within the radial zone EBC, the radial length is given by
$r=r_0e^{\theta\tan\phi}=\dfrac{B}{2\csc\alpha}e^{\theta\tan\phi}$ where θ is measured from EB. The derivation of the equation is based on principle of soil mechanics.

Length of AB is $\dfrac{B}{2}\cot\alpha$; Length of EC is $\dfrac{B}{2\csc\alpha}e^{\frac{\pi}{2}\tan\phi}$;

Length of DC is $\dfrac{B}{2\csc\alpha}e^{\frac{\pi}{2}\tan\phi}\sin\alpha=\dfrac{B}{2}e^{\frac{\pi}{2}\tan\phi}$

Length of ED is $\dfrac{B}{2\csc\alpha}e^{\frac{\pi}{2}\tan\phi}\cos\alpha=\dfrac{B}{2}\cot\alpha\times e^{\frac{\pi}{2}\tan\phi}$

Ignoring the weight of the soil and the horizontal stress under the footing is minor stress, the uniform pressure on AB is

$P_{AB}=q_uK_a-2c\sqrt{K_a}=q_u\tan^2\alpha-2c\tan\alpha$
where q_u is the ultimate stress immediately beneath the footing due to external load.

Uniform pressure on DC is
$P_{DC}=qK_p+2c\sqrt{K_p}=q\cot^2\alpha+2c\cot\alpha$.

The free body chosen for derivation of (Eqn 3.2) is shown in Figure A-2 as extracted from Figure A-1. The applied forces are also shown.

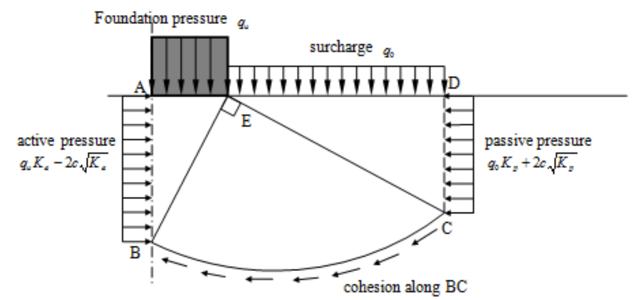


Figure A-2 – Assumed Failure Mechanism for Ultimate Bearing Capacity of Footing of Infinite Length

Considering rotational equilibrium of the soil mass ABCDEA in Figure A-2 and taking moment about E, the overturning moment is given by

$$M_{AE}=q_u\frac{1}{2}\left(\frac{B}{2}\right)^2=\frac{B^2}{8}q_f \quad \text{(Eqn A-1)}$$

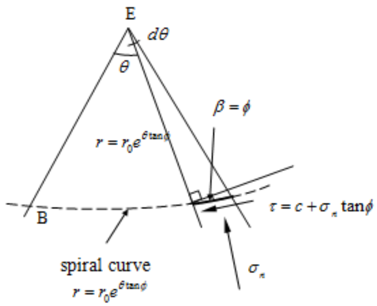
$$M_{AB}=P_{AB}\times\frac{1}{2}\times\left(\frac{B}{2}\cot\alpha\right)^2=\frac{B^2}{8}q_u-\frac{B^2}{4}c\cot\alpha \quad \text{(Eqn A-2)}$$

The restoring moments are

$$M_{ED}=q_0\times\frac{1}{2}\left(\frac{B}{2}\cot\alpha\times e^{\frac{\pi}{2}\tan\phi}\right)^2=\frac{B^2}{8}q_0\cot^2\alpha\times e^{\pi\tan\phi} \quad \text{(Eqn A-3)}$$

$$M_{DC}=\left(q_0\cot^2\alpha+2c\cot\alpha\right)\times\frac{1}{2}\times\left(\frac{B}{2}e^{\frac{\pi}{2}\tan\phi}\right)^2=\frac{B^2}{8}q_0\cot^2\alpha\times e^{\pi\tan\phi}+c\frac{B^2}{4}\cot\alpha\times e^{\pi\tan\phi}$$
(Eqn A-4)

For the contribution of the spiral curve BC, from Figure A-3, an elementary arc is under a normal stress σ_n and shear stress $\tau=c+\sigma_n\tan\phi$. As proven in Figure A-2 where $\beta=\phi$ the component perpendicular to the radius to E effective in restoring rotational equilibrium is $ds(\tau\cos\phi-\sigma_n\sin\phi)=cds\cos\phi$ where ds is the length of the arc and $ds=\dfrac{rd\theta}{\cos\phi}$. As the moment arm is r, the moment contributed is $dM_{BC}=crds\cos\phi=cr^2d\theta$



To find β,

$$\tan\beta = \frac{dr}{rd\theta} = \frac{1}{r}\frac{dr}{d\theta}$$

$$r = r_0 e^{\theta\tan\phi} \Rightarrow \frac{dr}{d\theta} = r_0 e^{\theta\tan\phi}\tan\phi$$

$$\therefore \frac{1}{r}\frac{dr}{d\theta} = \frac{r_0 e^{\theta\tan\phi}\tan\phi}{r_0 e^{\theta\tan\phi}} = \tan\phi$$

$$\therefore \beta = \phi$$

Figure A-3 – Proof of the Contribution of the Spiral BC

So for BC, $M_{BC} = c\int_0^{\frac{\pi}{2}} r^2 d\theta = \int_0^{\frac{\pi}{2}}\left(\frac{B}{2\csc\alpha}e^{\theta\tan\phi}\right)^2 d\theta = \frac{cB^2\csc^2\alpha}{8\tan\phi}\left(e^{\pi\tan\phi}-1\right)$

(Eqn A-5)

Equating $M_{AE} + M_{AB} = M_{ED} + M_{DC} + M_{AB}$

$$\frac{B^2}{8}q_u + \frac{B^2}{8}q_u - \frac{B^2}{4}c\cot\alpha = \frac{B^2}{8}q_0\cot^2\alpha\times e^{\pi\tan\phi} + \frac{B^2}{8}q_0\cot^2\alpha\times e^{\pi\tan\phi} + c\frac{B^2}{4}\cot\alpha\times e^{\pi\tan\phi}$$
$$+\frac{cB^2\csc^2\alpha}{8\tan\phi}\left(e^{\pi\tan\phi}-1\right)$$

(Eqn A-6)

Simplifying (Eqn A-6)

$$q_u = q_0\cot^2\alpha\times e^{\pi\tan\phi} + c\cot\alpha\times e^{\pi\tan\phi} + c\cot\alpha + \frac{c\csc^2\alpha}{2\tan\phi}\left(e^{\pi\tan\phi}-1\right)$$

$$\Rightarrow q_u = q_0 e^{\pi\tan\phi}\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right) + c\left[\cot\alpha\left(e^{\pi\tan\phi}+1\right) + \frac{\csc^2\alpha}{2\tan\phi}\left(e^{\pi\tan\phi}-1\right)\right]$$

Making use of the relation $\tan\phi = \frac{-1}{\tan\left(\frac{\pi}{2}+\phi\right)} = \frac{-1}{\tan 2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)} = \frac{\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)-1}{2\tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right)}$,

$\cot\alpha = \cot\left(\frac{\pi}{4}-\frac{\phi}{2}\right) = \tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right)$ and $\csc^2\alpha = 1+\cot^2\alpha = 1+\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)$

$$\therefore \left[\cot\alpha\left(e^{\pi\tan\phi}+1\right) + \frac{\csc^2\alpha}{2\tan\phi}\left(e^{\pi\tan\phi}-1\right)\right]$$

$$= \left[\cot\alpha\left(e^{\pi\tan\phi}+1\right)\tan\phi + \frac{\csc^2\alpha}{2}\left(e^{\pi\tan\phi}-1\right)\right]\cot\phi$$

$$= \left[\tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right)\left(e^{\pi\tan\phi}+1\right)\frac{\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)-1}{2\tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right)} + \frac{1}{2}\left[1+\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)\right]\left(e^{\pi\tan\phi}-1\right)\right]\cot\phi$$

$$= \left[\frac{1}{2}\left(e^{\pi\tan\phi}+1\right)\left[\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)-1\right] + \frac{1}{2}\left[1+\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)\right]\left(e^{\pi\tan\phi}-1\right)\right]\cot\phi$$

$$= \left[\begin{array}{l}\frac{1}{2}e^{\pi\tan\phi}\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right) + \frac{1}{2}\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right) - \frac{1}{2}e^{\pi\tan\phi} - \frac{1}{2} + \frac{1}{2}e^{\pi\tan\phi} \\ +\frac{1}{2}e^{\pi\tan\phi}\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right) - \frac{1}{2} - \frac{1}{2}\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)\end{array}\right]\cot\phi$$

$$= \left[e^{\pi\tan\phi}\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)-1\right]\cot\phi$$

Putting $N_q = e^{\pi\tan\phi}\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)$ (Eqn A-7)

$$\left[\cot\alpha\left(e^{\pi\tan\phi}+1\right) + \frac{\csc^2\alpha}{2\tan\phi}\left(e^{\pi\tan\phi}-1\right)\right] = \left(N_q-1\right)\cot\phi = N_c \quad \text{(Eqn A-8)}$$

$$\therefore q_u = q_0 N_q + cN_c \quad \text{(Eqn A-9)}$$

To include the effects of the weight of the soil mass ABCDEA, we need to cater for its weight and the eccentricity. The practice is to express its effect as a factor over the parameter $0.5\gamma B$ where γ is the unit weight of the soil mass and allow a coefficient N_γ to express its effects in restoring rotational equilibrium. The exact derivation of N_γ is complicated which cannot be expressed by a simple formula. Various researchers put forward simplified expression for this term. $N_\gamma = 2(N_q+1)\tan\phi$ is used by Vesic and adopted in GEOGUIDE 1 in Hong Kong.

Summing up, for an infinite strip footing of width B

$$q_u = 0.5B\gamma N_\gamma + q_0 N_q + cN_c \quad \text{(Eqn A-10)}$$

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

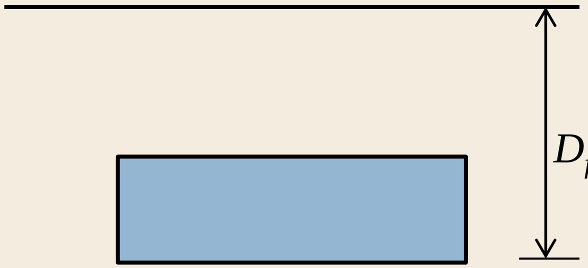
$N_c = \left(N_q-1\right)\cot\phi$

$N_\gamma = 2\left(N_q+1\right)\tan\phi$

2.2.4 – Bearing Capacity Equation Method

- The Code puts forward an equation in 2.2.4 (found in Craig's Soil Mechanics)

$$q_a = \frac{q_u - q_0}{F} + q_0 \qquad q_0 = \gamma_s D_f$$



- The equation is actually derived from $F = \frac{q_u - q_0}{q_a - q_0}$

- By the equation, it is applying factor of safety to the "net ultimate bearing pressure" and "net allowable bearing pressure" which are the values after deduction of the overburden soil pressure;

- It should be noted that the Code defines q_0 as the pressure of the soil originally exists above the base of the foundation, and the final value of q_a is regardless of whether there will be backfill after foundation construction.

2.2.4 – Bearing Capacity Equation Method

- The definition appears reasonable as both q_u and q_a are properties of the soil and foundation dimensions and should be independent of backfill.

- So in case there is backfill, the backfill should be taken as additional load on the soil stratum;

- The reason why the same q_0 is deducted from q_u and q_a in the definition of F is because q_0 can be determined with high certainty;

- Strictly speaking only q_u can be defined physically in terms of shear failure, rupture while q_a can entirely be defined by the equation;

- Now take an example when $q_u = 300\text{kPa}$ and $q_0 = 30\text{kPa}$, with $F = 3$, $q_a = 120\text{kPa}$ which is 40% q_u while by the current practice, it may become $300/3 = 100\text{kPa}$. Both do not include soil surcharge.

2.2.4 – Bearing Capacity Equation Method

- Nevertheless, it should be noted that “plate load test” is required when q_a is determined by this method unless the structure is a minor temporary one according to 4.2.2(2)(b).

A Worked Example on Determination of Ultimate Bearing Capacity of Inclined Footing by Vesic Equation

Ultimate bearing pressure of a footing of plan dimensions 8m × 6m under a load of 800kN of eccentricities 0.3m along its length and 0.4m along its width from the centre is to be found by the Vesic Equation (1975). The footing is founded at an inclination of 10° to the horizontal and is in close proximity of a 20° slope as shown in Figure HB-1. The soil parameters are also shown in Figure HB-1.

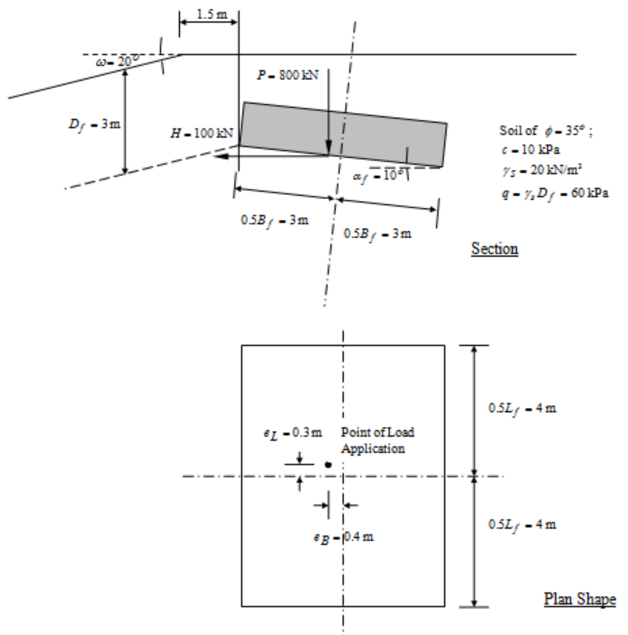


Figure HB-1 – Worked Example of Determination of Ultimate Bearing Capacity of Footing

The ultimate bearing capacity of the footing is determined by the equation

$$q_u = cN_c\zeta_{cs}\zeta_{ci}\zeta_{ct}\zeta_{cg} + 0.5B_{fe}\gamma_s N_\gamma\zeta_{\gamma s}\zeta_{\gamma i}\zeta_{\gamma t}\zeta_{\gamma g} + qN_q\zeta_{qs}\zeta_{qi}\zeta_{qt}\zeta_{qg} \qquad \text{(Eqn HB-1)}$$

where c is the cohesion of the soil, B_{fe} is the effective width of the footing, γ_s is the density of the soil, q is the surcharge on the area adjacent to the footing and the N factors are the bearing capacity factors.

The original equation in its simplest form is in fact

$$q_u = cN_c + 0.5B\gamma_s N_\gamma + qN_q \qquad \text{(Eqn HB-2)}$$

for a footing of infinite length without tiling and the footing is resisting vertical load only. The derivation is based on failure mode shown by Figure HB-2.

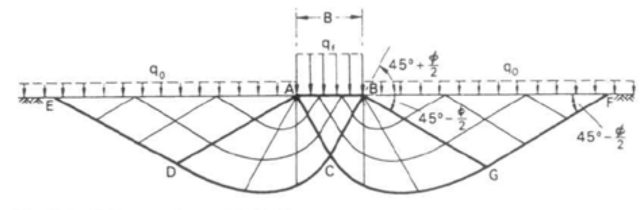


Figure HB-2 – Failure Mode under a Strip Footing

It can be seen that the ultimate bearing capacity by (Eqn B-2) is made up of three components with failures along the assumed failure surface : (i) the first one of which is given by cohesion of the soil; (ii) the second one is due to friction effected by weight of the soil mass; and (iii) the last one is due to friction effected by the adjacent surcharge.

For general use where a footing of finite dimensions which may be tilted and / or adjacent to slope, (Eqn HB-1) is derived based on (Eqn HB-2) with the ζ s as coefficients to account effects for (1) length width ratio of the footing; (2) inclination of resultant loads to the footing; (3) tiling of the footing; and (4) sloping of ground.

For the footing shown in Figure HB-2, the following factors are computed prior to arrival of the final ultimate bearing capacity.

Effective width and Lengths of the Footing

$$B_{fe} = B_f - 2e_B = 5.2\,\text{m}$$

$$L_{fe} = L_f - 2e_L = 7.4\,\text{m}$$

Bearing Capacity Factors

$$N_q = e^{\pi\tan\phi}\tan^2\left(\frac{\phi}{2} + 45^0\right) = 33.296$$

$$N_c = (N_q - 1)\cot\phi = 46.124$$

$$N_\gamma = 2(N_q + 1)\tan\phi = 48.029$$

Shape Factors

$$\zeta_{cs} = 1 + \frac{B_f}{L_f}\frac{N_q}{N_c} = 1.541$$

$$\zeta_{\gamma s} = 1 - 0.4\frac{B_f}{L_f} = 0.7$$

$$\zeta_{qs} = 1 + \frac{B_f}{L_f}\tan\phi = 1.525$$

Inclination Factors

$$m_i = (2 + B_{fe}/L_{fe})/(1 + B_{fe}/L_{fe}) = 1.587$$

$$\zeta_{qi} = \left(1 - \frac{H}{P + B_{fe}L_{fe}c\cot\phi}\right)^{m_i} = 0.885$$

$$\zeta_{ci} = \zeta_{qi} - \frac{1-\zeta_{qi}}{N_c\tan\phi} = 0.881$$

$$\zeta_{\gamma i} = \left(1 - \frac{H}{P + B_{fe}L_{fe}c\cot\phi}\right)^{m_i+1} = 0.819$$

Tilt Factors

$$\zeta_{\gamma t} = (1 - \alpha_f\tan\phi)^2 = 0.771$$

$$\zeta_{qt} = \zeta_{\gamma t} = 0.771$$

$$\zeta_{ct} = \zeta_{qt} - \frac{1-\zeta_{qt}}{N_c\tan\phi} = 0.763$$

Ground Sloping Factors

$$\zeta_{cg} = e^{-2\omega\tan\phi} = 0.613$$

$$\zeta_{qg} = (1-\tan\omega)^2 = 0.405$$

$$\zeta_{\gamma g} = \zeta_{qg} = 0.405$$

So the ultimate bearing capacity when the footing is at the edge of the slope is

$$q_u = cN_c\zeta_{cs}\zeta_{ci}\zeta_{ct}\zeta_{cg} + 0.5B_{fe}\gamma_s N_\gamma\zeta_{\gamma s}\zeta_{\gamma i}\zeta_{\gamma t}\zeta_{\gamma g} + qN_q\zeta_{qs}\zeta_{qi}\zeta_{qt}\zeta_{qg}$$

$$= 293.41 + 446.51 + 840.4 = 1580.4\text{kPa} < 3000\text{kPa}$$

It can be seen that the surcharge adjacent to the footing gives the greatest contribution in this example.

The ultimate bearing capacity of the same footing without tilting, sloping ground and resists vertical load only is re-calculated. The ultimate bearing pressure increases to $q_u = 5052$ kPa, (though the Code limits the value to 3000kPa as per Clause 2.2.4). So it can be seen that these factors have significant effects on reduction of ultimate bearing pressure.

When the footing is at ground level which can be assumed for the footing to be located at $4B_{fe} = 20.8$ m from the edge of the slope, the ground sloping factors are all unity, the ultimate capacity becomes

$$q_u = cN_c\zeta_{cs}\zeta_{ci}\zeta_{ct} + 0.5B_{fe}\gamma_s N_\gamma\zeta_{\gamma s}\zeta_{\gamma i}\zeta_{\gamma t} + qN_q\zeta_{qs}\zeta_{qi}\zeta_{qt}$$

$$= 478.39 + 1103.77 + 2077.64 = 3659.79\text{kPa}$$ and is limited to 3000kPa.

By Figure 2.3(c) of the Code, the ultimate bearing capacity of the footing which is at 1.5m from the edge of the slope can be interpolated between the ultimate values at edge = 1580.4m and that at $4B_{fe} = 20.8$ m which is 3659.79kPa. The linearly interpolated value is 1730.35kPa as demonstrated in Figure HB-3.

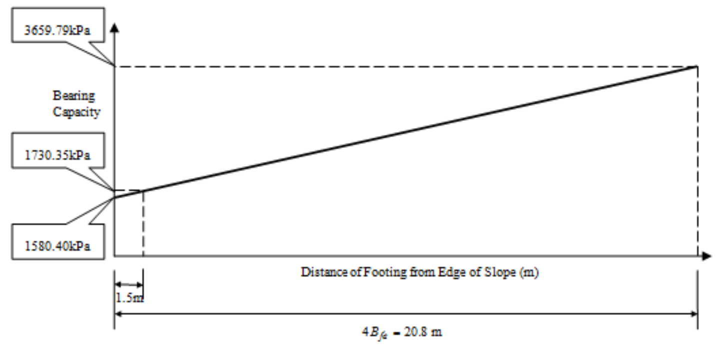


Figure HB-3 – Demonstration of Interpolation of Ultimate Bearing Capacity

So the ultimate bearing capacity of the footing is $1580.4 + (3659.79 - 1580.4)\times\frac{1.5}{20.8} =$ 1730.35kPa. With the overburden pressure $q_0 = \gamma_s D_f = 20\times 3 = 60$ kPa and by the equation listed in 2.2.4 of the Code, the allowable bearing capacity can be calculated as follows :

$$q_a = \frac{q_u - q_0}{F} + q_0 = \frac{1730.35 - 60}{3} + 60 = 616.78\text{ kPa.}$$

2.2.4 – Bearing Capacity Equation Method

- The 2017 Code contains a figure which extends the use of the Bearing Capacity Factor Approach to irregular footing;

- Smaller q_u will generally be resulted due to smaller dimension B value (contribution of $0.5B\gamma N_\gamma$);

- So the approach is conservative.

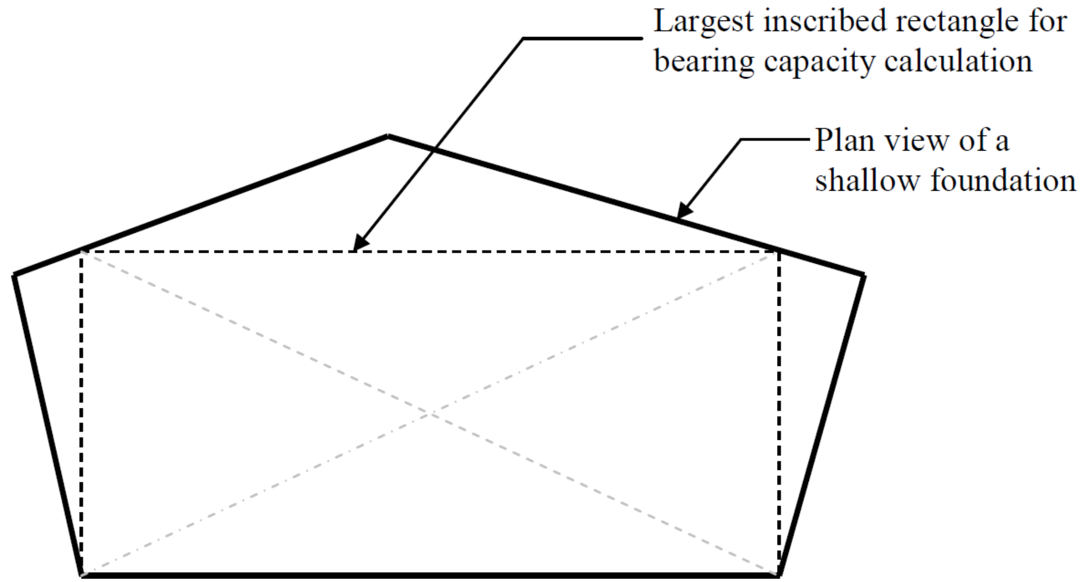


Figure 2.4 Shallow Foundation of Irregular Shape

2.3.1 – Estimation of Settlement

- General formulae in the determination of settlement for granular soils and cohesive soils are added in the 2017 Code. These are text book materials which are also current practice.

- That of granular soil are immediate settlement or elastic settlement by treating soil as an elastic materials; $S_e = \frac{q_{net} B_f F_0}{E_s}$

- Settlements of "Fine-Grained Soils" (or cohesive soils) are that due to consolidation. The Code gives formulae for "primary consolidation" but not for "secondary consolidation" (due to creep, as the phenomenon is not expulsion of water, some one consider this not a consolidation and Craig calls it secondary compression instead).

Extract from Bowles "Foundation Analysis and Design" for Secondary Consolidation

2-10.6 Secondary Consolidation

After primary consolidation the soil structure continues to adjust to the load for some additional time. This settlement is termed *secondary consolidation* or *secondary compression* and may continue for many years, but at an approximately logarithmic rate. At the end of secondary consolidation the soil has reached a new K_o state. The total settlement when accounting for both primary ΔH_p and secondary ΔH_s compression is

$$\Delta H_{\text{total}} = \Delta H_p + \Delta H_s$$

The slope of a plot of *deformation* versus log *time* beyond the D_{100} location is used (see Fig. 2-13*a*) to obtain the secondary compression index C_α, computed as

$$C_\alpha = \frac{\Delta H_{ls}/H_{li}}{\log t_2/t_1} = \frac{\Delta\epsilon}{\log t_2/t_1} \tag{2-48}$$

Now using this C_α index, the field secondary compression (or settlement) ΔH_s after some time $t_2 = t_1 + \Delta t$ is computed as

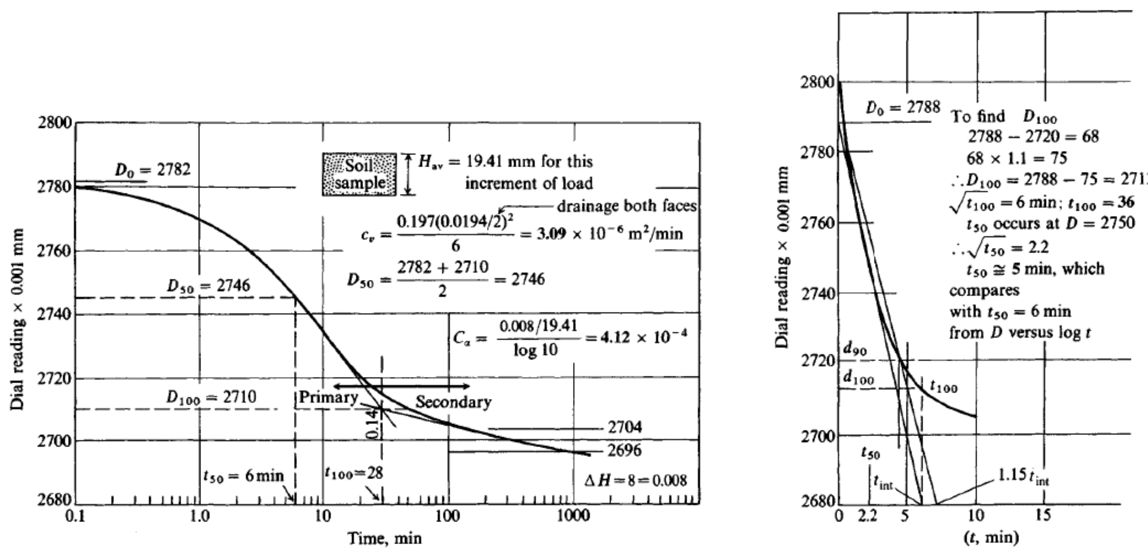
$$\Delta H_s = H_f C_\alpha \log \frac{t_2}{t_1} \tag{2-49}$$

where for the preceding two equations

H_{li} = thickness of laboratory sample at time t_i

ΔH_{ls} = change in sample thickness at some time t_2 taken from the *deformation* versus log *time* curve; try to use one log cycle

t_2 = time at end of primary consolidation $t_1 + \Delta t$ as just defined and consistent with c_v. Find the initial field time t_1 using Eq. (2-38), then rearrange to find t_{90} (use $T = 0.848$ from Table 2-4) and $t_{100} \approx t_{90}/0.9$; for Δt choose some convenient time lapse.



(a) Casagrande's semi log method of presenting time–settlement data. Method is required if it is necessary to obtain a secondary compression coefficient C_α as shown.

(b) Taylor's $\sqrt{\text{time}}$ method to directly obtain d_{90}

Figure 2-13 Two common methods of presenting time-settlement data from a consolidation (or oedometer) test. Note the use of dial readings instead of ΔH since the difference between any two dial readings × dial sensitivity (here 0.001 mm/div) is ΔH. If you directly read ΔH, plot that instead of dial readings.

2.3.1 – Estimation of Settlement

- Correlation of Young's Modulus of soil with SPT N in the absence of laboratory or insitu test is included in the 2017 Code, E_s = SPTN where E_s is in MPa, but limited to shallow foundations with bearing pressures $\leq$ 250kPa;

- The correlation of E with SPTN varies significantly among different researches and testing as revealed by Table 6.10 of GEO 1/2006 publication. Nevertheless, the discussion in 6.13.2.5 of the publication comments that the correlation can be used;

- As 250kPa refers to very light structures, do we need to carry out laboratory or insitu tests for the E values every time when bearing pressure > 250kPa?

Table 6.10 - Correlations between Drained Young's Modulus and SPT N Value for Weathered Granites in Hong Kong

Drained Young's Modulus of Weathered Granites (MPa)	Range of SPT N Values	Basis	Reference
0.2 N - 0.3 N	35 - 250	Plate loading tests at bottom of hand-dug caissons	Sweeney & Ho (1982)
0.6 N - 1 N	50 - 200	Pile and plate loading tests	Chan & Davies (1984)
1.8 N - 3 N	37 - >200	Pile loading tests	Fraser & Lai (1982)
0.6 N - 1.9 N	12 - 65	Pile loading tests	Evans et al (1982)
0.4 N -0.8 N 0.55 N - 0.8 N < 1.05 N	50-100 100 - 150 > 150	Pile loading tests	Holt et al (1982)
1 N - 1.4 N	50 - 100	Pile loading tests	Leung (1988)
2 N - 2.5 N	25 - 160	Pile loading tests	Lam et al (1994)
3 N	20 - 200	Pile loading tests	Pickles et al (2003)
1 N - 1.2 N	N/A	Settlement monitoring of buildings on pile foundations	Ku et al (1985)
1 N	50 - 100	Settlement monitoring of buildings on pile foundations	Leung (1988)
0.7 N - 1 N	50 - 75	Back analysis of settlement of Bank of China Building	Chan & Davies (1984)
3 N	47 - 100	Horizontal plate loading tests in hand-dug caissons (unload-reload cycle)	Whiteside (1986)
0.6 N - 1.9 N (average 1.2 N)	47 - 100	Horizontal plate loading tests in hand-dug caissons (initial loading)	Whiteside (1986)
0.8 N 1.6 N at depth	up to 170	Back analysis of retaining wall deflection	Humpheson et al (1986, 1987)
1 N	8 - 10 (fill and marine deposits)	Back analysis of movement of diaphragm wall of Dragon Centre	Chan (2003)
1.5 N – 2 N	35 - 200 (CDG)		
1.1 N 1.4 N 1.7 N	25 - 50 50 - 75 75 - 150	Multiple well pumping test and back analysis of retaining wall deflection	Davies (1987)

2.3.2 – Acceptable Settlement and Rotation

- The Code has listed limiting settlements and rotations for foundation design for buildings not "sensitive to movement" as
  - (a) maximum total settlement $\leq$ 30mm;
  - (b) differential settlement between vertical elements $\leq$ 1:500
  - (c) angular rotation due to transient loads $\leq$ 1:500

- Remark :

- The limiting values are generally in the same order of other national standards.
- (a) is deemed to be satisfied for buildings founded on rock (presumed value $\geq$3000kPa) and soil with SPTN $\geq$ 200. So practically not applicable to all high-rise buildings. But it may be a problem for light buildings of large plan dimensions founded on soil if the E_s value is low. Say a square plan building 30m x 30m with total bearing pressure 100kPa (2 storeys + raft footing) founded on soil of 50MPa (SPTN = 50), the settlement is $\delta = \frac{qB}{E} I_s = \frac{100 \times 30}{50000} \times 0.85 = 51\text{mm}$ > 30mm

2.3.2 – Acceptable Settlement and Rotation

- Remark :

- The differential settlement in previous code is 1:300, also in some old specifications (HD's old specifications) which is too large and not suitable for our comparatively rigid rc buildings. The value is now increased to 1:500. But even so, it is suspected many stiff structures like the deep lintel beams cannot tolerate such differential settlement : $M = 12EI\Delta/L^2$;

- The limiting angular rotation of foundation due to transient wind load has been taken as 1:750 as under table practice which was difficult to achieve for buildings of narrow base width. Now the criterion is relaxed to 1:500 but still sometimes it is not easy to satisfy, especially for the so called "tooth-pick" buildings.

- In addition, it is not explicitly stated that the rotation of the foundation needs not be included in the analysis of the superstructure. If the limit of 1:500 is reached by the foundation, no margin for the superstructure if to comply with Concrete Code requirement of 1:500 building top deflection.

Annex H
(informative)
Limiting values of structural deformation and foundation movement

(1) The components of foundation movement, which should be considered include settlement, relative (or differential) settlement, rotation, tilt, relative deflection, relative rotation, horizontal displacement and vibration amplitude. Definitions of some terms for foundation movement and deformation are given in figure H.1.

(2) The maximum acceptable relative rotations for open framed structures, infilled frames and load bearing or continuous brick walls are unlikely to be the same but are likely to range from about 1/2000 to about 1/300, to prevent the occurrence of a serviceability limit state in the structure. A maximum relative rotation of $\boxed{1/500}$ is acceptable for many structures. The relative rotation likely to cause an ultimate limit state is about 1/150.

(3) The ratios given in (2) apply to a sagging mode, as illustrated in figure H.1. For a hogging mode (edge settling more than part between), the value should be halved.

(4) For normal structures with isolated foundations, total settlements up to 50 mm are often acceptable. Larger settlements may be acceptable provided the relative rotations remain within acceptable limits and provided the total settlements do not cause problems with the services entering the structure, or cause tilting etc.

(5) These guidelines concerning limiting settlements apply to normal, routine structures. They should not be applied to buildings or structures, which are out of the ordinary or for which the loading intensity is markedly non-uniform.

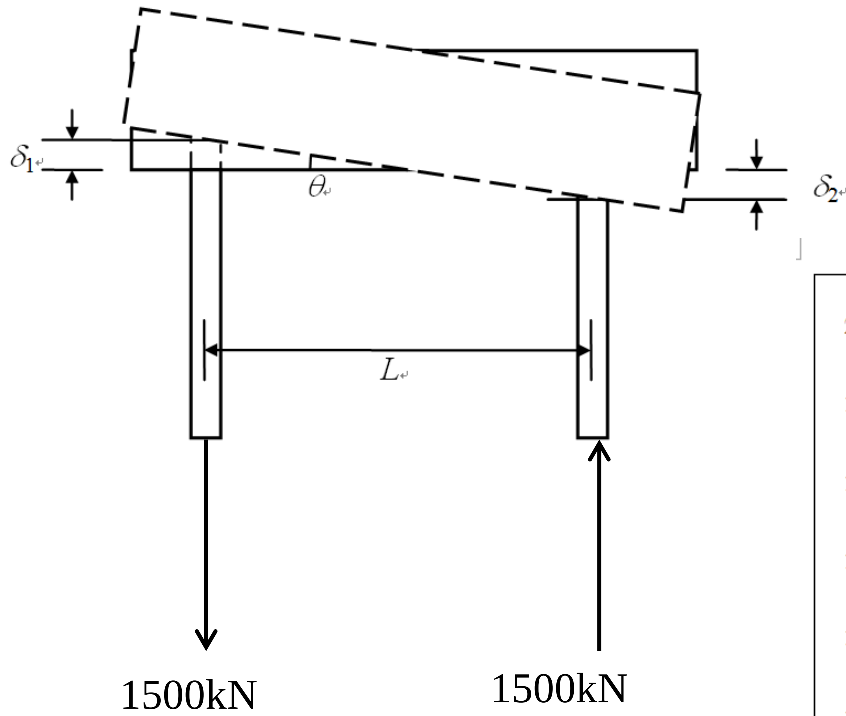
If pile load is ±1500kN due to wind load and pile Length = 40m, elastic shortening / lengthening for the 305×305×223 pile is

$$\frac{FL}{AE} = \frac{1500\times40}{0.0284\times205\times10^{6}} = 10.3\text{mm}$$

If the pile spacing is $L =$ 8m, the cap rotation is

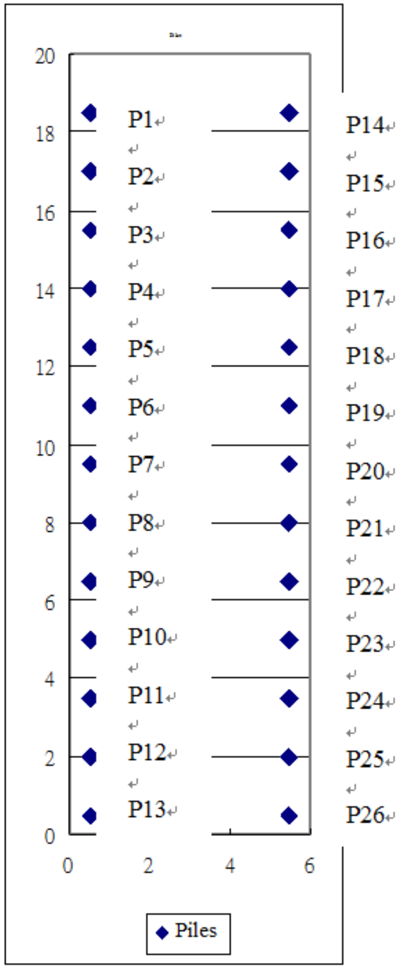
$$\frac{10.3 \times 2}{8000} = \frac{1}{388} > \frac{1}{500}$$

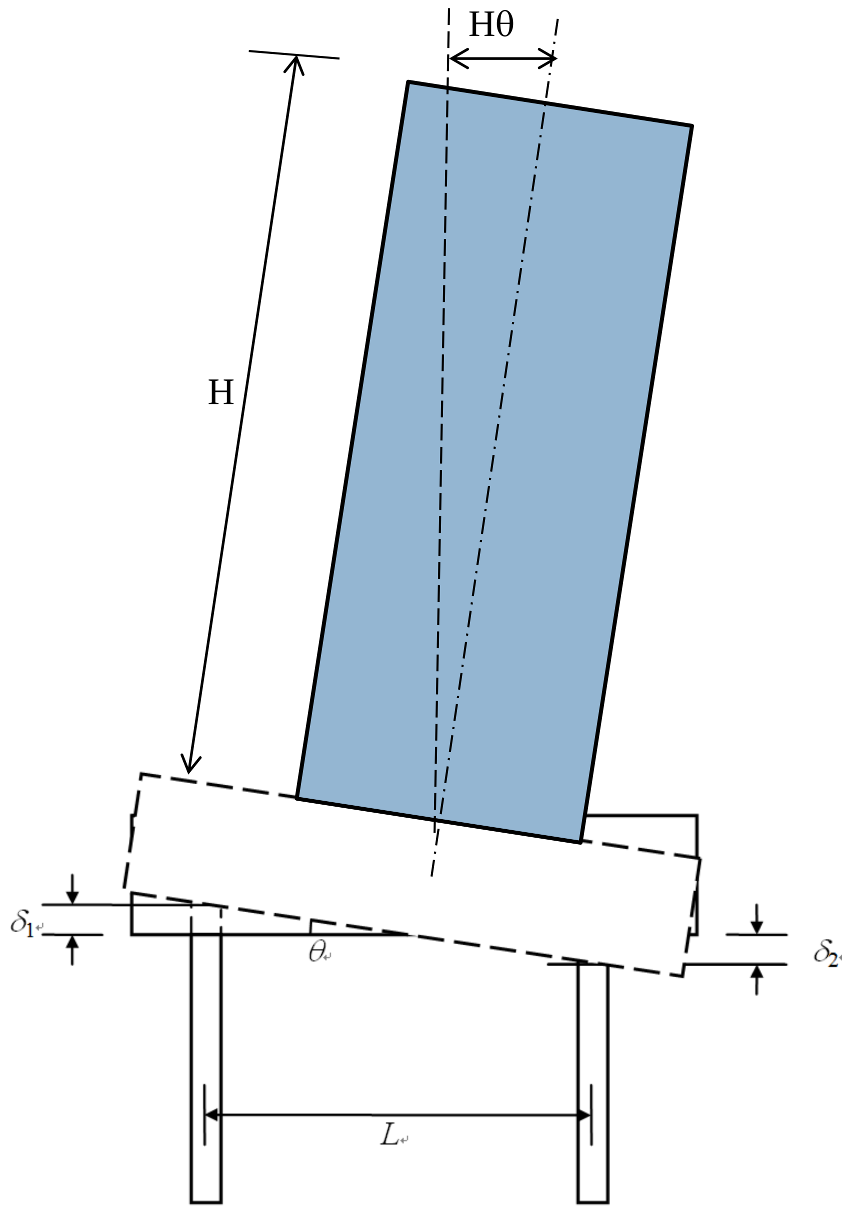
So fails the criterion



Cap Rotation

$= \theta = (\delta_1+\delta_2)/L$





Cap Rotation

$= \theta = (\delta_1+\delta_2)/L$

2.3.2 – Acceptable Settlement and Rotation

- Remark :

- Theoretically, the angular rotation should be included in the calculation of lateral deflection of the superstructure. The angular rotation magnifies the lateral deflection by $H \times \theta$.
If $\theta = 1{:}\,500$, then θ alone will exhaust all the limit in deflection imposed by the Concrete Code 2013.

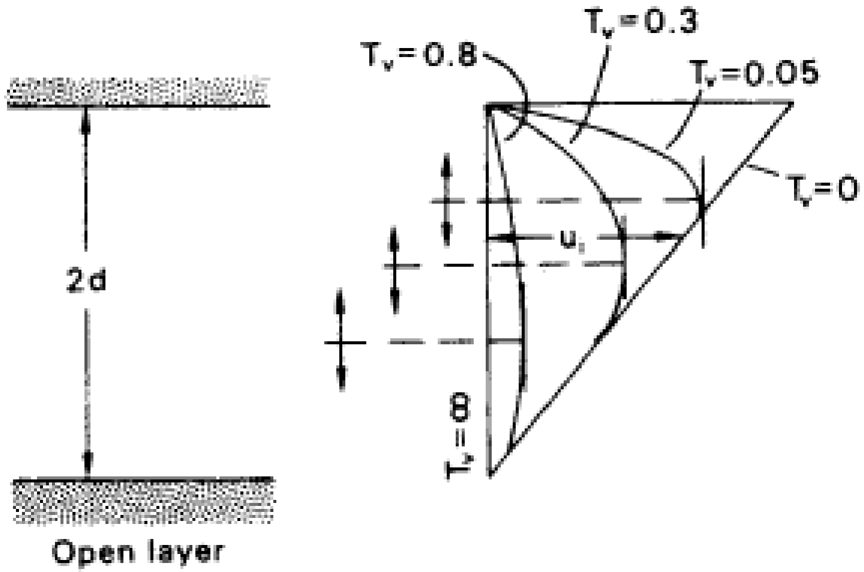
- But as a trade practice, the analysis of the superstructure is often independent of the foundation analysis and BD accepts the deflection purely by superstructure analysis in which the walls and columns at the foundation level are assumed to be total restrained from translation.

2.4.4 – Reclaimed land with consolidation substantially completed

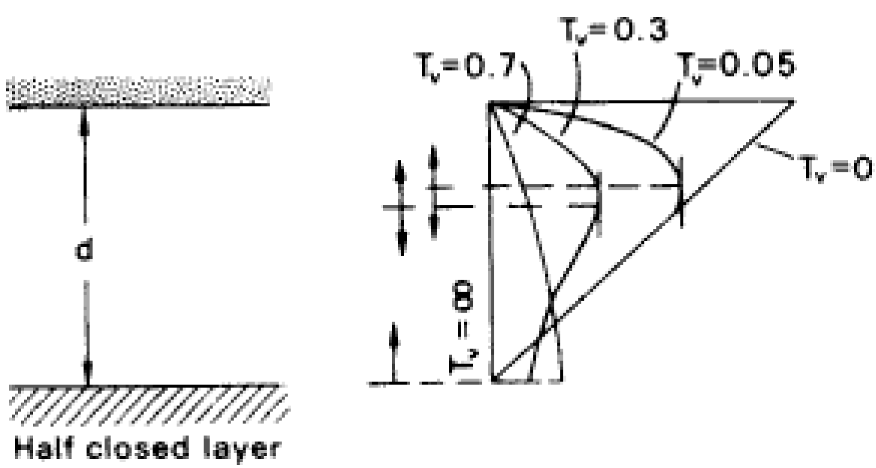
- This is a new sub-clause in the 2017 Code.
- A table indicating 95% degree of consolidation is contained in the Code which can be used in the absence of a detailed consolidation assessment

Thickness of clayey deposits without interbedding sand/silt layers, H	Number of years
H ≤ 5m	10
5m < H ≤ 10m	20
10m < H ≤ 15m	30

- Theoretically, it makes some difference in the rate of consolidation whether the bottom layer is permeable or impermeable in terms of the length of the drainage path. But this seems not considered by the 2017 Code.
- In addition, there is theoretical approach taught in text books to calculate consolidation under known value of coefficient of consolidation c_v.



(b)



(c)

Extract from Crag's Soil Mechanics,

u_i is the water pressure due to the surcharge which is a function of time.

2.5.4 – Resistance to Sliding, Uplift and Overturning

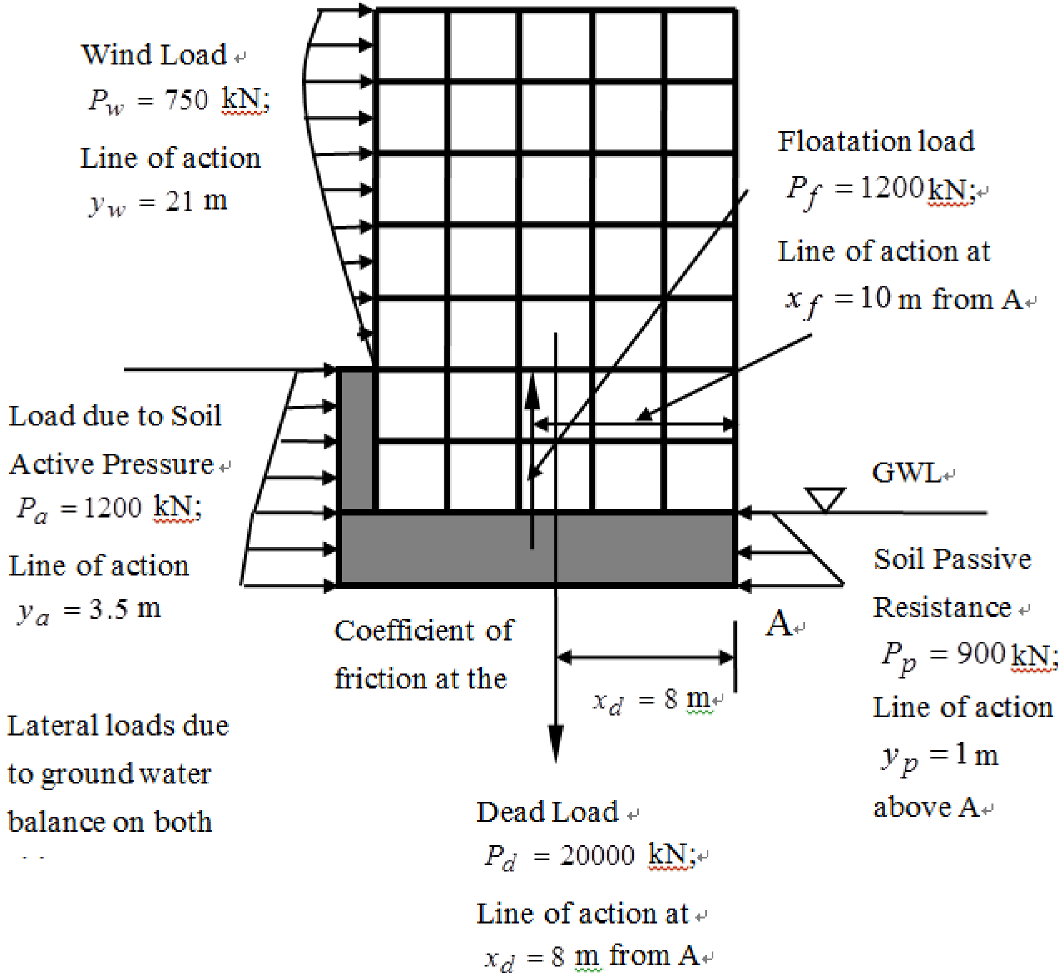
- The provisions are for stability check which are actually found in the old Building (Construction) Regulations. Basically the provisions are also identical in the 2004 and 2017 Codes;

- Since factors of safety against overturning are different for different types of loads, the following can be used to check adequacy of the stability moment (mostly provided by the dead weight of the structure which should be 90% of the total dead load and possibly anchors if any)

$$1.5(or\ 1.1) \times M_{upthrust} + 1.5M_{wind} + 2M_{soil} \leq M_{Dmin}$$

- Checks against sliding and uplift follow the same principle;

- This is global stability check by statics. Strictly speaking, the stiffness of the structure is not a concern, so long the structure remains intact.



Worked Example H2.1

(i) Check Sliding Stability

Sliding Force

$P_w + P_a = 750 + 1200 = 1950$ kN

Passive Soil Resistance = 900kN

Friction at the Base of the Building

$\mu(P_d - P_f) = 0.5 \times (20000 - 1200) = 9400$ kN

Factor of Safety against Sliding is

$(9400 + 900)/(1950 \times 1.5) = 3.52 > 1.0$ O.K.

(ii) Check Uplift Stability

$P_d \div P_f = 20000/(1200 \times 1.5) = 11.11 > 1.0$ O.K.

(iii) Check Overturning Stability (about A)

Overturning moment by wind about A is

$P_w \times y_w = 750 \times 21 = 15750$ kNm

Overturning moment by active soil pressure about A is $P_a \times y_a = 1200 \times 3.5 = 4200$ kNm

Overturning moment by floatation about A is

$P_f \times x_f = 1200 \times 10 = 12000$ kNm

Stabilizing moment by passive soil pressure about A is $P_p \times y_p = 900 \times 1 = 900$ kNm

Stabilizing moment by dead load about A is

$P_d \times x_d = 20000 \times 8 = 160000$ kNm

2.5.5– Materials and Stresses

The followings in the 2017 Code are highlighted :

- Comparing with the 2004 Code, in the 2017 Code, the restriction of concrete grades to 20D for design in cast-in-place concrete foundations of least lateral dimensions ≤ 750mm is removed;

- In the 2017 Code requirements for concrete to be applied to grout are added in 2.5.5(3);

- In the 2004 Code, the design of steel pile is based on the working stress method, i.e. the stresses in the pile due to working loads are worked out (by the elastic theory) and then compared with the allowable stresses of the steel. In such an approach, the allowable stresses due to purely axial loads are smaller than that with axial load coupled with bending because lesser area of the section is under maximum stress in the latter. So in the 2004 Code, the allowable working stress for driven piles due to axial load is $0.3f_y$ and can be increased to $0.5f_y$ if bending is included.

2.5.5– Materials and Stresses

- However, in the 2017 Code, the increase in allowable working stress for driven piles from $0.3f_y$ to $0.5f_y$ under bending is removed;

- In addition, the 2017 Code limits "axial stress" to 30% of f_y which is different from 2004 Code limiting "stress" to 30% of f_y instead. The 2017 Code intentionally refers "axial stress" to that due to axial load only (free from moment), leaving extra capacity to resist moment by the limit state method. (As otherwise axial load + bending moment can give combined stress can easily exceed $0.3f_y$ which will lead to more piles or larger sections). So by the 2017 Code should use limit state design to check axial load + bending;

- Instead, use of the limit state design in accordance with Steel Code 2011 is emphasized in the 2017 Code.

2.5.5– Materials and Stresses

Comparison in steel design by the working stress method and limit state design in case of a 305×305×223 S450 driven H-pile under full axial load of 3663kN:

Working Stress Method to Foundation Code 2004 :

The spared strength to resist bending about the major axis is $(0.5 - 0.3)f_y$. So the maximum working moment about the major axis that can be resisted

$M_w = (0.5 - 0.3)f_y Z_x = 0.2 \times 430 \times 3119 \times 10^3 \times 10^{-6} = 268.23\text{kNm}$

Limit State Method to Foundation Code 2017 (using Steel Code 2011):

$P_c = f_y A = 430 \times 28400 \times 10^{-3} = 12212\text{kN}$

$M_{cx} = \min(1.2 f_y Z_x, f_y S_x) = 430 \times 3653 \times 10^3 \times 10^{-6} = 1571\text{kNm}$

Assume overall $\gamma_f = 1.5$, check

$$\frac{P}{P_c} + \frac{M_x}{M_{cx}} \leq 1 \Rightarrow \frac{1.5 \times 3663}{12212} + \frac{1.5 M_x}{1571} \leq 1$$

$\Rightarrow M_x = 576\text{kNm} >> 268\text{kNm}$

2.5.5– Materials and Stresses

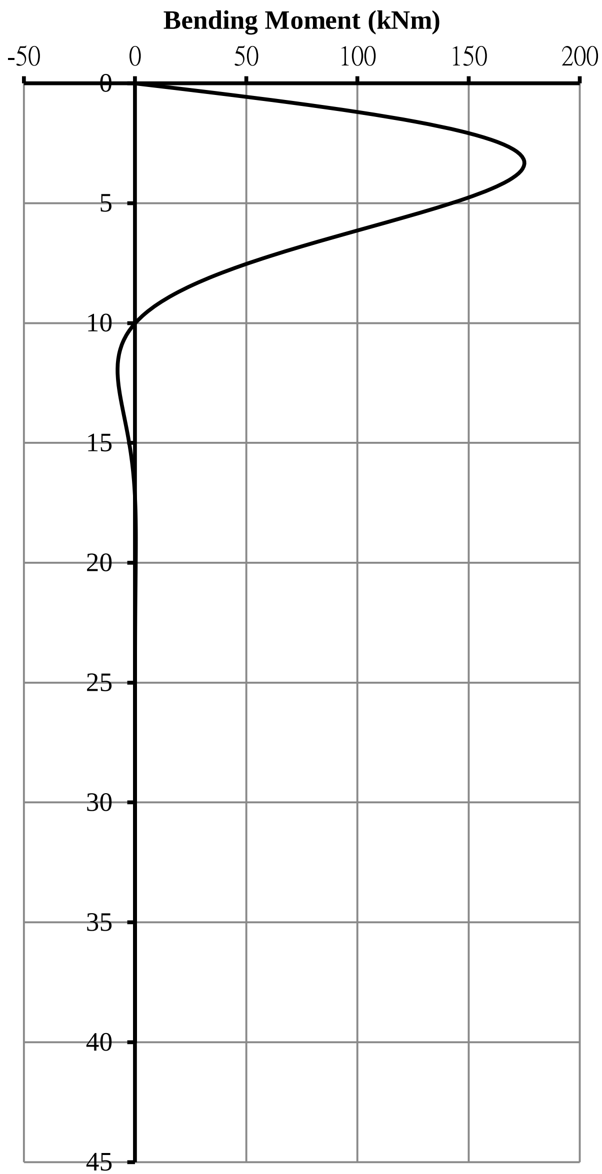
- Increase of design moment is similar for socketed pile. Socketed pile initially has no spared capacity for moment according to the 2004 Code if the axial load is up to $0.5f_yA$. However, according to the 2017 Code and using limit state design, the extra M_x is

$$\frac{P}{P_c} + \frac{M_x}{M_{cx}} \leq 1 \Rightarrow \frac{1.5 \times 6106}{12212} + \frac{1.5M_x}{1571} \leq 1$$

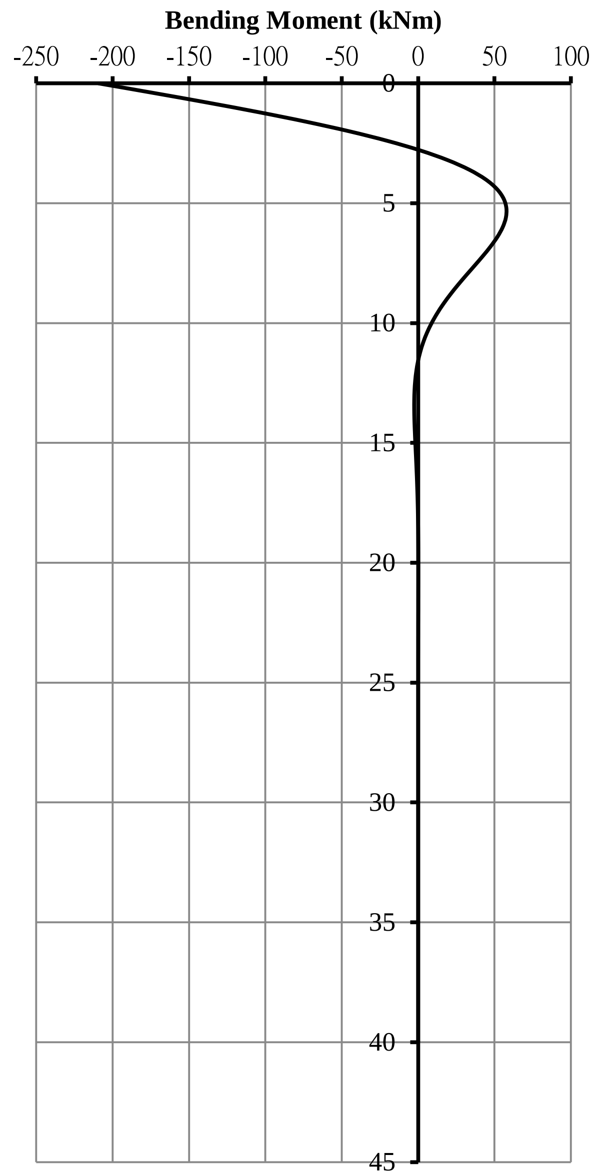
$$\Rightarrow M_x = 262\text{kNm}$$

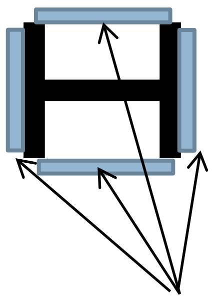
- So the 2017 Code allows more economical design in this case. Previously designers using the 2004 Code tend to use "stiffeners" at the top portion of the pile to resist moments instead of adding piles or using larger sections for the entire pile lengths if the section is not sufficient. Usually stiffeners are only required at top portions with short lengths as the moments diminish quickly down the piles.

Bending moment profiles of piles



Bending moment profiles of piles





Stiffeners to be welded at the upper portion of the piles where necessary to resist moments

2.5.5– Materials and Stresses

- Allowable bond stresses between steel and grout in the 2017 Code are decreased by one third as compared with the 2004 Code :

600kPa $\rightarrow$ 400kPa (when grouting in dry) ;

480kPa $\rightarrow$ 320kPa (when grouting in water)

- It is also clarified in the 2017 Code that the surface area for calculation of allowable steel/grout bond stress should be the "total external surface area" of the steel section, i.e. not the circumcised rectangular section. So for the 305 $\times$ 305 $\times$ 223 pile, the total surface area for one metre length is 1918 $\times$ 1000 $\times$ 10^{-6} = 1.918m^2).

2.5.5– Materials and Stresses

- In the 2017 Code, shear studs (or steel sections or other substitutes) be used to enhance bonding, but limited to overall allowable stress to 600kPa (when grouting in dry) and 480kPa (when grouting in water). So the maximum (residual) stress be provided by shears studs or other substitutes are 600 – 400 = 200kPa and 480 – 320 = 160kPa for dry and in water respectively.

- Often contractors prefer transverse bars along the length of the pile to shear studs likely because of the availability.

Use of Transverse Reinforcing Bars or Shear Studs on Flanges of Socketed H-piles for Enhancing Bonding between Grout and the Pile 305x305x223 socketed H-pile

Foundation Code 2017 Cl. 2.5.5(4) limits the bond strength of the pile to 480kN/m^2 (grouting under water) even with shear studs or other substitutes. So the minimum length of the pile with perimeter 1.918m under full load 6106kN (irrespective of whether there are shear studs or other substitutes) is

$$\frac{6106}{1.918\times 480}=6.63\,\text{m}$$

If without shear studs or other substitutes, the Code allows bonding strength of 320kN/m^2, the minimum length is

$$\frac{6106}{1.918\times 320}=9.95\,\text{m}$$

The length of socket (of diameter 550mm) within rock to achieve full load of 6106kN with bond strength to rock 700 kN/m^2 (Table 2.2 of the Foundation Code 2017) is

$$\frac{6106}{0.55\pi\times 700}=5.048\,\text{m}$$

So the minimum length above rock socket is $9.95-5.048=4.902\,\text{m}$ without the use of shear studs and other substitutes.

First Attempt by using Transverse Bars

The bonding results from bearing of the reinforcing bars on grout.

Ultimate bearing pressure on grout (Grade 30) can be taken conservatively as

$f_{bult}=\frac{f_{cu}}{\gamma_m}=\frac{30}{1.5}=20$ N/mm^2 as per Cl. 2.4.3.2 and Cl. 8.3 of the Concrete Code 2013.

So for a bar of diameter ϕ (in mm) and length 300mm, the ultimate bearing offered by a bar is

$f_{bult}\phi\times 300=6000\phi\text{kN}$ 6ϕkN or allowable load of 3ϕkN (conservatively taken

$\gamma_f=2.0$ instead of 1.4). Summarizing, we have

Bar Diameter (mm)	Allowable Bearing (kN)
$\phi=20$	60
$\phi=25$	75
$\phi=32$	96

If a pile has a minimum length of 1.6m above rock socket so that the total length is 6.648m (>6.63m). The bonding between the pile and grout alone is

$1.918\times 6.648\times 320=4080\,\text{kN}$

The excess load to be taken by transverse bars is

$6106-4080=2026\,\text{kN}$

Using $\phi 20$ bar, $\frac{2026}{60}=34$ nos, i.e. 17 rows (each row consists of 2 bars on the 2 flanges) of bars evenly distributed at intervals of $\frac{6.6}{17}=388$, say 350mm.

Theoretically, smaller enhancement need be used for long piles. But as a conservative design, use Y20 – 300mm long transverse bars at spacing 350mm on flanges of all piles.

Re-attempt by using Shear Studs

Shear Studs (Class 1) : nominal shank diameter $d=22$ mm;
nominal height $h=100\,\text{mm}$

Ultimate Tensile Strength of Stud : $f_u=450\,\text{MPa}$

Grout Cube Strength : $f_{cu}=30\,\text{MPa}$

Young's Modulus of Grout : Taken conservatively as 30% of concrete of the same grade in accordance with E4.2.4 of the Explanatory Report to the Code of Practice for the Structural Use of Steel issued by the Buildings Department which becomes

$E_{cm}=0.3\times 22200=6660\,\text{MPa}$

Allowable bond stress between steel and grout (concreting under water) = 480kPa

Characteristic strength of a shear stud is (in accordance with Eqn (10.20) of the Code of Practice for the Structural Use of Steel 2011)

$$P_k = 0.29 d^2 \alpha \sqrt{0.8 f_{cu} E_{cm}} = 0.29 \times 22^2 \times 1 \sqrt{0.8 \times (0.8 \times 30) \times 6660} \times 10^{-3} = 50.19 \text{ kN}$$

$$\leq 0.8 f_u \left(\frac{\pi d^2}{4} \right) = 0.8 \times 450 \times \left(\frac{\pi \times 22^2}{4} \right) \times 10^{-3} = 136.85 \text{ kN (shear resistance of the stud)}$$

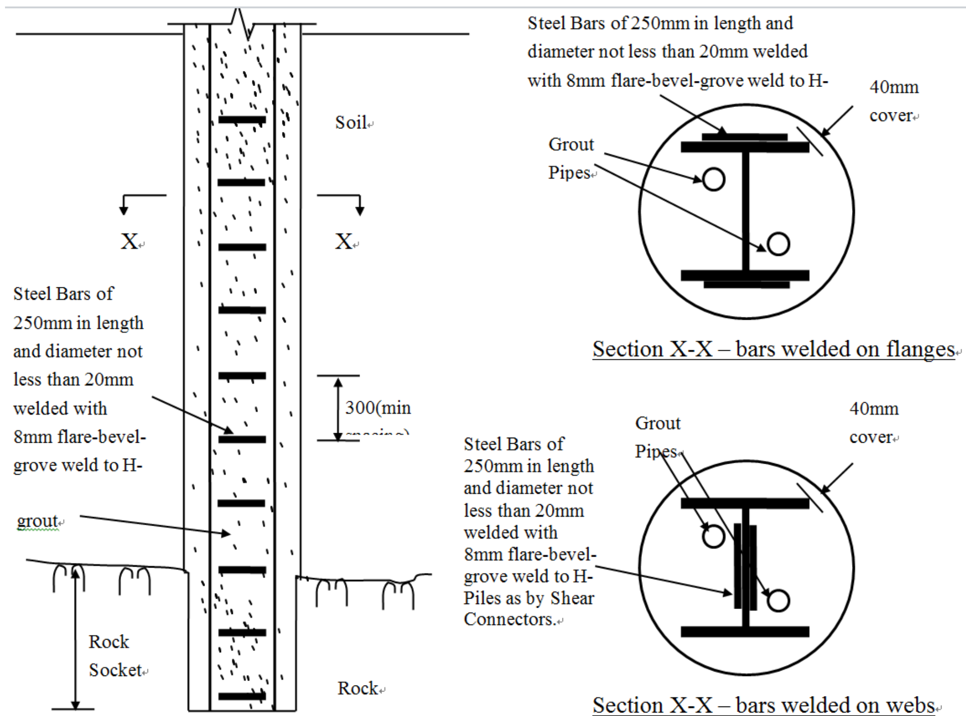
as $\frac{h}{d} = \frac{100}{22} = 4.55 > 4 \Rightarrow \alpha = 1$ (Note: f_{cu} of grout is discounted by 20% to allow for grouting in water).

As the shear stud is Class 1 material, the partial strength factor $\gamma_{m1} = 1.0$ is used in accordance with Table 4.1 of the Code of Practice for the Structural Use of Steel 2011. So the design strength of the shear stud is $50.19 / \gamma_{m1} = 50.19$ kN.

Treating the shear stud as if it is in slab under negative moment as per Cl. 10.3.2.1 of the Code of Practice for the Structural Use of Steel 2011, its design resistance is $P_n = 0.6 P_k = 0.6 \times 50.19 = 30.11$kN

Returning to the design by using steel bars and for a total minimum pile length of 6.63m, the number of shear studs required to take up the excess bond is $\frac{2026}{30.11} = 67$ nos, say 68nos.

So arrange as 2 nos. per row per flange with total 17 rows or 350mm row spacing.



2.8 – Foundations Design in Scheduled Areas

- The 2017 Code has significant enrichment in giving some details for foundation design in Scheduled Area Nos. 2 and 4 – marble area.

- Some aspects are highlighted “

1) Determined Bulk Excavation Limit (DEBL) – limiting extent of bulk excavation ;
2) A pile redundancy is provided for the uncertainties which the driven piles can be affected by karst features beneath the pile toe or damaged sustained during driving;
3) Limit on increase of vertical stress at marble surface.

3.3.(2) – Geological Study (under Site Survey)

- The 2017 Code has a requirement of developing a geological model and ground model. “Key elements” of the geological and ground models in respect of foundations are given mainly on the recognition of potential geological and/or geotechnical complexity together with uncertainties and factors warrant attentions.

3.4 – Ground Investigation

- A paragraph is added at 3.4.1 requiring an existing GI not complying with CoPSS should be ascertained before use.

- 3.4.4 is a new clause requiring good qualities samples for testing and the testing requirements are listed.

- 3.4.6 – The last paragraph is a new one, stating that more extensive ground investigation works are normally required for Scheduled Areas No. 2 and 4.

3.6 – Ground Investigation within the Scheduled Area

- 3.6 is a new clause in the 2017 Code giving some general requirements on ground investigation within the Scheduled Area.

4.2 – Allowable Bearing Pressures and Settlement

- A flow chart has been added in the 2017 Code in the use of “plate load test” for design of shallow foundations.

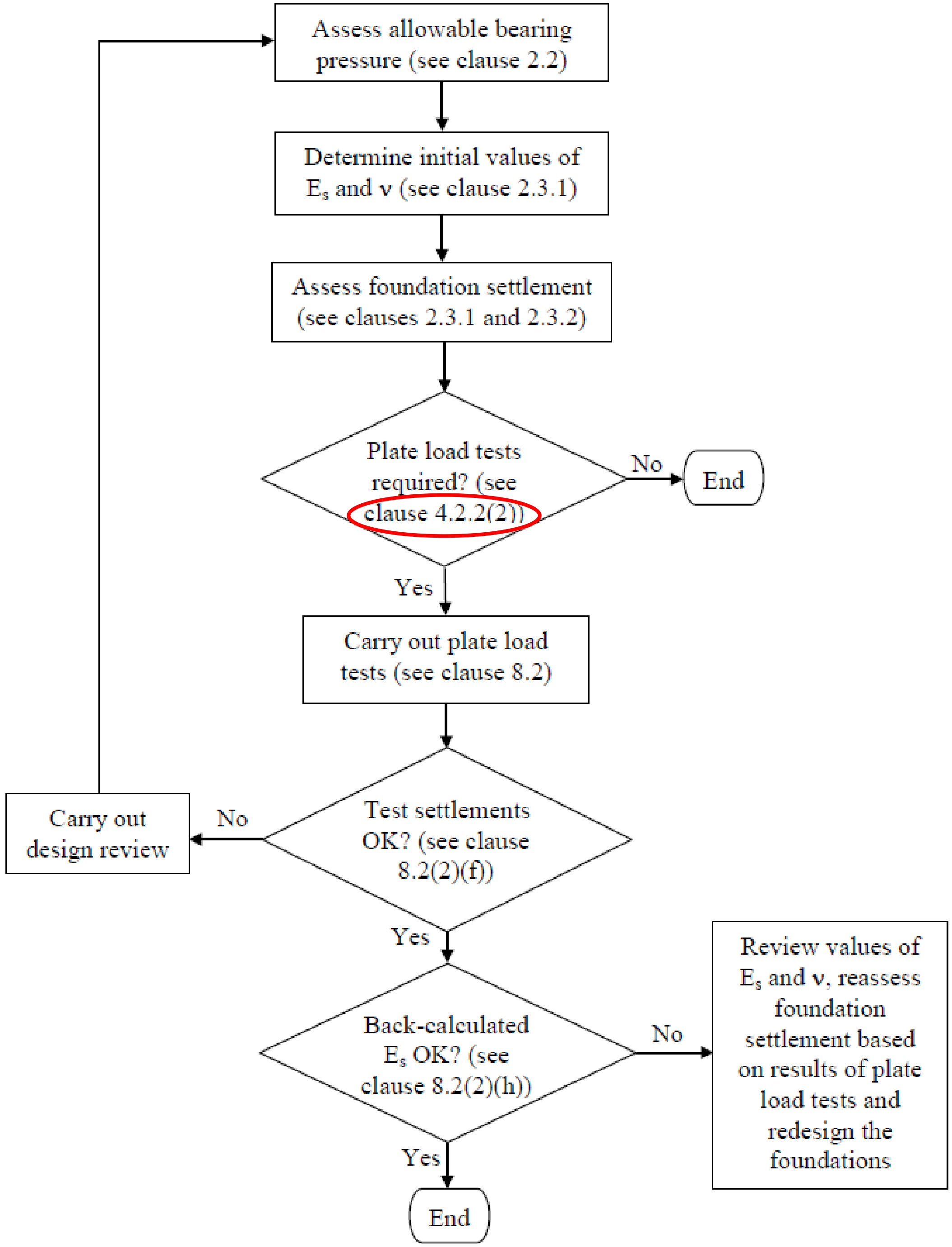
- By 4.2.2(2), the followings require plate load test

a) q_a based on Table 2.1 > 300kPa (only Cat. 4(a) and 4(b) when dry, SPTN>30) unless $q_a - q_0 < 50$kPa (if $q_a = 299$, $q_0 > 250$ implying 250/20 = 12.5m soil surcharge if $\gamma = 20$ – difficult to achieve)

b) q_a determined by the bearing capacity equations except for footings of minor temporary structure;

c) Determination of E_s greater than 1 times the SPTN value.

- So plate load test is difficult to escape.


Assess allowable bearing pressure (see clause 2.2)
Determine initial values of E_s and ν (see clause 2.3.1)
Assess foundation settlement (see clauses 2.3.1 and 2.3.2)
Plate load tests required? (see clause 4.2.2(2))
No
End
Yes
Carry out plate load tests (see clause 8.2)
Test settlements OK? (see clause 8.2(2)(f))
No
Carry out design review
Yes
Back-calculated E_s OK? (see clause 8.2(2)(h))
No
Review values of E_s and ν, reassess foundation settlement based on results of plate load tests and redesign the foundations
Yes
End

5.1.6 – Piles providing resistance against uplift, overturning and buoyancy

- Two conditions of "deemed to satisfy" requirements for stability check of individual piles against uplift and overturning for pile foundations are stated in this clause.

a) $D_{min} + 0.9R_u - 2.0I_a - 1.5U_a(\text{or} 1.1U_p) - 1.5W_k \geq 0$ (check by ultimate load)

b) $D_{min} + R_a - I_a - U_a - W_k \geq 0$ (checked by allowable load)

- Both conditions are listed in the 2004 Code, but in different clauses. The 2017 Code puts them together under the same clause 5.1.6.

- Comparing with 2.5.4 on global stability, the check is more stringent.

- Currently both a) and b) are checked. Generally, a) is more critical.

Checking of Piles against Uplift, Overturning and Buoyancy of a Hypothetical Pile Group

Worked Example HG-1

The pile group comprises 26 nos. of 305x305x223 Grade S460 H-piles. No piles have adverse live load (i.e. uplift due to live load). The R_u values are derived from cohesion (c = 20 kPa as the ultimate value being twice the allowable value of 10kPa) on the pile shaft.

Pile No.	D_{min} (kN)	TL (DL+LL) (kN)	Wind Axial Load			Critical Load Combination		Uplift Checking						
			Wind X (kN)	Wind Y (kN)	W_{max} (kN)	TL (kN)	TL $+W_{max}$ (kN)	Upthrust U_t (kN)	D_{min} $-W_{max}$ $+U_t$ (kN)	R_s (kN)	R_u (kN)	$D_{min}-W_{max}$ $+U+R_s$ (kN)	$D_{min}-1.5W_{max}$ $+U_t$ (kN)	$D_{min}+0.9R_s-$ $1.5(U_t+W_{max})$ (kN)
P1	1566	2211	-483	-987	987	2211	3198	-70	509	633	1227	1142	16	1085
P2	1450	2030	-83	-1033	1033	2030	3063	-69	348	633	1227	981	-169	901
P3	1438	2007	338	-1148	1148	2007	3155	-66	224	633	1227	857	-350	721
P4	1581	2217	982	-1407	1407	2217	3624	-58	116	633	1227	749	-588	488
P5	1671	2366	-808	-870	870	2366	3236	-70	731	633	1227	1364	296	1365
P6	1732	2458	-1154	-743	1154	2458	3612	-68	510	633	1227	1143	-67	1003
P7	1462	2036	986	-1131	1131	2036	3167	-58	273	633	1227	906	-293	783
P8	1641	2308	-1238	-513	1238	2308	3546	-68	335	633	1227	968	-284	786
P9	1389	1923	1029	-863	1029	1923	2952	-57	303	633	1227	936	-212	864
P10	1584	2207	-1365	-211	1365	2207	3572	-65	154	633	1227	787	-529	543
P11	1370	1890	1144	-455	1144	1890	3034	-53	173	633	1227	806	-399	679
P12	1429	1968	342	-62	342	1968	2310	-58	1029	633	1227	1662	858	1933
P13	1513	2096	-1469	143	1469	2096	3565	-60	-16	633	1227	617	-751	324
P14	1501	2068	-65	213	213	2068	2281	-58	1230	633	1227	1863	1124	2199
P15	1412	1951	1312	113	1312	1951	3263	-46	54	633	1227	687	-602	479
P16	1454	2008	878	399	878	2008	2886	-47	529	633	1227	1162	90	1171
P17	1473	2036	-1547	465	1547	2036	3583	-55	-129	633	1227	504	-903	174
P18	1518	2096	422	656	656	2096	2752	-49	813	633	1227	1446	485	1565
P19	1483	2054	1465	654	1465	2054	3519	-40	-22	633	1227	611	-755	330
P20	1530	2111	-1240	710	1240	2111	3351	-53	237	633	1227	870	-383	695
P21	1592	2199	-40	884	884	2199	3083	-50	658	633	1227	1291	216	1295
P22	1536	2127	985	934	985	2127	3112	-42	509	633	1227	1142	17	1100
P23	1451	2002	-1600	821	1600	2002	3602	-50	-199	633	1227	434	-999	80
P24	1596	2204	-893	995	995	2204	3199	-50	551	633	1227	1184	54	1133
P25	1603	2219	488	1177	1177	2219	3396	-43	383	633	1227	1016	-206	877
P26	1579	2194	1616	1258	1616	2194	3810	-35	-72	633	1227	561	-880	207

Table HG-1 – Example of Checking of Piles against Uplift, Overturning and Buoyancy of a Hypothetical Pile Group

5.1.6 – Piles providing resistance against uplift, overturning and buoyancy

- The last paragraph in 5.1.6 of the 2017 Code states that “global stability” check in accordance with 2.5.4 requires consideration of stiffnesses of all structural members and the interaction of the structural members with the subgrade and bearing strata. However, the stability check is in fact a “static problem”. The stabilizing forces depends generally on the weight of the structure and the gravity centre. They can be fully effected so long the structure does not disintegrate. Similarly the de-stabilizing forces such as the wind loads, soil loads and water uplift are also independent of the stiffnesses of the structure. So the stiffnesses of the structure and their interaction of the ground should not be a concern.

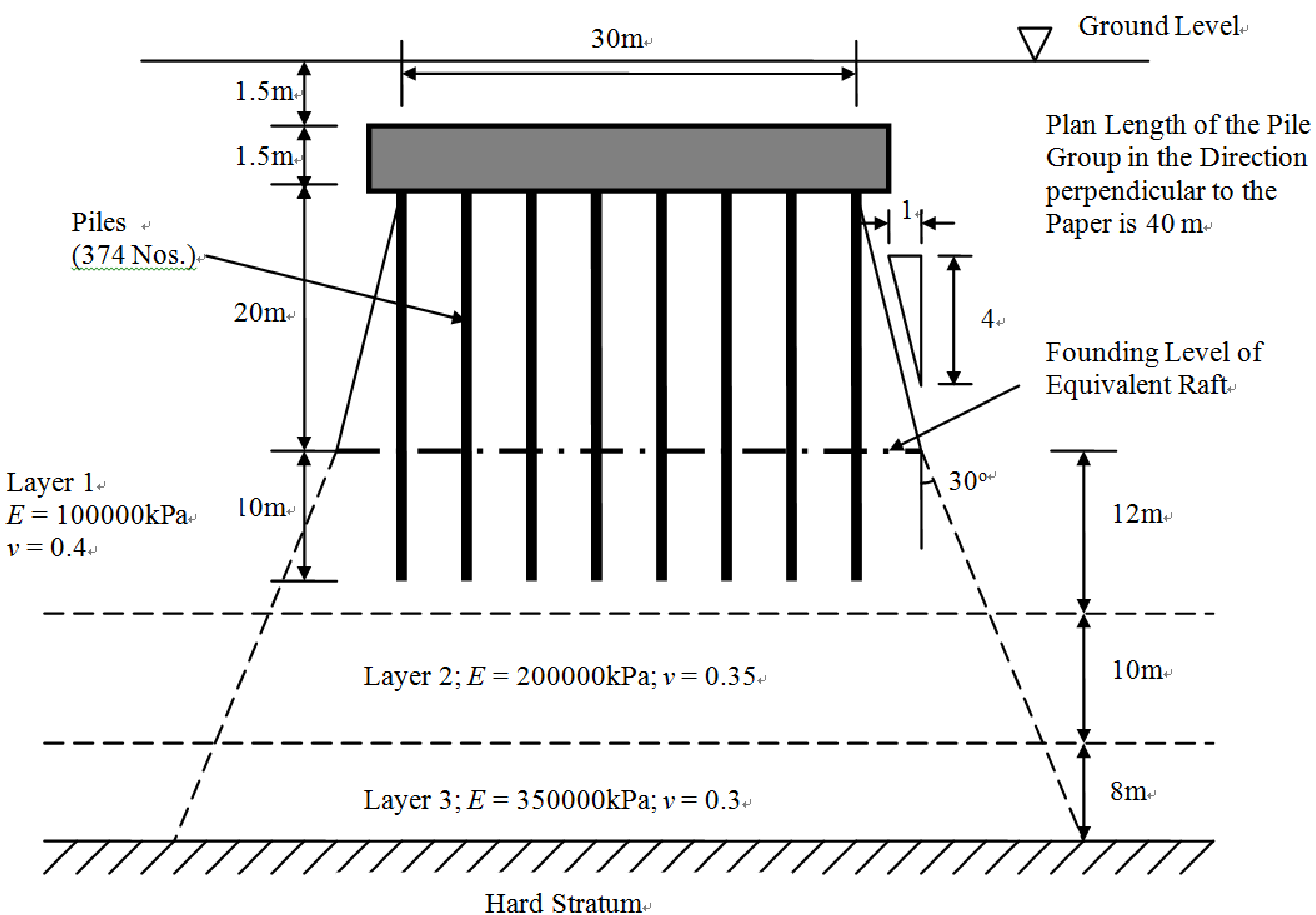
5.1.7 – Pile Group Settlement

- Comparing with the 2004 Code, 5.1.7 is a new clause in the 2017 Code.

- It is specified in the clause that the “equivalent raft” method may be used to estimate pile group settlement. (The method is NOT the only one accepted by the Code);

- The method is fully described in Tomlinson’s “Pile Design and Construction” and has been in use in the current trade practice;

- Strictly speaking, the method can only be used for soil strata with no horizontal variation in soil geology.



Ground Level
30m
1.5m
1.5m
Plan Length of the Pile Group in the Direction perpendicular to the Paper is 40 m
Piles (374 Nos.)
1
4
20m
Founding Level of Equivalent Raft
Layer 1
E = 100000kPa
v = 0.4
10m
30°
12m
Layer 2; E = 200000kPa; v = 0.35
10m
Layer 3; E = 350000kPa; v = 0.3
8m
Hard Stratum

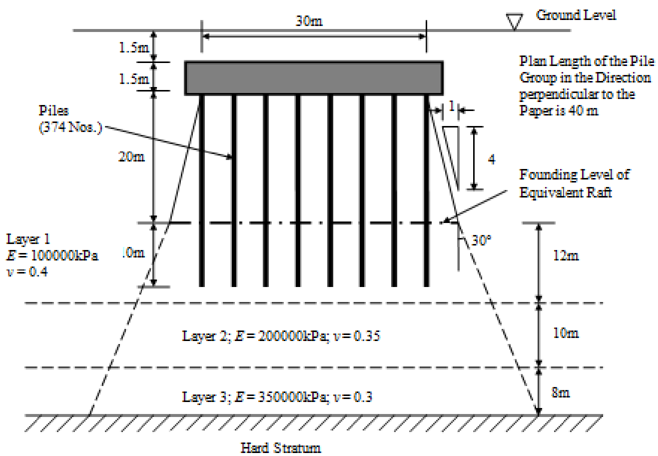


Figure HH-2 – Worked Example HH-1

The plan dimensions of the "Equivalent Raft" are worked out as
$B_1 = 30 + 20/4 \times 2 = 40$ m and $L_1 = 40 + 20/4 \times 2 = 50$ m;

The plan dimensions at the base level of Layer 1 are
$B_2 = 40 + 12\tan 30^0 \times 2 = 53.856$ m and $L_2 = 50 + 12\tan 30^0 \times 2 = 63.856$ m;

Similarly, the plan dimensions at base levels of Layer 2 and Layer 3 are
$B_3 = 40 + 22\tan 30^0 \times 2 = 65.403$ m and $L_3 = 50 + 22\tan 30^0 \times 2 = 75.403$ m;
$B_4 = 40 + 30\tan 30^0 \times 2 = 74.641$ m and $L_3 = 50 + 30\tan 30^0 \times 2 = 84.641$ m.

The average plan width and length of Layer 1 are $(40 + 53.856)/2 = 46.928$ m and $(50 + 63.856)/2 = 56.928$ m giving a plan area of $46.928 \times 56.928 = 2671.52\,\text{m}^2$.

Under the applied load of 100000 kN, the settlement of Layer 1 is

$$\frac{PL}{AE} = \frac{100000 \times 12}{2671.52 \times 100000} = 0.00449 .$$

Settlement of other layers are similarly worked out worked out and summarized in Table HH-1.

Table HH-1 – Summary for Calculation for Worked Example HH-1

Layer	Average Width (m)	Average Length (m)	Average Plan Area (m^2)	Depth (m)	Young's Modulus of Soil (kN/m^2)	Settlement (mm)
1	46.928	56.928	2671.52	12	100000	4.49
2	59.630	69.630	4152.04	10	200000	1.20
3	70.022	80.022	5603.30	8	350000	0.41
					Sum	6.10

So the total settlement is 6.10mm.

Alternatively, a more accurate estimation is that by the approach in Appendix HC as presented in Table 5.2. In Table HH-2, D' which is the depth of the "sunken footing" being simulated by the pile group is 23m.

Table HH-2 – Calculation for Worked Example HH-2 by the Approach in Appendix HC (the I_S and I_F coefficients are read from Tables HC-1 and HC-5 of Appendix HC). In the last column, the settlement is calculated by $(q_0 B'/E_s) I_S I_F$. The items with * are subtractive items.

Step	Description	E (MPa)	v	M	H' (m)	N	D'/B'	I_S	I_F	Settlement (mm)
1	Due to 1st layer,	100	0.4	1.333	12	0.4	0.767	0.19	0.76	3.61
2	Due to 1st + 2nd layers	200	0.35	1.333	22	0.733	0.767	0.36	0.74	3.33
3	Due to 1st layer *	200	0.35	1.333	12	0.4	0.767	0.21	0.74	−1.94
4	Due to 1st + 2nd + 3rd layers	350	0.3	1.333	30	1.0	0.767	0.48	0.72	2.47
5	Due to 1st + 2nd layers *	350	0.3	1.333	22	0.733	0.767	0.39	0.72	−2.01
									Sum	5.46

The settlement determined is 5.46 mm which is less than that by Tomlinson of 6.10 mm. One factor to account for the difference is the lack of consideration of the Poisson's effect by Tomlinson's approach.

Nevertheless, the above only caters for the settlement of the soil, additional settlement should also be allowed for the elastic shortening of the piles. Conservative estimation can be made through the *AE/L* approach which assumes the greatest shortening by ignoring skin friction along the pile shaft. As the cross sectional area of each of the pile is 0.0284m^2, conservatively ignoring the skin friction on the piles and assuming each pile have each share of the load from the pile cap, the elastic shortening of the piles are

$$\frac{PL}{AE} = \frac{100000 \times 30}{374 \times 0.0284 \times 205 \times 10^6} = 0.00138\text{ m} = 1.38\text{mm}$$

This elastic shortening of the pile may be added to the settlement due to the compression of the soil.

5.3.2(1) – Driven Piles

- Set criteria (already implemented in current practice) for H-piles are listed in the 2017 Code :

1. Final set ≥ 25mm per 10 blows and ≤ 100mm per 10 blows
2. When the calculated final set is between 50mm and 100mm per 10 blows, the final set should be taken as 50mm per 10 blows;
3. Final set will not be accepted if $(c_p+c_q)/L > 1.15$ where c_p, c_q are in mm and L in m.

- Some discussions :

1. Final set ≥ 25mm per 10 blows was from Civil Engineering Practice 4 (ICE, 1954) so as to discourage Contractors from using light hammer with large drop height which can easily damage concrete piles.
2. The Final set ≥ 25mm per 10 blows criterion was later abandoned in CP2004 and BS8004 and replaced by Final set ≤ 50mm per 10 blows with unknown reasons;
3. Under Final set ≥ 25mm per 10 blows and ≤ 50mm per 10 blows, the set table becomes very narrow. So the 2017 Code allows the upper limit to extend to ≤ 100mm per 10 blows but capped at 50mm per 10 blows;
4. The criterion $(c_p+c_q)/L > 1.15$ is to control the driving stress to be < 80% of yield stress.

Pile Length (m)	Temporary Compression c_p+c_q																									
	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
15	146	141	136	131	126	121	116	111	106	101	96	91	86	81	76	71	66	61	56	51	46	41	36	31	26	21
16	144	139	134	129	124	119	114	109	104	99	94	89	84	79	74	69	64	59	54	49	44	39	34	29	24	19
17	142	137	132	127	122	117	112	107	102	97	92	87	82	77	72	67	62	57	52	47	42	37	32	27	22	17
18	141	136	131	126	121	116	111	106	101	96	91	86	81	76	71	66	61	56	51	46	41	36	31	26	21	16
19	139	134	129	124	119	114	109	104	99	94	89	84	79	74	69	64	59	54	49	44	39	34	29	24	19	14
20	137	132	127	122	117	112	107	102	97	92	87	82	77	72	67	62	57	52	47	42	37	32	27	22	17	12
21	136	131	126	121	116	111	106	101	96	91	86	81	76	71	66	61	56	51	46	41	36	31	26	21	16	11
22	134	129	124	119	114	109	104	99	94	89	84	79	74	69	64	59	54	49	44	39	34	29	24	19	14	9
23	133	128	123	118	113	108	103	98	93	88	83	78	73	68	63	58	53	48	43	38	33	28	23	18	13	8
24	131	126	121	116	111	106	101	96	91	86	81	76	71	66	61	56	51	46	41	36	31	26	21	16	11	6
25	130	125	120	115	110	105	100	95	90	85	80	75	70	65	60	55	50	45	40	35	30	25	20	15	10	5
26	129	124	119	114	109	104	99	94	89	84	79	74	69	64	59	54	49	44	39	34	29	24	19	14	9	4
27	127	122	117	112	107	102	97	92	87	82	77	72	67	62	57	52	47	42	37	32	27	22	17	12	7	2
28	126	121	116	111	106	101	96	91	86	81	76	71	66	61	56	51	46	41	36	31	26	21	16	11	6	1
29	124	119	114	109	104	99	94	89	84	79	74	69	64	59	54	49	44	39	34	29	24	19	14	9	4	-1
30	123	118	113	108	103	98	93	88	83	78	73	68	63	58	53	48	43	38	33	28	23	18	13	8	3	-2
31	122	117	112	107	102	97	92	87	82	77	72	67	62	57	52	47	42	37	32	27	22	17	12	7	2	-3
32	121	116	111	106	101	96	91	86	81	76	71	66	61	56	51	46	41	36	31	26	21	16	11	6	1	-4
33	119	114	109	104	99	94	89	84	79	74	69	64	59	54	49	44	39	34	29	24	19	14	9	4	-1	-6
34	118	113	108	103	98	93	88	83	78	73	68	63	58	53	48	43	38	33	28	23	18	13	8	3	-2	-7
35	117	112	107	102	97	92	87	82	77	72	67	62	57	52	47	42	37	32	27	22	17	12	7	2	-3	-8
36	116	111	106	101	96	91	86	81	76	71	66	61	56	51	46	41	36	31	26	21	16	11	6	1	-4	-9
37	115	110	105	100	95	90	85	80	75	70	65	60	55	50	45	40	35	30	25	20	15	10	5	0	-5	-10
38	113	108	103	98	93	88	83	78	73	68	63	58	53	48	43	38	33	28	23	18	13	8	3	-2	-7	-12
39	112	107	102	97	92	87	82	77	72	67	62	57	52	47	42	37	32	27	22	17	12	7	2	-3	-8	-13
40	111	106	101	96	91	86	81	76	71	66	61	56	51	46	41	36	31	26	21	16	11	6	1	-4	-9	-14
41	110	105	100	95	90	85	80	75	70	65	60	55	50	45	40	35	30	25	20	15	10	5	0	-5	-10	-15
42	109	104	99	94	89	84	79	74	69	64	59	54	49	44	39	34	29	24	19	14	9	4	-1	-6	-11	-16
43	108	103	98	93	88	83	78	73	68	63	58	53	48	43	38	33	28	23	18	13	8	3	-2	-7	-12	-17
44	107	102	97	92	87	82	77	72	67	62	57	52	47	42	37	32	27	22	17	12	7	2	-3	-8	-13	-18

Table HK-1 – Final Set Values per 10 blows with No Restriction on the *S* values

Pile Length (m)	Temporary Compression c_p+c_q																									
	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
15	-	-	-	-	-	-	-	-	-	-	50	50	-	-	-	-	-	-	-	-	-	-	-	-	-	-
16	-	-	-	-	-	-	-	-	-	50	50	50	50	-	-	-	**Zone C1** -	-	-	-	-	-	-	-	-	-
17	-	-	-	-	-	-	-	-	-	50	50	50	50	50	-	-	-	-	-	-	-	-	-	-	-	-
18	-	-	-	-	-	-	-	-	-	50	50	50	50	50	50	-	-	-	-	-	-	-	-	-	-	-
19	-	-	-	-	-	-	-	-	50	50	50	50	50	50	50	50	-	-	-	-	**Zone C2** -	-	-	-	-	-
20	-	-	-	-	-	-	-	-	50	50	50	50	50	50	50	50	50	-	-	-	-	-	-	-	-	-
21	-	-	-	-	-	-	-	-	50	50	50	50	50	50	50	50	50	50	46	-	-	-	-	-	-	-
22	-	-	-	-	-	-	-	50	50	50	50	50	50	50	50	50	50	49	44	39	-	-	-	-	-	-
23	-	-	-	-	-	-	-	50	50	50	50	50	50	50	50	50	50	48	43	38	33	-	-	-	-	-
24	-	-	**Zone A** -	-	-	-	-	50	50	50	50	50	50	50	50	50	50	46	41	36	31	26	-	-	-	-
25	-	-	-	-	-	-	50	50	50	50	50	50	50	50	50	50	50	45	40	35	30	-	-	-	-	-
26	-	-	-	-	-	-	50	50	50	50	50	50	50	50	50	50	49	44	39	34	29	-	-	-	-	-
27	-	-	-	-	-	-	50	50	50	50	50	50	50	50	50	50	47	42	37	32	27	-	-	-	-	-
28	-	-	-	-	-	-	50	50	50	50	50	50	50	50	50	50	46	41	36	31	26	-	-	-	-	-
29	-	-	-	-	-	50	50	50	50	50	50	50	50	50	50	49	44	39	34	29	-	-	-	-	-	-
30	-	-	-	-	-	50	50	50	50	50	50	50	50	50	50	48	43	38	33	28	-	-	-	-	-	-
31	-	-	-	-	-	50	50	50	50	50	50	50	50	50	50	47	42	37	32	27	-	-	**Zone B** -	-	-	-
32	-	-	-	-	-	50	50	50	50	**Zone E** 50	50	50	50	50	50	46	41	36	31	26	-	-	-	-	-	-
33	-	-	-	-	50	50	50	50	50	50	50	50	50	50	49	44	39	34	29	-	-	-	-	-	-	-
34	-	-	-	-	50	50	50	50	50	50	50	50	50	50	48	43	38	33	28	-	-	-	-	-	-	-
35	-	-	-	-	50	50	50	50	50	50	50	50	50	50	47	42	37	32	27	-	-	-	-	-	-	-
36	-	-	-	-	50	50	50	50	50	50	50	50	50	50	46	41	36	31	26	-	-	-	-	-	-	-
37	-	-	-	50	50	50	50	50	50	50	50	50	50	50	45	4 **Zone D**	35	30	-	-	-	-	-	-	-	-
38	-	-	-	50	50	50	50	50	50	50	50	50	50	48	43	38	33	28	-	-	-	-	-	-	-	-
39	-	-	-	50	50	50	50	50	50	50	50	50	50	47	42	37	32	27	-	-	-	-	-	-	-	-
40	-	-	-	50	50	50	50	50	50	50	50	50	50	46	41	36	31	26	-	-	-	-	-	-	-	-
41	-	-	-	50	50	50	50	50	50	50	50	50	50	45	40	35	30	25	-	-	-	-	-	-	-	-
42	-	-	50	50	50	50	50	50	50	50	50	50	49	44	39	34	29	-	-	-	-	-	-	-	-	-
43	-	-	50	50	50	50	50	50	50	50	50	50	48	43	38	33	28	-	-	-	-	-	-	-	-	-
44	-	-	50	50	50	50	50	50	50	50	50	50	47	42	37	32	27	-	-	-	-	-	-	-	-	-

Only Zones D and E are applicable zones.
Zone A : $S \geq 100$;
Zone B : $S \leq 25$;
Zone C including Zone C1 and C2 is for $c_p+c_q > 1.15L$. Zone C1 is with $S \geq 50$ and Zone C2 is with $50 \geq S \geq 25$ which is acceptable in the previous practice when the new restraint $c_p+c_q > 1.15L$ is not imposed;
Zone D is for $50 \geq S \geq 25$ and $c_p+c_q \leq 1.15L$;
Zone E is for $100 \geq S > 50$ and $c_p+c_q \leq 1.15L$.

5.3.2(1) – Driven Piles

- The Hiley Formula is listed in the 2017 Code with :

1. the hammer efficiency E_h set to ≤ 0.7 for drop hammer;
2. c_c, the temporary compression of the hammer cushion $\leq$ 5mm when plastic cushion < 200mm thick is used.

5.3.2(2) – Non-driven Piles

- Bearing derived from ground for non-driven piles:

- The 2004 Code generally allows piles socketed into rock to derive bearing from a combination of end-bearing and friction / bond. The 2017 Code however, explicitly allows only LDBP to do so (under some limitations). For other piles, the use of both shaft friction and end-bearing would requires justification that settlements under working load conditions are acceptable and adequate to mobilize the required shaft friction and end bearing simultaneously;

5.3.3 – Ground Resistance for Piles subjected to Uplift Forces

- The clause states clearly methods to determine ground resistance from as the lesser of

1. Shaft friction / bond;
2. Anchorage by effective weight of soil mass/rock cone

- The dimensions of the anchorage weight of soil mass/rock cone are given. However, for exact determination, the geometry is very complicated.

- It should be noted that the weight of soil mass/rock cone so assessed contribute to the “ultimate anchorage”.

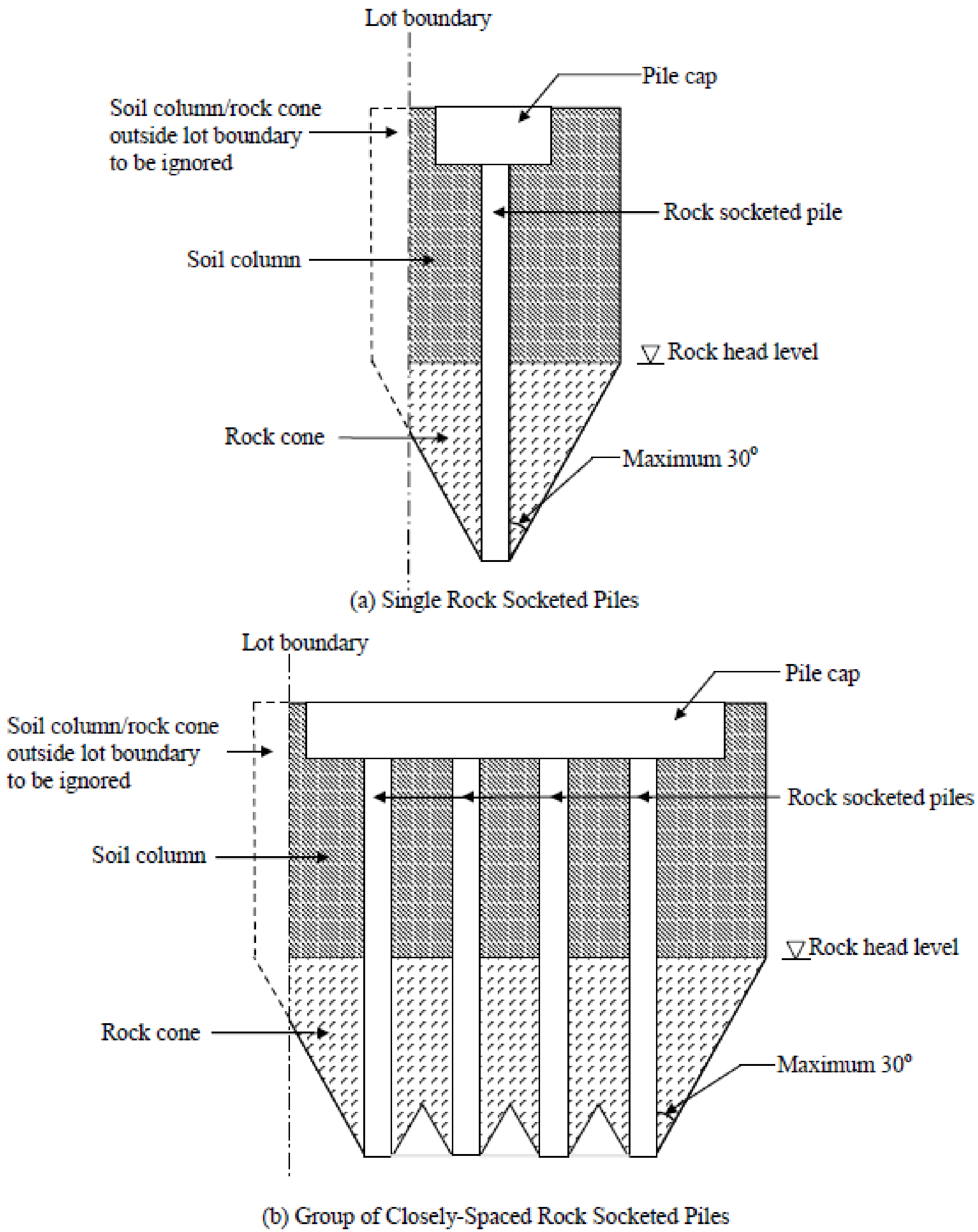


Figure 5.1 Configuration of Rock Cone/Soil Column for Rock Socketed Piles

HN.1 Useful Mathematical Expressions for Determination of Geometry of Anchorages by Soil and Rock

Due to overlapping of the soil columns and rock cones for piles against uplift as illustrated by Figures 5.1 and 5.2 of the Code, the shaded portions of the soil column and rock cone on the top right portion of Figure HN-1 should be deducted from calculation of effective weight for a pile against uplift. Mathematical expressions for the determination of the geometric volumes of the overlapped portions are presented in the first part of this appendix.

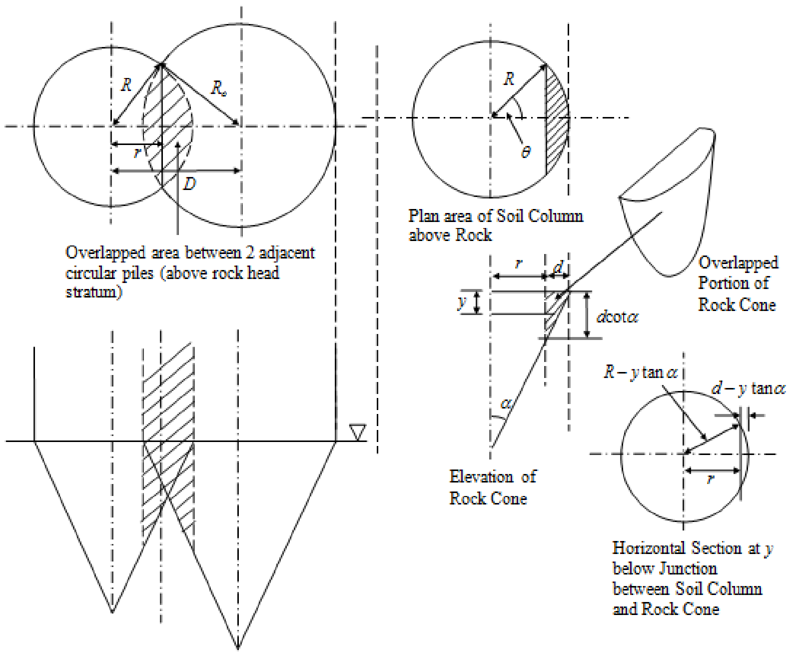


Figure HN-1 – Geometrical Shape due to Overlapping of Two Adjacent Circular Piles

Consider a circular pile with R as its radius of the soil column as shown in Figure 5.1 of the Code. If its soil column overlaps with another pile of radius of soil column R_o with centre to centre distance D as shown in the top left portion of Figure HN-1, the distance r can be determined through the application of the cosine rule as

$$r = R\cos\theta = \frac{R^2 + D^2 - R_o^2}{2D} \qquad \text{(Eqn HN-1)}$$

Referring to the top right portion of Figure HN-1, the area of shaded portion is

$$A_s = \frac{1}{2}R^2(2\theta) - r\sqrt{R^2-r^2} = R^2\sin^{-1}\left(\frac{\sqrt{R^2-r^2}}{R}\right) - r\sqrt{R^2-r^2} \qquad \text{(Eqn HN-2)}$$

The volume of the overlapped portion of the soil column is

$$V_s = A_s L \qquad \text{(Eqn HN-3)}$$

Below the rock head level, the overlapped portion will be that beyond a vertical plane at r from the centre a height of $d\cot\alpha$ as illustrated in Figure HN-2, with α limited to 30º by the Code. Consider an "elementary slice" at depth y beneath rock head stratum, the radius of the elementary slice is $R - y\tan\alpha$ and the width beyond "touching line" with the adjacent circular pile is $d - y\tan\alpha$ as illustrated in Figure HN-2. So the area of the overlapped portion by (Eqn HN-2) for one pile is

$$A(y) = [R - y\tan\alpha]^2 \sin^{-1}\left(\frac{\sqrt{(R-y\tan\alpha)^2 - r^2}}{R - y\tan\alpha}\right) - r\sqrt{(R-y\tan\alpha)^2 - r^2}$$

The volume of the elementary slice will be $A(y)dy$. Integrating over the height of overlapping of $d\cot\alpha$, the volume of the overlapping portion of the rock cone is

$$V_r = \int_0^{d\cot\alpha} A(y)dy$$

Analytically, the result is

$$V_r = \frac{\cot\alpha}{3}\left[r^3\ln\left(\frac{r}{R-\sqrt{R^2-r^2}}\right) + R^3\sin^{-1}\left(\frac{\sqrt{R^2-r^2}}{R}\right) - 2Rr\sqrt{R^2-r^2}\right] \qquad \text{(Eqn HN-4)}$$

Nevertheless, an estimate will be treating the overlapping portion as a pyramid which results in a smaller value and subsequently a smaller deduction for the final effective weight, so

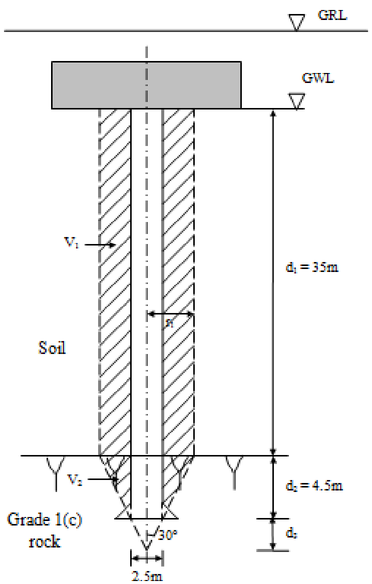
$$V_r \approx \frac{d\cot\alpha}{3}A_s \qquad \text{(Eqn HN-5)}$$

where A_s is taken from (Eqn HN-2).

The above geometric expressions can be used to evaluate the volume of soil / rock columns / cones as required by clause 5.3.3 of the Code.

Worked Examples for Determination of Ultimate Uplift Resistance of Piles

Worked Example HN-1 – Ultimate Uplift Resistance of Large Diameter Bored Pile



<u>Pile Loading</u>

D.L. = 30000kN
D_{min} = 26000kN
LL = 12000kN
Wind W = 12000kN
Uplift, U = 14500kN

Density of Soil taken as $19kN/m^3$
Density of Rock taken as $22\ kN/m^3$
Density of concrete taken as $24.5kN/m^3$
Density of water taken as $9.8kN/m^3$

Ground Water Level : at Underside of Pile Cap

Rock anchoring the pile is Grade 1(c)

Figure HN-2 – Worked Example for Checking Uplift of Large Diameter Bored Pile

<u>Check for Allowable Bond Resistance to Table 2.2 of the Code (under permanent tension condition)</u>
Allowable Bond Resistance from Rock $R_b = 2.5 \times \pi \times 4.5 \times 350 = 12370.02kN$

<u>Check for Ground Resistance to Figure H5.3(a)</u>
From the above Figure HN-2 $d_3 = 2500 \div 2 \div \tan 30^\circ = 2165.06mm$
$r_1 = (4500 + 2165.06) \times \tan 30^\circ = 3848mm$
Volume $V_1 = (3.848^2 - 1.25^2) \times \pi \times 35 = 1456.32\ m^3$
Volume $V_2 = [(3.848^2 \times (4.5 + 2.165) - 1.25^2 \times 2.165)] / 3 \times \pi - 1.25^2 \times 4.5 \times \pi = 77.72\ m^3$
Weight of Soil Column $W'_1 = V_1 \times (19 - 9.8) = 13398.14kN$
Weight of Rock Cone $W'_2 = V_2 \times (22 - 9.8) = 948.18kN$
Weight of Pile $W'_p = 1.25^2 \times \pi \times (35 + 4.5) \times (24.5 - 9.8) = 2850.26\ kN$
$W'_1 + W'_2 = 14346.32\ kN$
Allowable Anchorage Resistance $R_a = (W'_1 + W'_2)/F + W'_p = 17196.58/2 = 10023.42kN < R_b = 12370kN$

Ultimate Anchorage Resistance $R_o = W'_1 + W'_2 + W'_p = 17197kN$

<u>Check for Ultimate Anchorage to Cl. 5.1.6 and Allowable Anchorage to Cl. 5.3.3</u>
$D_{min} + 0.9R_o - 2.0I_a - 1.5U_a - 1.5W = 26000 + 0.9 \times 17197 - 2 \times 0 - 1.5 \times 14500 - 1.5 \times 12000 = 1727\ kN > 0$
$I_a + U_a + W_k - D_{min} = 0 + 14500 + 12000 - 26000 = 500 \le R_a = 8598kN$
So both conditions are satisfied.

Worked Example HN-2 – Ultimate Uplift Resistance of Group of Large Diameter Bored Piles

Consider a group of 4 large diameter bored piles, each of same geometry as in Worked Example HN-2 with centre to centre 6.0m as shown in Figure HN-3.

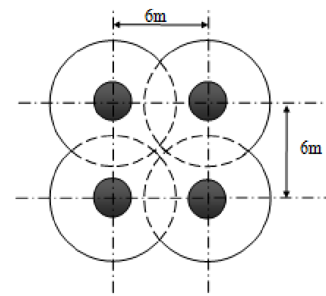


Figure HN-3 – Overlapping of Soil / Rock Column / Cone of Large Diameter Bored Pile for Worked Example HN-2

Consider one single pile, the volumes of the overlapped portion of the soil column and rock cone of a pile are to be worked out.

By (Eqn HN-1), the distance r is calculated as

$$r = \frac{R^2 + D^2 - R_o^2}{2D} = \frac{3.848^2 + 6^2 - 3.848^2}{2 \times 6} = 3\,\text{m}$$

(In fact it can readily be seen that $r = 0.5D$ if $R = R_o$)

By (Eqn HN-2), the area of the overlapped portion in the soil column is

$$A_s = R^2 \sin^{-1}\left(\frac{\sqrt{R^2 - r^2}}{R}\right) - r\sqrt{R^2 - r^2} = 2.791\,\text{m}^2;$$

By (Eqn HN-3), the volume of an overlapped portion soil column is

$$V_s = A_s L = 2.791 \times 35 = 97.685\,\text{m}^3;$$

As there are 2 overlaps, the total volume of soil to be deducted is $97.685 \times 2 = 195.37\ m^3$ and the weight of the soil column is

$W_s = 195.37 \times (19 - 9.8) = 1797.404$ kN

By (Eqn HN-4), volume of an overlapped portion of the rock cone is

$$V_r = \frac{\cot\alpha}{3}\left[r^3 \ln\left(\frac{r}{R - \sqrt{R^2 - r^2}}\right) + R^3 \sin^{-1}\left(\frac{\sqrt{R^2 - r^2}}{R}\right) - 2Rr\sqrt{R^2 - r^2}\right] = 1.6\,\mathrm{m}^3;$$

Or by (Eqn HN-5),

$$V_r \approx \frac{d\cot\alpha}{3} A_s = 1.366\,\mathrm{m}^3$$

The weight of the rock cone (again 2 no.) is $W_r = 1.6 \times (22 - 9.8) \times 2 = 39.04$ kN.

So total weight to be deducted as the anchorage of a pile becomes

$W_s + W_r = 1797.404 + 39.04 = 1836.444$ kN

So the corrected weight for anchorage by soil and rock is

$17197 - 1836 = 15361$ kN

Worked Example HN-5 – Effective weight of the soil cone/soil column

The effective weight of the soil cone/soil column of a pile group comprising 9 driven H-piles (305×305×223 kg/m) against uplift is as shown in Figure HN-4. The piles are 4m centre to centre apart.

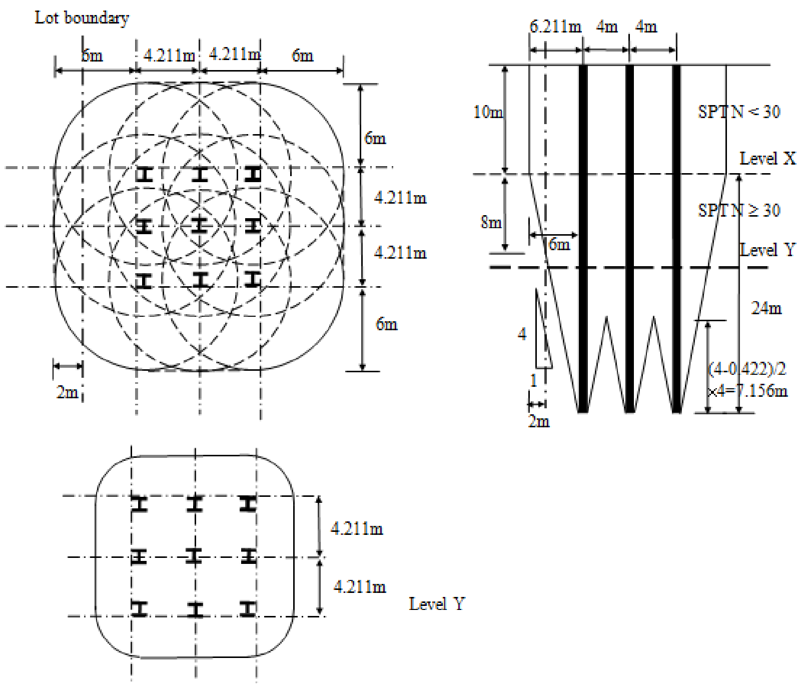


Figure HN-4 – Pile Layout for Worked Example HN-5

Ignoring the lot boundary first, the equivalent diameter of the H-pile d_p is first to be determined by equating the perimeter of the H-pile to a circle of diameter d_p as

$\pi d_p = 1.326 \Rightarrow d_p = 0.422$

So the radius of soil column by each pile above Level X (above which SPT N-value < 30) is $6 + 0.422/2 = 6.211\,\text{m}$

The plan area of the soil column at ground level is

$8.422^2 + 4\times 8.422\times 6 + \pi\times 6^2 = 386.155\,\text{m}^2$

So volume of the soil column above Level X is $386.155\times 10 = 3861.55\,\text{m}^3$.

The soil volume beneath Level X can be regarded as comprising a square cylinder, 4 prisms and 4 quadrant circular cones at the corners as shown in Level Y in Figure HN-4. So the volume is

$$8.422^2\times 24 + 4\times\frac{1}{2}\times 6\times 8.422\times 24 + 4\times\frac{1}{4}\times\frac{1}{3}\times\pi\times 6^2\times 24 = 1702.322 + 2425.536 + 904.779$$
$$= 5032.637\,\text{m}^3$$

However, there are 4 "pyramids" of plan area $(4-0.422)^2 = 12.802\,\text{m}^2$ and height 7.156m among the pile tip that need to be deducted. The volume of these 4 "pyramids" is

$$4\times\frac{1}{3}\times 12.802\times 7.156 = 122.148\,\text{m}^3$$

Adding up, volume of soil without consideration of the lot boundary is $3861.55 + 5032.637 - 122.148 = 8772.039\,\text{m}^3$

If the lot boundary is considered, volume beyond the lot boundary is to be determined for deduction. The plan area of the portion is (with the use of (Eqn HN-2)

$$8.422\times 2 + 6^2\sin^{-1}\left(\frac{\sqrt{6^2-4^2}}{6}\right) - 4\sqrt{6^2-4^2} = 29.234\,\text{m}^2$$, giving a volume of

$29.234\times 10 = 292.339\,\text{m}^3$ above Level X.

For the portion below Level X comprising a prism and a cut cone with volume that can be calculated by (Eqn HN-4), the volume is

$$\frac{1}{2}\times 8.422\times 2\times 8 + \frac{4}{3}\left[4^3\ln\left(\frac{4}{6-\sqrt{6^2-4^2}}\right) + 6^3\sin^{-1}\left(\frac{\sqrt{6^2-4^2}}{6}\right) - 2\times 6\times 4\sqrt{6^2-4^2}\right] = 95.514\,\text{m}^3.$$

So volume of soil beyond lot boundary to be deducted is $292.339 + 95.514 = 387.853\,\text{m}^3$.

Net soil volume after deduction of the portion beyond lot boundary is $8772.039 - 387.853 = 8384.186\,\text{m}^3$.

If the buoyant unit weight of soil is $19 - 9.8 = 9.2\,\text{kN/m}^3$, the total soil weight for balancing uplift is $8384.186\times 9.2 = 77134.511\,\text{kN}$.

Shared among the 9 identical piles, the effective weight available for each pile against uplift is $77134.511/9 = 8570.501\,\text{kN}$.

So very likely the bond friction of soil on pile shaft is the controlling factor in this case.

5.3.3 – Ground Resistance for Piles subjected to Uplift Forces

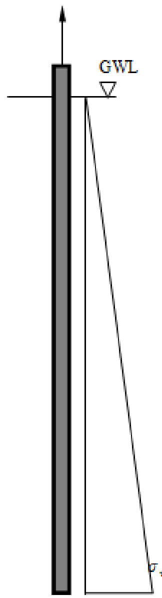
- For the friction and bond along the pile shaft, Table 2.1 can be used for bond with rock. For friction of pile with soil, 3 methods are indicated

1. Uniform shaft friction : allowable 10kPa for SPTN $\geq$ 10, ultimate value : 20kPa;
2. Effective stress method (the beta approach) τ_s(ultimate value) =$\beta\sigma_v$< 120kPa with SPTN $\geq$20, soil density $\geq$20kN/m^3 and $\beta \leq 0.2$
3. Empirical method by correlation with SPTN values, basically τ_s(ultimate value) =$0.75N$< 60kPa (with no trial pile)

Worked Example HN-3 –Uplift Resistance of Driven H-Pile – Effective Stress Method

A driven H-pile (305×305×223 kg/m) 46m long encounters soil of N-values as shown in Table HN-1. Adopting the Effective Stress Method $\tau_s = \beta\sigma_v'$ capped at 120kPa in accordance with clause 5.3.3(3)(ii), (also subject to the conditions stated in the clause so that trial pile is not used). With $\beta = 0.2$, the unit skin frictions worked out are shown in Table HN-1.

Depth of Pile Below Ground (m)	N-value	σ_v' (kPa)	Unit Skin Friction $\tau_s = \beta\sigma_v'$ (kPa)	Friction Force per unit Perimeter of Pile (kN/m)
0	0	0.0	0.00	0.00
2	0	18.4	0.00	0.00
4	11	36.8	0.00	0.00
6	11	55.2	0.00	0.00
8	51	73.6	14.72	29.44
10	50	92.0	18.40	36.80
12	15	110.4	0.00	0.00
14	41	128.8	25.76	51.52
16	48	147.2	29.44	58.88
18	54	165.6	33.12	66.24
20	62	184.0	36.80	73.60
22	69	202.4	40.48	80.96
24	75	220.8	44.16	88.32
26	60	239.2	47.84	95.68
28	58	257.6	51.52	103.04
30	59	276.0	55.20	110.40
32	68	294.4	58.88	117.76
34	71	312.8	62.56	125.12
36	73	331.2	66.24	132.48
38	77	349.6	69.92	139.84
40	81	368.0	73.60	147.20
42	80	386.4	77.28	154.56
44	83	404.8	80.96	161.92
46	82	423.2	84.64	169.28
			Sum	1943.04



Pile Loading

D.L. = 4500kN
D_{min} = 3600kN
LL = 500kN
Wind W_k = 1300kN
Uplift, U_a =1800kN

Unit weight of soil
$\gamma = 19$ kN/m^3
< 20kN/m^3;

Effective weight of soil
$\gamma' = 19 - 9.8 = 9.2$ kN/m^3
< 10kN/m^3

Table HN-1 – Computation of Ultimate Uplift Resistance of a Driven Pile

As the perimeter of the pile is 1.326m, the total ultimate tension capacity against uplift for transient load on the pile is 1943.04kN/m × 1.326m = 2576kN.

The allowable tension capacity against uplift for permanent load is 2576/2 = 1288kN.

Taking a factor of safety = 3, the allowable uplift resistance of the pile is 2576/3 = 859kN for transient load and 1288/3 = 429kN for permanent load.

Check for Ultimate Anchorage and Allowable Anchorage to Cl. 5.1.6

By $[2.0I_a + 1.5U_a(\text{or } 1.1U_p) - D_{min}]/R_{u\text{-permanent}} + 1.5W_k/R_{u\text{-transient}} = 0.83 < 0.9$

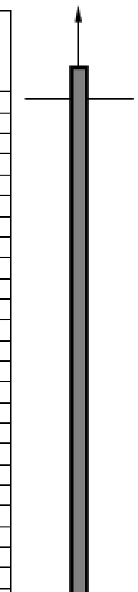
By $[I_a + U_a - D_{min}]/R_{a\text{-permanent}} + W_k/R_{a\text{-transient}} = -1.51 < 1$

So both conditions are satisfied.

Worked Example HN-4 – Uplift Resistance of Driven H-Pile – Empirical Method by SPT N-values

A driven H-pile (305×305×223 kg/m) 46m long encounters soil of N-values as shown in Table HN-2. Adopting the empirical correlation with SPT N-values as ultimate value $\tau_s = 0.75N$ in accordance with clause 5.3.3(3)(iii), with τ_s capped at 60kPa (or SPT N capped at 80), the unit skin frictions worked out are shown in Table HN-2.

Depth of Pile Below Ground (m)	Actual N-Value	Design N-value	Unit Skin Friction (kPa)	Friction Force per unit Perimeter of Pile (kN/m)
0	0	0	0	0
2	0	0	0	0
4	11	11	8.25	16.5
6	11	11	8.25	16.5
8	51	51	38.25	76.5
10	50	50	37.5	75
12	15	15	11.25	22.5
14	41	41	30.75	61.5
16	48	48	36	72
18	54	54	40.5	81
20	62	62	46.5	93
22	69	69	51.75	103.5
24	75	75	56.25	112.5
26	60	60	45	90
28	58	58	43.5	87
30	59	59	44.25	88.5
32	68	68	51	102
34	71	71	53.25	106.5
36	73	73	54.75	109.5
38	77	77	57.75	115.5
40	81	80	60	120
42	80	80	60	120
44	83	80	60	120
46	82	80	60	60
			Sum	1849.5



Pile Loading

D.L. = 4500kN
D_{min} = 3600kN
LL = 500kN
Wind W_k = 1300kN
Uplift, U_a =1800kN

Table HN-2 – Computation of Ultimate Uplift Resistance of a Driven Pile

As the perimeter of the pile is 1.326m, the total ultimate tension capacity against uplift for transient load on the pile is 1849.5kN/m × 1.326m = 2452.44kN.

The ultimate tension capacity against uplift for permanent load is 2452.44/2 = 1226.22kN.

Taking a factor of safety = 3, the allowable uplift resistance of the pile is 2452.44/3 = 817.48kN for transient load and 1226.22/3 = 407.74kN for permanent load.

Check for Ultimate Anchorage to Cl. 5.1.6 and Allowable Anchorage to Cl. 5.3.3

By $[2.0I_a + 1.5U_a(\text{or } 1.1U_p) - D_{min}]/R_{u\text{-permanent}} + 1.5W_k/R_{u\text{-transient}} = 0.88 < 0.9$
By $[I_a + U_a - D_{min}]/R_{a\text{-permanent}} + W_k/R_{a\text{-transient}} = -1.59 < 1$

So both conditions are satisfied.

5.3.4 – Ground Resistance for Piles subjected to Lateral Load

- Two tables are added (Actually in use in the industry for long time) :

Table 5.1 Correlation of Constant of Horizontal Subgrade Reaction with SPT N-values for Granular Soil

SPT N-value	n_h for dry or moist sand ($kN/m^2/m$)	n_h for submerged sand ($kN/m^2/m$)
4 to 10	2200	1300
11 to 30	6600	4400
31 to 50	17600	10700

From GEO Publication 1/2006 (Table 6.11)

Table 5.2 Reduction Factor for Constant of Horizontal Subgrade Reaction for Laterally Loaded Pile Group

Ratio of pile spacing to pile diameter	Reduction factor for n_h
3	0.25
4	0.40
6	0.70
8	1.00

Notes: (1) Pile spacing normal to the direction of loading has no influence, provided that the spacing is greater than 2.5 pile diameter.
(2) Subgrade reaction is to be reduced in the direction of loading.

From GEO Publication 1/2006 (Table 7.2). The variation is a linear one as $y = 0.15x - 0.2$. Actually from Canadian Foundation Engineering Manual 1978, but not found in the updated versions (1992 & 2006)

5.4.2 – Socketed Steel H-piles

- Item (a) under "Design Principle" in the 2004 Code indicating that the allowable axial working stress due to combined axial load and bending to 50% of the steel yield stress has been removed.

- The removal is OK as the original provision in 2004 Code is rather meaningless as the allowable axial working stress for socketed pile is already $0.5f_y$ (stated in 2.5.5(4)).

- Anyhow, inclusion of check for bending can be carried out by limit state design.

5.4.6 – Small Diameter Bored Piles

- Design and construction of Continuous Flight Auger Pile (CFA Pile) has been added as sub-clause (2).

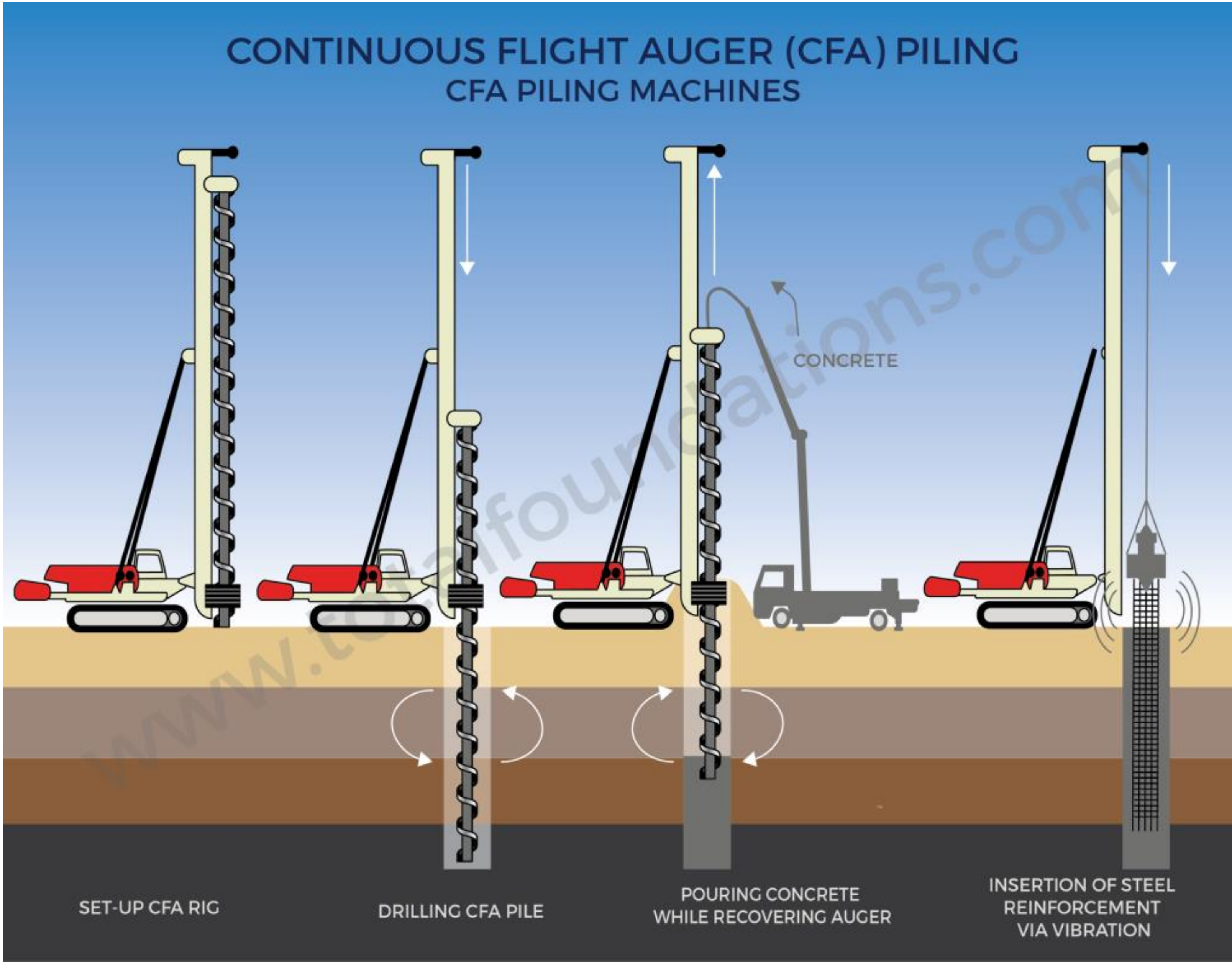
- CFA pile is formed by augering into the ground, lower a reinforcement cage into the hole and subsequently fill up by grout.

- In the design aspect, both end-bearing and friction are used.

$$R_{bc} = \mu N_{av} pL + 5N_b A_b$$

without trial pile : $\mu = 1$; with trial pile can be 1.6

- This type of pile becomes a popular one for low-rise building.


CONTINUOUS FLIGHT AUGER (CFA) PILING
CFA PILING MACHINES
CONCRETE
SET-UP CFA RIG
DRILLING CFA PILE
POURING CONCRETE
WHILE RECOVERING AUGER
INSERTION OF STEEL
REINFORCEMENT
VIA VIBRATION

610mm Dia. CFA pile

Depths Below Ground	Soil Description	Layer of depth (m)	SPT N-value from G.I. report	Design SPT N-value	Shaft Resistance / Depth (kN/m)	Shaft Resistance (kN)
0m – 6m	Fill or marine deposit	–	–	–	Neglected	–
6m – 21m	Completely Decomposed Granite	1.50	18	18	28.8	82.79
		1.50	25	25	40.0	114.98
		1.50	36	36	57.6	165.57
		1.50	48	40	64.0	183.97
		1.50	57	40	64.0	183.97
		1.50	62	40	64.0	183.97
		1.50	78	40	64.0	183.97
		1.50	91	40	64.0	183.97
		1.50	104	40	64.0	183.97
		1.50	118	40	64.0	183.97
		Allowable Friction resistance				1,651.15
		Allowable End bearing resistance = $5 \times 40 \times 0.61^2 \pi/4$				58.45
		Total loading capacity of pile				1,709.60

Table H5.3 – Determination of Geotechnical Capacity of a Small Diameter Bored Pile

5.4.7 – Large Diameter Bored Piles

- LDBP can utilize both end-bearing and rock socket friction in total pile capacity

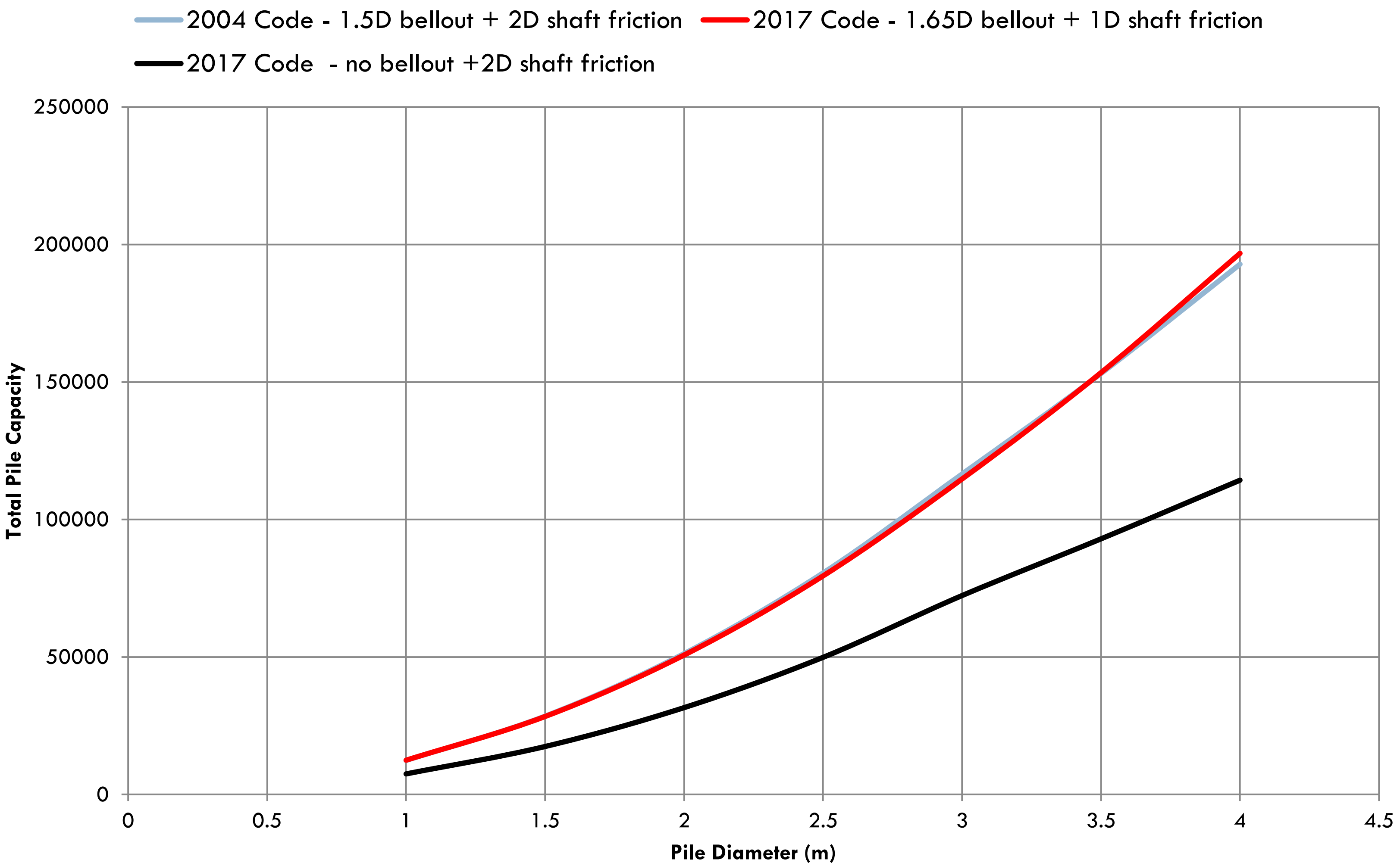
- Main revisions of the 2017 Code, comparing with the 2004 Code are on the limited diameter of bell-out and depth of rock socket

1. Maximum Bell-out dia. from 1.5D to 1.65D;
2. If using bell-out, socket length from 2D < 6m to 1D <3m ;
3. If no bell-out, socket length can remain as 2D

(1 and 3 are very close)

- The revision arises from doubts in mobilization of both end-bearing and friction.

Comparing LDBP Capacities - 2004 Code vs 2017 Code

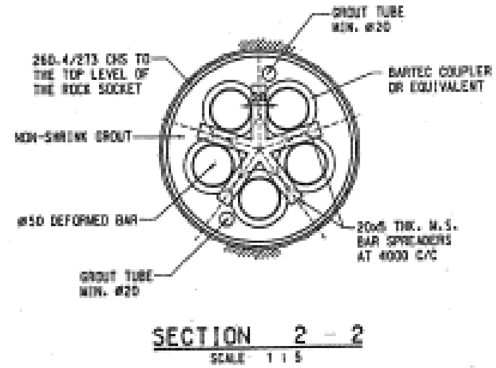


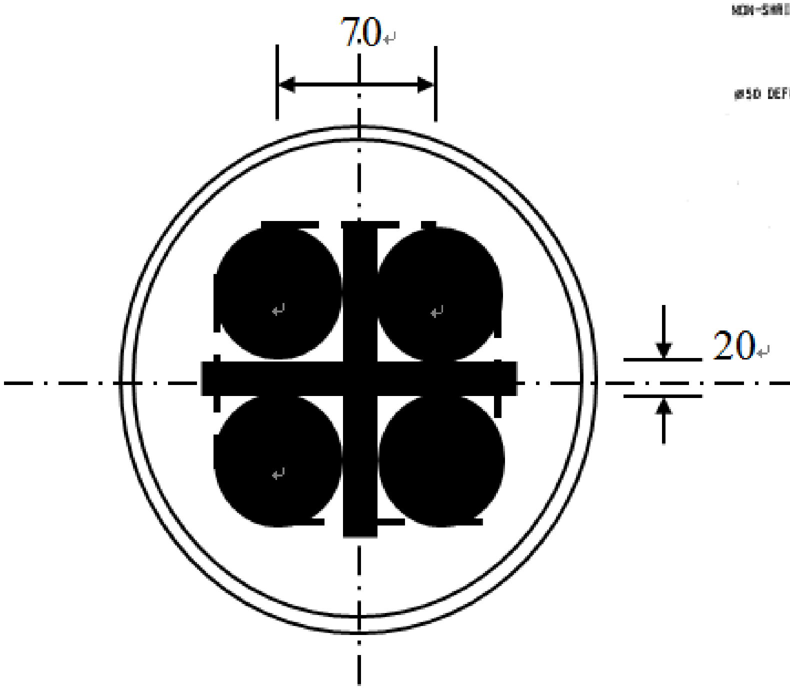
1.5D bell-out +2D shaft friction (2004) very close to 1.65D bell-out +1D shaft friction (2017)

5.4.8 – Mini-Piles

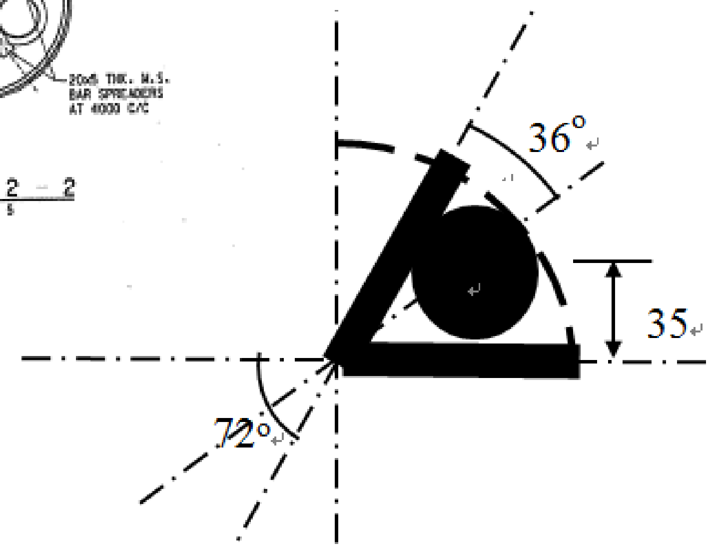
Comparing with the 2004 Code, the following revisions / additions are found in the 2017 Code :

- Imposing limits on bar no. (5 no.) and pile dia. (450mm) and load bearing capacity (2350kN);
- Clarify that the allowable stress of steel bars as 0.475fy. 0.475 likely from 0.5x0.95 with 0.5 to account for loading test (twice working load) and 0.95 as the partial load factor γ_f. So 5T50 gives 2331kN;
- Imposing minimum bar spacing (20mm) and minimum bar cover (30mm);
- Specifying geometric shape of parameter (round square for 4T50 and circular for 5T50);
- Trial pile required if bearing relies on friction on soil;
- Highlighting that lateral stability be duly considered if pile passes through weak soil and cavity;
- Limiting use of raking pile in consolidating soil.



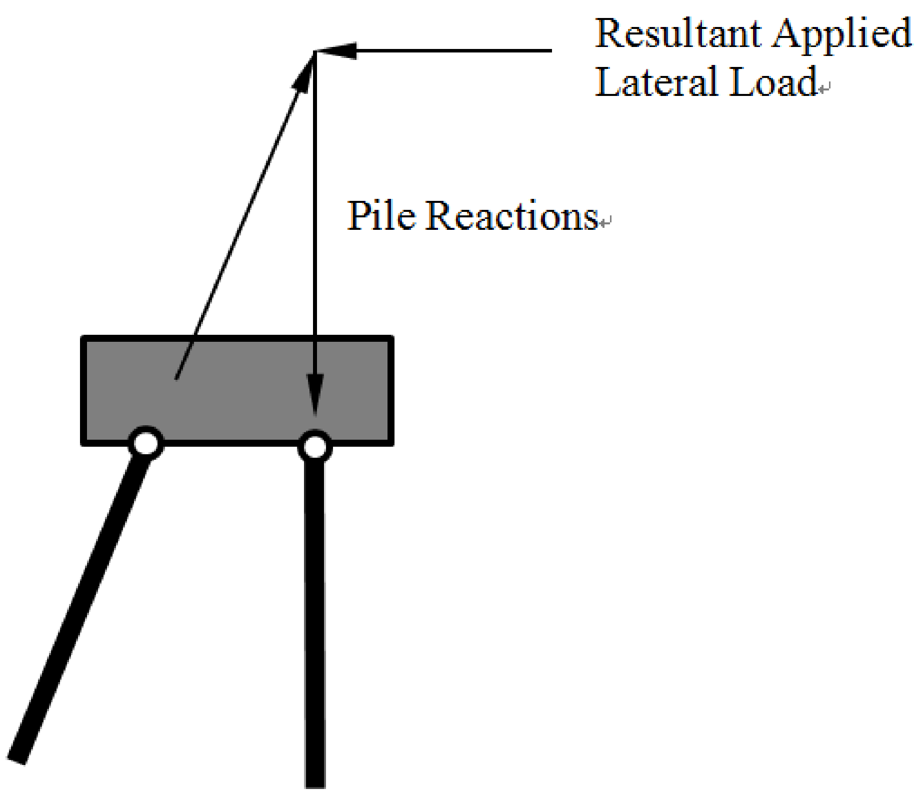


Perimeter of the shear plane (dotted line)
$= 4 \times 0.25 \times 50 \times \pi + 70 \times 4 = 437\text{mm}$



For 5T50 minipile :
Radius of enclosing circle (dotted line)
is $35 \div \sin 36^{\circ} + 25 = 84.546$
So perimeter of the enclosing circle is
$2 \times 84.546 \times \pi = 531\text{mm}$

Figure H5.15 – Shear Plane for Checking Bond Stress between Steel Bars and Grout for 4T50 and 5T50 Mini-pile



The raking pile and the vertical pile should be so arranged that the lines of actions (axial loads) of the piles and the resultant applied lateral load are concurrent so as to achieve rotational equilibrium.

Figure H5.16 – Structural Configuration of Raking Mini-pile Resulting in Pure Axial Loads in the Piles

5.4.11 – Steel H-pile driven to bedrock

The following revisions / additions are found in the 2017 Code as compared with the 2004 Code

- Stating that the clause applies only to steel H-pile driven to bedrock not steeper than 25^{o} to the horizontal;
- The allowance of increase of allowable stress to $0.5f_y$ with bending is deleted and requires reference to 2.5.5(4) for design (limit state design)
- Under sub-clause 2(b), the requirement of monitoring driving stress by PDA (or other methods) is moved to sub-clause (5) with substantial enhancement ;
- Sub-clause 3(c) is added to caution avoidance of constructing end-bearing piles on steeply sloping bedrock surface. Also, construction of H-pile at bedrock (even not steeper than 25^{o} to the horizontal) should be by gradual increase of drop height.
- Sub-clause (5)(d) has been substantially enhanced in the use of dynamic load tests (PDA and CAPWAP) to (i) verify 10% of the working piles (half selected from piles of greater depth); (ii) measure driving stress to $\leq 0.75fy$; (iii) to verify integrity of 20% or more working piles (at lower stress level but $\geq 0.3f_y$; (iv) in case of weak soil ($SPTN_{ave} \leq 20$ for at least 5m).

5.4.12 – Steel H shear pile

This is a new clause for the use of a pile type popularly in use in the industry for resisting shear. It is not required to found on strong stratum.

The clause cautions

- shear piles should be adequately embedded in the pile cap to ensure compatibility of rotation and displacement – rigid joint;
- compatibility with other foundations – load bearing pile, pile cap etc.

- For study of embedment, a finite element analysis by SAP2000 shows that an embedment of 400mm for the popularly used 305 × 305 × 223 pile in thick pile cap is adequate, under the case of adequate surround on 4 sides and even one side with some 500mm concrete side cover;

5.4.12 – Steel H shear pile

Study of Embedment Length of H-Shear Pile in Pile Cap

- For study of adequate length of embedment of the H-pile to achieve "rigid joint connection", a finite element analysis by SAP2000 in which the pile cap is simulated as assembly of "brick elements" and the popularly used 305×305×223 pile as assembly of plate elements;

- Different embedment lengths have been tried and the rotations of the pile under the same bending moment are plotted. At 400mm embedment length where the curve "flattens", it is reasonable to assume that the "maximum embedding capacity" of the pile has been reached by which we can imply that "rigid joint connection" has been achieved as further increase in embedment length will not decrease rotation. It should be borne in mind that zero rotation meaning strict fixed connection can never been attained because the pile cap can never be infinitely stiff (even with great structural depth) and the rotation of the pile actually takes place with local deformation of the pile cap which is near the bottom level. The upper portion of the cap is not anticipated to help decrease the rotation.

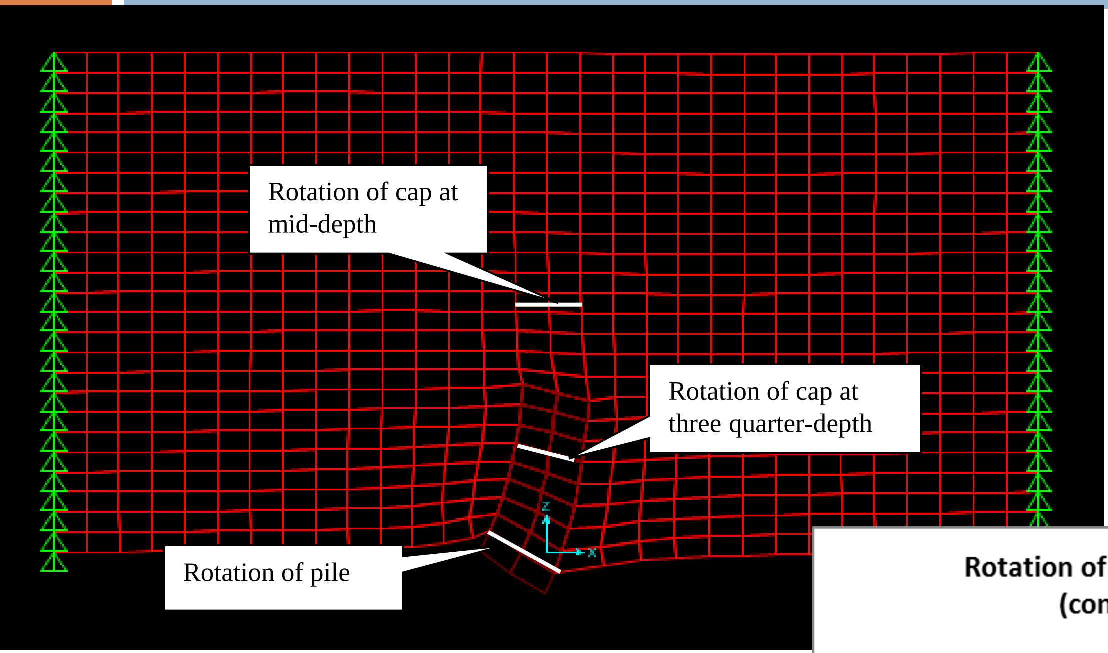
5.4.12 – Steel H shear pile

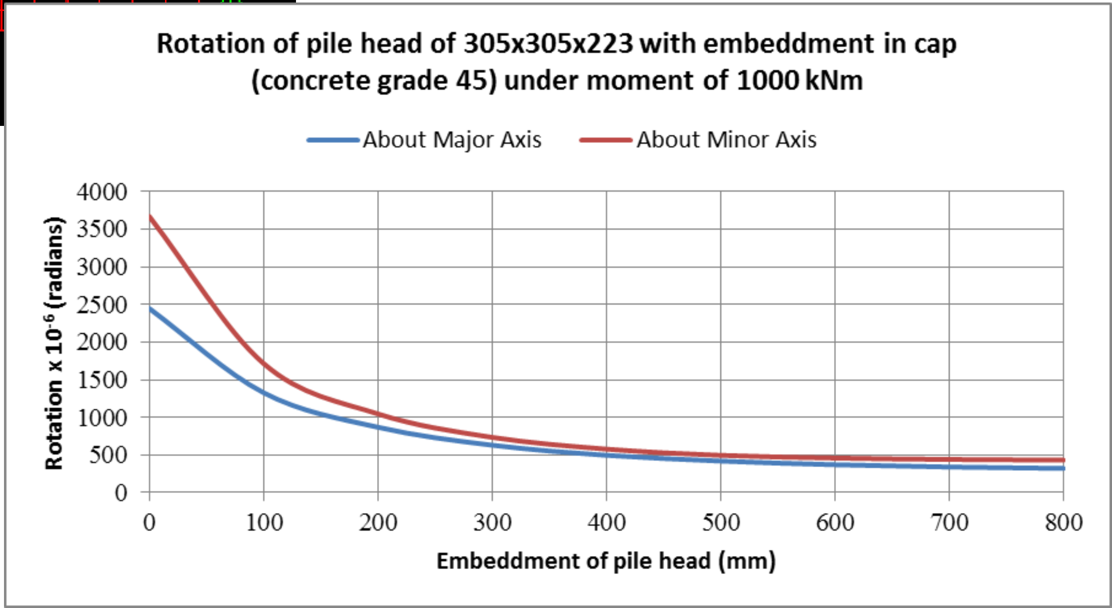
Study of Embedment Length of H-Shear Pile in Pile Cap

- In addition, it is not possible to achieve the conventional "rigid joint" connection by equating the rotation of the pile cap to the rotation of the pile. As shown in the Figure, the rotation of the pile cap as a 3-dimensional structure can have different rotations (or curvatures) at different levels. In the Figure, the rotation decreases from bottom to top and is almost zero at mid-depth. However, our conventional analysis of a piled foundation is by simulating the pile cap as a "plate element" having unique rotations for any points on the same vertical line irrespective of the depth. So we have to rely on the "flattened" rotation in the plot for determination of the required embedment depth for "rigid joint" connection;

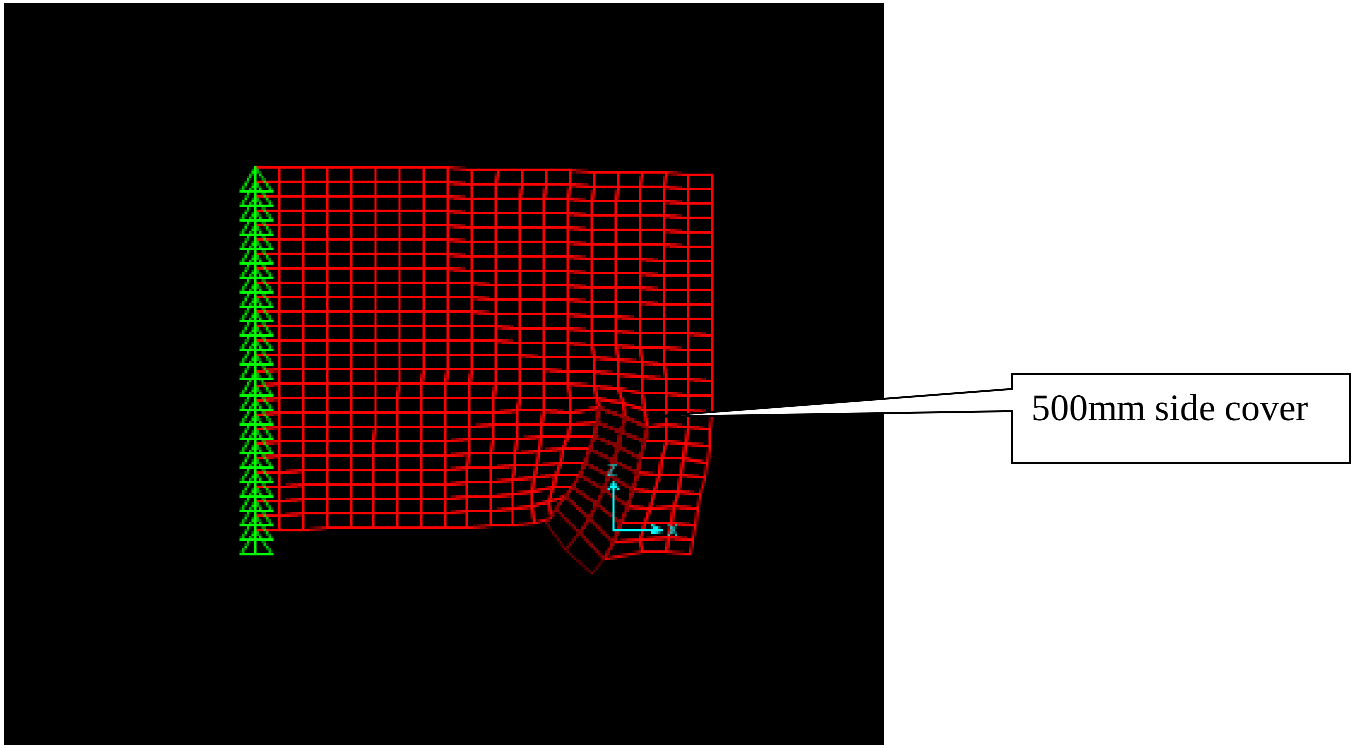
- The scenario with 500mm cover on a vertical side of the pile is also tried. The rotations do not differ significantly from that of 4 sides under thick cover. So the 500mm vertical cover should be considered adequate.

5.4.12 – Steel H shear pile





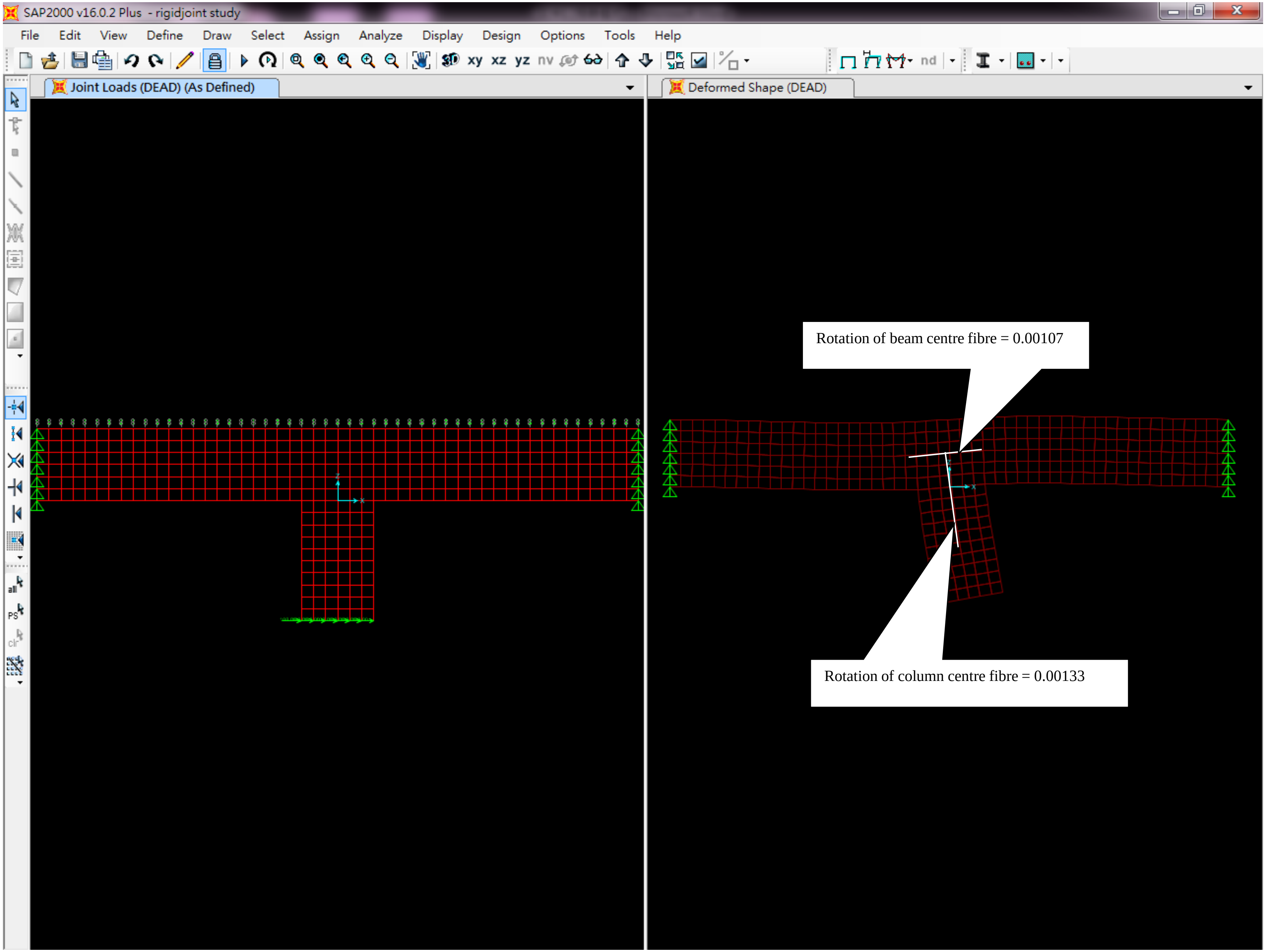
5.4.12 – Steel H shear pile



5.4.12 – Steel H shear pile

Study of Embedment Length of H-Shear Pile in Pile Cap

- A comparison with an ordinary frame structure is also carried out. It seems the rigid joint assumption adopted in simulating a frame structure by line elements can be well justified;


SAP2000 v16.0.2 Plus - rigidjoint study
File Edit View Define Draw Select Assign Analyze Display Design Options Tools Help
Joint Loads (DEAD) (As Defined)
Deformed Shape (DEAD)
Rotation of beam centre fibre = 0.00107
Rotation of column centre fibre = 0.00133

5.5 – Pile Caps

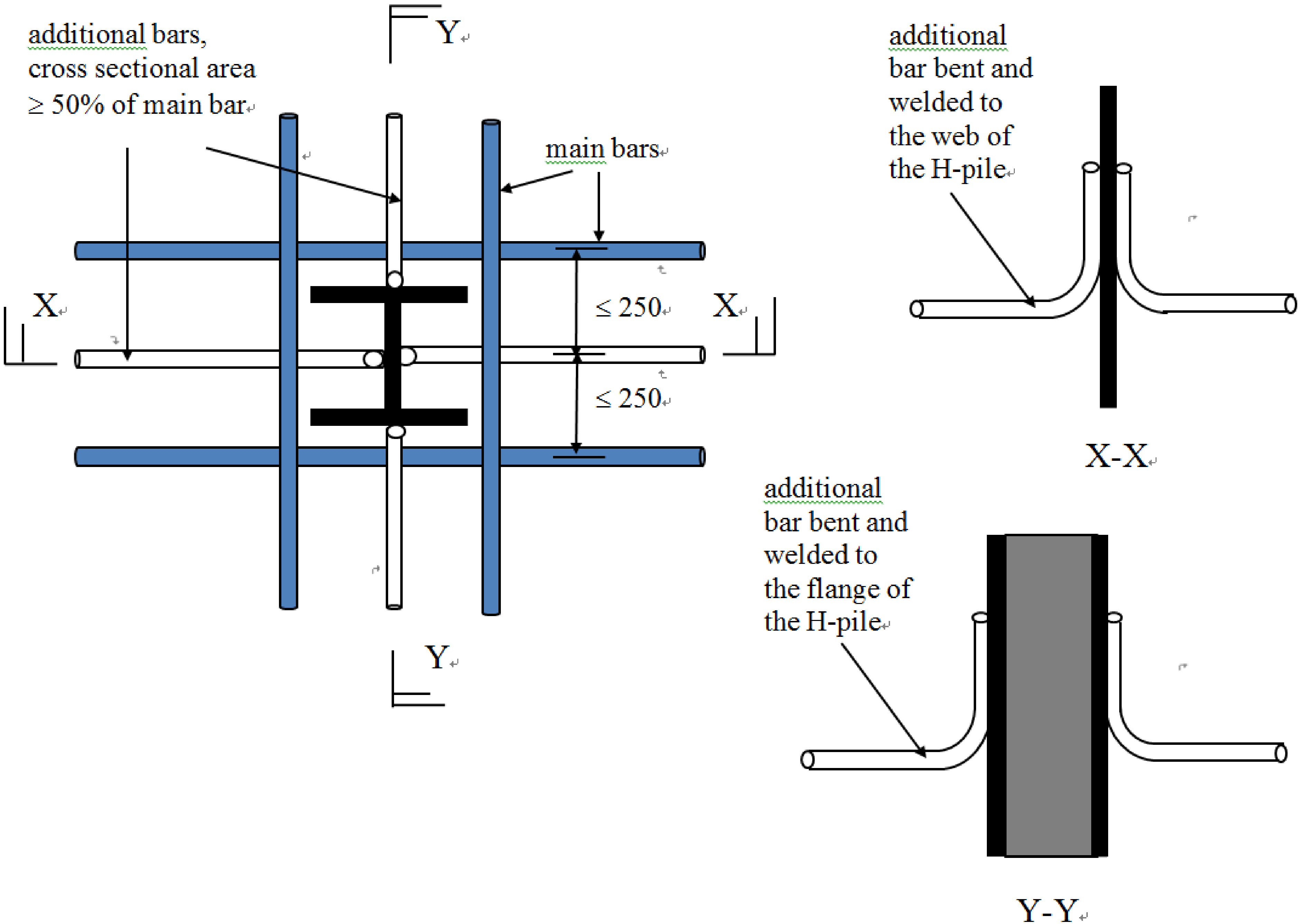
This is a new clause for “Pile Cap” in the 2017 Code :

The clause emphasizes on :

- the use of “flexible cap” analysis with consideration of interactions among piles, pile caps and soil (seems to forbid the use of rigid cap). (However, it should be noted that the rigid cap can achieve more economical layout for pile as by careful planning, every pile under the cap can be stressed to the maximum stress level close to the maximum). The clause says “The distribution of pile loads through the pile cap should generally be carried out by flexible cap analysis ……”;
- However, the Concrete Code 2013 6.7.3.1 says “Pile caps can be designed as rigid or flexible, taking into account various factors including pile spacing and arrangement and pile cap thickness ……..”

5.5 – Pile Caps

- compatibility with other foundations – load bearing pile, pile cap etc.
- Minipiles must be designed as pinned to the pile caps and stability be achieved;
- Reinforcement design to follow Concrete Code 2013. Reinforcement spacing for pile cap thickness $\geq$ 800mm be increased to 400mm but with trimming down to 250mm by additional bars.


additional bars, cross sectional area ≥ 50% of main bar
Y
main bars
X
≤ 250
≤ 250
X
Y
additional bar bent and welded to the web of the H-pile
X-X
additional bar bent and welded to the flange of the H-pile
Y-Y

6.5 – Re-use of Existing Foundation

The clause has been completely re-written in the 2017 Code as compared with the 2004 Code with emphasizes on the “Comprehensive testing schemes” to verify the existing foundation prior to re-use.

Requirements are given to different types of foundations : Large diameter bored piles; small size concrete pile; steel pile; concrete footings. The requirements include visual inspections, core-drilling, dynamic test, re-driving of pile as appropriate.

7.2.3 – Monitoring Plan

- The 3 “A” – “Alert”, “Alarm” and “Action” Levels are explicitly added as contents of the “Monitoring Plan” with illustrations by examples. The 3 “A” levels are already in use in the industry for long time (PANP APP-18)

Table 7.1 Example of the Contingency Measures for Three Triggering Levels

Triggering level	Contingency measures
Alert	The monitoring should be enhanced by increasing the frequency of monitoring measurements and the number of check points.
Alarm	The method of installation of the pile foundation should be reviewed with the purpose of mitigating the detrimental effects arising from vibration or ground settlement.
Action	The corresponding site works should be suspended. Construction activities should not be resumed until the necessary remedial and preventive measures have been completed satisfactorily.

7.2.4 – Ground Settlement

- This is a new clause in the 2017 Code stressing the importance of ground settlement monitoring and gives typical numerical examples for normal buildings for the 3 A Levels.

Table 7.2 Typical Values for the Three Triggering Levels on Nearby Buildings, Structures or Services that are not Sensitive to Settlement

Monitoring check points	Triggering level		
	Alert	**Alarm**	**Action**
Ground settlement	12 mm	18 mm	25 mm
Services settlement / angular rotation	12 mm or 1:600	18 mm or 1:450	25 mm or 1:300
Building tilting	1:1000	1:750	1:500

7.2.6 – Vibrations

- This clause has been substantially enhanced in the 2017 Code, with categorization of vibrations – “continuous”, “transient” and “intermittent”.
- Limits of ppv for protection of buildings are listed;
- Again the limits have been adopted in the industry for long time (PNAP APP-137).

Table 7.3 Limits of ppv for Protection of Buildings

Building condition	Limits of ppv (mm/s)	
	Transient or intermittent vibration	Continuous vibration
Robust and stable buildings in general	15	7.5
Vibration-sensitive or dilapidated buildings	7.5	3

7.2.6 – Vibrations

- A formula, copied from BS5228-2:2009 $v_{res} = k_p \left[\frac{\sqrt{W_e}}{r^{1.3}} \right]$ is listed for assessment of ground-borne vibration. The value k_p may be estimated as 1.5 as a start (3.0 when driven to bedrock) and then verified by back analysis of field measurement.

- Though it is often a specification to limit the ground velocity as a control of vibration caused by construction, acceleration should be a more direct measure which is more directly related to "forces" created to the structures. In fact, acceleration should be an input parameter in dynamic analysis of structures. Vibrograph can measure acceleration directly.

7.4.2 – Test Boring

- This is a new clause in the 2017 Code explaining why test boring is required when the drilling bit advances ahead of the steel casing and the drill hole is larger than 450mm – excessive overbreak and water drawdown;

- Parameters to be assessed by test boring – (i) safety and suitability of the boring method; (ii) water drawdown and ground settlement; (iii) range of anticipated rates of advancement;

- Contents of boring proposal are listed in the clause.

7.5 – Construction Tolerances

- Permissible deviations are listed in the 2017 Code, clarifying that not all piles have 75mm construction deviations on plan as adopted in the previous practice.

Table 7.4 Construction Tolerances

Foundation type	Permissible deviation[1]		
	Position on plan	Verticality	Dimensions
1. Mini-piles	± 15 mm[2]	1 in 100	± 3%
2. Piles for marine structures	± 150 mm	1 in 25	± 3%
3. Piles other than items 1and 2	± 75 mm	1 in 75	± 3%
4. Rafts, ground beams, pile caps and footings	± 50 mm	N/A	± 3%

Notes:

(1) The permissible deviation should not result in any part of the foundation element extending outside the site boundary.

(2) Subject to justification, the value may be increased to a value not exceeding ± 75 mm.

7.8 – Foundation Works in Scheduled Areas

- A new sub-clause in relation to performance review and settlement monitoring for Scheduled Area Nos. 2 and 4 has been added in the 2017 Code.

8.1 – Testing of Foundations and Ground – General

- The clause describes the purpose of testing of foundations and ground in general. A new paragraph is added in the 2017 Code requiring that except SPT and proof test by core-drilling, all tests specified in Chapter 8 should be carried out by a HOKLAS accredited laboratory.

8.2 – Plate Load Test

- Both 2004 and 2017 Codes stipulate that plate load test is to determine allowable bearing capacity and to estimate settlement;

- The procedures for the loading test comprising increment and time for maintaining loads are essentially identical;

- The 2004 Code however only stipulates the test will be acceptable if the maximum settlement of the plate does not exceed S_p given by an equation :

$$S_p = 3 \times S_f \times \left(\frac{B+b}{2B}\right) \times \frac{m+0.5}{1.5m}$$

where S_f is the allowable settlement of footing under allowable working load.

The equation is actually correlating the actual settlement of the footing to that of a plate of much smaller dimensions. It might be the oldest one proposed by Terzaghi (1955).

But S_f is not well defined by the 2004 Code.

8.2 – Plate Load Test

- Plate Load Test in the 2017 Code serves two purpose :

1. To verify allowable bearing capacity; and
2. To back-analyze Young's Modulus for settlement estimation of the foundation.

- For verification of bearing capacity, the 2017 Code appears to be clearer by stipulating that so long the settlement does not exceed 0.15B (B is the dimension of the test plate which is either square or circular) under the test load, the test will pass with arrival of the allowable bearing capacity with the use of the "bearing capacity factor"

$$q_u = cN_c + qN_q + 0.5\gamma BN_\gamma$$

But for cohesionless soil, $c = 0$ and during plate load test, no adjacent surcharge

$$q = 0, q_u = 0.5\gamma BN_\gamma$$

8.2 – Plate Load Test

Ultimate Bearing Capacity

For the formulae given in the 2017 Code

Square plate : $\zeta_{\gamma s} = 1 - 0.4\frac{B}{D} = 1 - 0.4 \times 1 = 0.6$

So the ultimate bearing capacity is $q_u = \zeta_{\gamma s} 0.5\gamma B N_\gamma = 0.3\gamma B N_\gamma$

As $q_a = \frac{q_u}{3} = 0.1\gamma B N_\gamma \Longrightarrow \frac{W_t}{B^2} = 0.1\gamma B N_\gamma \Longrightarrow W_t = 0.1\gamma B^3 N_\gamma$

Circular plate : Approximation is by finding a square of same area of the circle.

The equivalent side of the square is $\sqrt{\frac{\pi}{4}}B$,

$$\therefore W_t = 0.1\gamma\left(\sqrt{\frac{\pi}{4}}B\right)^2 B N_\gamma = 0.025\pi\gamma B^3 N_\gamma$$

8.2 – Plate Load Test

<u>Back-calculation of soil E (Young's modulus) value</u>

There are established coefficients I by the continuum theory for calculation of settlement of soil by a rectangular footing as $S = \frac{pB}{E}(1 - v^2)I$

I depends on the L/B ratio of the rectangular plate. From Poulos & Davis "Elastic Solutions for Soil and Rock Mechanics" $I = 1/1.13$

For circular plate $S = \frac{pB}{E}(1 - v^2)\frac{\pi}{2}$ (from Equation 7.8 of Poulos & Davis "Elastic Solutions for Soil and Rock Mechanics")

These formulae are listed in the 2017 Code.

So by the measured S, E can be back-calculated. And E will be used to verify against design values used in settlement calculations.

7.4 Rectangle on Semi-Infinite Mass

In all cases below, the rectangle is smooth.

7.4.1 SYMMETRICAL VERTICAL LOADING.

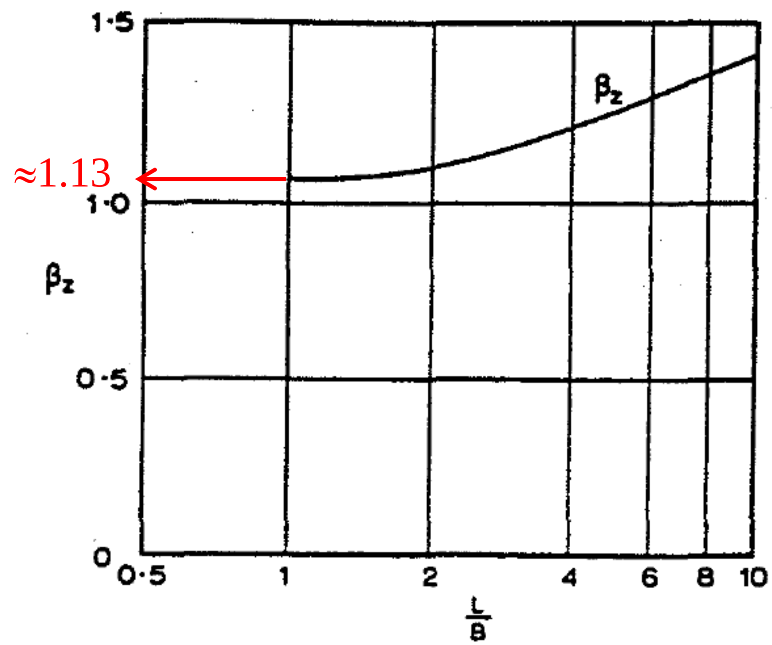
The following approximate solution for the vertical displacement ρ_z is quoted by Whitman and Richart (1967):

$$\rho_z = \frac{P(1-\nu^2)}{\beta_z \sqrt{BL}\, E} \quad \dots (7.17)$$

where P = total vertical load

B, L = rectangle dimensions

β_z = factor dependent on L/B and plotted in Fig.7.8.



IG.7.8 Coefficient β_z for rigid rectangle (Whitman and Richart, 1967).

8.6 – Sonic Logging

- Comparing with the 2004 Code, the last sentence in the first paragraph of the 2004 Code reading "The test should be carried out by a HOKLAS accredited laboratory" is deleted in the 2017 Code. However, it was declared in 8.1 only SPTN and core-drilling are exempted from HOKLAS accredited laboratory. But HOKLAS accredited laboratory should be required for the sonic logging test as BD has stated the requirement added in their approval appendix.

8.7 – Sonic echo tests

- The sentence "The test should be carried out by a HOKLAS accredited laboratory" originally at the end of the first paragraph of the 2004 Code is removed in the 2017 Code. However, by 8.1 which states that exemption from HOKLAS accredited laboratory applies only to SPTN and core-drilling, HOKLAS accredited laboratory is required for sonic echo tests.

8.9 – Dynamic load test for driven piles

- The following changes are identified in the 2017 Code as compared with the 2004 Code:

1. Title revised with the words “for driven piles” added;
2. Though the 1st and 2nd paragraphs have been rewritten, but effectively the same contents;
3. An item (c) as a purpose of the test “to differentiate piles with lower pile capacities within a large group of piles” is added.

8.10 – Dynamic load test for driven piles

- The following changes are identified :

1. Title revised from “Tension Test” to “Tension Loading Test”;
2. The requirement of keeping reaction pile be the larger of 3 pile dia. and 2m remains but “correction of pile interactions be made if the requirement cannot be met”;
3. Test procedures added, essentially same as compression tests with load reversed;
4. The requirement of ignoring grout but including casing for the calculation of elastic extension of mini-pile and socketed piles added;
5. The use of “reaction pads” in lieu of “reaction piles” added.

Estimation of Interactions of Piles in Close Proximity under Vertical Loads by Randolph's Approach

HV.1 Introduction

A pile under vertical load will settle and "drag" the soil around it downwards and thus will create further settlement on adjacent piles. So the settlement of a pile is that due to its own and the effects from others. The phenomenon is termed as "pile interactions".

The mechanism is difficult to quantify even under the elastic theory by which the soil is idealized as an elastic continuum. Poulos & Davis (1980) has developed an approach based on Mindlin's Equations originated from effects due to point loads in an elastic continuum. However, the approach involves lengthy and tedious mathematical manipulations. Randolph (1977), nevertheless, has developed a much simplified approach which has been popularly used by designers and researchers.

HV.2 Description of Randolph's Approach

Basically, Randolph assumes the soil deforms by shear due to the shear stress on the pile shaft and that the effects on soil will be considered negligible beyond a distance r_m (defined below) for determination of pile settlement due to soil friction. The following Figure HV-1 and (Eqn HV-1) are extracted from GEO Publication 1/2006 (Figure 6.26) which explains the assumption. It should be noted that the formula is based on linearly varying shear modulus of soil with depth.

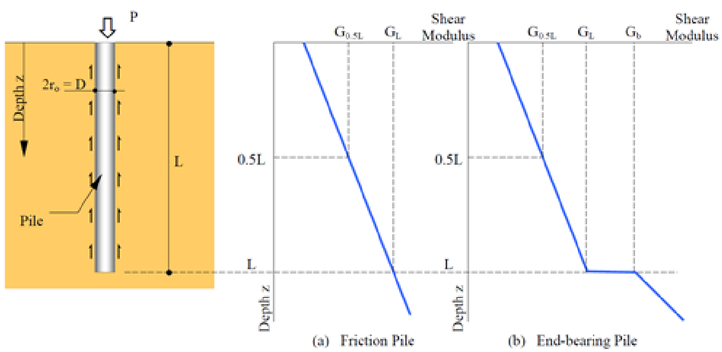


Figure HV-1 – Extract from GEO 1/2006 (Figure 6.26) to Explain Randolph's Approach in Settlement Determination

$$\frac{P}{\delta_t r_o G_L} = \frac{\dfrac{4\eta}{(1-\nu_s)\xi} + \dfrac{2\pi\rho}{\zeta}\dfrac{\tanh(\mu L)}{\mu L}\dfrac{L}{r_o}}{1 + \dfrac{4\eta}{\pi\lambda(1-\nu_s)\xi}\dfrac{\tanh(\mu L)}{\mu L}\dfrac{L}{r_o}} \qquad \text{(Eqn HV-1)}$$

where P is the load applied at the pile head
δ_t is the settlement of the pile head
r_o as the pile radius at the pile shaft
r_b as the radius of the underream for underreamed pile
L as the length of embedment of the pile
$\eta = r_b/r_o$ (ratio of underream for underreamed pile)
G_L as the shear modulus of soil at the pile tip
$G_{0.5L}$ as the shear modulus of soil at mid-depth
G_b as the shear modulus of soil under the pile tip
$\rho = G_{0.5L}/G_L$ (variation of soil modulus with depth)
$\xi = G_L/G_b$
E_p Young's modulus of Pile
$\lambda = E_p/G_L$ (pile-soil stiffness ratio)
ν_s as the Poisson's ratio of the soil
$r_m = [0.25 + (2.5\rho(1-\nu) - 0.25)\xi]L$
$\zeta = \ln(r_m/r_o)$ (measure of radius of influence of pile)
$\mu L = \sqrt{2/\zeta\lambda}\,(L/r_o)$ (measure of pile compressibility)

Re-writing (Eqn HV-1), we may list

$$\delta_t = \left(\frac{1 + \dfrac{8\eta}{\pi\lambda(1-\nu_s)\xi}\dfrac{2\tanh(\mu L)}{\mu L}\dfrac{L}{r_o}}{\dfrac{4\eta}{(1-\nu_s)\xi} + \dfrac{2\pi\rho}{\zeta}\dfrac{\tanh(\mu L)}{\mu L}\dfrac{L}{r_o}}\right)\frac{P}{r_o G_L} \qquad \text{(Eqn HV-2)}$$

Again by Randolph's approach, the settlement at the soil surface at distance S_{pi} away is

$$\delta_i = \delta_t \frac{\ln(r_m/S_{pi})}{\zeta} \qquad \text{(Eqn HV-3)}$$

where $\zeta = \ln(r_m/r_o)$ as defined above.

By the elastic theory, this settlement can be imposed onto a pile at the same distance from the original pile. The phenomenon is explained by Figure HV-2 which is an extract of Figure 7.9 of GEO Publication 1/2006.

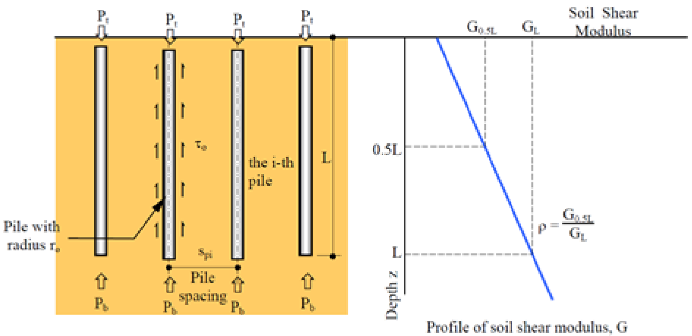


Figure HV-2 – Extract from GEO 1/2006 (Figure 7.9) to Explain Randolph's Approach in Pile Interaction of Settlement

So for the application to piles undergoing tension test, the settlement due to a test pile carrying a compression load created onto the test pile will be calculated which should be deducted from the up-rise of the pile.

HV.3 Worked Example HV-1

Consider a socketed H-pile of cross section undergoing tension load test in soil as indicated in Figure HV-3.

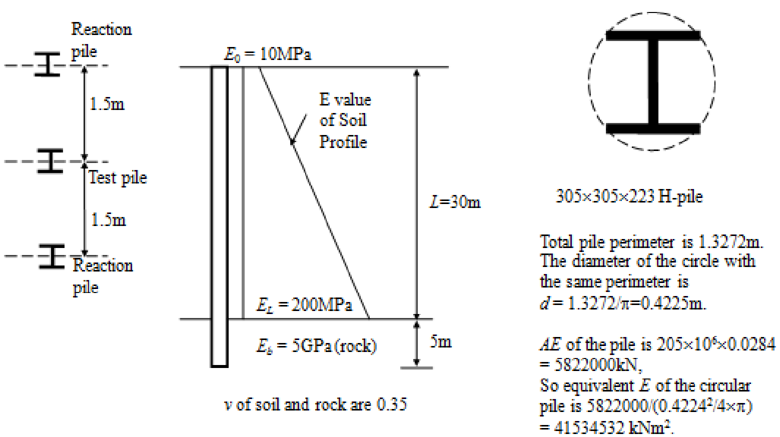


Figure HV-3 – Worked Example HV-1 for a Tension Loading Test

The maximum test tension load on the socketed pile is 6100kN so that each reaction pile carries 3050kN compression. The followings are calculated for estimation of settlement of the reaction pile by (Eqn HV-1) :

$$\eta = d_b/d = 1.0$$

$$E_0 = 10000\text{ kPa} \Rightarrow G_0 = \frac{E_0}{2(1+\nu)} = \frac{10000}{2(1+0.35)} = 3703.7\text{ kPa}$$

$$E_L = 200000\text{ kPa} \Rightarrow G_L = \frac{E_L}{2(1+\nu)} = \frac{200000}{2(1+0.35)} = 74074.07\text{ kPa}$$

$$E_b = 5000000\text{ kPa} \Rightarrow G_b = \frac{E_b}{2(1+\nu)} = \frac{5000000}{2(1+0.35)} = 1851851.85\text{ kPa}$$

$$\xi = G_L/G_b = 0.04 \quad \text{(ratio of end-bearing for end-bearing piles)}$$

$$\rho = \overline{G}/G_L = (3703.7 + 74074.07)/2/74074.07 = 0.525$$

$$\lambda = E_p/G_L = 41534532/74074.07 = 560.716$$

$$r_m = [0.25 + (2.5\rho(1-\nu) - 0.25)\xi]L$$

$$= [0.25 + (2.5\times 0.525(1-0.35) - 0.25)\times 0.04]30 = 8.224\text{ m}$$

$$\zeta = \ln(2r_m/d) = \ln(2\times 8.224/0.4225) = 3.6618$$

$$\mu L = 2\sqrt{2/\zeta\lambda}\,(L/d) = 2\sqrt{2/(3.6618\times 560.716)}(30/0.4225) = 4.4326$$

$$\text{So}\quad \delta_t = \left(\frac{1 + \dfrac{8\eta}{\pi\lambda(1-\nu)\xi}\dfrac{\tanh(\mu L)}{\mu L}\dfrac{L}{d}}{\dfrac{2\eta}{(1-\nu)\xi} + \dfrac{2\pi\rho}{\zeta}\dfrac{\tanh(\mu L)}{\mu L}\dfrac{L}{d}}\right)\frac{P}{dG_L} = 0.00405\text{ m}$$

By (Eqn HV-3), the settlement induced on the test pile at $S_{pi} = 1.5$ m away is

$$\delta_i = \delta_t\frac{\ln(r_m/S_{pi})}{\zeta} = 0.00405\times\frac{\ln(8.224/1.5)}{3.6618} = 0.00405\times 0.4647 = 0.00188\text{ m}$$

As there are 2 reaction piles, an adjustment by a downward settlement of $1.88\times 2 = 3.76$ mm should be made to the test pile.

(Note : in reality the variation of the G values of soil is not linear. An approximation can be made by fitting a best line with errors through the G values measured at different levels by the mathematical technique of linear regression.)

8.11 – Lateral load test

- Comparing with the 2004 Code, a standard loading procedures for lateral load test is added in the 2017 Code as Table 8.1.
- The test takes approximately 4 hours which is significantly less than vertical load tests.

The End, Thank You