Common "Mistakes" in Designing of Piles Subjected to Negative Skin Friction

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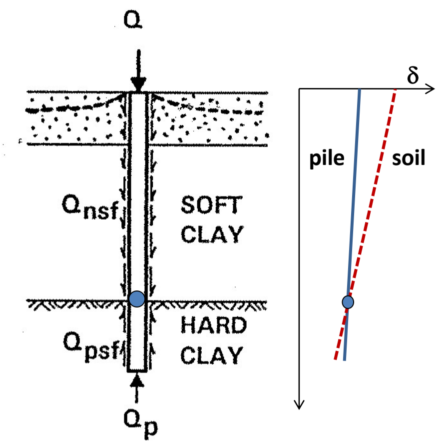
Common "Mistakes" in Designing of Piles Subjected to Negative Skin Friction

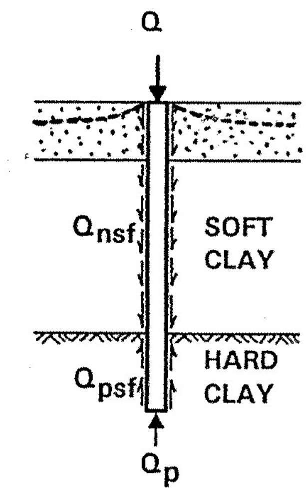
- ✓ **Mistakes**
- ✓ **Misconceptions**
- ✓ **Misunderstandings**
- ✓ **Misinterpretation of CP4:2003**
- ✓ **Controversial issues**
- ✓ **Clarifications**
- ✓ **Proposals**

Known Facts about Negative Skin Friction



1. Q_{NSF} develops when the soil settles more than the pile.
2. Q_{PSF} develops when the pile settles more than the soil.
3. There exists a neutral point which divides Q_{NSF} and Q_{PSF}.
4. It only takes a few mm of relative movement to fully mobilise Q_{NSF} and Q_{PSF}.

Issue # 1 -- Drag Load vs Downdrag



Negative Skin Friction
Shear stress on pile due to downward soil movement relative to pile

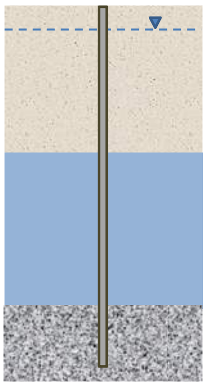
Drag Load
Force on pile caused by NSF

Downdrag
Settlement of pile due to drag load

Issue #2

The soft clay layer is over-consolidated or fully consolidated under the existing fill.

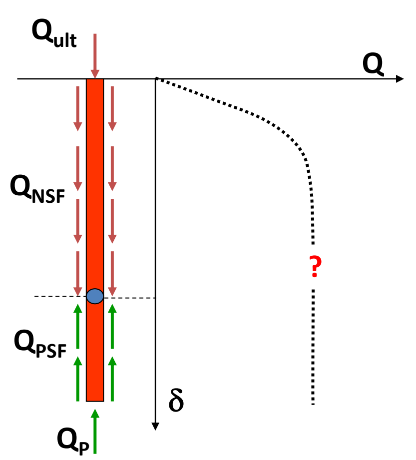
Therefore, NSF is not an issue.



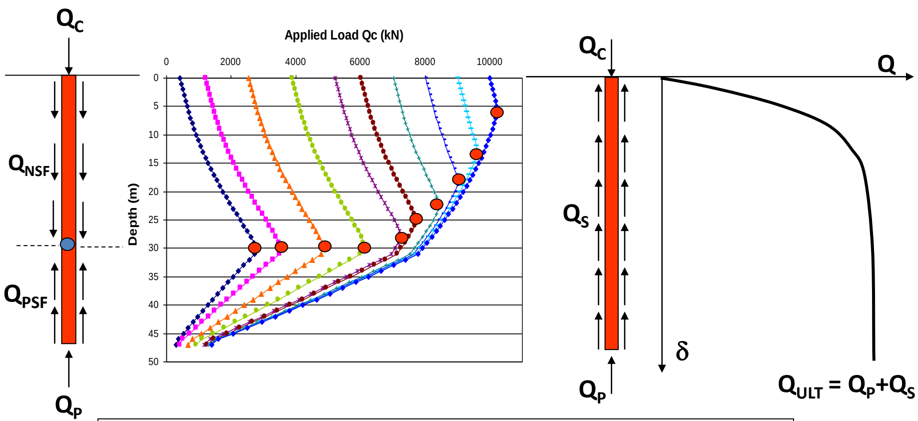
1. **Will there be settlement under future loading?**
2. **Do you have control over future developments?**

Issue #3

What happen when $Q + Q_{nsf} > Q_p + Q_{psf}$?



What will happen when ($Q_C + \eta Q_{NSF}$ > $Q_P + Q_{PSF}$) ?

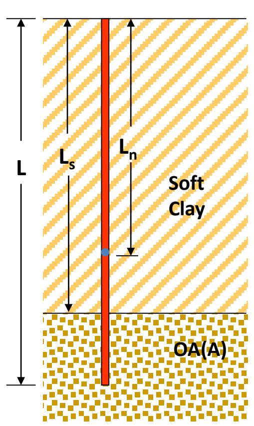


1. **No plunging failure until ($Q_C + \eta Q_{NSF}$) = ($Q_P + Q_S$).**
2. **NSF is a settlement problem.**
3. **Ultimate geotechnical capacity = ($Q_P + Q_S$).**

Issue #4

Soft Toe → $Q_p = 0$ → Friction Pile

Therefore $L_n = 0.6L_s$



Neutral Point Location

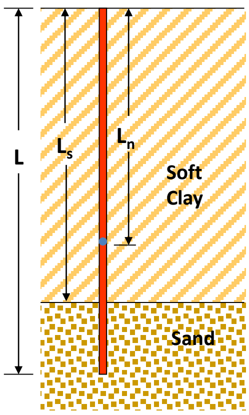
CP4:2003
Friction Pile: $L_n = 0.6L_s$
End Bearing Pile: $L_n = 1.0L_s$

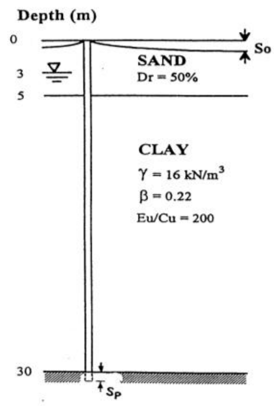
L_s = thickness of consolidating soil
L_n = Distance from cut-off level to n.p.

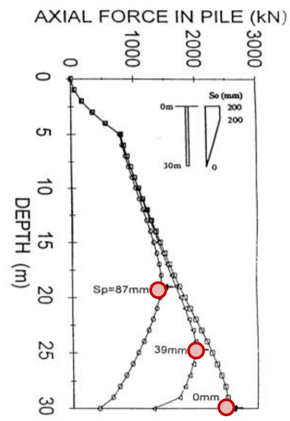
CP4:2003
Friction Pile: $L_n = 0.6L_s$
End Bearing Pile: $L_n = 1.0L_s$

Neutral Point Location

L_s = thickness of consolidating soil
L_n = Distance from cut-off level to n.p.







Neutral point of pile end bearing in clay

Friction Pile Floating in Soft Clay

L_n = 0.6 L

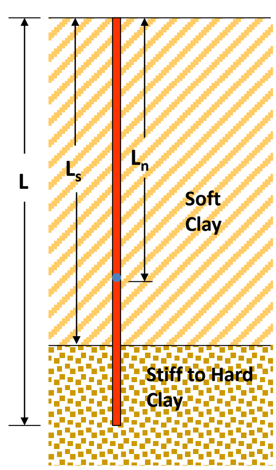
Pile End Bearing in Stiff to Very Stiff Clay

L_n = 0.7 to 0.9 L_s

Pile End Bearing in Hard Clay

L_n = 0.9 to 1.0 L_s

where L = total pile length
L_n = location of neutral point below pile top
L_s = distance from pile top to stiff stratum



End Bearing Layer : Hard Rock

$L_n = 1.0\ L_s$

where L_n location of neutral point below surface

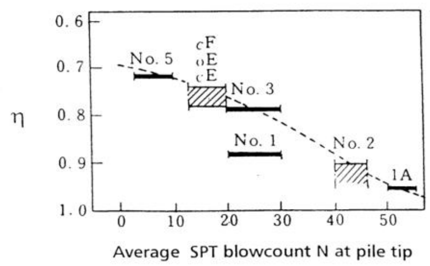
L_s thickness of compressible stratum

End Bearing Layer : Dense Sand $N \geq 50$

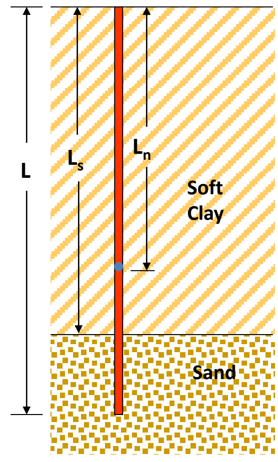
$L_n = 0.95\ L_s$

End Bearing Layer : Sand with $N \leq 50$

$L_n = \eta\ L_s$

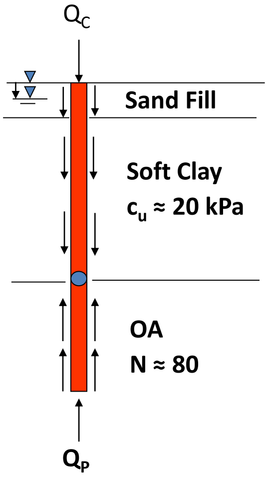


Neutral point of driven pile end bearing in sand & rock



Issue #5

NSF should be computed using effective stress method only.



β-method

$f_s = \beta\ \sigma'_v$

Total stress method can also be used for clayey soils.

α- method

$f_s = \alpha\ c_u$

Determination of Negative Skin Friction in Clay

CP4 : 2003 (Singapore)
Use either α or β method

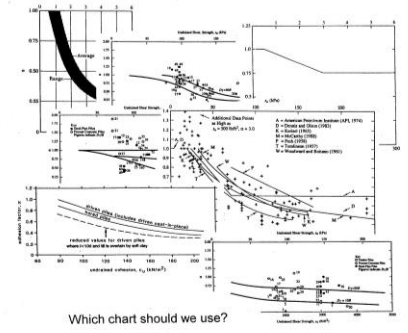
LTA
Use effective stress method

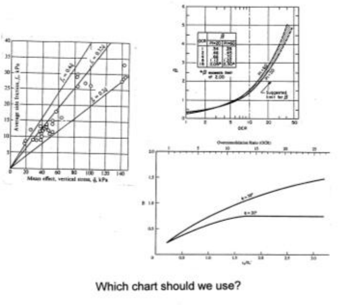
α- method

$$f_s = \alpha c_u$$

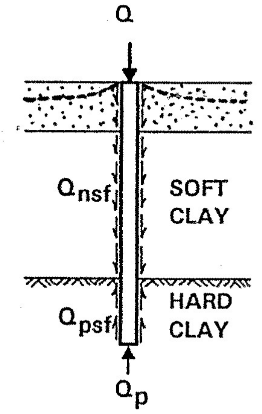
β-method

$$f_s = \beta \sigma'_v$$





Determination of NSF in Clay using Total Stress Method



Clay (Fleming et al., 1987)

$$f_s = \alpha c_{uf}$$

where $\alpha = 0.5 / (c_{uf} / \sigma_{vf}')^{0.5}$ for $c_{uf} / \sigma_{vf}' \leq 1$

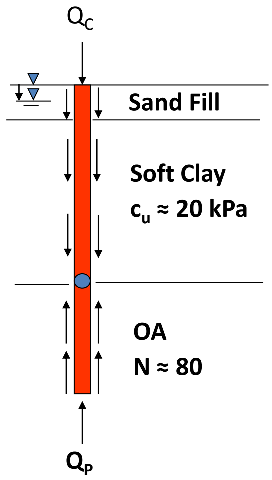
$\alpha = 0.5 / (c_{uf} / \sigma_{vf}')^{0.25}$ for $c_{uf} / \sigma_{vf}' > 1$

c_{uf} = final undrained shear strength

σ_{vf}' = final effective stress

Issue #6

NSF can be computed using c_u values from SI report.

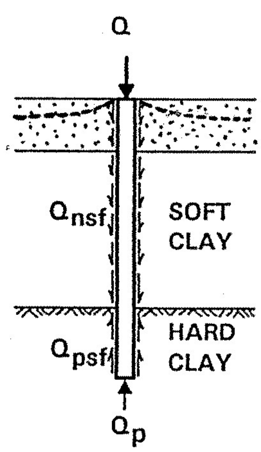


α- method

$f_s = \alpha\ c_u$

1. Use final c_u.
2. May not be appropriate to use current c_u. It depends on the state of consolidation and the long term effective stress.

Determination of NSF in Clay using Effective Stress Method



Clay (Wong and Teh, 1995)

$f_s = \beta\ \sigma_{vf}'$

where $\beta = (c_u / \sigma_v')_{NC}\ OCR_f^{\ 0.5}$

$(c_u / \sigma_v')_{NC} \sim 0.22$ for many clays

$OCR_f = \sigma_p' / \sigma_{vf}'$

σ_p' = preconsolidation pressure

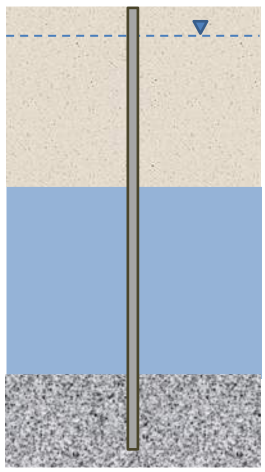
σ_{vf}' = final effective stress

For conservative estimation of Q_{nsf}, higher unit weights and lower ground water table should be used.

For conservative estimation of Q_{psf}, the opposite trend should be used.

Issue #7

How to determine NSF in Sand?



<u>β-method</u>

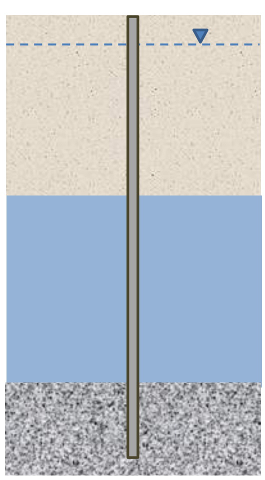
$f_s = \beta\ \sigma'_v$

$\beta = K_s \tan \delta$

Use one of following:

1. $\beta = 0.35$?
2. $f_s = N$?
3. $f_s = 2N$ to $5N$?
4. $f_s = q_c / 200$?
5. $f_s = q_c / 400$?

Determination of Negative Skin Friction in Sand



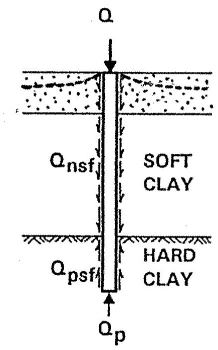
1. $\beta = 0.35$?

   1. May be appropriate for sand above water where OCR>>1.
   2. May be too conservative below water where OCR≈ 1.

2. $f_s = N$?
3. $f_s = 2N$ to $5N$?
4. $f_s = q_c / 200$?
5. $f_s = q_c / 400$?

   1. Conservative for PSF.
   2. Unconservative for NSF.

Determination of Negative Skin Friction in Sand



β-method

$$f_s = \beta \, \sigma'_v = K_s \tan \delta$$

K_s = lateral earth pressure coefficient

δ = interface friction angle in degrees

σ_v' = average effective vertical stress along the pile shaft

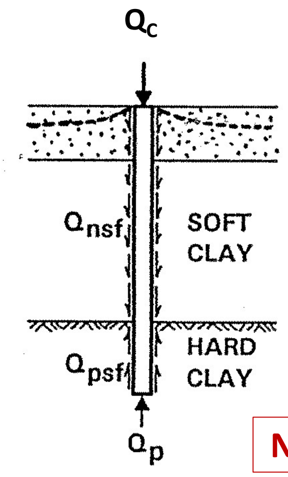
$K_o \sim (1 - \sin \phi') \, OCR^{0.5}$

	K_s / K_o	
	Loose	Dense
Large displacement piles	1	2
Small displacement piles	0.75	1.25
Jetted piles	0.5	0.67
Bored piles	0.67	1

	δ / ϕ'	
	Loose	Dense
sand / rough concrete	1	1
sand / smooth concrete	0.8	1
sand / rough steel	0.7	0.9
sand / smooth steel	0.5	0.7
sand / timber	0.8	0.9

Issue #8

For piles subjected to NSF we only need to check the following according to CP4:2003.



Structural

$Q_C + \eta \, Q_{NSF} \leq Q_{ALL,ST} = f_{cu} A_c / F_S$

where $F_s = 4$ for concrete

Geotechnical

$Q_C + \eta \, Q_{NSF} \leq (Q_P + Q_{PSF}) / F_s$

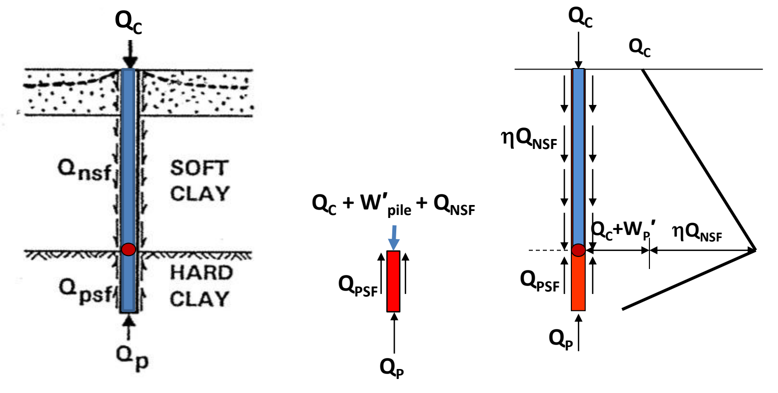
where $F_s = 2.0$ or 2.5

$\eta = 0.67$ or 1

Need to check : $Q_C = (Q_P + Q_S) / Fs$

Issue #9

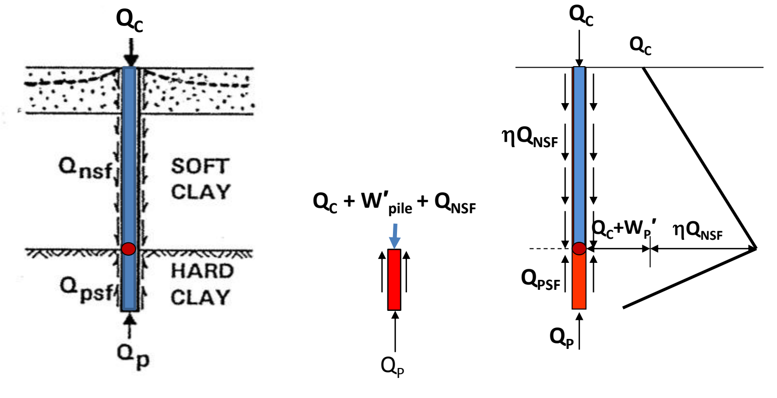
Is it necessary to consider self-weight of pile?



For 300x300 RC pile with L_n=20m, W'_{pile} ≈ 2.5 t.

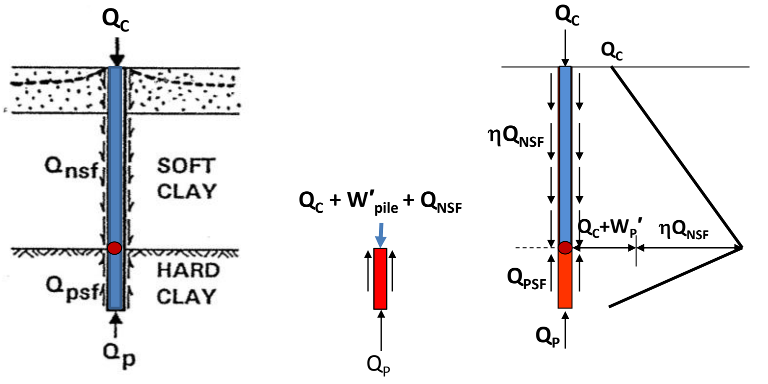
For 1.2m diameter bored pile with L_n=30m, W'_{pile} ≈ 50 t.

Issue #9



Theoretically W'_{pile} should be included in the design.

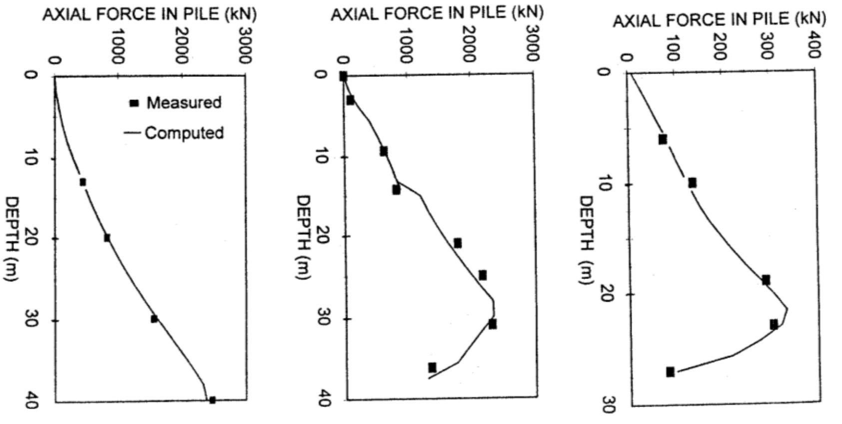
Issue #9



Practically, it may not be necessary to include W'_{pile}. It depends on how we compute Q_{NSF}. A conservative Q_{NSF} can easily covered W'_{pile}.

Evaluation of Soil Parameters for Negative Skin Friction

(Wong and Teh, 1995)



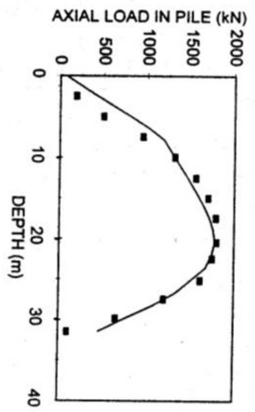
Norway (Bjerrum et al., 1969)

Tokyo Bay (Fukuya et al., 1982)

Bangkok (Indraratna et al., 1992)

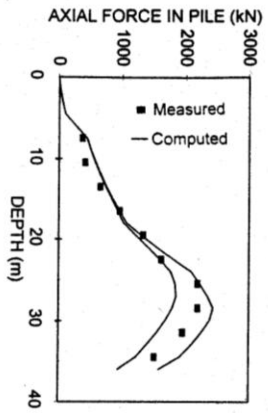
Evaluation of Soil Parameters for Negative Skin Friction

(Wong and Teh, 1995)

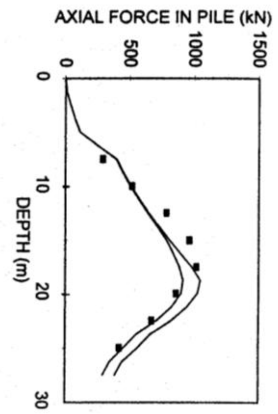


Melborne

(Walker & Darvall, 1969)



End bearing pile

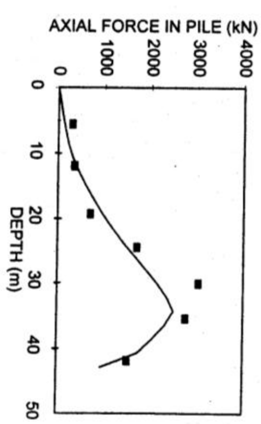


Floating friction pile

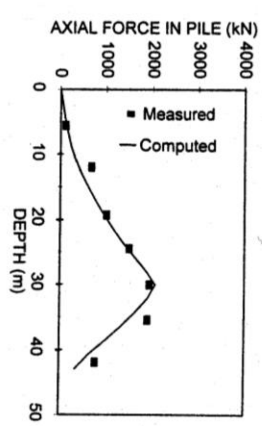
Japan (Nishi & Esashi, 1982)

Evaluation of Soil Parameters for Negative Skin Friction

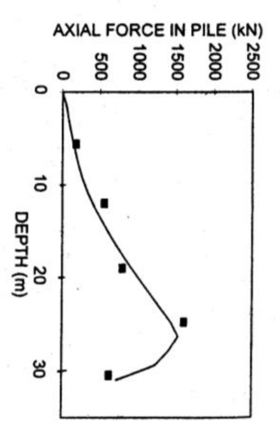
(Wong and Teh, 1995)



Closed-end pipe pile (End bearing)

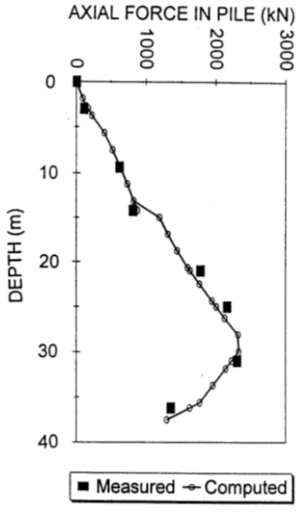


open-end pipe pile (End bearing)



Closed-end pipe pile (Floating friction)

Tokyo (Endo et al., 1969)



Tokyo Bay, Japan
(Fukuya et al., 1982)

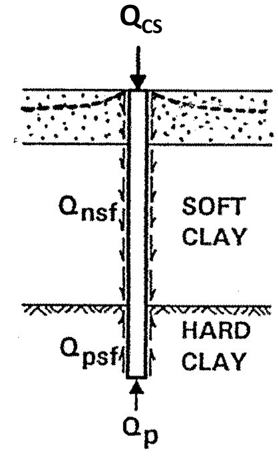
Issue #9

The above comparisons show that the current method of NSF computation has implicitly included the self-weight of pile.

No need to include W'_{pile} !

Issue #10

Bored pile is limited to Grade 30 concrete.



Structural

$$Q_{CS} + \eta\, Q_{NSF} \le Q_{ALL,ST} = f_{cu} A_c / F_S$$

where $F_s = 4$ for concrete

CP4 allows only grade 30 concrete?

$$f_{cu} / F_s = 7.5 \text{ MPa}$$

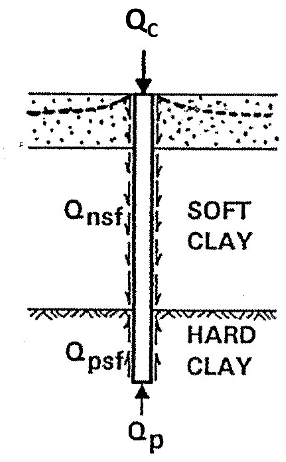
For piles subjected to NSF ⇨

Bored Piles
Use Grade 40 or higher

Driven Piles
Use as high as possible

Issue #11

Difficulties with η and Fs



Geotechnical

$Q_C + \eta\, Q_{NSF} \leq (Q_P + Q_{PSF}) / F_s$

where F_s = 2.0 or 2.5

η = 0.67 or 1

What is "η" ?

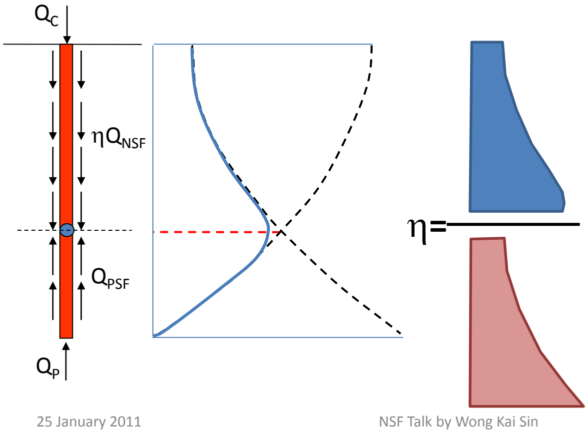
When to use η = 0.67 or 1?

When to use F_s = 2.0 or 2.5?

Can we use F_s = 1.5 if Q_p =0?

CP4:2003 - Degree of Mobilization "η"

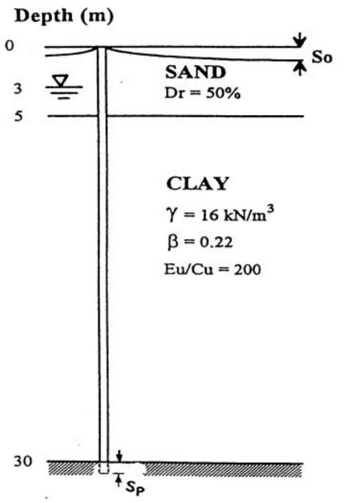
"The negative unit friction along the pile section above the neutral plane may vary between the fully mobilization value on the top and a small value close to the neutral plane."

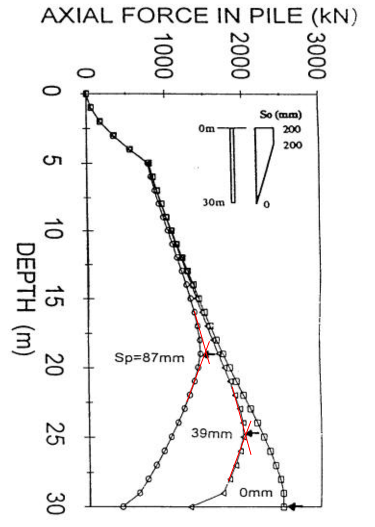


η= 1.0
for low capacity piles in highly compressible clay

η = 0.67
for all other cases

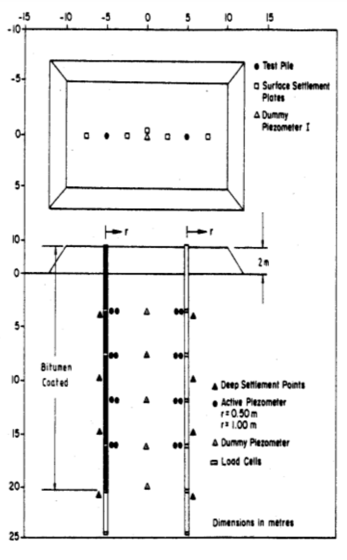
Effect of Bearing Stratum Stiffness on Neutral Point

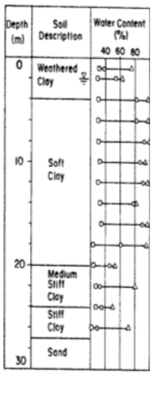


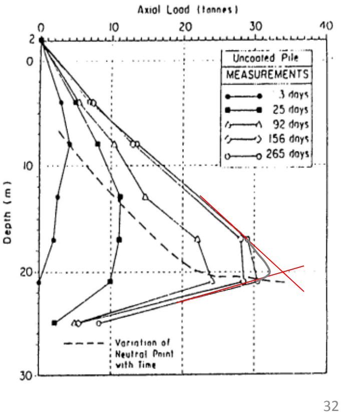


Bangkok, Thailand

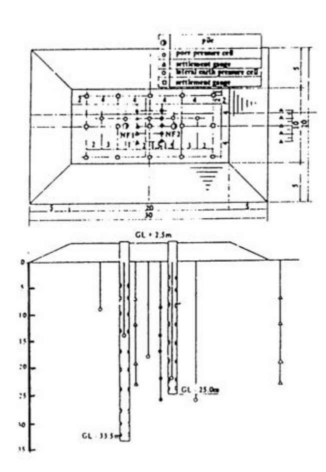
(Indraratna et al., 1992)



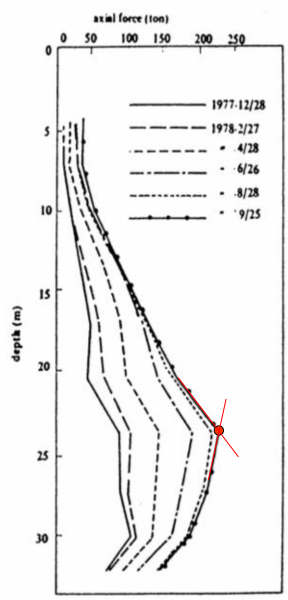




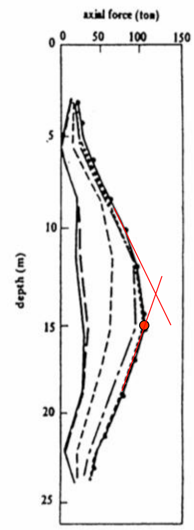
Nishi & Esashi (1982)



End Bearing Pile

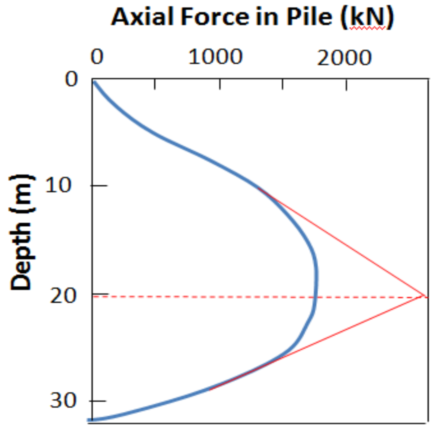


Floating Pile



Melborne, Australia

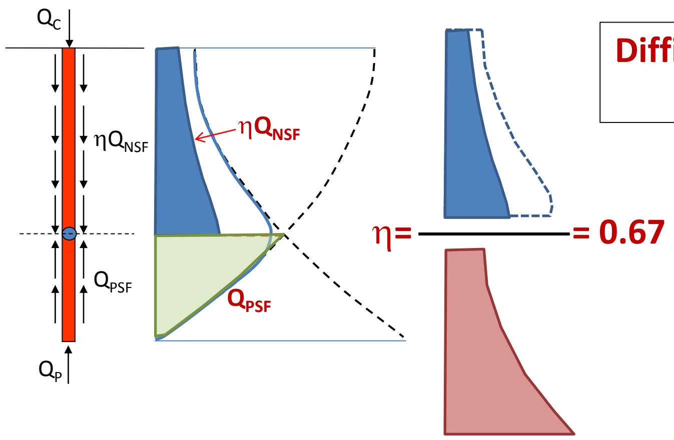
(Walker & Darvall, 1969)



Difficult to justify $\eta = 0.67$!

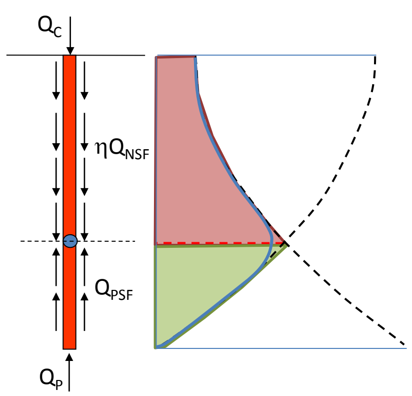
Degree of Mobilization "η"

The negative unit friction along the pile section above the neutral plane may vary between the fully mobilization value on the top and a small value close to the neutral plane.



Difficult to justify η = 0.67 !

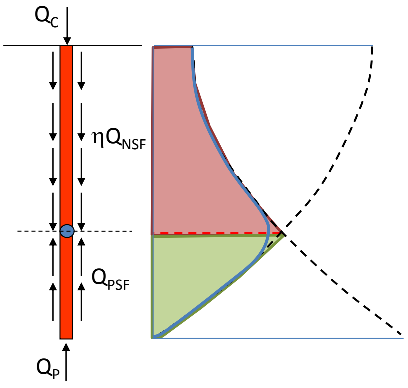
Degree of Mobilization "η"



For single pile or piles in group with large spacing:

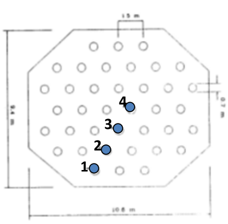
Use η = 1.0

Many LTA tunnels fall into this category!



Degree of Mobilization "η"

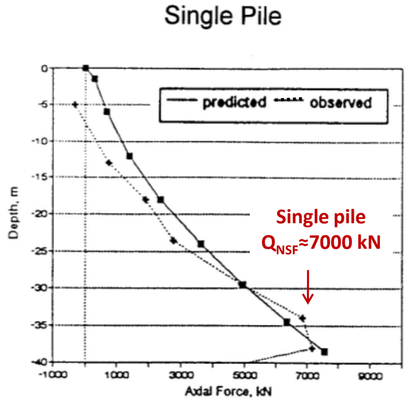
Does that mean we should not use
η =0.67 ?

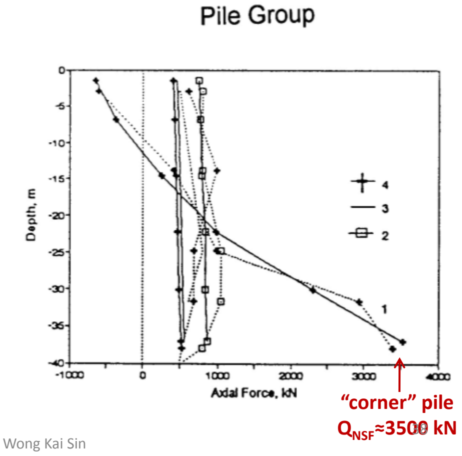


Validation of 3-D Analysis for Pile Group Subjected to Downdrag

(after Jeong and Briaud, 1994)

(After Okabe, 1977)

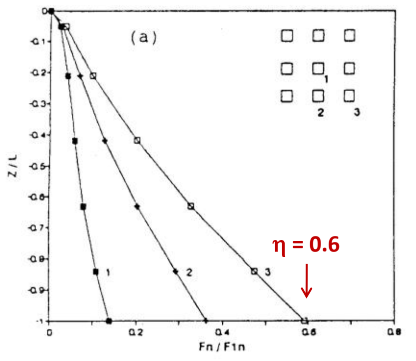




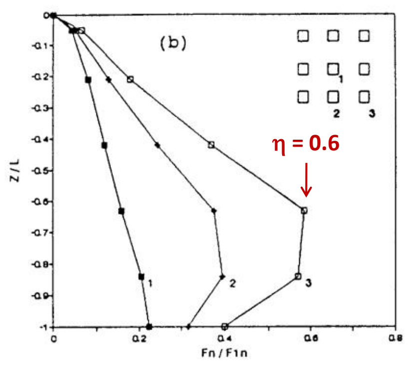
Effect of Bearing Stratum Stiffness

(Jeong & Briaud, 1994)

End Bearing Pile
(s/d = 2.5)

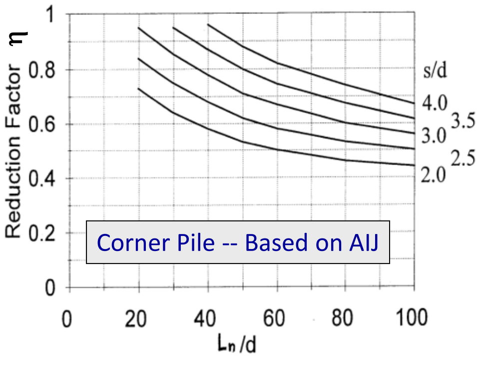


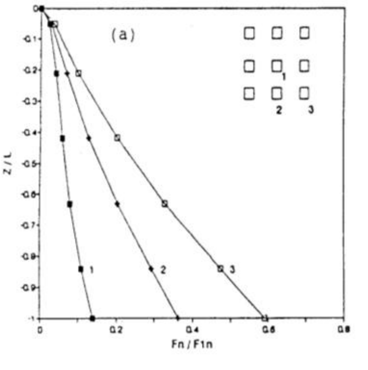
Friction Pile
(s/d = 2.5)



Group Reduction Factor "η"

Downdrag = η Q_{NSF} where η **~ 0.5 to 1**

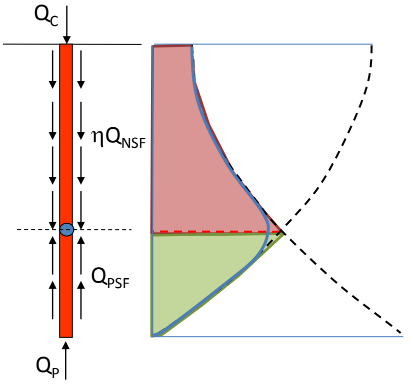




Foundation Code 2004 (Hong Kong)

Use group reduction factor **η** = 0.85

~~Degree of Mobilization~~ "η"
Group Efficiency Factor



For piles in group with small spacing:

Use η = 0.67

Degree of Mobilization "η"
or
Group Efficiency Factor "η"

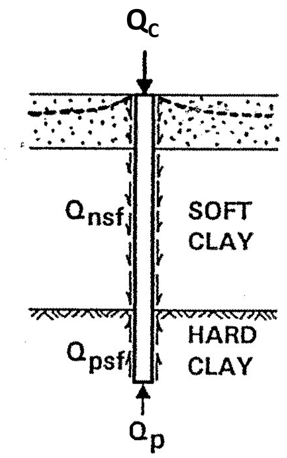
Single Piles or Piles in Group with Large Spacing

η = 1.0

Piles in Group with Small Spacing

η = 0.67

Geotechnical Capacity of Piles subjected to NSF



No NSF

$Q_{ALL} = (Q_P + Q_{PSF}) / 2.5$

or

$Q_{ALL} = Q_P / 3 + Q_S / 1.5$

With NSF

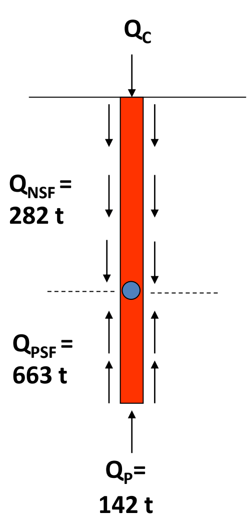
$Q_C + \eta\, Q_{NSF} \leq (Q_P + Q_{PSF}) / (2 \text{ or } 2.5)$

Can we also use:

$Q_C + \eta\, Q_{NSF} \leq Q_P / 3 + Q_S / 1.5$?

1. What are the implications on pile capacity?
2. Is it supported by code?

1m ϕ bored pile
Grade 40 concrete



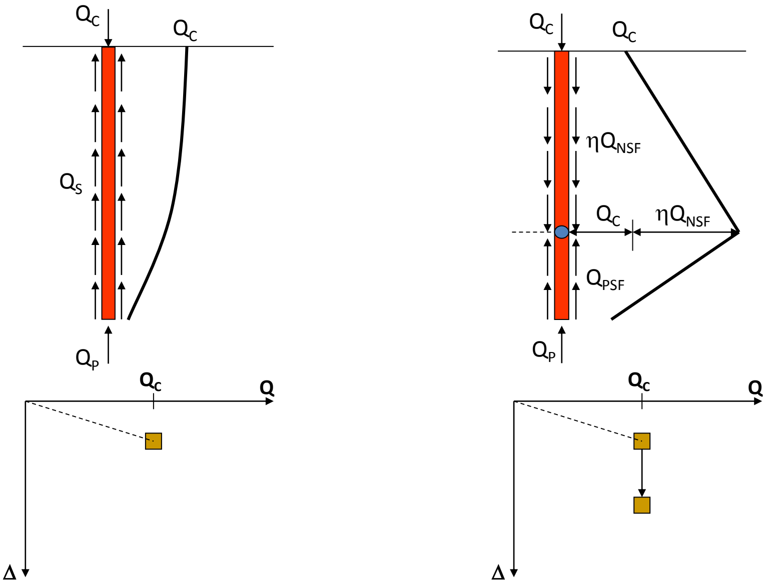
Which factor of safety should we use ?

$Q_C + \eta Q_{NSF} \leq (Q_P + Q_{PSF}) / F_s$

F_S	Q_c (t) η=1.0	Q_c (t) η=0.67
1.5	253	346
2.0	119	212
2.5	39	132

What are the corresponding settlements?

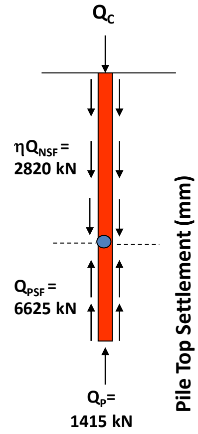
Pile Settlement due to NSF

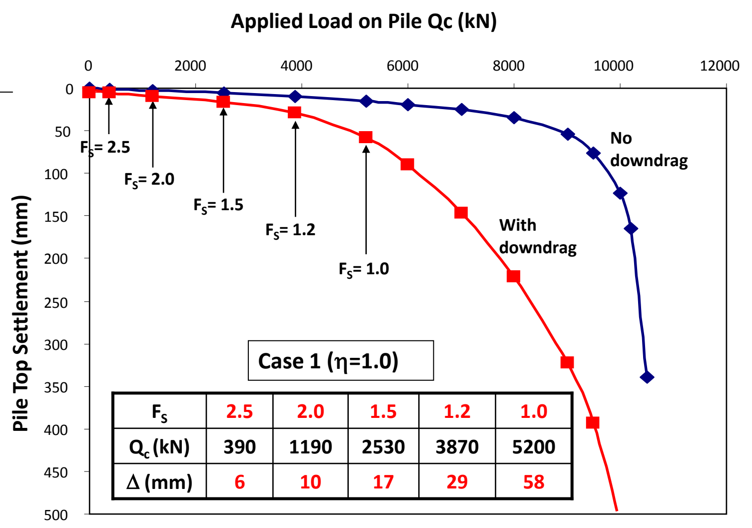


What factor of safety should we use ?

1m ϕ bored pile
Grade 40 concrete

$$Q_C + \eta Q_{NSF} \le (Q_P + Q_{PSF}) / F_s$$



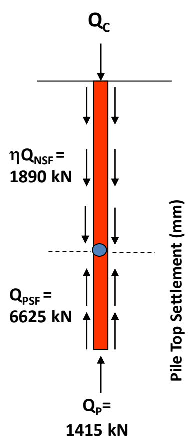


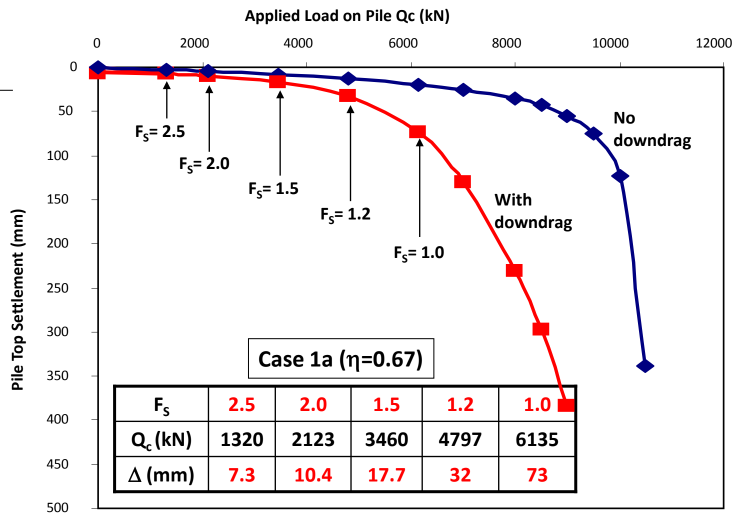
F_s	2.5	2.0	1.5	1.2	1.0
Q_c (kN)	390	1190	2530	3870	5200
Δ (mm)	6	10	17	29	58

What factor of safety should we use ?

1m ϕ bored pile
Grade 40 concrete

$$Q_C + \eta Q_{NSF} \le (Q_P + Q_{PSF}) / F_s$$





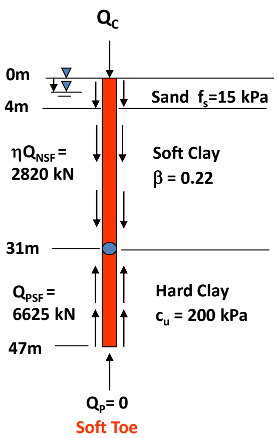
F_S	2.5	2.0	1.5	1.2	1.0
Q_c (kN)	1320	2123	3460	4797	6135
Δ (mm)	7.3	10.4	17.7	32	73

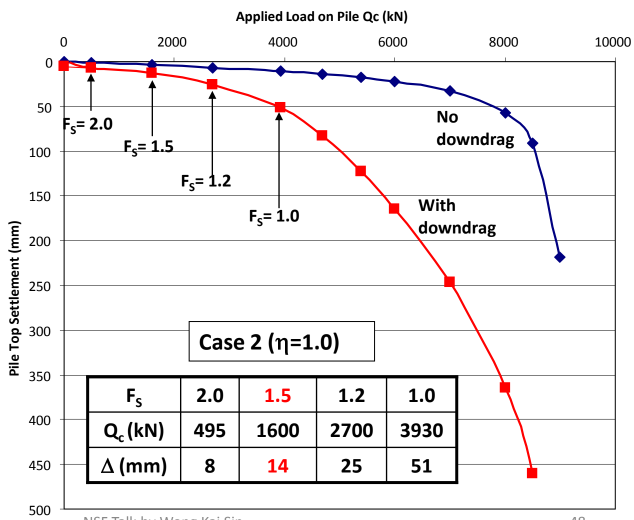
What factor of safety should we use ?

1m ϕ bored pile
Grade 40 concrete

$$Q_C + \eta Q_{NSF} \le (Q_P + Q_{PSF}) / F_s$$

Ground water lowered by 2m.

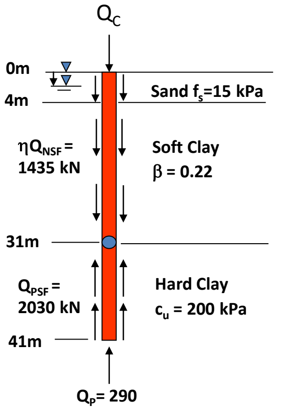




F_S	2.0	1.5	1.2	1.0
Q_c (kN)	495	1600	2700	3930
Δ (mm)	8	14	25	51

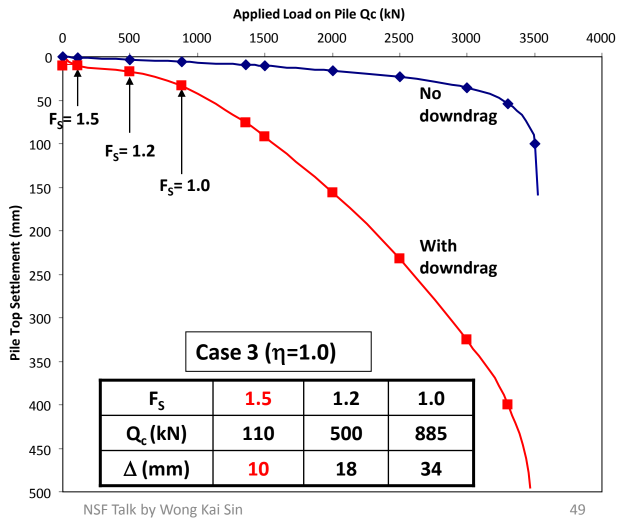
400x400 RC Pile
Grade 50 concrete

Ground water lowered by 2m.



What factor of safety should we use ?

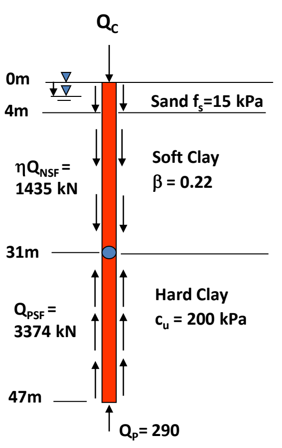
$$Q_C + \eta Q_{NSF} \le (Q_P + Q_{PSF}) / F_S$$



F_S	1.5	1.2	1.0
Q_c (kN)	110	500	885
Δ (mm)	10	18	34

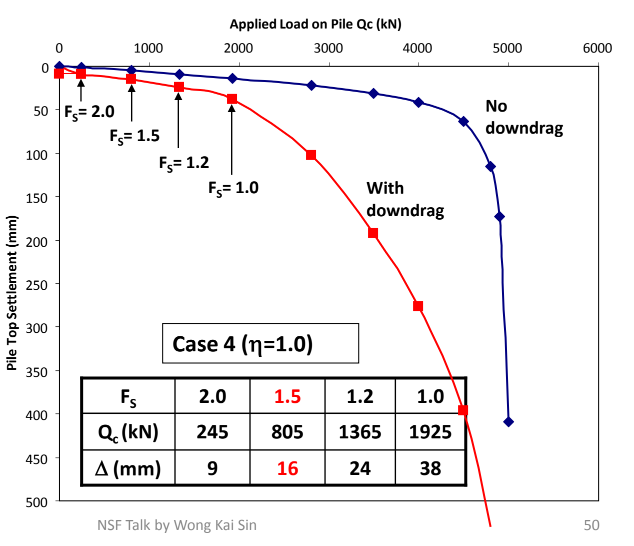
400x400 RC Pile
Grade 50 concrete

Ground water lowered by 2m.



What factor of safety should we use?

$$Q_C + \eta Q_{NSF} \le (Q_P + Q_{PSF}) / F_S$$



F_S	2.0	1.5	1.2	1.0
Q_c (kN)	245	805	1365	1925
Δ (mm)	9	16	24	38

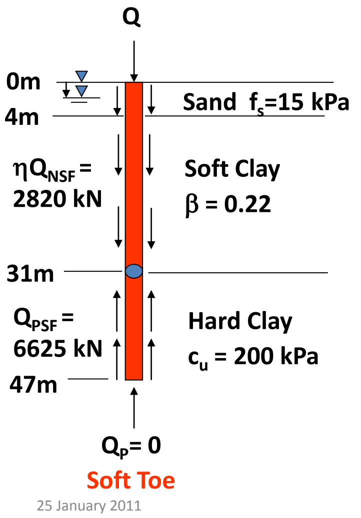
For Serviceability (SLS) Consideration:

Single Pile or Pile Group with Large Spacing

η = 1.0
Fs = 1.5 (non-compliance with CP4)
or
η = 0.67
Fs = 2.0 (compliance with CP4)

1m ϕ bored pile
Grade 40 concrete

Ground water lowered by 2m.



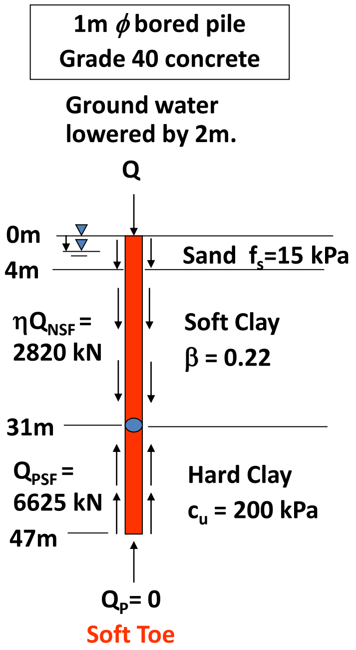
Example illustrating effect of Fs and η on Q_{NSF}

$$Q_C + \eta Q_{NSF} \le (Q_P + Q_{PSF}) / F_s$$

η = 0.67
Fs = 2.0 (compliance with CP4:2003)
Q_C = 1420 kN

η = 1.0
Fs = 1.5 (non-compliance with CP4)
Q_C = 1620 kN

Therefore, η= 0.67 and Fs = 2 yield reasonable results for single piles & piles in group with large spacing.



Example illustrating effect of Fs and η on Q_{NSF}

$$Q_C + \eta Q_{NSF} \leq (Q_P + Q_{PSF}) / F_s$$

η = 0.67
Fs = 2.0 (compliance with CP4:2003)
Q_C = 1420 kN

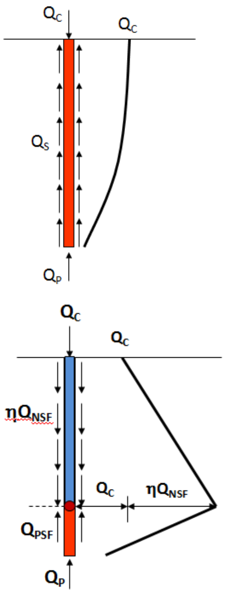
η = 0.67
Fs = 1.5 (non-compliance with CP4)
Q_C = 2530 kN

If η=0.67 and Fs=2 are used for pile group with small spacing, the computed Q_C is conservative. There is plenty of "fat" in the design.

For Serviceability (SLS) Consideration:

Pile Group with Small Spacing

η = 0.67
Fs = 1.5 (non-compliance with CP4)
or
η = 0.67
Fs = 2.0 (compliance with CP4)



For Failure (ULS) Consideration:

Geotechnical

$Q_C \le (Q_P + Q_S)/2.5$

or

$Q_C \le Q_P/3 + Q_S/1.5$

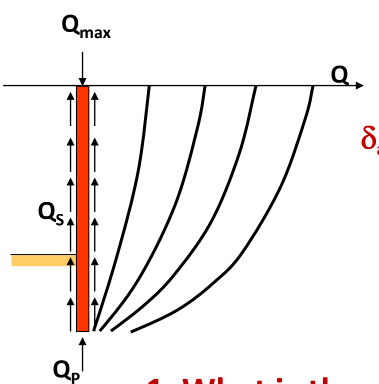
Structural

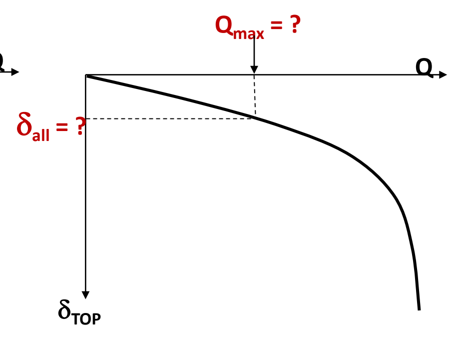
$Q_C + \eta\, Q_{NSF} \le Q_{ALL,ST} = f_{cu} A_c / F_S$

where $F_s = 4$ for concrete

Issue #12

Difficulties with Load Test on Piles subjected to NSF





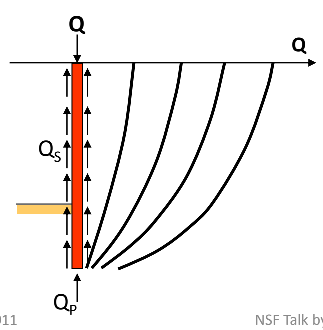
1. What is the maximum test load?
2. What is the allowable settlement?
3. What is the settlement under NSF?

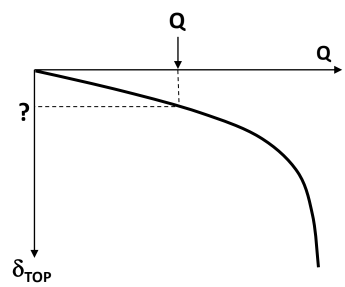
CP4 : 2003 & HDB - Settlement at $(WL + 2Q_{NSF}) \leq 10$ mm

Eurocode EC7 : Maximum test load = $WL + 2Q_{NSF}$
Allowable settlement not specified.

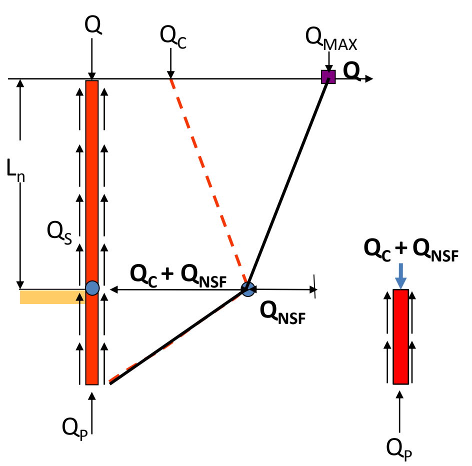
Found. Code 2004 (Hong Kong): Max. test load = $2Q_C + Q_{NSF}$
Allowable settlement not specified.

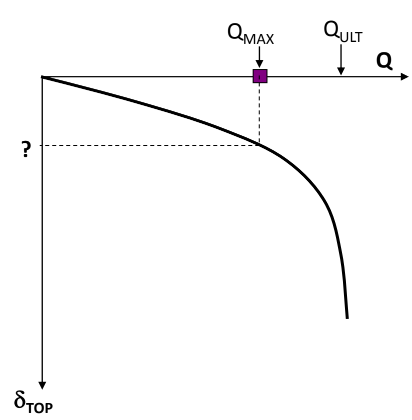
Other - Settlement at $(WL + Q_{NSF}) \leq 10$ mm





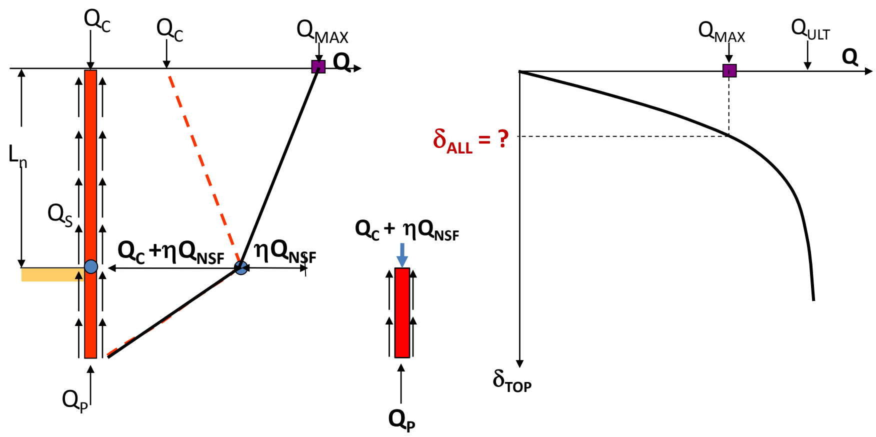
Why Maximum Test Load = $WL + 2Q_{NSF}$?





Working Pile: $Q_{MAX} = WL + 2\eta Q_{NSF}$

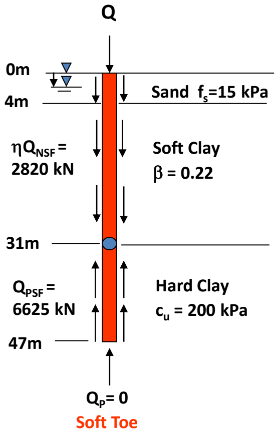
Preliminary Pile: $Q_{MAX} = Q_{ULT} = Q_P + Q_S$



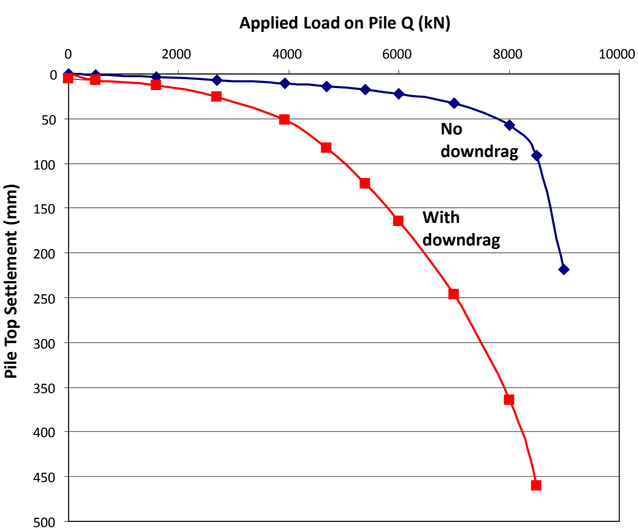
What is the allowable settlement?

1m ϕ bored pile
Grade 40 concrete

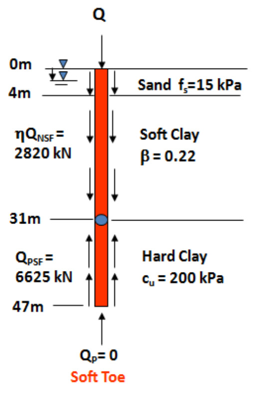
Ground water lowered by 2m.

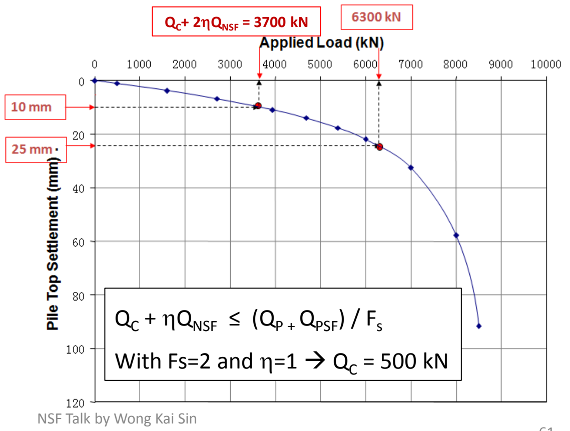


Example illustrating different code requirements

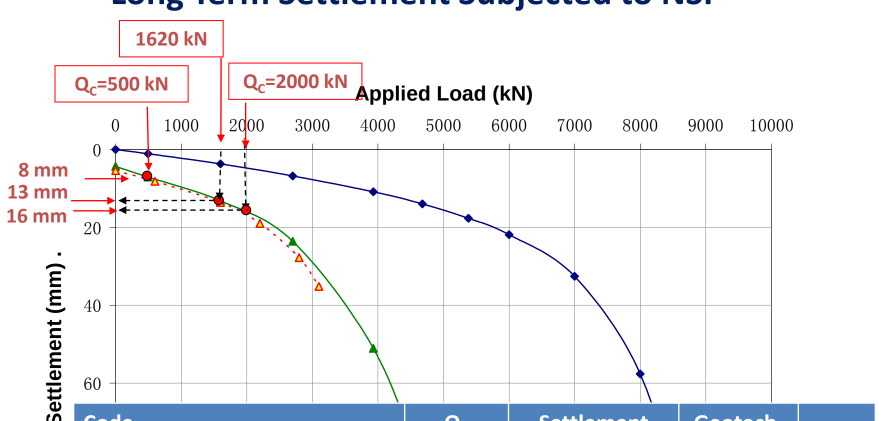


Code (η=1)	Maximum Load	Allowable Settlement (mm)	Allowable Load Qc (kN)
CP4:2003 & HDB	$Q_C + 2Q_{NSF}$	10	-980
Eurocode EC7	$Q_C + 2Q_{NSF}$	25 (assumed)	1620
Fdn. Code of H.K. 2004	$2Q_C + Q_{NSF}$	25 (assumed)	2000



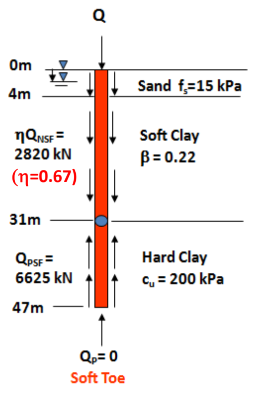


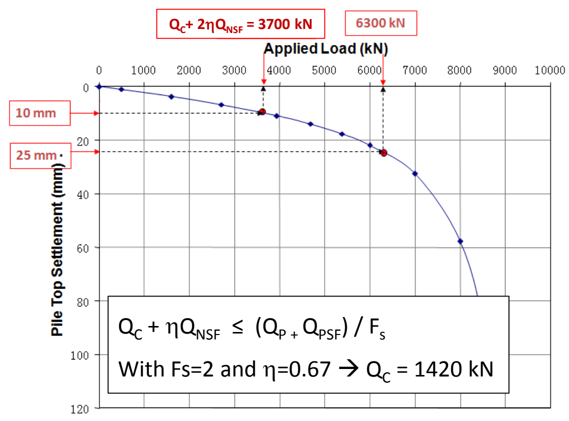
Long Term Settlement Subjected to NSF



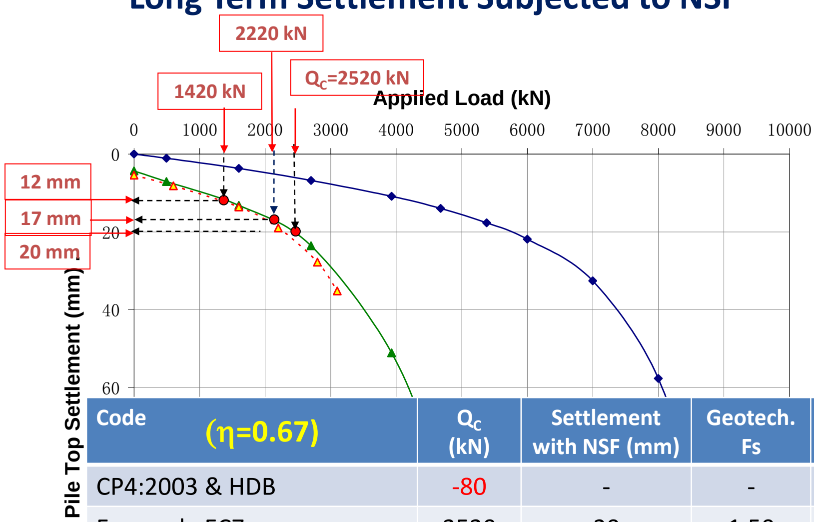
Code (η=1)	Q_C (kN)	Settlement with NSF (mm)	Geotech. Fs	η
CP4:2003 & HDB	-980	-	-	
Eurocode EC7	1620	13	1.50	1.00
Fdn Code of H.K. 2004	2000	16	1.37	1.00
$Q_C + \eta Q_{NSF} \le (Q_P + Q_{PSF}) / F_s$	500	8	2	1.00

Code (η=0.67)	Maximum Load	Allowable Settlement (mm)	Allowable Load Qc (kN)
CP4:2003 & HDB	$Q_C + 2\eta Q_{NSF}$	10	-80
Eurocode EC7	$Q_C + 2\eta Q_{NSF}$	25 (assumed)	2520
Fdn. Code of H.K. 2004	$2Q_C + \eta Q_{NSF}$	25 (assumed)	2200





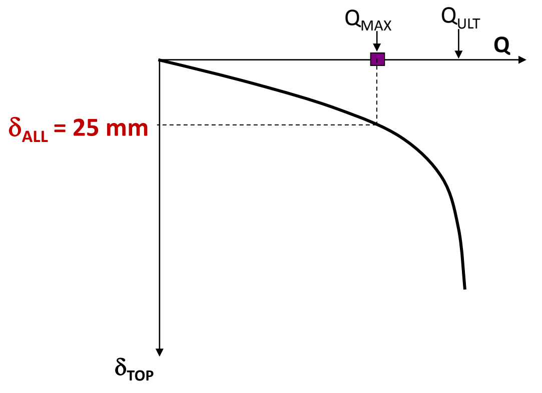
Long Term Settlement Subjected to NSF



Code (η=0.67)	Q_C (kN)	Settlement with NSF (mm)	Geotech. Fs	η
CP4:2003 & HDB	-80	-	-	
Eurocode EC7	2520	20	1.50	0.67
Fdn Code of H.K. 2004	2200	17	1.62	0.67
$Q_C + \eta Q_{NSF} \leq (Q_P + Q_{PSF}) / F_s$	1420	12	2	0.67

Load Test on Piles Subjected to NSF

Proposed Allowable Settlement at $Q_C + 2Q_{NSF} \leq 25$ mm



Conclusions & Recommendations

1. NSF should be considered, if future settlement is real regardless of current state of consolidation.
2. NSF is a settlement problem.
3. Ultimate geotechnical capacity → $Q_{ULT} = Q_P + Q_S$
4. Neutral point → top of competent stratum

Conclusions & Recommendations (con't)

5. Both α and β method can be used.
6. Use η=0.67 and Fs=2 in all cases.
7. For bored piles, use Grade 40 concrete or higher.
8. Self-weight of pile need not be considered in design.

Conclusions & Recommendations (con't)

9. For load test on preliminary piles:
    - $Q_{MAX} = Q_S + Q_P$
    - Allowable settlement at 1.5WL ≤ 15 mm
    - Allowable settlement at 2WL ≤ 25 mm
    - Allowable settlement at $Q_C + 2\eta Q_{NSF}$ ≤ 25 mm

10. For load test on working piles:
    - $Q_{MAX} = Q_C + 2\eta Q_{NSF}$
    - Allowable settlement at 1.5WL ≤ 15 mm
    - Allowable settlement at 2WL ≤ 25 mm
    - Allowable settlement at $Q_C + 2\eta Q_{NSF}$ ≤ 25 mm

Thank You for your attention !

If you have any comments and suggestions, please email to me at wks@wks.sg.