



# **GUIDELINES ON LOCAL PRACTICES FOR PILE FOUNDATION DESIGN AND CONSTRUCTION**

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Professionals in charge of each project are strictly advised to do an independent assessment and verification to determine if the information provided in this guide is adequate and sufficient for the needs of their project.

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# **1. Introduction**

## **1.1 Background**

Foundation design in Singapore has been carried out in accordance with Singapore Standard Code of Practice for Foundations, SS CP4 (Spring Singapore, 2003) since 1976. The latest version of SS CP4 issued in 2003 had incorporate some local practices which were referred extensively and formed the basis in foundation design before the implementation of Eurocodes in Singapore. From 1<sup>st</sup> April 2015, Eurocode 7: Geotechnical Design (SS EN 1997, in short EC7) which comprises the main code SS EN 1997-1 and its accompanying National Annex, NA to SS EN1997-1, have replaced SS CP4 as the official code of practice for the foundation design in Singapore.

In 2019, Technical Committee of Civil and Geotechnical Works (TC CGW) of Enterprise Singapore (ESG) set up a committee to review and retain suitable local design practices in SS CP4 and produce a guideline for local practitioners.

## **1.2 Scope**

This guide aims to provide a compilation of local design practices, from SS CP4, with suitable updates, which could serve as Non-Contradictory Complementary Information (NCCI) documents to EC7.

The provisions of this guide apply to design of pile foundations in Singapore with reference to Section 7 of SS EN 1997-1.

# **2. Engineering Classification of Soils and Rocks**

In compliance with EC7, Designers are responsible for the planning of the geotechnical investigation and accountable for their decisions in specifying quantity and tests required to determine the geological profiles and geotechnical design parameters. Geotechnical investigation for pile foundation design shall be planned in such a way as to ensure that relevant geotechnical information and data are available at the various stages of the project. The geotechnical investigation shall be carried out in accordance with Eurocode 7: Geotechnical design – Part 2 Ground investigation and testing (“SS EN 1997-2”).

Prior to the field work, the Designers shall carry out desktop study to understand the geological setting of the project site. This could be done by referring to the Singapore Geological Map (BCA, 2021), literature review of conference and journal paper, and/or reviewing existing boreholes at the vicinity available through public domain, such as SLA Integrated Land Information Service (INLIS).

The aim is to understand the potential subsurface soil conditions to produce an estimate pile length, which will aid in the subsequent planning of geotechnical investigation of the specific site. In 2015, GeoSS has published a guide on “Ground Investigation and Geotechnical Characteristic Values to Eurocode 7”, which the Designer should refer to for the planning of ground investigation depending on the Geotechnical Categorisation of Projects as required in EC7.

The engineering classification of soils and rocks shall be based on the description of Rock Mass Weathering Grade as per the EN ISO 14689-1:2018. It is recommended that the Weathering Grade Symbols for igneous and sedimentary rocks such as Bukit Timah Centre Granite and Jurong Group shall follow Approach 2 of Annex B in EN ISO 14689-1:2018, where Roman numerals (I, II, III, IV, V & VI) are used to classify the weathering of rock. At the same time, the Terms and Description of rocks shall follow Table 15 of EN ISO 14689-1:2018. Table 2.1 summarises the approach recommended to suit local practice.

Table 2.1 – Weathering Grade of Rock Mass (Modification to Table 15 Of EN ISO 14689-1:2018)

Term	Grades Symbols [revised]	Description Classification of Rock Mass Weathering grade
Fresh	I	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces
Slightly weathered	II	Discoloration indicates weathering of rock material and discontinuity surfaces
Moderately weathered	III	Less than half of the rock material is decomposed or disintegrated. Fresh or discoloured rock is present either as a continuous framework or as core stones
Highly weathered	IV	More than half of the rock material is decomposed or disintegrated. Fresh or discoloured rock is present either as a continuous framework or as core stones
Completely weathered	V	All rock material is decomposed &/or disintegrated to soil. The original mass structure is still largely intact.
Residual Soil	VI	All rock material is converted to soil. The mass structure & material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported

For classification of Bedok Formation (Old Alluvium Formation), Approach 4 of Annex B in EN ISO 14689-1:2018 is recommended. It divides Bedok Formation into Class OA to OE based on weathered classifier and typical characteristics as summarised in Table 2.2. The local practice of classifying Old Alluvium Formation using SPT N-value in SS CP4 has been retained and shown together in Table 2.2.

Table 2.2 – Weathering classification of Bedok Formation (Old Alluvium) (see Annex B of EN ISO 14689-1:2018)

Class	Classifier	Typical Characteristics	Indicative SPT N-value (Blows/300 mm)
OA	Unweathered	Original Strength, colour, fracture spacing	> 100
OB	Partially Weathered	Slightly reduced strength, slightly closer fracture spacing, weathering penetrating in from fractures, brown oxidation	> 50
OC	Distinctly Weathered	Further weakened, much closer fracture spacing grey reduction	30 to 50
OD	Destructed	Greatly weakened, mottled, ordered litho-relics in matrix becoming weakened and disordered, bedding disturbed	10 to 30
OE	Residual or Reworked	Matrix with occasional altered, random or 'apparent' litho-relics, bedding destroyed. Classified as reworked when foreign inclusions are present as a result of transportation	<10

### 3. Geotechnical Design of Pile Foundations

#### 3.1 Partial Factors

The analysis and design for pile foundation shall be undertaken in accordance with SS EN 1997-1 Section 7 Pile Foundation. To demonstrate that the pile foundation is able to support the design actions with adequate safety against compressive failure, the following inequality shall be satisfied for all ULS load cases and load combinations:

$$F_{c,d} \leq R_{c,d} \quad \text{for axially loaded pile in compression} \quad (3.1)$$

$$F_{t,d} \leq R_{t,d} \quad \text{for axially loaded pile in tension} \quad (3.2)$$

where,

$F_{c,d} / F_{t,d}$  is the design value of the action

$R_{c,d} / R_{t,d}$  is the design value of the resistance

Design Approach 1 (DA1), as recommended in the NA to SS EN 1997-1, shall be adopted. The following sets of partial factors shall be adopted for axially loaded pile:

Combination 1 (DA1-C1): *A1 "+" M1 "+" R1*

Combination 2 (DA1-C2): *A2 "+" (M1 or M2) "+" R4*

where

*A* – Action;

*M* – Material;

*R* – Resistance;

“+” implies: “to be combined with”

The partial factors for Action and Material are presented in Table 3.1. In DA1-C2, set *M1* is used for calculating resistances of piles and set *M2* for calculating unfavourable actions on piles owing to e.g. negative skin friction or transverse loading. The partial resistance factors for driven and bored piles, as provided under NA to SS EN197-1, are presented in Table 3.2.

Table 3.1 Partial factors for ULS Design

	Combination 1 (DA1-C1)	Combination 2 (DA1-C2)
<b>Action</b>	<b>A1</b>	<b>A2</b>
Permanent Action - Unfavourable	1.35	1.00
Permanent Action - Favourable	1.00	1.00
Variable Action - Unfavourable	1.50	1.30
Variable Action – Favourable	0	0
<b>Soil Parameter</b>	<b>M1</b>	<b>M2</b>
Angle of internal friction, $\phi'$	1.00	1.25
Effective cohesion, $c'$		1.25
Undrained shear strength, $C_u$		1.40
Unconfined compressive strength, $q_u$		1.40
Density, $\gamma$		1.00

Table 3.2 Partial resistance factors for axially loaded piles

Resistance	Symbol	R1	Driven Piles		Bored Piles	
			R4 without explicit verification of SLS <sup>^</sup>	R4 with explicit verification of SLS <sup>^</sup>	R4 without explicit verification of SLS <sup>^</sup>	R4 with explicit verification of SLS <sup>^</sup>
Base	$\gamma_b$	1.0	1.7	1.5	2.0	1.7
Shaft (compression)	$\gamma_s$	1.0	1.5	1.3	1.6	1.4
Total/Combined (compression) <sup>#</sup>	$\gamma_t$	1.0	1.7	1.5	2.0	1.7
Shaft in tension	$\gamma_{st}$	1.0	2.0	1.7	2.0	1.7

<sup>^</sup> The lower g values in R4 may be adopted (a) if serviceability is verified by load test (preliminary and/or working) carried out on more than 1% of the constructed piles to loads not less than 1.5 times the representative load.

<sup>#</sup>  $\gamma_t$  shall be used when the base and shaft resistances could not be independently verified.

EC7 allows the compressive resistance of an individual pile to be determined from any of the following methods in accordance with SS EN 1997-1 clauses 7.4.1 and 7.6.2:

- a) static pile formulae based on ground parameters;
- b) direct formulae based on the results of field tests;
- c) the results of static pile load tests;
- d) the results of dynamic impact tests;
- e) pile driving formulae; and
- f) wave equation analysis.

Alternative design procedure in accordance with SS EN1997-1 clause 7.6.2.3(8) is commonly adopted for pile design in Singapore using the following empirical equations:

$$R_{c,d} = (Q_{s,k} + Q_{b,k}) / (MF \times \gamma_t) \quad (3.3)$$

$$R_{c,d} = Q_{s,k} / (MF \times \gamma_s) + (Q_{b,k} / (MF \times \gamma_b)) \quad (3.4)$$

where,

$Q_{s,k}$	-	total shaft resistance = $\sum(f_s \times A_s)$ ;
$Q_{b,k}$	-	total base resistance = $q_b \times A_b$ ;
$q_s$	-	unit shaft resistance = $K_s \times N_p$ ;
$A_s$	-	effective pile shaft surface area;
$A_b$	-	effective pile shaft base area;
$q_b$	-	unit base resistance = $K_b \times (40 N_p)$ ;
$K_s$	-	empirical coefficient for shaft resistance;
$K_b$	-	empirical coefficient for base resistance;
$N_p$	-	design SPT N-value;
$MF$	-	model factor.

The design  $q_s$  and  $q_b$  shall be the “cautious estimate of the values affecting the occurrence of the limit state” (see GeoSS Guide on Ground Investigation and Geotechnical Characteristic Values to Eurocode 7).

Besides the partial resistance factors given in Table 3.2, the characteristic geotechnical resistance will need to be corrected by a model factor ( $MF$ ) larger than 1.0. As stipulated in NA+A1:2018 to SS EN 1997-1:2010+A1:2018 Cl A.3.3.2, the value of the model factor should be **1.55** if the resistance, i.e. design parameters, are not verified by instrumented maintained load test, or reduced to **1.35** if the resistance is verified by instrumented maintained load test.

Designer shall comply with recommendations stipulated in Joint BCA/IES/ACES/GEOSS Circular “Requirements on Ground Investigation, Load Test and Quality Control Test for Foundation or its updated version. In accordance with Cl 5.2 of the Joint Circular, the total characteristic shaft resistance of bored piles should not be less than **1.3** times the characteristic actions unless

method of enhancing the pile base resistance, such as base grouting, is employed at site with verification by testing., i.e.  $(Q_{s,k} + Q_{b,k}) > 1.3 (G_k + Q_k)$ .

### 3.2 Design parameters $K_s$ and $K_b$

If the pile shaft resistance and base resistance are derived using empirical coefficients  $K_s$  and  $K_b$  in conjunction with the SPT N-values, the empirical coefficients, which form the key component of pile design parameters should be verified by instrumented ultimate pile load tests (ULT).

The value of  $K_s$  depends on method of pile installation, i.e. replacement or displacement. For replacement pile, the following factors, among others, shall be considered:

- a) method and tools used for boring,
- b) type of stabilising fluid, and
- c) method of concreting.

Similarly, the value of  $K_b$  depends on method of pile installation and the following factors:

- a) the depth of embedment in the bearing strata,
- b) the effect of loosening of the soil at the base due to boring,
- c) the effect of soil softening due to ingress of groundwater, and
- d) the methodology to clean the base of the borehole prior to casting.

In situations where the design parameters are not verified by ultimate load test, the recommendations provided in Tables 3.3 and 3.4 should be used.

Table 3.3. Recommended values of  $K_s$  and  $K_b$  for cast in-situ bored piles without ULT

Shaft Resistance	Base Resistance
For stiff to hard cohesive soil, including residual soils of Bukit Timah Central Granite and Jurong Group: $K_s = 1.5$ $q_s = K_s N_p \leq 150\text{kPa}$	For all stiff to hard formations: $K_b = 1$ $q_b = K_b (40 N_p) \leq 4\text{MPa}$
For dense or hard cemented soil in Bedok Formation (Old Alluvium Formation): $K_s = 2.0$ $q_s = K_s N_p \leq 200\text{kPa}$	

Table 3.4 Recommended values of  $K_s$  and  $K_b$  for displacement piles without ULT

Shaft Resistance	Base Resistance
$K_s = 2.5$ $q_s = K_s N_p \leq 250\text{kPa}$	For piles set on hard soils: $K_b = 6$ $q_b = K_b (40 N_p) \leq 18\text{MPa}$  For piles set on rock, $q_b$ may be taken as the strength of the pile material or the unconfined compressive strength of the rock, whichever is lower.

In addition to Table 3.4, the following criteria shall be complied for displacement pile:-

- a) driven pile based on total energy input with set criteria, hammer weight and drop height;
- b) jacked-in based on 2.25WL

To avoid discrepancy between design and as-built pile length, it is advisable to carry out instrumented ultimate load test to verify the design parameters, instead of using the conservative parameters provided in Tables 3.3 and 3.4.

### 3.3 Pile spacing

It is known that the resistance of individual pile in a pile group reduces as the spacing between the piles reduces. This phenomenon is commonly known as “pile group effect”. In order to limit the reduction in resistance of individual pile in a pile group, minimum spacing between the piles as specified in clause 6.3.3 of BS 8004 (2015) need to be complied with. The relevant spacing vary depending on the contribution of the total shaft resistance,  $Q_{s,k}$  to the total geotechnical resistance,  $Q_t = Q_{s,k} + Q_{b,k}$ .

When viewed on plan, the centre-to-centre pile spacing,  $S$  should comply with the following criteria.

for “friction piles” where the  $Q_{s,k} > 75\% Q_t$ :

- $S \geq 3D$  for circular piles or;
- $S \geq P$  for non-circular piles

for piles where  $50\% Q_t \leq Q_{s,k} \leq 75\% Q_t$ :

- $S \geq 2.5D$  for circular piles or;
- $S \geq \frac{3}{4} P$  for non-circular piles

for piles where  $Q_{s,k} < 50\% Q_t$ :

- $S \geq 2D$  for circular piles or;
- $S \geq \frac{2}{3} P$  for non-circular piles

Where:

P is the perimeter of the larger of two adjacent piles: and  
D is the outside diameter of the larger of two adjacent piles.

Pile group interaction shall be considered in the design when closer spacings are adopted. It should be shown that any increased settlement of the piles arising from their interaction does not lead to a limit state being exceeded. For special circumstances where the pile foundations form part of an earth retaining structure, the Designer shall consider the shaft and/or base resistance based on the effective shaft or base resistance. The choice of pile spacing should take into account the pile installation method, particularly when dealing with displacement piles such as driven piles.

## 4. Pile Design Subjected to Drag Force

### 4.1 Consideration of Drag Force or Negative Skin Friction

Ground in which piles are located may be subject to displacement caused by consolidation, swelling, adjacent loads, creeping soils, landslides or earthquakes. Consideration shall be given to these phenomena as they can affect the piles by causing drag force (negative skin friction), heave, stretching, transverse loading and displacement.

The term “Negative Skin Friction (NSF)” is used in this guide to refer to the phenomenon in pile design whereby the ground surrounding a pile settles a significant amount relative to the pile head.

NSF is particularly relevant to piles installed in a low strength clay [*NOTE: low strength clay refers to clay with undrained shear strength  $c_u < 40\text{kPa}$  based on EN ISO 14689-1:2018*] or in loose granular soils subject to upfilling or a lowering of the groundwater table (BS8004:2015). For piles installed in a low strength clay, the Designer needs to consider the NSF force (the drag force) in the pile design if no in-situ stress check is carried out. For cases where the designer strongly

believe that NSF may not be applicable, the Designer shall determine the in-situ stress history and stage of consolidation of the surrounding ground under the designed vertical effective stress, to ascertain if drag force need to be considered in the pile design. This can be done by determining the pre-consolidation pressure,  $P_c$  of the soft soils using laboratory tests such as oedometer test and constant rate of strain (CRS) test; and field test such as cone penetration test (CPT). The  $P_c$  value is then used to compute over-consolidation ratio (OCR) using the following equation:

$$\text{OCR} = P_c / \sigma'_v \quad (4.1)$$

Where,  $\sigma'_v$  is the designed vertical effective stress.

Drag force on piles shall be considered when OCR is less than 1.0, where the clay will undergo consolidation and settle relative to the piles during the design life of the piles.

Table 4.1 provides the minimum requirement for laboratory and/or field tests to ascertain if drag force design is applicable.

For piles subjected to drag force, the designer may consider the following design approaches:

1. Piles design for NSF (no debonding)
2. Piles design without NSF (with debonding)

Table 4.1: Minimum test requirement to determine if NSF design is applicable

Soft Clay Thickness	Minimum Test Requirements (i) OR (ii)
≥10m	(i) Tests to determine $P_c$ and OCR shall be carried out for at least 50% of the boreholes. For each borehole chosen, undisturbed soil samples should be retrieved at vertical spacing of 3-5m, with minimum 3 samples, for oedometer or CRS tests. (ii) CPT of the same quantity as the minimum number of boreholes, in replacement for lab tests.
<10m	(i) Tests to determine $P_c$ and OCR shall be carried out for at least 50% of the boreholes. QP to determine and justify the number of lab test required. (ii) CPT of the same quantity as the minimum number of boreholes, in replacement for lab tests

## 4.2 Determination of Neutral Plane

Figure 4.1 illustrates the soil-pile interaction under settling ground scenario. When soil surrounding a pile settles relative to the pile, drag force will be induced and accumulated along the pile shaft within the settling soil layer through “negative skin friction”. At the same time, pile

will move downward under the design action and drag force. The point along the pile shaft where the pile movement and the soil movement are equal, i.e. no relative movement between pile and soil, is defined as Neutral Plane, as shown in Figure 4.1.

The actual location of the neutral plane depends on many factors and is significantly affected by the thickness of the consolidating soil ( $L_s$ ), the end-bearing condition, and the axial load, among others. Theoretical studies of negative skin friction on piles with no other load being applied indicated a depth to neutral plane,  $L_{dd}$ , ranging from  $0.5L_s$  for a friction pile to  $1.0L_s$  for end-bearing pile, although field data suggested a range of  $0.7L_s$  to  $1.0L_s$  (Kog, et al., 1986; Wong and Teh, 1995). The location of the neutral plane is expected to move up as the axial load in the pile increases.

For design purpose, the neutral plane location can be assumed at  $0.6L_s$  for friction piles in a generally homogeneous consolidating stratum and  $1.0L_s$  for end bearing piles. The neutral plane could also be determined from numerical analysis using finite element program or other pile-soil interaction program.

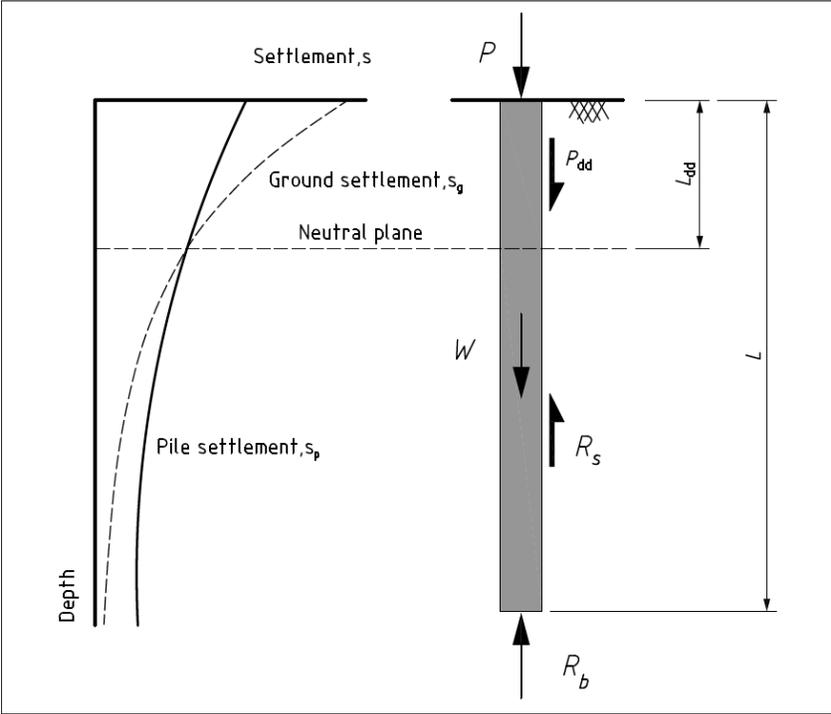


Figure 4.1. Soil-pile interaction in settling ground

### 4.3 Calculation of Drag Force

The characteristic compressive force ( $F_{c,k}$ ) acting on a pile that is subject to drag force could be expressed using the following equation:

$$F_{c,k} = P_{c,k} + W_k + \eta P_{dd,k} \quad (4.2)$$

where:

$P_{c,k}$  is the characteristic compressive force applied to the pile by the structure (or the characteristic column load);

$W_k$  is the characteristic self-weight of the pile;  
 [Note: need to consider as per EC7 cl. 7.6.2.1(2) if (i) downdrag is significant, (ii) soil is very light or (iii) pile extends above the surface of the ground]

$\eta$  is the degree of mobilised value of the negative unit friction along the pile section. The values above the neutral plane may vary between the fully mobilised value on the top and a small value close to the neutral plane. The value may be assumed as 0.67, although a value of 1.0 may be considered for special cases involving low-capacity piles in highly compressible clay stratum; and

$P_{dd,k}$  is the additional characteristic compressive force due to drag force (see Figure 4.1), given by:

$$P_{dd,k} = \int_0^{L_{dd}} (C_s \times q_{s,k,sup}) dz \quad (4.3)$$

where:

$L_{dd}$  is the length of pile subject to drag force (defined in Figure 4.1);

$C_s$  is the circumference of the pile shaft at depth  $z$ ; and

$q_{s,k,sup}$  is the “superior” characteristic unit shaft friction at depth  $z$ .

For determination of drag force, effective stress method (i.e. beta method) should be adopted. The “superior” characteristic unit shaft friction ( $q_{s,k,sup}$ ) should be selected as a cautious upper estimate of the mean shaft friction acting over the length of pile that is subject to drag force. [NOTE: A cautious upper estimate of the mean is one that has a 5% probability of being exceeded during the design working life].

The characteristic shaft resistance ( $R_{s,dd,k}$ ) or the positive skin friction of a pile subject to drag force should be calculated from:

$$R_{s,dd,k} = \frac{\int_0^L (C_s \times q_{s,k,inf}) dz}{\gamma_{Rd}} \quad (4.4)$$

where:

$L$  is the total length of the pile;

$L_{dd}$  is the length of pile subject to drag force (see Figure 5.1);

$C_s$  is the circumference of the pile shaft at depth  $z$ ;

$q_{s,k,inf}$  is the “inferior” characteristic (moderately conservative) unit shaft friction at depth  $z$ ; and

$\gamma_{Rd}$  is a model factor.

The “inferior” characteristic unit shaft friction ( $q_{s,k,inf}$ ) should be selected as a cautious lower estimate of the mean shaft friction acting over the length of pile that is not subject to drag force.

[NOTE: A cautious lower estimate of the mean is one that has a 5% probability of not being achieved during the design working life]

#### 4.4 Design Compressive Force at Ultimate Limit State

The design compressive force ( $F_{c,d}$ ) applied to an individual pile at its ultimate limit state should be calculated from:

$$F_{c,d} = \sum_i (\gamma_{F,i} \psi_i P_{c,k,i}) + \gamma_G (W_k + \eta P_{dd,k}) \quad (4.5)$$

where:

- $P_{c,k,i}$  is the  $i$ th characteristic compressive force applied to the pile by the structure;
- $W_k$  is the characteristic self-weight of the pile;
- $\eta P_{dd,k}$  is the mobilised additional characteristic compressive force owing to drag force;
- $\psi_i$  is the corresponding combination factor for the  $i$ th force;
- $\gamma_{F,i}$  is the corresponding partial factor on actions for the  $i$ th force; and
- $\gamma_G$  is the partial factor on permanent actions.

For building load, the equation above could be simplified to:

$$F_{c,d} = 1.35G_k + 1.5Q_k + 1.35(W_k + \eta P_{dd,k}) \quad \text{DA1-C1} \quad (4.6)$$

$$F_{c,d} = 1.0G_k + 1.3Q_k + 1.0(W_k + \eta P_{dd,k}) \quad \text{DA1-C2} \quad (4.7)$$

In EC7 Cl 7.3.2.2 (7), drag force and transient loading need not be considered simultaneously in load combinations. The transient loading generally referred to temporary conditions of the structure, of use, or exposure, e.g. during construction or repair (EC0 Cl1.5.2.3). The wind load and live loads are not defined as transient load.

#### 4.5 Geotechnical and Structural Resistances

The design calculation of geotechnical resistance for pile segment below the neutral plane is similar to a typical pile foundation as described in Section 3 of this Guide. For structural resistance of pile subject to drag force, Section 5 of this Guide should be referred.

## 5. Structural Capacity of Bored Piles

### 5.1 Structural capacity of bored piles in accordance with SS EN 1992-1-1

The structural design of concrete piles shall be referred to SS EN 1992-1-1. The design axial capacity of a concrete pile  $N_{c,d}$  may be calculated using the following equation: -

$$N_{c,d} = \alpha_{cc} f_{ck} / (\gamma_c \cdot k_f) A_c + 0.87 f_y A_{sc} \quad (5.1)$$

where

$$\alpha_{cc,p} = 0.85 \text{ (reinforced concrete); } = 0.60 \text{ (un-reinforced concrete)}$$

$$f_{ck} = \text{characteristic cylinder strength of concrete}$$

$$\gamma_c = 1.5 \text{ (SS EN 1992-1-1 Table 2.1N)}$$

$$k_f = 1.1 \text{ for cast in place piles without permanent casing, otherwise } k_f = 1.0.$$

$$A_c = \text{cross section area of concrete}$$

$$A_{sc} = \text{total area of steel reinforcement bars}$$

For nominally reinforced concrete bored pile, the contribution from steel reinforcement bars shall be ignored. For segment of pile with reinforcement, the following design condition shall be complied with:

$$N_{c,d} = 0.515 f_{ck} * A_c > N_{Ed} = 1.35G_k + 1.5Q_k \quad (5.2)$$

For segment of pile without reinforcement, the following design condition shall be complied with:

$$N_{c,d} = 0.364 f_{ck} * A_c > N_{Ed} = 1.35G_k + 1.5Q_k \quad (5.3)$$

In the event where the contribution of steel reinforcement bars is to be considered in the structural capacity, such as micropile, the total design axial capacity of the concrete pile computed using Equation (5.1) shall be divided by a factor of 1.5, as shown in Equation (5.4).

$$N_{c,d} = (0.515 f_{ck} * A_c + 0.87 f_y A_{sc}) / 1.5 > N_{Ed} = 1.35G_k + 1.5Q_k \quad (5.4)$$

This is to account for the geometric imperfections and nominal eccentricity in bored cast-in-place concrete pile, to comply with EN1992-1-1 Cl 5.2 and Cl 6.1 (4).

## 5.2 Structural capacity of bored piles in accordance with SS CP4

Alternative to the structural capacity check described in Section 5.1, the following simplified design method, adapted from SS CP4, can be used.

For nominally reinforced bored cast-in-place pile the allowable structural capacity  $(Q_a)_{st}$  can be taken as

$$(Q_a)_{st} = 0.25f_{cu} A_c, < (G_k + Q_k)$$

where  $0.25f_{cu}$  should not exceed 7.5MPa.

For rock socketed reinforced bored cast-in-place pile with full length steel reinforcement, the allowable structural capacity of the piles may be determined in accordance with Clause 3.8.4.3 of SS CP 65 as axially loaded short column, which could be taken as

$$(Q_a)_{st} = (0.4 f_{cu} A_c + 0.75 f_y A_{sc}) / F_s$$

where

- $f_{cu}$  = characteristic cube strength of concrete
- $f_y$  = yield stress of steel (limited to 500 MPa)
- $F_s$  = factor of safety ( $\geq 2$ )

## 5.3 Allowable concrete compressive stress of bored pile – SS CP4 vs Eurocode

While SS CP4 has specified a maximum average concrete compressive stress not exceeding 7.5 MPa, no such limitation has been imposed in Eurocode design for pile foundation.

As a result, Designers may tend to maximise the concrete compressive stress in the process of optimising (i.e. reducing) the pile size. In some instances, this is done solely based on the Equations (5.2) to (5.4) without consideration of geotechnical resistance, in particular the base resistance. It should be noted that the base resistance is highly governed by geology, pile construction methodology and pile toe cleanliness etc.

Designers that adopt concrete compressive stress  $> 7.5$ MPa shall check whether the settlement criteria can be complied with. In addition, Designer also to assess whether the higher quality/standards in workmanship to commensurate with the design requirement can be consistently achieved at the site. For pile design adopts concrete compressive stress under working load condition greater than 7.5 MPa, the Designer shall refer to Joint BCA/ IES/ ACES/

GeoSS Circular APPBCA-2016-08 for the required tests, for instance Concrete Core Test below pile cut-off level till a depth where the anticipated stress is equal or less than 7.5MPa.

#### **5.4 Bored pile reinforcement**

For bored piles subject to compression, the minimum longitudinal reinforcement,  $A_{s, \text{bmin}}$ , for different pile cross-section area shall comply with SS EN 1992-1-1 Cl 9.8.5 Table 9.6N. For bored piles with diameters not exceeding 600mm, the minimum longitudinal reinforcement,  $A_{s, \text{bmin}}$ , given in Table 9.6N of SS EN 1992-1-1 is still applicable.

SS EN 1992-1-1 has recommended values for minimum bar diameter, minimum number of main bars and clear spacing between bars as reference for Designers. As a good practice, Designer shall adopt large enough bar diameter and adequate quantity of longitudinal bar to ensure the reinforcement cage rigidity during lifting and lowering into the pile bore. Further reference shall also be made to EN1536 for detailing requirement of longitudinal and transverse reinforcement in bored piles.

EN1536 recommends that the length of reinforcement shall go beyond the soft or loose soil, subject to a minimum of 10 m from pile cut-off-level. This is consistent with SS CP4's recommendation of providing longitudinal reinforcement to at least 10 m below the pile cut-off-level or full length of the pile if the pile is less than 10 m long.

### **6. Reuse of Existing Piles under Demolished Building**

In certain circumstances, some or all of the existing piles under a demolished building may be reused if detailed records, such as records of pile design, pile installation, load test and as-built plans, are available and the existing condition of the pile can be evaluated by testing. Stringent quality checks should be carried out to confirm the strength, durability and integrity of the existing piles. Tests may include but not limited to static load test, dynamic load tests, pile integrity tests and pile material strength tests.

Design of the foundation system should take into consideration the relative stiffness of the existing piles in the foundation system especially when new piles and existing piles are in the same pile group or when the new piles and existing piles are of different pile types.

To ensure that piles to be reused are left intact after the building is demolished, the demolition work has to be properly planned and closely supervised by competent personnel.

The guides on reused piles can be referred to Joint BCA/IES/ACES/GEOSS Circular “Guidelines on Reuse of Existing Piles” issued on 2<sup>nd</sup> September 2019.

## **7. Position and Alignment Tolerances**

Piles shall be constructed as accurately as possible to the vertical or the specified rake. For vertical piles, a deviation of 1 in 75 should not normally be exceeded, although in special cases a closer tolerance may be necessary. Straining the pile into position can damage it, and the leaders and driving equipment should be adjusted as much as possible to follow the position of the pile. For projects that requires more stringent measures such as high-rise building, the designer could specify a verticality tolerance of 1:100 or even more stringent, as a recommendation.

Piles should not deviate 75 mm from their designated position at the working level of the piling rig. Greater tolerance may be prescribed for pile driven over water and for raking piles. For piles to be cut off at a substantial depth, the design should recognise the worst combination of the above tolerances in position and inclination.

Any pile deviating beyond these limits and to such an extent that the resulting eccentricity cannot be taken care of by redesign of the pile cap or tie beams, should, at the discretion of the engineer, be replaced or supplemented by compensating pile(s).

## **8. Allowable Settlement under Pile Load Test**

When pile design parameters are determined based on empirical correlations to soil strengths from laboratory or field tests, the parameters must be verified by instrumented ultimate pile load tests (ULT). In local practice, this principle should be applied as follows:

1. For buildings of 10-storeys or more, ULT must be carried out, as prescribed in Joint BCA/ IES/ ACES/ GeoSS Circular APPBCA-2016-08.
2. For other cases, the Qualified Person for Structural Works (QP(ST)) is to assess the necessity of ULT.

The serviceability limit states should be verified according to SS EN 1997 Cl 2.4.8(1) and 2.4.9(1) via pile testing. For project with major building work, working load test should be carried out.

### Working pile load test

As per BCA/ IES/ ACES/ GeoSS Circular APPBCA-2016-08, for working pile load test in which the pile is tested to 1.5 or 2.0 times the characteristic load (equivalent to unfactored column load), the allowable maximum settlement measured at the pile top under full test load should be taken as 15 mm or 25 mm, respectively. Table 8.1 provides the allowable pile settlement for working load test under various testing conditions for piles not subjected to NSF.

For piles subject to negative skin friction, the working load test can be performed in the normal manner and its acceptance subject to satisfaction of an additional criterion summarised in Table 8.2.

### Ultimate Pile Load Test

For ultimate pile load test for which the pile is normally tested to 2.5 to 3.0 times characteristic load, failure is usually defined as the test load at which pile settlement achieves 10% of pile diameter, or continues without further load increment. Specific value of allowable settlement need not be specified because the objective of such load tests is to test the pile until soil resistances are fully mobilised so that the design parameters can be verified.

Table 8.1(a): Allowable Pile Settlement for Working Load Test without NSF – Case 1

<p><u>Case1 (without NSF)</u></p> <p>Notes:          WL = unfactored column load          COL = cut-off level.          GL = ground level</p>	<p><b><u>Case 1 (without NSF):</u></b></p> <ul style="list-style-type: none"> <li>• Pile COL near or at GL.</li> <li>• Maintained load test from pile top with measurement of the pile load and settlement at GL.</li> </ul> <p>➔ Allowable maximum settlement of 15mm or 25mm at 1.5WL or 2WL, respectively.</p>
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Table 8.1(b): Allowable Pile Settlement for Working Load Test without NSF – Case 2

<p><u>Case2(a) (without NSF)</u> <u>Direct Measurement of Load and Settlement at COL</u></p>	<p><u>Case2(b) (without NSF)</u> <u>Conventional Testing Method</u></p>	<p><b>Case 2 (without NSF):</b></p>
<p>1.5WL or 2WL</p> <p>GL</p> <p>Fill</p> <p>← debonding</p> <p>Measured load &amp; pile settlement @ COL</p> <p>Stiff soil</p> <p>Residual Soil</p> <p>Qs</p> <p>Qb</p>	<p>1.5WL or 2WL</p> <p>GL</p> <p>Fill</p> <p>Measured load &amp; pile settlement @ GL</p> <p>← debonding</p> <p>COL</p> <p>Stiff soil</p> <p>Residual Soil</p> <p>Qs</p> <p>Qb</p>	<ul style="list-style-type: none"> <li>• Pile COL at some distance below GL.</li> <li>• Test pile cast up to GL with debonding up to COL.</li> </ul> <p><b>Case 2(a) measurement of load and displacement at Cut-off Level:</b></p> <p>Allowable maximum settlement of 15mm or 25mm at 1.5WL or 2WL, respectively</p> <p><b>Case 2(b) measurement of load and displacement at Ground Level:</b></p> <p>Allowable maximum settlement of 15mm or 25mm at pile top for 1.5WL or 2WL, respectively. (Note: elastic shortening of pile within the debonding length will result in more stringent settlement criteria at COL.)</p>

Table 8.1(c): Allowable Pile Settlement for Working Load Test without NSF – Case 3

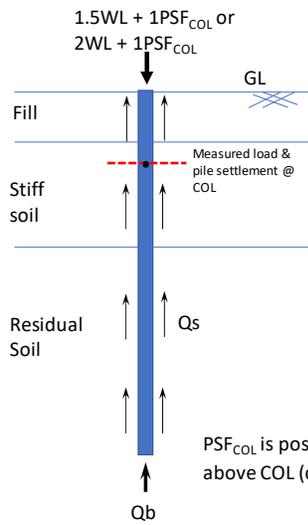
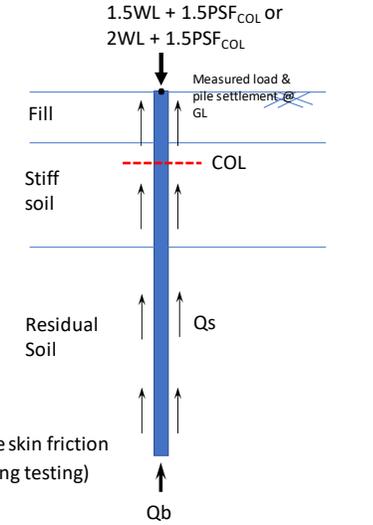
<p><u>Case3(a) (without NSF)</u> <u>Direct Measurement of Load and Settlement at COL</u></p>  <p>1.5WL + 1PSF<sub>COL</sub> or 2WL + 1PSF<sub>COL</sub></p> <p>GL</p> <p>Fill</p> <p>Stiff soil</p> <p>Residual Soil</p> <p>Measured load &amp; pile settlement @ COL</p> <p>Qs</p> <p>Qb</p> <p>PSF<sub>COL</sub> is positive skin friction above COL (during testing)</p>	<p><u>Case3(b) (without NSF)</u> <u>Conventional Testing Method</u></p>  <p>1.5WL + 1.5PSF<sub>COL</sub> or 2WL + 1.5PSF<sub>COL</sub></p> <p>GL</p> <p>Fill</p> <p>Stiff soil</p> <p>Residual Soil</p> <p>Measured load &amp; pile settlement @ GL</p> <p>COL</p> <p>Qs</p> <p>Qb</p>	<p><b>Case 3 (without NSF):</b></p> <ul style="list-style-type: none"> <li>• Pile COL at some distance below GL.</li> <li>• Test pile cast up to GL without debonding.</li> </ul> <p><b>Case 3(a) measurement of load and displacement at Cut-off Level:</b></p> <p>Allowable maximum settlement of 15mm or 25mm at 1.5WL or 2WL respectively (Note: The load and displacement measurement at COL shall be ensured)</p> <p><b>Case 3(b) measurement of load and displacement At Ground Level:</b></p> <p>Acceptable settlement of 15mm or 25mm at 1.5WL+1.5PSF<sub>COL</sub> or 2WL+1.5PSF<sub>COL</sub> respectively. Where PSF<sub>COL</sub> = Positive shaft resistance between GL and COL</p>
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Table 8.2(a): Allowable Pile Settlement for Working Load Test with NSF – Case 1

<p><u>Case1 (with NSF)</u></p> <p><math>1.5WL + 2NSF</math> or <math>2WL + 2NSF</math></p> <p>Measured load &amp; pile settlement @ <math>GL=COL</math></p> <p>Notes:          WL = unfactored column load          COL = cut-off level.          GL = ground level          NSF = negative skin friction</p>	<p><b>Case 1 (with NSF):</b></p> <ul style="list-style-type: none"> <li>• Pile COL near or at GL.</li> <li>• Maintained load test from pile top with measurement of the pile load and settlement at GL.</li> </ul> <p>➔ Allowable maximum settlement of 15mm at <math>1.5WL+2NSF</math> or 25mm at <math>2WL+2NSF</math>.</p>
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Table 8.2(b): Allowable Pile Settlement for Working Load Test with NSF – Case 2

<p><u>Case2(a) (with NSF)</u>  <u>Direct Measurement of Load and Settlement at COL</u></p>	<p><u>Case2(b) (with NSF)</u>  <u>Conventional Testing Method</u></p>	<p><b><u>Case 2 (with NSF):</u></b></p> <ul style="list-style-type: none"> <li>• Pile COL at some distance below GL.</li> <li>• Test pile cast up to GL with debonding up to COL.</li> </ul> <p><b>Case 2(a) measurement of load and displacement at Cut-off Level:</b></p> <p>Allowable maximum settlement of 15mm at 1.5 WL + 2NSF or 25mm at 2WL + 2NSF</p> <p><b>Case 2(b) measurement of load and displacement at Ground Level:</b></p> <p>Allowable maximum settlement of 15mm at 1.5WL + 2NSF or 25mm at 2WL + 2NSF</p> <p>(Note: elastic shortening of pile within the debonding length will result in more stringent settlement criteria at COL.)</p>
<p>1.5WL + 2NSF or 2WL + 2NSF</p> <p>GL</p> <p>Fill</p> <p>← debonding</p> <p>Measured load &amp; pile settlement @ COL</p> <p>Clay (soft)</p> <p>↓ Qs (NSF)</p> <p>Residual Soil</p> <p>↑ Qs</p> <p>↑ Qb</p>	<p>1.5WL + 2NSF or 2WL + 2NSF</p> <p>GL</p> <p>Fill</p> <p>Measured load &amp; pile settlement @ GL</p> <p>← debonding</p> <p>COL</p> <p>Clay (soft)</p> <p>↓ Qs (NSF)</p> <p>Residual Soil</p> <p>↑ Qs</p> <p>↑ Qb</p>	

Table 8.2(c): Allowable Pile Settlement for Working Load Test with NSF – Case 3

<p><u>Case3(a) (with NSF)</u> <u>Direct Measurement of Load and Settlement at COL</u></p> <p>1.5WL + 2NSF + 1PSF<sub>COL</sub> or 2WL + 2NSF + 1PSF<sub>COL</sub></p> <p>GL</p> <p>Fill</p> <p>Clay (soft)</p> <p>Measured load &amp; pile settlement @ COL</p> <p>Qs (NSF)</p> <p>Residual Soil</p> <p>Qs</p> <p>PSF<sub>COL</sub> is positive skin friction above COL (during testing)</p> <p>Qb</p>	<p><u>Case3(b) (with NSF)</u> <u>Conventional Testing Method</u></p> <p>1.5WL + 2NSF + 1.5PSF<sub>COL</sub> or 2WL + 2NSF + 1.5PSF<sub>COL</sub></p> <p>Measured load &amp; pile settlement @ GL</p> <p>GL</p> <p>Fill</p> <p>Clay (soft)</p> <p>COL</p> <p>Qs (NSF)</p> <p>Residual Soil</p> <p>Qs</p> <p>Qb</p>	<p><b>Case 3 (with NSF):</b></p> <ul style="list-style-type: none"> <li>• Pile COL at some distance below GL.</li> <li>• Test pile cast up to GL without debonding.</li> </ul> <p><b>Case 3(a) measurement of load and displacement at Cut-off Level:</b></p> <p>Allowable maximum settlement of 15mm at 1.5 WL + 2NSF + 1PSF<sub>COL</sub> ; or 25mm at 2.0 WL + 2NSF + 1PSF<sub>COL</sub> (Note: The load and displacement measurement at COL shall be ensured)</p> <p><b>Case 3(b) measurement of load and displacement At Ground Level:</b></p> <p>Acceptable settlement of 15mm at 1.5 WL + 2NSF + 1.5PSF<sub>COL</sub> ; or 25mm at 2.0 WL + 2NSF + 1.5PSF<sub>COL</sub>. Where PSF<sub>COL</sub> = Positive shaft resistance between GL and COL</p>
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## **9. Procedures on Pile Load Tests**

For pile load test, designer shall refer to BS EN ISO 22477-1:2018 for load step sequence, which supersedes the old practice based on SS CP4. The subsequent sections provide a brief summary of the testing procedure for the reference of designers.

### **9.1 Clause 5.2.1 of BS EN ISO 22477-1: 2018 on Pile Load Tests**

Load test to BS EN standards should be executed following either a one single loading / unloading cycle, or alternatively multiple loading / unloading cycles. Table 9.1 provides a summary of the single and multiple loading and unloading cycle testing procedure.

For building structure that is not likely to experience cyclic loading, it is suggested that the load test can be executed via a one single loading/unloading cycle. However, for structure that will experience multiple loading and unloading cycle through the structure life span, rebound data is useful for analysis of the rebound behaviour where minimum 2 cycles for pile load test should be carried out.

### **9.2 Test loads for pile load test**

The designer should design the pile load test based on the intent and purpose of the test. For example, if the designer has assessed that there is potential to optimise the pile design parameters, the test load for investigation test should be adequate to enable the designer to substantiate that the design parameter can a) be mobilised and b) satisfy the settlement performance criteria.

It is recommended that project sites carrying out multiple investigation test should plan to carry out subsequent investigation tests after assessing the results from the first test, especially where there is a change in the design parameter. This will enable the designer to optimise the investigation test to verify the performance of the revised design parameter.

To be compatible with the settlement requirements as per Circular “Requirements on Ground Investigation, Load Test and Quality Control Test for Foundation” issued on 22 Sep 2016, the test load and the pile load test report should be based on working load (defined as unfactored column load) where the settlement performance criteria can be directly compared against.

Table 9.1: - Loading and unloading procedure

Description	Single Loading/unloading Cycle	Multiple Loading/unloading Cycle
Pile load test cycles	Single unloading after completion of maximum loading stage	Maximum test load is reached in a minimum of two steps
Loading and unloading steps $F_p$ = max test load. $F_{c,k}$ = characteristic pile load	Start by a load of maximum $0.05F_p$ Minimum 8 steps of generally equal magnitude in loading and 4 steps in unloading.	Start by a load of maximum $0.05F_p$ Minimu. 4 steps <sup>^</sup> in loading for the first cycle (up to $F_{c,k}$ ) and 2 steps in first unloading cycle. Min. 8 steps <sup>^</sup> in loading for the second load cycle (up to $F_p$ ) and minimum 4 steps in unloading cycle.
Load duration	Refer to Figure 9.1 for the min. recommended load duration for each load steps. The 1 <sup>st</sup> loading step may have a shortened duration when the pile displacement rate is lower than 0.1 mm/20 min. Duration should be extended if the creep rate is still increasing, or displacement rate is greater than 0.1 mm/10 min.	Refer to Figure 9.2 for the min. recommended duration for each load steps. Duration shall be extended if displacement rate is greater than 0.1 mm/10 min when load is greater than $F_{c,k}$ , and 0.1 mm/5 min subsequently.

<sup>^</sup> - Load increments should be of equal magnitude

Between  $0.05 F_p$  and the maximum load of the first cycle;

Between the maximum load of the first cycle and  $F_p$ .

Magnitude of increments between  $0.05 F_p$  and the maximum load of the first cycle and between the maximum load of the first cycle and  $F_p$  are usually different.

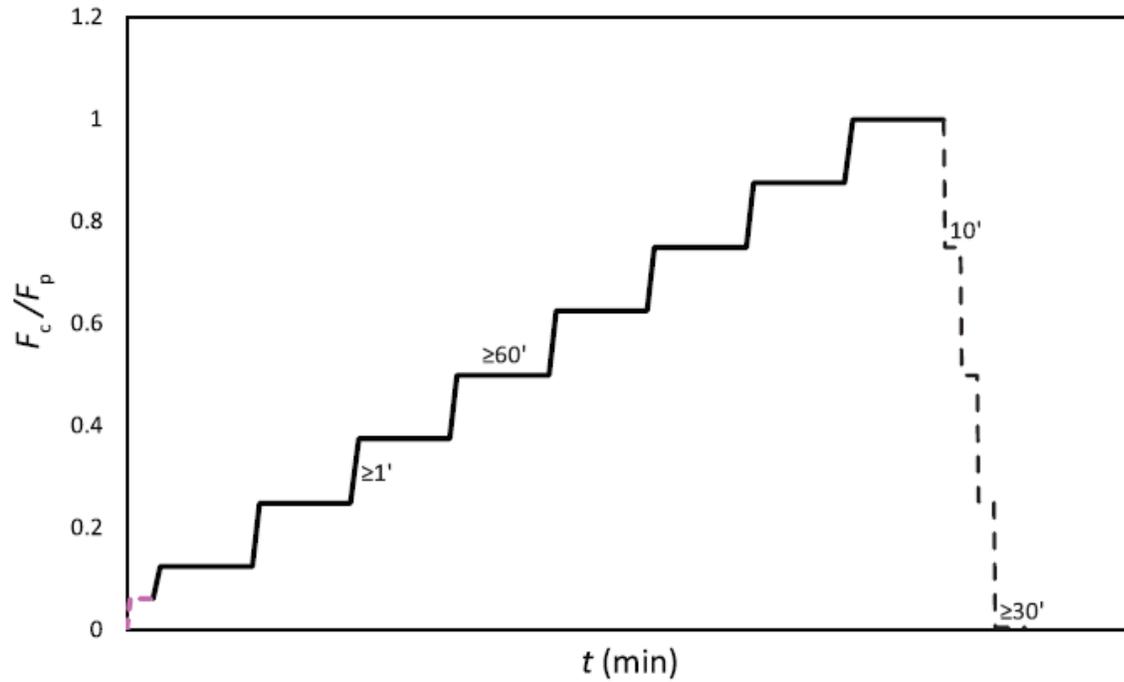


Figure 9.1 – Load step sequence for one cycle procedure

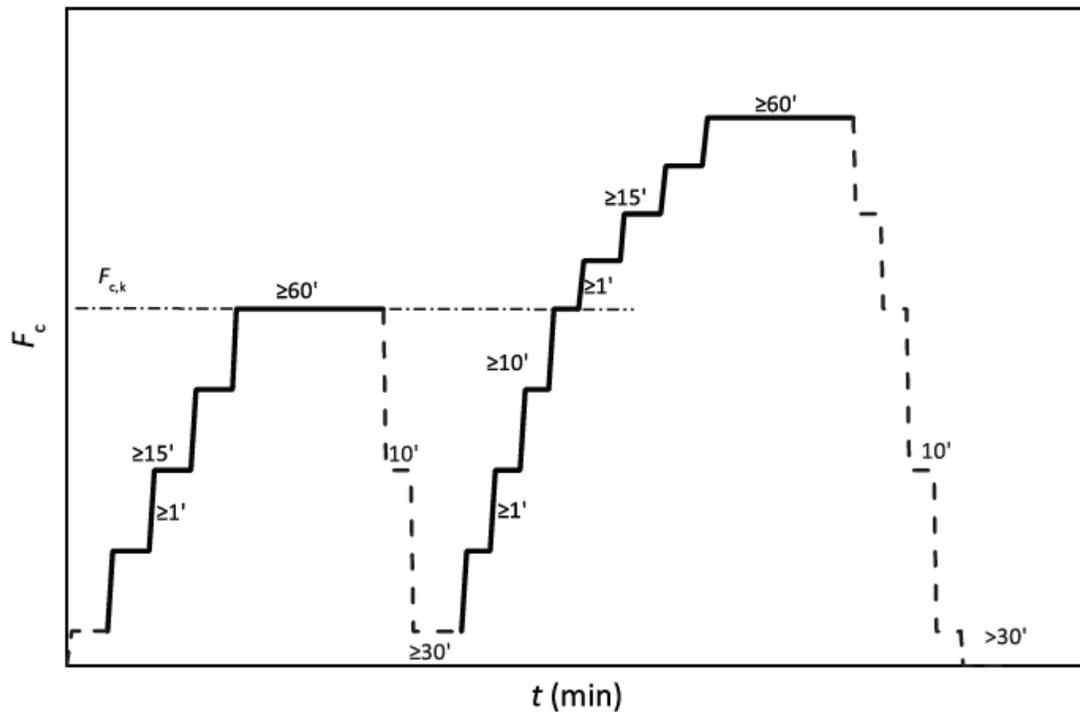


Figure 9.2 – Load step sequence for two cycle procedure

## References

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