A new approach for assessing offshore piles subjected to cyclic axial loading[1]

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Summary

It is a requirement from BSH ("Bundesamt für Seeschifffahrt und Hydrographie", the permitting authority for offshore wind farms in Germany) that the effect of cyclic loading on the capacity of foundations supporting offshore wind turbines is assessed. So far practical design methods are rare. In this paper a new approach is presented for assessing offshore piles subjected to cyclic axial loading which combines two known methods, the approach for static loading as developed by the Imperial College of London (termed ICP approach) and a load-transfer-approach as incorporated in software called "RATZ" for cyclic loading. Results using this method are compared against tests performed for Health & Safety Executive (HSE) in Dunkirk and very good agreement is found.

1. Introduction

Consideration of cyclic loading is a new requirement for offshore wind turbines erected in Germany under legislative of BSH [1], details can be found in [2]. Currently, only very few methods exist to fulfill this requirement for axial and/or lateral loading. This paper proposes a new approach for axial loading which combines the "Imperial College Pile" (ICP) method for statically loaded piles with a load-transfer approach as implemented in the software RATZ. Both methods are well accepted and tested in the offshore business, hence they form an ideal basis for assessment of piles supporting offshore wind turbines. The newly combined method has been validated against tests performed in Dunkirk.

2. Existing calculation methods as basis for the new approach

A very brief summary of the ICP method as described in [3] is given. It is outside the scope of this paper to give further justification or description of this method.

2.1. Static pile capacity

The capacity of a pile under static (monotonic) loading is the basis for any subsequent assessment of cyclic loading. It is therefore essential to start with a realistic estimate of axial capacity, which is deemed to be provided by the ICP method. Additional information can be found in numerous publications and textbooks about pile design, e.g. [3].

[1] This is the pre-peer reviewed version, which has been published in final form at http://onlinelibrary.wiley.com/doi/10.1002/gete.201100007/abstract

2.1.1. ICP design method for driven piles in sand

The ultimate load capacity of a pile Q_u is often the governing criterion when designing driven piles. It is given as the sum of the load carried at the pile tip (end bearing capacity) and the total frictional resistance (shaft friction) derived from the soil-pile interface:

$$Q_r = Q_f + Q_p$$

where:
Q_p is the representative[2] value of the end bearing capacity (for compression loads only)
Q_f is the representative value of the total skin friction resistance

A wide range of empirical approaches are currently used to calculate the representative values for end bearing and skin friction for piles in sand. Four CPT based design methods for design of driven piles in siliceous sand are discussed in ISO 19902:2007 [4]. For the present paper the ICP method [3] is used for the calculation of the ultimate pile capacity. In ISO 19902 a simplified version of this method has been included, but here the original publication has been used.

The experiments carried out by ICP showed that a failure of the local shear stresses acting on the pile shaft, τ_f, in sand follows the simple Coulomb failure criterion:

$$\tau_f = \sigma'_{rf} \cdot \tan \delta_{cv}$$

δ_{cv} is the ultimate value of the operational interface angle of friction which is developed when the soil at the interface has ceased dilating or contracting. This value is adopted for sands, which are investigated here.

It was also recognized that the radial effective stress is a function of the radial stress after installation and equalization σ'_{rc} (when pore pressures and radial stresses are relatively stable), the change in radial effective stress developed during pile loading $\Delta\sigma'_{rd}$ and the interface friction angle δ_{cv}:

$$\sigma'_{rf} = \sigma'_{rc} + \Delta\sigma'_{rd}$$

$$\tau_f = (\sigma'_{rc} + \Delta\sigma'_{rd}) \cdot \tan \delta_{cv} \qquad \text{(in compression)}$$
$$\tau_f = 0.9 \cdot (0.8 \cdot \sigma'_{rc} + \Delta\sigma'_{rd}) \cdot \tan \delta_{cv} \qquad \text{(in tension)}$$

The change in the radial effective stress, $\Delta\sigma'_r$, is associated with constrained interface dilation during pile loading and induced by a radial displacement, Δr, that must develop in the thin interface shear zone for slip to occur. Δr is similar in magnitude to the average peak-to-trough centre-line roughness of the pile surface, $2R_{cla}$. The method is explained in detail in [3].

The outcome of this method are peak (failure) shaft friction values along the length of the pile. These peak shaft friction values are used as the basis for the cyclic loading assessment.

[2] In ISO 19902 it is defined that the characteristic value is the main representative value

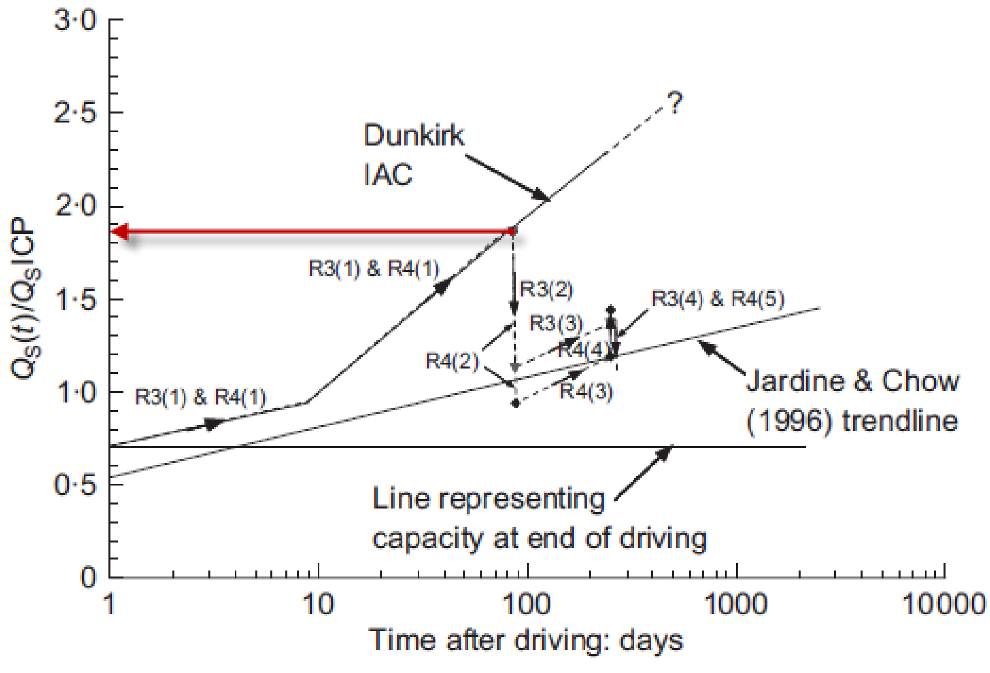
For this study only tension tests have been evaluated. End-bearing and internal friction are therefore not discussed further, although they can also be included in the model as well.

2.1.2. Time dependent effects

The ICP method predicts capacity of a pile around 50 days after driving. The effect of time on pile capacity can be quite significant, see e.g. [3], [5] and [6]. Fig. 1 shows normalized pile capacities against time as presented in [6] along with explanations on the effect of different loading histories. "Ageing"-factors of more than 2 can be achieved compared to the 50-day-capacity. In comparison to the capacity directly after driving, a factor of more than 4 has been found in tests.

It has also been found that "fresh" piles with intact ageing characteristics (IAC) show higher capacity compared to piles which have been tested to failure. This is important for interpretation of results later.

For piles R4 & R6, an ageing factor of around 1.85-1.90 is read from Fig. 1 (a factor of 1.92 is stated in [10]). It should be noted that this ageing factor has been determined by back-calculation against calculations from Jardine/Standing, i.e. it can not be directly transferred to other methods.



R3(1): virgin path for pile R3 (end point estimated to be similar to that for R6)
R3(2): decrease in capacity from two phases of cycling testing (end point proven by second test)
R3(3): increase in capacity (end point proven by third test)
R3(4): decrease in capacity indicted by brittle response of third test to failure

R4(1): virgin path for pile R4 (end point estimated to be similar to that for R6)
R4(2): decrease in capacity from two phases of high-level cycling (end point proven by second test)
R4(3): increase in capacity (end point estimated by drawing line parallel to that proven for R3(3))
R4(4): increase in capacity from low-level cycling (end point proven by third test)
R4(5): decrease in capacity indicated by brittle response of third test to failure

Fig. 1: Increase of pile capacity with time (from [6])

2.2. Load transfer algorithm as implemented in RATZ

Randolph (2003) has implemented a load transfer approach in a computer program RATZ [7]. The following description has been taken from the RATZ manual [7], see also [8].

Figures and formulas are also taken from [7], hence notation is not fully consistent throughout this paper. The main difference is that pile displacement is typically given by parameter “z” (as in t-z-curves), but “w” is used in RATZ.

The load transfer (or t-z) method is probably the most widely used technique to design single axially loaded piles and to predict the load-settlement relationships of piles subjected to axial load because of the simplicity and explicit concept. This method is useful in particular when the soil surrounding the pile is stratified because the approach offers great flexibility to handle soil non-homogeneity. Each soil layer is idealized through his own load transfer curve.

The overall deformation response of an axially loaded pile depends on the axial compressibility of the pile as well as on the mechanical properties of the surrounding soil. A load transfer curve, which describes the relationship between the unit resistance transferred from the pile to the surrounding soil and the displacement of the pile relative to each soil layer, is required in the analysis. The method models the reaction of soil surrounding the shaft using localized springs: a series of springs along the shaft (the t-z curves) and a spring at the tip or bottom of the shaft (the Q-z curves), that represents the tip resistance of the pile. Preferably, the nonlinear stress-strain behavior of the soil should be incorporated into the t-z curve. Adequate t-z and Q-z curves are essential in this method to obtain reliable settlement and load transfer calculations for axially loaded single piles.

The input parameters for RATZ are the following:

- Basic geometric parameters of the pile (inner and outer diameter, embedded depth and Young’s modulus of the pile material)
- Load and load type

The key load transfer parameters that affect the pile performance are:

- The peak and residual values of shaft friction (τ_p, τ_r) of each soil layer, if the soil is stratified
- The initial and unload-reload gradients of the load transfer curve (k)
- The rate of degradation under monotonic and cyclic loading
- The level of cyclic loading threshold (ξ) below which no degradation will occur, i.e. a yield criterion that models unloading and reloading.

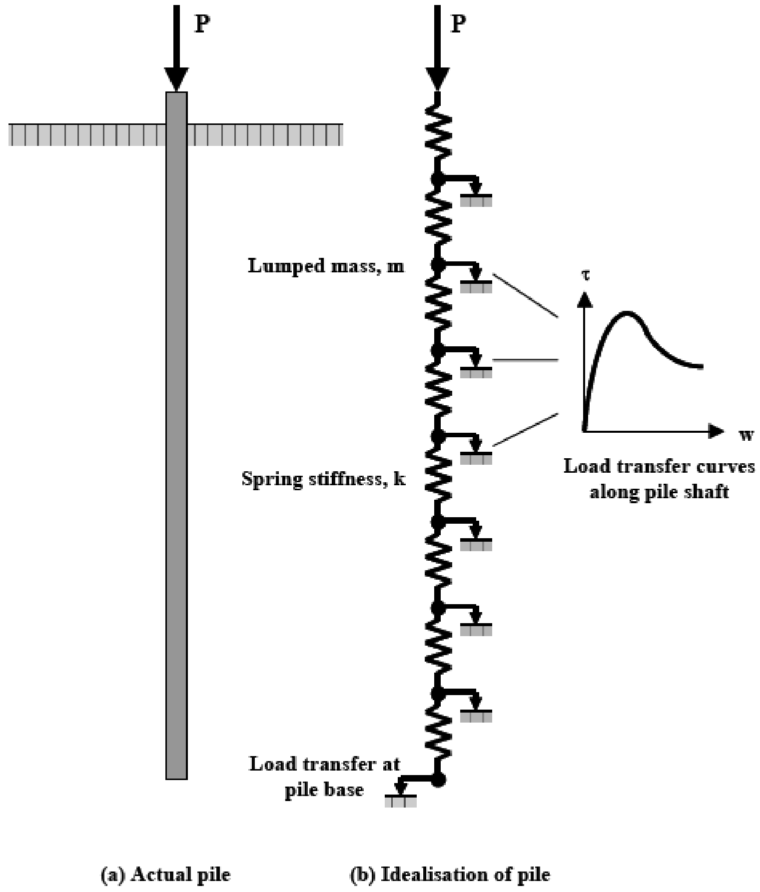


Fig. 2: Idealisation of pile in load transfer analysis [7]

In the program a general form of load transfer curve has been adopted. The form of the curve has no precise theoretical basis. There are three main stages to the curve, see Fig. 3:

1. A linear stage (marked as "linear" in Fig. 3) where the soil deforms as a continuum, with shear strains induced according to the shear stress applied at the pile shaft. For this stage of loading, τ_0 is directly proportional to w/r_0 (the normalized pile displacement), which extends from zero shear stress up to a fraction ξ of the peak shear stress, τ_P.

2. A parabolic stage (up to Point "B" in Fig. 3), with initial gradient, k, and a final gradient of zero when $\tau_0 = \tau_P$. The peak shaft friction is reached after a displacement equivalent to 1% of the pile diameter. A hyperbolic pre-peak load transfer curve has also been implemented for monotonic loading.

3. A strain softening stage (beyond Point "B" in Fig. 3), where the current value of shaft friction is related to the absolute pile displacement by:

$$\tau_0 = \tau_p - 1.1 \cdot (\tau_p - \tau_r) \cdot \left[1 - \exp\left(-2.4 \cdot (\Delta w / \Delta w_{res})^{\eta}\right)\right]$$

where:

τ_p is the peak shaft friction

Δw is the post-peak displacement
Δw_{res} is the additional displacement required to reach the residual shaft friction
η controls the shape of the strain softening curve and lies between 0.7 (for most degradation) and 1.3 (for least degradation).

Under cyclic loading, by reversed shearing, if there has been no previous history of unloading, yield is assumed to occur at the same point as for loading, which is at a shear stress of $-\xi\cdot\tau$. The yield point for reloading and further unloading (when previous unloading has occurred) is calculated in RATZ with a standard yield criterion:

$$\tau_c = \tau_{\min} + 0.5\cdot(1+\xi)\cdot(\tau_{c-1} - \tau_{\min})$$

where:

ξ is the yield threshold
τ_c is the current value of shaft friction
τ_{c-1} is the previous value of shaft friction
τ_{min} is the minimum value of shaft friction

The yield point is assumed to move down the unloading path at a rate of 0.5(1-ξ) times the rate of the current stress-displacement point. Consequently if the pile reverses direction such that the mobilized shear stress decreases by $\Delta\tau$, then the yield stress reduces by 0.5(1-ξ) $\Delta\tau$.

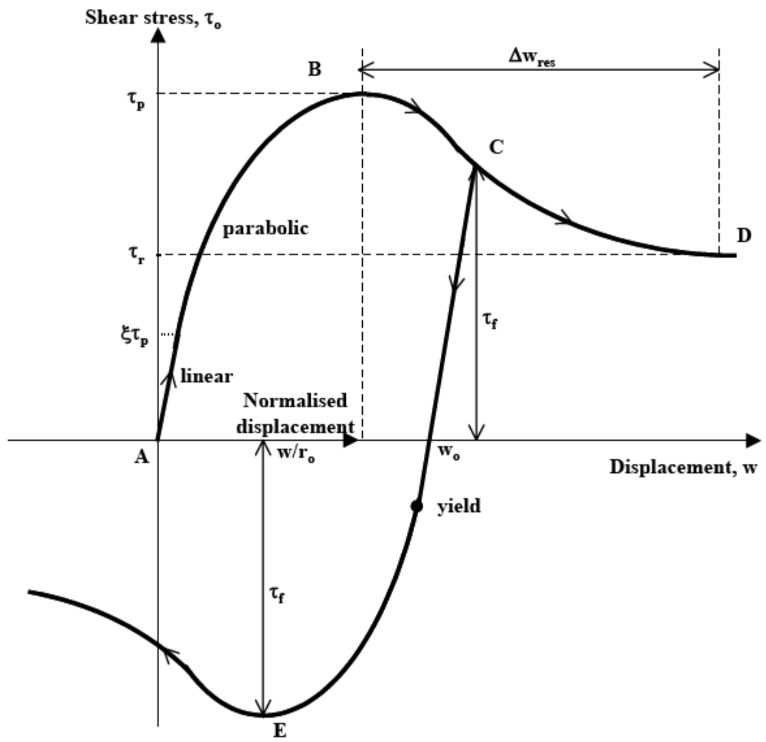


Fig. 3: Details of load transfer curve [7]

The effect of different parameters on the results has been intensively studied. Table 1 provides an overview about the parameters selected for this study. It is outside the scope of this paper to present results from these parametric studies and further justification for the individual parameters.

The shear modulus has been calculated after Baldi et al. as suggested in [3]. For static loading, the shear modulus is reduced to 30% of the dynamic shear modulus value. This is in accordance with general recommendations for reduction of dynamic vs. static shear modulus.

Parameter	Symbol	Value	Comment
Strain softening algorithm	–	–	Exponential shape adopted.
Cyclic yielding control parameter	–	–	Parabolic algorithm adopted.
Yield parameter	ξ	0.10	Conservative value chosen, larger value may be realistic for sands, see Poulos [12].
Residual shaft friction	τ_r	$0.70 \cdot \tau_p$	Taken as 70% of the peak shaft friction according to the recommendations from Poulos [12].
Displacement to residual	Δw_{res}	0.10m	This parameter has relative small influence; value assumed to be realistic.
Strain softening parameter	η	0.70	Conservative value chosen.
Cyclic residual shaft friction (see [7] for details)	$\tau_{r,cyc}$	$0.10 \cdot \tau_p$	Conservative value chosen.

Table 1: Parameters for RATZ calculation

3. New design approach to assess cyclic loading

The approach to assess cyclic loading is proposed as follows:

1. Determine peak value of shaft friction along the pile acc. to ICP method [3]
2. Apply ageing factor on peak values (if found appropriate)
3. Import peak values of shaft friction in RATZ and run cyclic loading history
4. Determine reduction in capacity from degradation ($\Delta Q_{f,cyc}$) over length of the pile
5. Determine characteristic and design capacity

One of the advantages of RATZ esp. for design of offshore wind turbines is that multi-stage loadings can be processed. No simplification to one-stage-loading is necessary, different stages with varying mean and amplitude can be used.

The reduction in capacity due to cyclic loading is performed with characteristic loads, i.e. no safety factors are applied at this stage.

Both the predicted capacity under static loading and the reduction due to cyclic loading have statistical variation. Theoretically, the distribution functions for both must be combined to derive adequate partial safety factors. This has not been done yet. It seems to be a reasonable assumption at this stage that the coefficient of variation (COV) for the predicted capacity after reduction due to cyclic loading is not significantly larger than without taking cyclic reduction into account, i.e. the safety factor remains unchanged.

The design capacity of the pile does then become:

$$Q_d = \frac{Q_r - \Delta Q_{f,cyc}}{\gamma_{R,Pe}}$$

The partial safety factor as per ISO 19902 is $\gamma_{R,Pe}$ = 1.25. This appears to be more than adequate for offshore wind turbines as Jardine et al. [3] state that a safety factor of 1/0.85=1.18 has been determined to achieve an annual probability of failure of $P_f=5\cdot10^{-4}$ when using the ICP method. Higher safety factors for ULS, as required in Germany, do not appear to be required from a safety point of view. In this context it is interesting to note that requirements as stipulated for offshore wind turbine foundations in Germany, e.g. load tests, are not foreseen in ISO 19902.

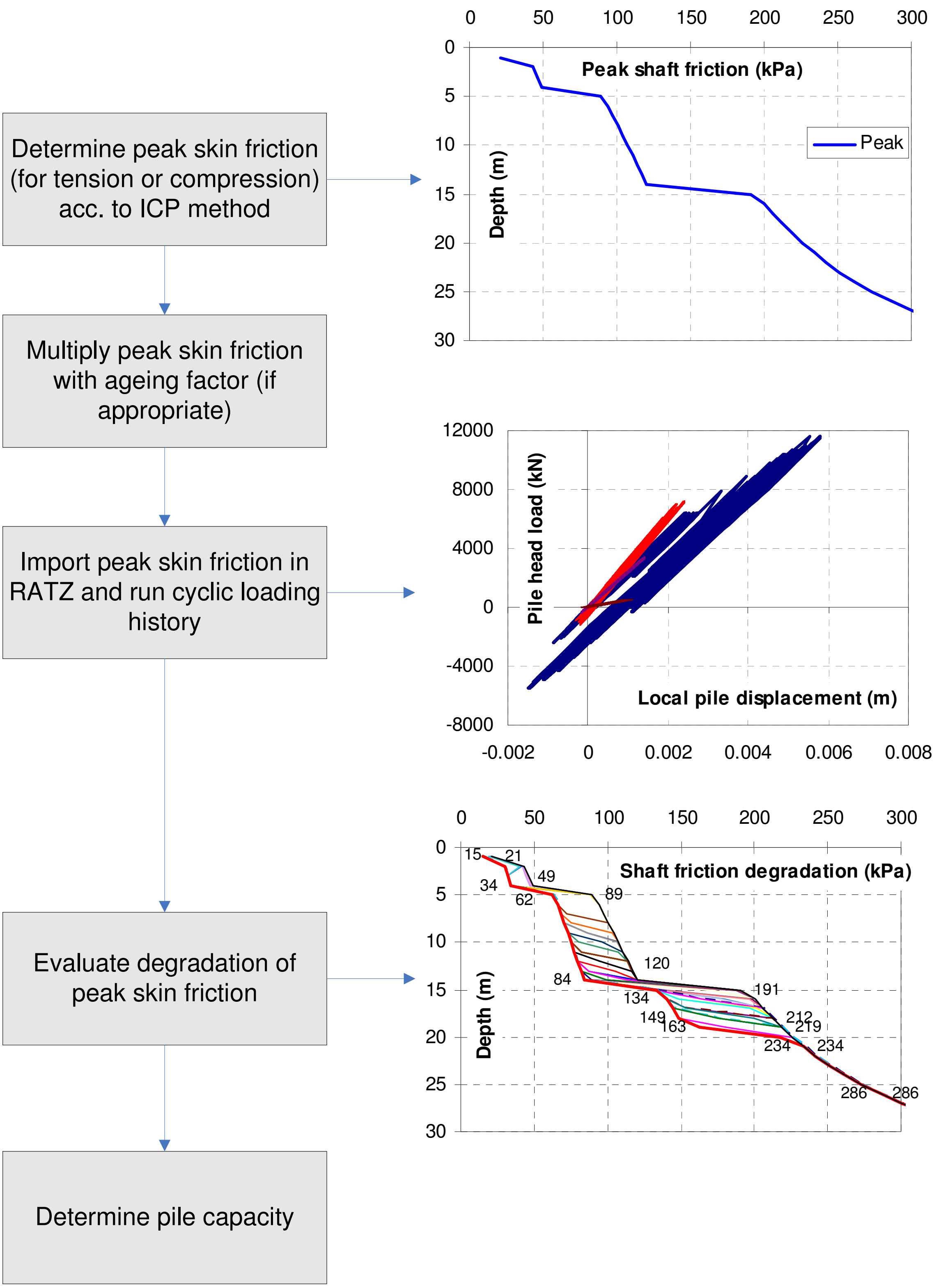


Fig. 4: Flow-chart for proposed design approach

4. Dunkirk tests

The pile tests which are described in [9] and [10] are part of the GOPAL (Grouted Offshore Piles for Alternating Loading) research project, which has been extended by a special study funded by HSE on cyclic loading.

The pile load tests performed in Dunkirk are described in detail in [9] and [10]. A summary of the most relevant data for pile R3 is given in the following.

Pile data:

- Outer diameter: D=457mm
- Wall thickness: 20mm (top 2.5m); 13.5mm (lower 18m)
- Embedded length: 19.24m
- Steel grade: ST52 (top 2.5m), ST44 (below)

Comparative calculations have been made for piles C1, R2, R3, R4, R5 and R6. The results were generally of similar quality. Pile R3 is documented in more detail in this paper.

The soil conditions (below an upper layer of sand fill) in the upper 30m consist of Flandrian Sand, which is a marine sand of varying density. Details can be found in [9]. Fig. 5 shows the CPT profile for site R3, together with the approximation adopted for the ICP capacity prediction.

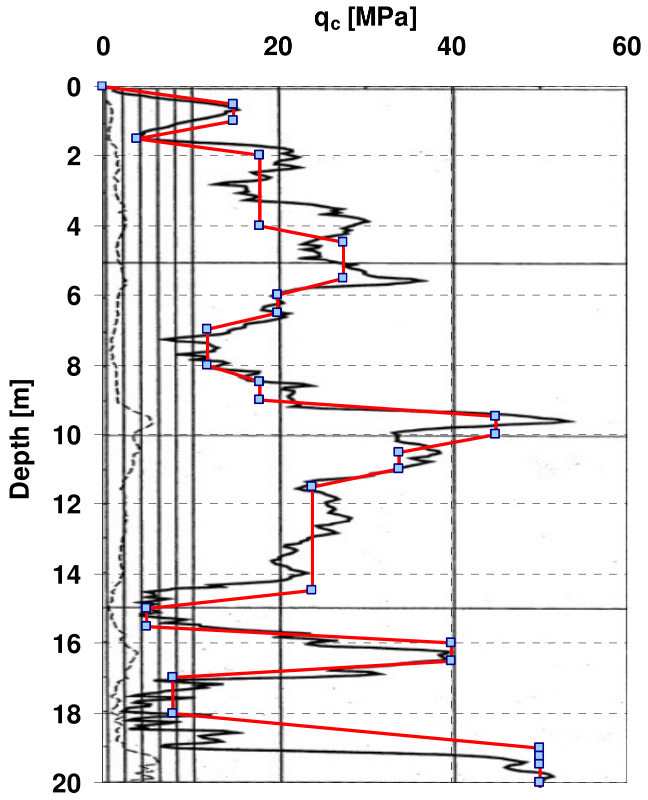


Fig. 5: CPT profile for location R3 and idealization for ICP method

The loading history for pile R3 is summarized in Table 2.

Date	Test	Applied loading	Outcome	Comment
20/08/98		None		Pile driven
29-30/10/98	–	Tension from loading reaction JP1		Reaction forces (Loading 70 days after time of first driving)
13/11/98	2.R3.T1	Tension test	Slow static maintained loading test; max. load of 2000kN applied, pile head displacement 10.3mm	Loading stopped before failure (Loading 85 days after time of first driving)
14/11/98	2.R3.CY2	Cyclic loading 0 to 1400 kN (tension)	Cyclic loading with loading rate of app. 1 minute per cycle; N=200 with a permanent displacement of 6.8mm	Test stopped after 200 cycles without failure
15/11/98	2.R3.CY3	Cyclic loading 0 to 1900 kN (tension)	Cyclic loading with loading rate of app. 2 minutes per cycle; failure (based on mean displacement, cyclic displacement and load) after N=12 with a rapid drop of capacity to 1700kN on 13th cycle	Unable to reach 1900kN at end of test. Capacity on rapid loading afterwards ~1650kN max.
20/04/99	2.R3.T5	1986 kN		Re-test after 156 days, substantial recovery can be seen

Table 2: Loading history of pile R3 (after [10])

5. Validation of the proposed approach with the Dunkirk test results

Pile R3 has been used for this paper for comparison. This is one of the piles which have not been tested to failure prior to the cyclic tests. This avoids any difficulties when trying to interpret results from a pre-failed pile.

In general, interpreting the results from [9] and [10] requires careful consideration of the loading sequence and the initial capacity of the pile at the beginning of each test. As many of the tests (static or cyclic) have been executed to failure, the capacity following one or two days after such failure must be assessed. While it is possible to determine a degraded capacity after cyclic loading with RATZ, this can not be expected to be reliable when the pile has completely failed and has undergone several centimetres of displacement.

Nevertheless, an attempt has been made to analyze the sequence of tests with relatively good success (as will be shown in the following).

5.1. Static load test on pile R3

Acc. to the description in the report, the test 2.R3.T1 is a "slow static maintained loading test". It can be seen in the load-displacement curve that the displacement becomes nonlinear beyond around 1000 kN, but failure at maximum load of 2000 kN is apparently not yet reached.

Regarding the pile capacity, the following can be stated:

- Initial estimates of pile capacity were made by Jardine & Chow acc. to the procedure proposed in 1996 [11]. The "85-day first test" capacity estimated for pile R3 with this method was 1431 kN acc. to [10], which is far less than the actual capacity of the pile. In [10] the estimated capacity of the pile is stated as 2320 kN, i.e. there is a factor of 1.62 compared to the 1431 kN estimate.

- The capacity predicted with the ICP method [3] is 1560 kN, incl. 60kN pile weight and internal soil. Comparing this with the estimated capacity stated in [10] yields an increase factor of 1.49, which is the ageing factor for this method.

- If an aging factor of 1.85 as per Fig. 1 is applied to the ICP prediction, then the estimated capacity becomes 2775 kN (in RATZ the weight of soil and pile is disregarded, so here only 1.85·1500 = 2775 kN are considered).

The load-displacement curve is computed in RATZ, using the peak value of shaft friction as determined with the ICP method as input. Additionally, an ageing factor is applied. A comparison using an ageing factor of 1.85 is shown in Fig. 9, left chart. It can be seen that the deformation is captured well until 1500 kN, but the deformations at the maximum load are under predicted. This indicates that the total assumed capacity of 2775 kN is too high.

When the predicted capacity as stated in [9] of 2320 kN is used as an input (see Fig. 9, upper right chart), then the deformation at maximum load of 2000 kN is slightly overpredicted.

For the following comparisons both ageing factors are considered and results are compared.

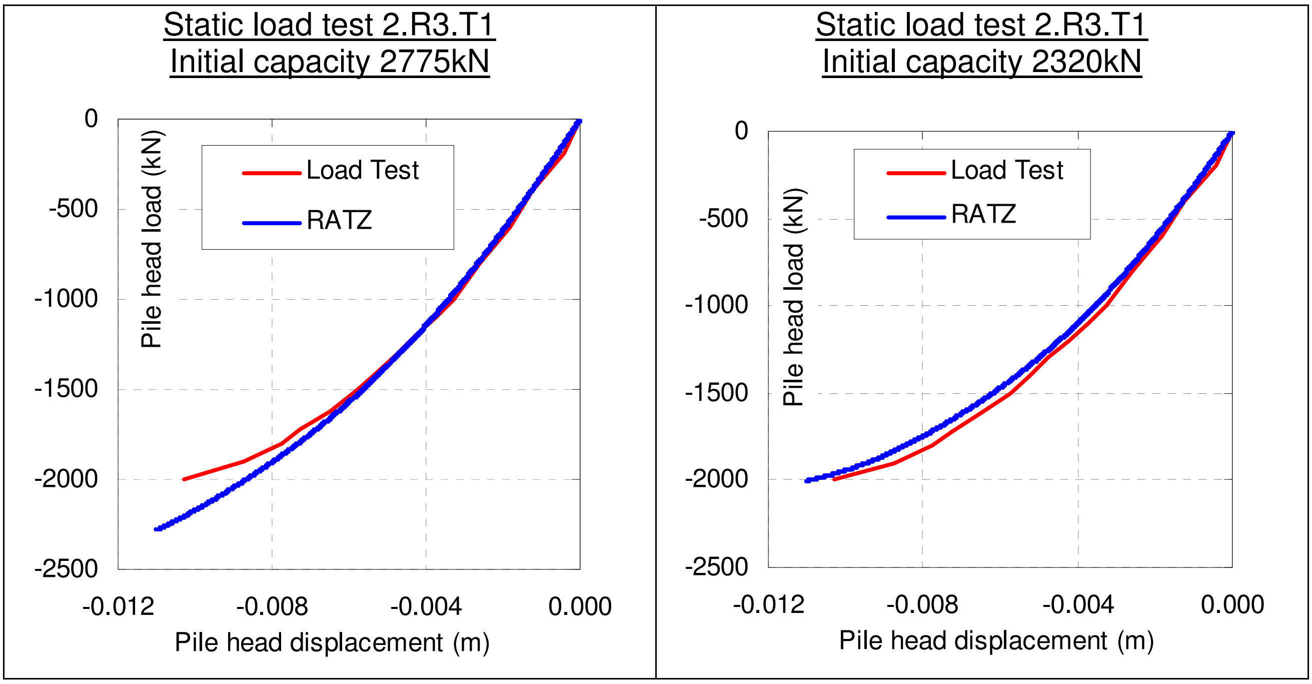


Fig. 6: Comparison of results from RATZ with tests for pile R3 (static test)

5.2. Cyclic loading tests on pile R3

5.2.1. Test 2.R3.CY2

For the cyclic loading tests, the input value for the initial capacity has been adjusted such that the results from the cyclic testing were best reproduced. Results are shown in Fig. 7. For the results computed with RATZ, only minimum and maximum values for each cycle are shown. These data points appear to be connected because they are so close together.

For test 2.R3.CY2 it was found that a reduction to 94% of the initial capacity of 2775 kN gives a good fit (left part of Fig. 7). This appears to be a reasonable input based on the assessment of static capacity discussed above. The test is qualitatively and quantitatively well reproduced, with a tendency to somewhat conservative results. The additional displacement per cycle is very well reproduced, while the displacement range is underestimated. This may be related to some degradation in the upper layers from the previous high static load. As the reduction of 94% has been applied evenly along the length of the pile, such an effect can not be captured here.

Using the predicted capacity of 2320 kN as initial capacity returns even more conservative results as shown in the right part of Fig. 7. Still the general behaviour is reproduced well.

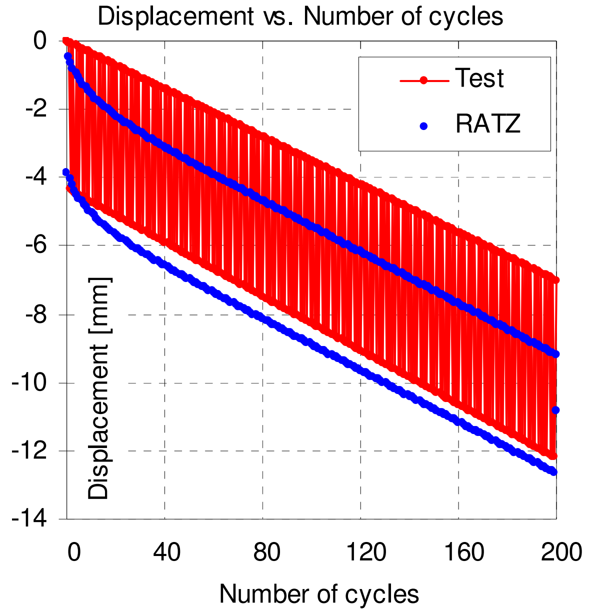
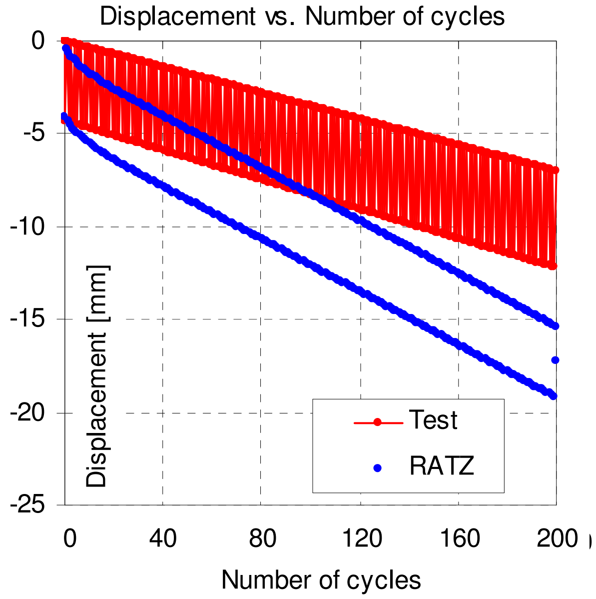
Cyclic load test 2.R3.CY2 Initial capacity 94% · 2775 kN = 2606 kN	Cyclic load test 2.R3.CY2 Initial capacity 2320kN
	
Degraded capacity: 2237 kN (-14%)	Degraded capacity: 1820 kN (-12%)
Estimated capacity after test as stated in [9] or [10]: 2050 kN – the results obtained here are hence +-10% to the estimated value, depending on chosen initial capacity.	

Fig. 7: Comparison of results from RATZ with tests for pile R3 (cyclic test 2.R3.CY2)

The skin friction over depth (initial peak values and degraded values) for this test is shown in Fig. 8. It can be seen that degradation is most pronounced in the upper layers, while lower layers show less degradation. This is to be expected due to the elastic behaviour of the pile, which leads to larger displacements near pile top. Integration of the degraded skin friction values along the pile yields the degraded capacity as given below the time histories of cyclic loading in Fig. 7.

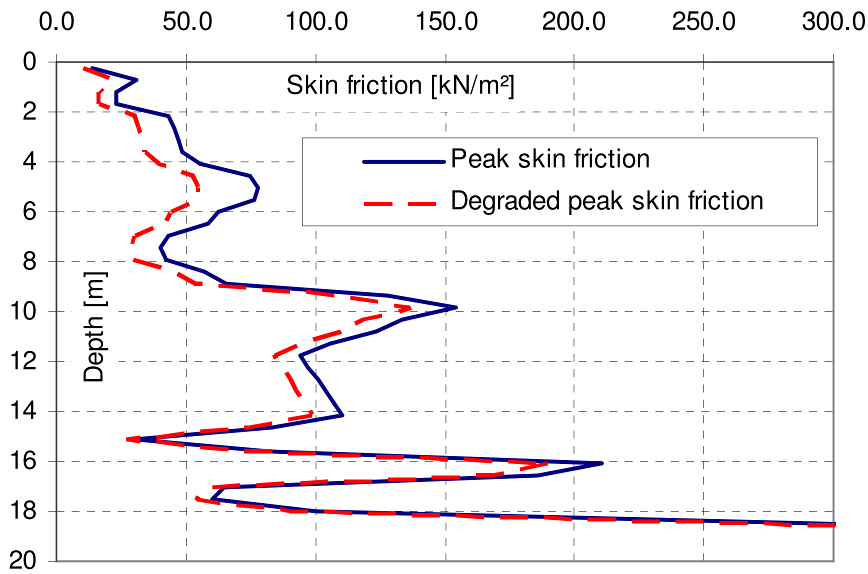


Fig. 8: Skin friction profiles over depth after test 2.R3.CY2

5.2.2. Test 2.R3.CY3

The degraded capacity received from test 2.R3.CY2 was used as input for the next calculation. The reduction was applied as a constant factor over depth, rather than applying the reduction in each depth. This may be the more logical approach and could be an interesting alternative to investigate for further studies.

The displacement vs. number of cycles for test 2.R3.CY3 is shown in Fig. 9. The following can be seen:

- The agreement between the test result and the RATZ simulation is extremely good for the initial capacity of 2237 kN. The progressive failure is predicted correctly.
- For the lower initial capacity of 1820 kN, the pile fails completely after the first cycles and excessive deformations are computed (note the different scale on the y-axis). The pile is completely degraded and only a reduced load is actually applied in each time step.

The exact prediction of displacements is quite sensitive to the initial (peak) capacity near failure. The load-displacement-curve can actually be matched exactly be slightly increasing the initial capacity over the value shown here.

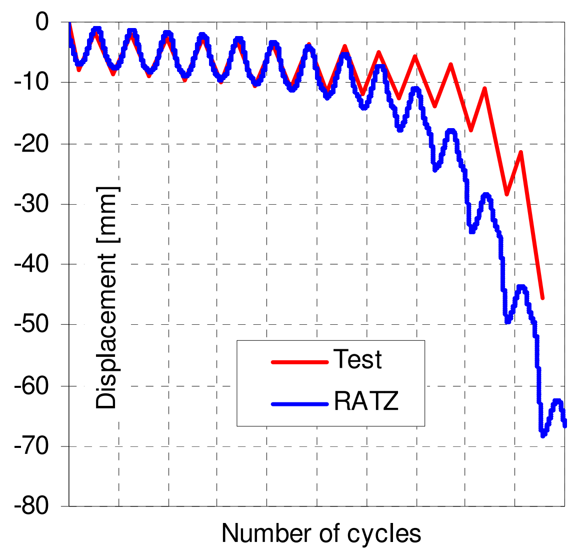
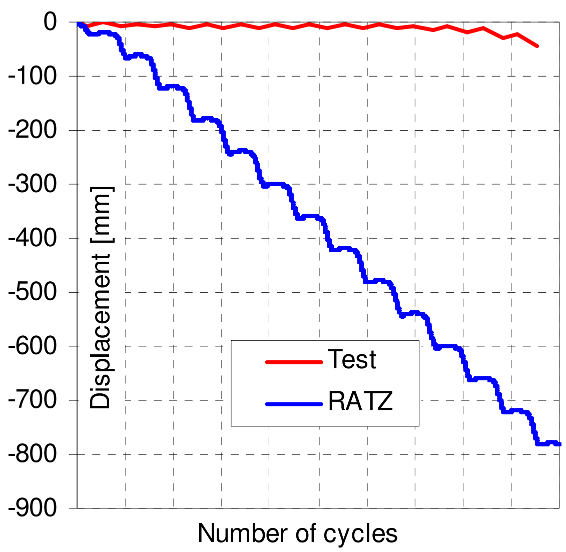
Cyclic load test 2.R3.CY3 Initial capacity: 2237 kN	Cyclic load test 2.R3.CY3 Initial capacity: 1820 kN
	
Degraded capacity: 1595 kN (71%)	Degraded capacity: 1281 kN (70%)
Estimated capacity after test as stated in [9] or [10]: 1650 kN – results obtained here are hence conservative	

Fig. 9: Comparison of results from RATZ with tests for pile R3 (cyclic test 2.R3.CY3)

In [9] it is stated that the load of 1900 kN could not be reached after the 12th cycle. The same is apparent in the RATZ simulation, see Fig. 10. Here, the full load can only be mobilized in the first eight cycles. A drop to 1620 kN is seen in the last cycle, which again corresponds nicely to the test result, where 1700 kN are reached.

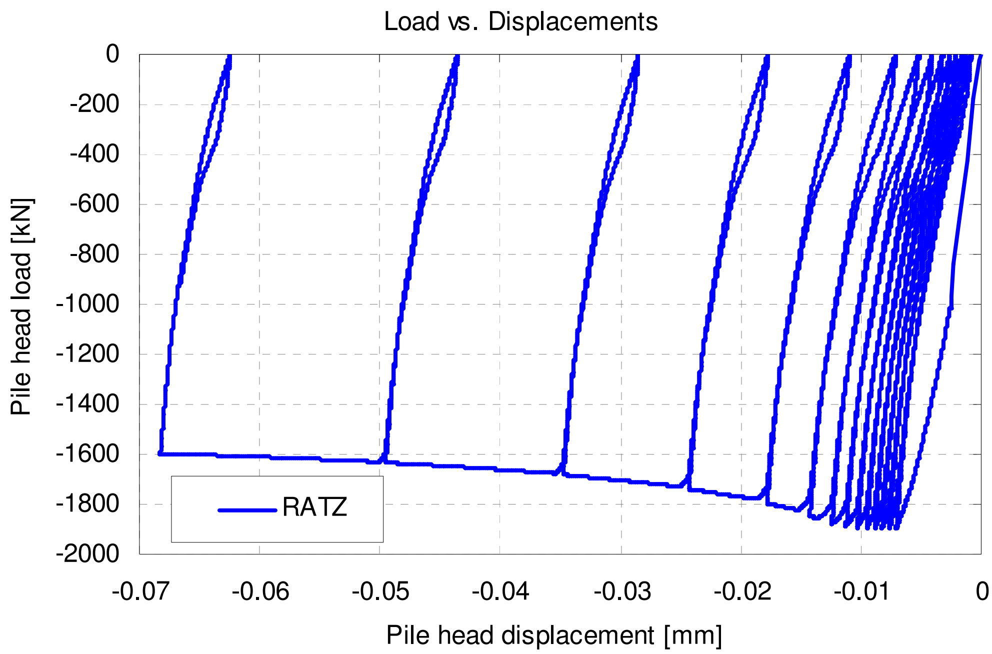


Fig. 10: Load vs. displacement as simulated with RATZ for test 2.R3.CY3

5.3. Comparison to other tests

More results are shown in Table 3 for piles R2, R4 and R5. The results can be commented as follows:

- Line 4: Estimated capacity has been given in [9], Table 5.3
- Line 9: The initial capacity for the RATZ calculations has been fitted such that the cyclic test is qualitatively well reproduced. It can be seen that for tests 2.R4.CY2 and 2.R5-CY2 slightly higher values compared to the estimated capacity have been used. This shows that the approach is conservative – had the estimated capacities been used, then the failure had occurred earlier.
  For test 3.R2.CY2 the input capacity is 72% of the previous failure load of the pile. As the pile has been tested to failure with 30mm displacements, it is not unexpected that the initial capacity for a subsequent test is substantially less than the previous failure load.
- Line 10: Both tests show high degradation, for test 2.R5.CY2 the pile is nearly fully degraded to the residual skin friction of 70% peak skin friction.
- Line 15: Here, initial capacity for the next calculation had to be chosen lower than calculated in the previous run (result from line 10). While this could be criticized as non-conservative, it should be borne in mind that the piles had previously failed and had undergone relatively large displacements (2.R4.CY2: 28mm, 2.R5.CY2: 32mm). As mentioned before, it is unlikely under such circumstances that the capacity of the pile can be reliably judged. Also disruption of the ageing process as discussed in section 2.1.2 can be of relevance. Seen in this light, the result is quite good.
- Line 20: Static capacity after the last cyclic test is predicted very well, with around 10% error on the conservative side.

<table>
<tr><th>Line</th><th>Test</th><th>3.R2.T1</th><th>2.R4.T1</th><th>2.R5.T1</th></tr>
<tr><td>1</td><td>Date</td><td>1999-04-18</td><td>1998-11-16</td><td>1998-11-19</td></tr>
<tr><td>2</td><td>Type</td><td>Static test
(failure)</td><td>Static test
(no failure)</td><td>Static test
(no failure)</td></tr>
<tr><td>3</td><td>Applied load</td><td>3147 kN
@ 30mm</td><td>2000 kN
@ 8.73mm</td><td>2000 kN
@ 8.86mm</td></tr>
<tr><td>4</td><td>Est. capacity [9]</td><td>N/A</td><td>2960 kN</td><td>2464 kN</td></tr>
<tr><td>5</td><td>Test</td><td>3.R2.CY2</td><td>2.R4.CY2</td><td>2.R5.CY2</td></tr>
<tr><td>6</td><td>Date</td><td>1999-04-18</td><td>1998-11-17</td><td>1998-11-20</td></tr>
<tr><td>7</td><td>Type</td><td>Cyclic
(failure)</td><td>Cyclic
(failure)</td><td>Cyclic
(failure)</td></tr>
<tr><td>8</td><td>Applied load</td><td>0 kN
2000 kN</td><td>0 kN
2000 kN</td><td>500 kN
2000 kN</td></tr>
<tr><td>9</td><td>Initial capacity for RATZ</td><td>2276 kN
(72% of
failure load)</td><td>3026 kN
(102% of
est. capacity)</td><td>2780 kN
(113% of
est. capacity)</td></tr>
<tr><td>10</td><td>Degraded capacity</td><td>1714 kN
(75% of
initial capacity)</td><td>2325 kN
(77% of
initial capacity)</td><td>1991 kN
(72% of
initial capacity)</td></tr>
<tr><td>11</td><td>Test</td><td rowspan="6">Only one cyclic test performed for this pile</td><td>2.R4.CY3</td><td>2.R5.CY3</td></tr>
<tr><td>12</td><td>Date</td><td>1998-11-18</td><td>1998-11-21</td></tr>
<tr><td>13</td><td>Type</td><td>Cyclic
(failure)</td><td>Cyclic
(failure)</td></tr>
<tr><td>14</td><td>Applied load</td><td>500 kN
2000 kN</td><td>0 kN
1400 kN</td></tr>
<tr><td>15</td><td>Initial capacity for RATZ</td><td>2058 kN
(89% of
last result)</td><td>1674 kN
(84% of
last result)</td></tr>
<tr><td>16</td><td>Degraded capacity</td><td>1533 kN
(74%)</td><td>1230 kN
(73%)</td></tr>
<tr><td>17</td><td>Test</td><td>2.R3.T3</td><td>2.R4.T4</td><td>2.R5.T4</td></tr>
<tr><td>18</td><td>Date</td><td>1999-04-18</td><td>1998-11-18</td><td>1998-11-21</td></tr>
<tr><td>19</td><td>Type</td><td>Static test
(failure)</td><td>Static test
(failure)</td><td>Static test
(failure)</td></tr>
<tr><td>20</td><td>Applied load</td><td>1655 kN
(97% of
predicted)</td><td>1625 kN
(106% of
predicted)</td><td>1350 kN
(110% of
predicted)</td></tr>
</table>

Table 3: Comparison for piles R2, R4 and R5

6. Conclusions and Outlook

A new approach to assess cyclic loading for axially loaded piles has been presented. The method combines well proven methods for static and cyclic performance.

Very encouraging results have been achieved with this approach. Deflections, cyclic failure mechanisms and magnitude of reduction in capacity are all predicted with good accuracy. Partially results are strikingly identical to the tests. It can thus be concluded that the tendency of a pile to fail under cyclic loading can be reliably predicted with RATZ when the input values for the peak skin friction are realistic. This is considered to be the case when the peak skin friction as predicted by the ICP method is used.

The methodology can of course be further refined. It would e.g. be logical to use the degraded profile from one test as input for the next. In this study, the reduction was applied based on overall reduction and peak shaft friction was reduced proportionally over the entire length of the pile. The difficult question how pile capacity after failure is assessed must then be answered.

One of the main assumptions for the approach is the choice of the maximum reduction due to cyclic loading, which is established by the post-peak residual value of skin friction. This has been set to 70% of the peak value for this study and very good agreement with the tests performed in Dunkirk could be demonstrated. As this value also complies with recommendations made by Poulos [12], it seems to be a reasonable value to adopt for design purposes, until further research gives more guidance.

The presented methodology does appear an appropriate tool to fulfill BSH requirements regarding assessment of cyclic loading for offshore wind turbines in typical North Sea sands.

Further research should investigate the very high loading rate as seen for offshore wind turbines, see [13]. This could have a substantially positive effect on capacities. Furthermore, recovery from (temporary) loss of capacity due to high cyclic loading should be investigated.

7. References

[1] BSH: Standard "Design of Offshore Wind Turbines", BSH No. 7005, 20 December 2007 ("BSH Standard Design"), available for download at: http://www.bsh.de/en/Products/Books/Standard/index.jsp (English)

[2] BSH: Standard Konstruktion, Geotechnische Anwendungshinweise sowie EA Pfähle 2011 Entwurf Kap. 13 einschl. Anhang D http://www.bsh.de/de/Produkte/Buecher/Standard/Anlage1.pdf

[3] Jardine, R., Chow, F., Overy, R. and Standing J. ICP Design methods for driven piles in sands and clays. Thomas Telford, London 2005.

[4] DIN EN ISO 19902:2008-07: Petroleum and natural gas industries - Fixed steel offshore structures (ISO 19902:2007); English version EN ISO 19902:2007

[5] Chow, F.: Time effects on piles and on their design methods. International Symposium on Frontiers in Offshore Geotechnics, Perth 2005. http://www.isfog.civil.uwa.edu.au/chow.pdf

[6] Jardine, R. J.; Standing, J. R: Chow, F. C.: Some observations of the effects of time on the capacity of piles driven in sand. Géotechnique 56, No. 4, 2006.

[7] Randolph, M. F.: RATZ Version 4-2: Load Transfer Analysis of Axially Loaded Piles, 2003.

[8] Erbrich, C. T. et al.: Axial and lateral pile design in carbonate soils. In: Frontiers in Offshore Geotechnics II, Taylor & Francis Group: London 2011.

[9] Jardine, R.J.; Standing, J.R.: OTO 2000 008 - Pile Load Testing Performed for HSE Cyclic Loading Study at Dunkirk, France; Volume 1 www.hse.gov.uk/research/otopdf/2000/oto00008a.pdf

[10] Jardine, R.J.; Standing, J.R.: OTO 2000 007 - Pile Load Testing Performed for HSE Cyclic Loading Study at Dunkirk, France; Volume 2 www.hse.gov.uk/research/otopdf/2000/oto00007.pdf

[11] Jardine, R.J.; Chow, F.C.: New design methods for offshore piles. MTD publication 96/103, Marine Technology Dept. London 1996.

[12] Poulos, H. G.: Marine Geotechnics. Unwin Hyman, London, 1988.

[13] Seidel, M.: Load characteristics of axially loaded jacket piles supporting offshore wind turbines. Proceedings for Workshop „Gründung von Offshore-Windenergieanlagen“, Karlsruhe 2010. http://www.marc-seidel.de/Papers/Seidel_Workshop_Foundations_2010.pdf