Earthquake Geotechnical Engineering for Protection and Development of Environment and Constructions – Silvestri & Moraci (Eds)
 ISBN 978-0-367-14328-2

Simple performance-based pile foundation assessments for risk-based evaluations of existing structures

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ABSTRACT: Extensive gas production in the Groningen field, The Netherlands, has caused an increasing seismicity in the region. Local buildings have not been designed to withstand any seismic load, emerging the necessity for evaluation of their seismic performance. The newly developed Dutch seismic design code, NPR 9998:2018 allows for damages, as long as life safety risks remain within acceptable limits. The same counts for the foundations, the performance of which is expressed in terms of maximum acceptable distortions/settlements, such that they don't pose risk for building collapse. The large number of buildings to be assessed suggests the development of a simple performance based assessment framework, instead of elaborate nonlinear time history analysis for each individual building. This paper describes some recent developments in approaches to GEO and STR limit state evaluations for pile foundations, focusing on consequences of foundation performance for the superstructure.

1 INTRODUCTION

1.1 *Background*

Due to soft soil conditions in the Groningen gas field region, buildings often have piled foundations. The Groningen seismic hazard levels are moderate and events are characterized by relatively high peak ground accelerations, low energy content and short duration. Many pile foundations have been designed using superseded design codes and most pile foundations have not been designed with significant shear capacity. Accordingly, foundation upgrading would be required based on the outcome of limit equilibrium type of calculations, both with respect to geotechnical bearing capacity and shear capacity. Foundation upgrading measures for existing structures are difficult to implement, costly and have major impact on society. Therefore foundation upgrading assessments are only acceptable if contributing to mitigation of life safety risks. Besides, foundation upgrading may in some cases have a negative effect on the performance of the superstructure, which supports a critical view towards large scale foundation upgrading programs.

1.2 *Scope*

This paper addresses simple performance based foundation assessment methods that can contribute to efficient risk-based evaluations of foundations. Two recent advantages in the definition of simple performance based verification methods towards a future NPR 9998 update are discussed. Firstly, a method to estimate seismic and post-seismic geotechnical bearing capacity and potential settlements of piles including liquefaction effects is discussed. Secondly, a system based approach to evaluate the lateral performance of pile foundation systems subject to seismic base shear is described.

2 SEISMIC AND POST-SEISMIC VERTICAL PILE BEARING CAPACITY

Specific points of attention for the seismic and post-seismic bearing capacity of piles are the occurrence of excess pore water pressure generation or liquefaction, the impact of static negative skin friction and the differentiation between shaft and base mobilization. All these factors have been integrated in a straight forward floating pile calculation framework (Fellenius, 1984), which is modified to be fully compatible to the Dutch pile bearing capacity design approach prescribed by NEN 9997-1:2016.

2.1 *Geotechnical bearing capacity*

The geotechnical bearing capacity of piles is provided by the pile tip resistance and the pile shaft resistance, who are in equilibrium with the load acting on the pile head and the negative skin friction that may act along part of the pile (Figure 1). The maximum normal force is found at the neutral plane: that location at which the relative movement between soil and pile is zero. It is possible to find the location of the neutral plane by performing an interaction calculation (e.g. Fellenius, 1984). Seismic action may lead to a transient increase of pile loads and liquefaction may reduce bearing capacity or increase negative skin friction. NPR9998 prescribes a methodology is presented that is aligned with the Dutch code of practice. Following NPR9998 the seismically induced pile settlement during the earthquake and the post-seismic pile settlement shall be calculated and added to obtain the total seismic settlement.

2.2 *Pile settlement during the earthquake*

During the earthquake, depending on the dynamic behaviour of the superstructure, there may be an increase of the pile load, while no seismic settlements have occurred. At minor downward displacement of the pile (several mm) the cumulative negative friction force F_{nk} will reverse and result in positive friction. From this perspective the following reasoning has been proposed for NPR9998 update:

- If the quasi-static temporal vertical load increment F_{dyn} is smaller than the initial cumulative negative friction force F_{nk} no analysis has to be performed and no additional settlements are expected to occur.
- If the quasi-static temporal vertical load increment F_{dyn} is larger than the initial cumulative negative friction force F_{nk}, the displacement can be determined by considering the pile settlement as a function of the combined SLS load and F_{dyn} accounting for positive skin friction along the entire shaft.

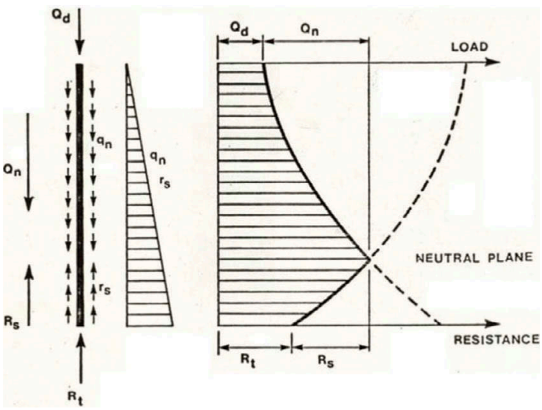

Figure 1. Pile equilibrium state comprising both tip and shaft resistance.

- If the calculated settlement from this procedure is smaller than the initial (pre-seismic) situation, then the component F_{dyn} has no influence on the pile settlement and there will be no additional settlement compared to the current situation.
- If the calculated settlement from this procedure is larger than the initial (pre-seismic) situation, then the different between the calculated settlement and the initial situation can be taken as seismic settlement.

2.3 *Pile settlement after the earthquake*

After the earthquake two phenomena are expected which, slightly conservatively, can be expected to occur simultaneously, a strength reduction represented by a reduction of the cone tip resistances by a factor of $(1\text{-r}_{u:100\%})^{1/2}$, in which r_u is the excess pore pressure ratio estimated by liquefaction triggering analysis and post-seismic consolidation settlement as a consequence of dissipating excess pore pressures. These two effects may have an effect on the post-seismic displacement of the pile s_{after}. To quantify this displacement an interaction calculation is needed in which the pile behaviour including the two phenomena listed above, are compared to the pre-seismic pile behaviour. In the NPR9998 background document a flowchart is presented that can explains the analysis procedure, which will be explained using an example presented in Figure 2. The example comprises a prefabricated rectangular 400x400 mm driven pile which is centrically loaded by a SLS load F_{rep} of 400 kN.

Figure 3a presents the cone tip resistances over depth, from which the following is obtained:

- A measure of the shaft resistance $q_{s,max}$ through the shaft friction factor α_s and cut-off limits following NEN-EN 9997-1.
- A measure of the base resistance $q_{b,max}$ according the Koppejan rule.
- A measure for the negative skin friction, which in contrast to positive friction, in the Dutch code is determined based on soil effective stresses and friction angle using a K0 approach.

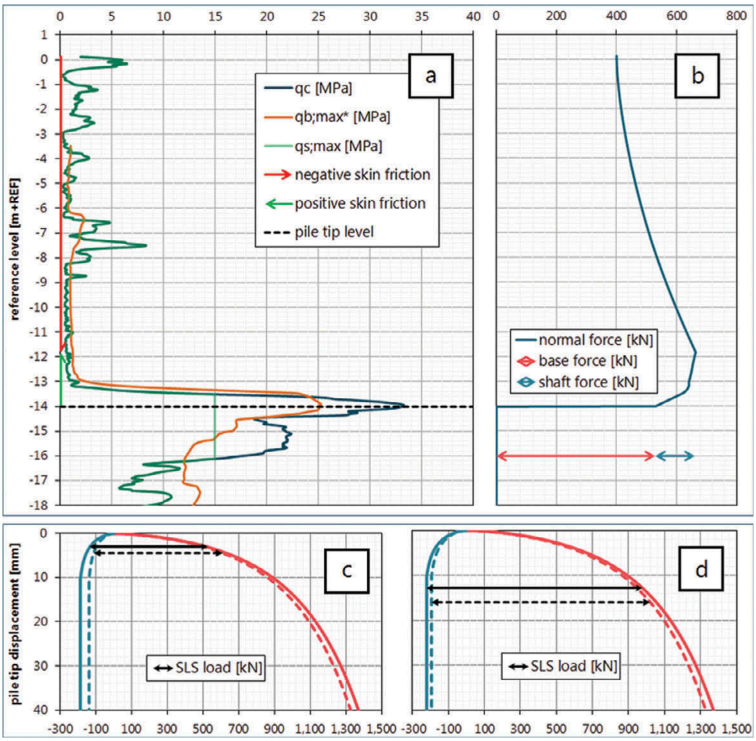

Figure 2. Graphical representation of the pile interaction calculation.

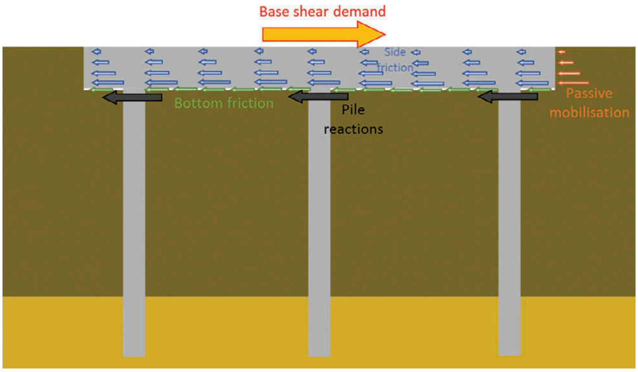

Figure 3. Mechanisms contributing to the lateral stability of a pile foundation system.

Figure 3b presents the normal force in the pile. The distribution of forces along the pile shaft and pile base is determined by the combined mobilisation curves given in the Dutch code as shown by the continuous curves in Figure 2c:

– Shaft mobilisation is a function of displacement of the pile and is obtained by integration of $q_{s,max}$ over the length of the positive friction zone.
– Base mobilisation is a function of the ratio of s_b over the (equivalent) pile diameter and represents a percentage of the base capacity $q_{b,max}$.
– The summation of F_{rep} and F_{nk} is represented by the continuous black arrow in Figure 2c and has to be taken by the shaft and base, resulting in a pile tip displacement s_b.

Adding the pile deformation to the pile tip displacement yields the expected vertical displacement at each depth. Given an estimated or calculated (seismic) settlement profile, the location of the neutral can be adjusted such that at the neutral plane the relative displacement between the soil and the pile is equal to zero. This interaction calculation allows for the inclusion of the liquefaction-induced strength reduction and settlement. In Figure 3c this is shown for an earthquake with a PGA of 0.30g using a 1D semi-empirical triggering procedure (Idriss & Boulanger, 2008) and the settlement estimation procedure by Yoshimine et al. (2006) where the dashed curves represent the post-seismic situation. Figure 3d shows for the same CPT and configuration the static and post-seismic pile tip displacement for the case where F_{rep} = 1000 kN. Due to liquefaction induced soil settlements the neutral plane drops, thereby reducing the maximum shaft capacity and increasing the negative friction force. In general the post-seismic pile displacements due to a strength reduction and liquefaction-induced settlement appear to be limited (< 1 cm) even for relatively high seismic loads. Liquefaction close to- or under the pile tip level may have a significant effect on the pile behaviour; significant reduction of the base resistance will lead to insufficient bearing capacity and the pile will naturally follow settlement that occurs under the pile tip level.

3 PERFORMANCE OF PILE FOUNDATION SYSTEMS SUBJECT TO SEISMIC BASE SHEAR

The work done in relation to lateral system performance evaluation of pile foundation systems comprises a foundation pushover framework for near collapse verifications. Post-shear failure behavior of piles, the effects of pile head damage to their load bearing function and the possibilities for load redistribution among piles and to foundation beams are integrated. An analogy with the Elwood & Moehle (2003) approach for reinforced concrete columns is made, with specific adjustments to cover typical conditions for foundation piles. The evaluation of potential load redistribution requires a coupling pile damage and corresponding deformations to resistance mobilization by foundation beams. This is done based on estimated pile crack angle and soil mobilization characteristics following NEN 9997-1 extended with Plaxis 2D finite element analyses.

3.1 *Contributing mechanisms*

Base shear resistance is of main interest for buildings for which overturning is irrelevant. It is often assumed that the total seismic base shear needs to be taken by the piles. Neglecting all the mechanisms apart from the piles can be considered a safe approach which is reasonable for the design of new structures. EN 1998-5 sets the requirement that piles shall be designed to remain elastic (with some exceptions for the pile head zone provided ductile structural detailing), which implies small deformation levels and therefore limited possibility for load redistribution. A more realistic approach however would be to consider all mechanisms that plays a role for a pile foundation system subject to base shear (see Figure 3). Apart from lateral pile reactions (pile shear forces) also passive mobilization to- and friction along the sides and the bottom of embedded foundation beams contribute to total system equilibrium.

3.2 *Analytical framework for lateral response of concrete piles*

The analogy of concrete piles to reinforced concrete columns and the Elwood & Moehle analytical framework has been made by several authors. In the present work this has been done as well where the column "length" parameter has been calculated as the equivalent depth of fixity equalizing 5 to 10 times the pile diameter, as function of the soil stiffness. Based on Elwood and Moehle work two possible failure mechanisms for typical reinforced concrete piles in Groningen could be likely; a shear failure mechanism and a combination of shear and flexure failure, recognizing that a pure flexural mechanism is unlikely given the low reinforcement ratio of piles in Groningen. Distinction between a shear-critical pile and a shear-flexure critical pile is made in accordance with Elwood & Moehle. In the present work the approach of EN 1992 has been followed with respect to cumulative shear capacity calculation considering concrete, shear reinforcement and dowel action. Eurocode does not allow to add these components where other codes, e.g. the Greek code for assessment of existing structures (EPPO, 2012), do allow dowel action contribution to the total capacity.

Shear capacity of concrete elements is known to be function of the axial compression stress in the member. In addition, the drift at axial failure following the Elwood & Moehle framework is function of axial load. This implies that redistribution of load from piles to foundation beams the shear capacity decreases but the "ductility" of the piles acting as vertical load bearing elements increases. The bearing capacity of the piles with respect to the drift ratio can be found by solving the equation of the drift at axial failure for the unknown P for several drift ratios. This is illustrated by Figure 4 and has been incorporated in the present pushover framework.

The rate with which the settlement (u_y) is progressing with respect to the lateral displacement at this stage is assumed equal to the lateral displacement (u_x) times tangent of the shear failure angle (θ) of the concrete pile.

For wooden piles with concrete extensions the Elwood & Moehle framework is concluded to not be directly applicable based on full scale pile test that have been executed. The results of these tests were analysed as part of the present study. Lateral deformation levels at the onset of pile shear failure observed from these tests reach values that indicate that the connections between the wooden pile and the concrete extension do not behave like they are rotationally fixed. The observed failure mechanism is a shear failure of the concrete extensions. For this reason the post shear failure relation between horizontal and vertical deformations like for concrete piles is assumed to be function of the shear failure angle.

3.3 *Analytical framework for foundation beam - soil interaction mechanisms*

The contribution of foundation beams grid to the total system response comprises different mechanisms. Side friction and passive mobilisation contribute to the total system response. The ultimate resistance and resistance mobilisation are taken in accordance with NEN 9997-1:2016.

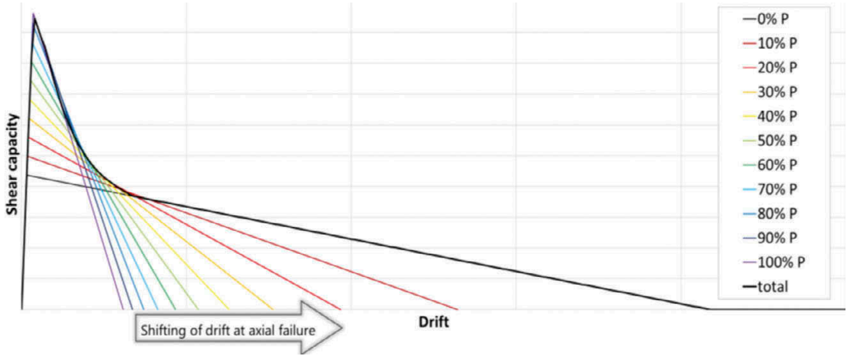

Figure 4. Interaction diagram indicating the effect of decreasing axial load on shear failure behaviour.

Friction below the bottom of foundation beams contributes to lateral response under the condition that the beams are in contact with the soil. Due to soil settlements that may have developed since construction of the building, in the pre-earthquake state there could be a gap between the foundation beam and the soil underneath. In this case foundation settlement after pile damage is required prior to mobilization of any bottom friction below a foundation beam. After the foundation beams are in contact with the soil, redistribution of load through the bottom contact area of the beams occurs. Numerical studies did indicate that this load redistribution can be very significant, causing a reduction of the initial pile load by 50%. Ultimate bearing capacity of foundation beams in the present study has been defined in accordance with strip footing bearing capacity calculated by NEN 9997-1:2016. Mobilisation characteristics for vertical and horizontal mobilisation were retrieved from numerical simulations using Plaxis 2D software. Little deformation of footings is required to mobilize interaction forces close to the bearing capacity.

3.4 *A foundation pushover framework for Near Collapse assessments*

Having identified the contributing mechanisms it is possible to include them all in a foundation pushover framework, that quantifies their contribution in terms of resistance as function of relative displacement between foundation and soil. In the present work the lateral displacement was adopted as the main deformation measure. Vertical settlements were indirectly coupled to horizontal deformations based on crack patterns of piles. Based on the evaluation of the total system capacity then the damage potential and the residual bearing capacity of piles can be evaluated. In this paragraph two cases are presented and analysed.

The first case comprises a building with plan dimensions 8.5x6.5 m, supported by 6 290x290 shear-flexure critical concrete piles and a soft clay shallow soil layer (φ'=15°, c'=1kPa). Foundation beams along the sides of the building have 60 cm embedment in the ground and width 50 cm. An initial pile axial load 200 kN, 8Φ14 longitudinal reinforcement and Φ6/150 transverse reinforcement are applied. The depth of equivalent fixity is defined to be 3.0 m (~10 pile diameters) since present soil conditions for this case classify to be very soft soil. The horizontal and vertical system performance that is calculated using the analytical pushover formulation is illustrated by Figure 5. The graphs show that in the small displacement range side friction and passive mobilisation acting on foundation beams brings approximately 50% additional lateral capacity to the system compared to solely considering the contribution of the piles. The capacity of solely the 6 piles would be approximately 250 kN when calculated based on EN 1992-1 and is limited by the plastic moment capacity. At approximately 0.1 m lateral displacement pile shear-flexural failure initiates, based on the Elwood & Moehle calculation using the parameters listed above. From this point on the shear capacity of the piles starts decreasing with increasing lateral displacement. For lateral displacements 0.25 m the piles reach their axial failure limit. In between the onset of shear failure and axial failure the vertical settlements that are associated with pile failure are calculated based on a pile shear crack angle of

25°. A relatively low angle has been selected because the failure mode is shear-flexural failure. Zero gap was considered for this case, implying that any vertical settlement results in a mobilisation of bearing capacity of the strip footings and thus in a horizontal mobilization as well. In addition, side friction and passive mobilisation against the foundation beams adds to the total lateral mobilized resistance. Combining the two graphs of Figure 5 one can conclude that for this case an exceedance of the base shear of 400 kN would imply pile failure, but both horizontal and vertical equilibrium are guaranteed due to the redistribution to beams acting as embedded strip footings, which based on calculation following NEN 9997-1:2016 provide a vertical bearing capacity equal to about 1050 kN. Long term consolidation settlements are neglected and irrelevant for seismic safety risk considerations. The calculated total vertical settlements are in the order of a few cm indicating that this will not trigger a Near Collapse limit state of the building because these do not exceed NPR9998:2017 threshold values.

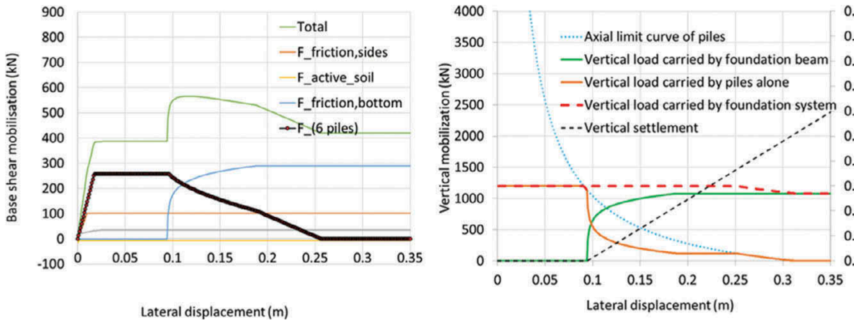

Figure 5. System response diagrams for case 1.

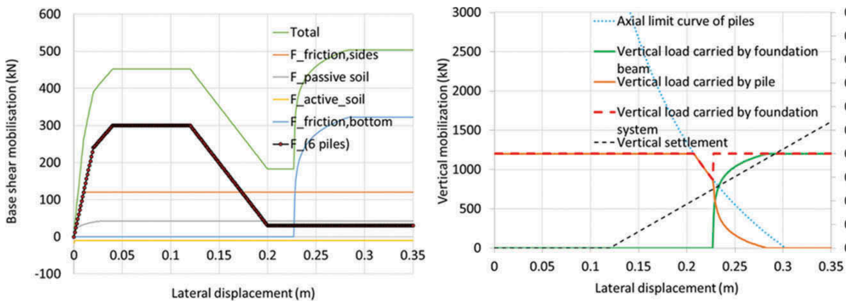

Figure 6. System response diagrams for case 2.

The second case presented in this paper comprises a 8.5x10 m building with 6 shear-critical wooden piles with concrete extensions, having diameter 290 mm. The exact reinforcement detailing of the concrete extensions is unknown. The overall lateral response characteristic of this pile type is taken in accordance with available full scale test results from the field. This in contrary to prestressed concrete piles for which Elwood & Moehle empirical relations for columns are applied. The field test data was available and is considered to give more reliable input. Soil conditions (a shallow very soft clay layer) and foundation beam depth and width are assumed to be identical to the first case presented. In this second case a gap of 5 cm between the foundation beams and the soil underneath is assumed. This implies that larger lateral deformations after shear failure of the concrete extensions are required to result foundation beams acting as strip footings. The onset of shear failure is approximately 0.12 m and axial failure of the piles themselves has been tested to happen at around 0.20 m. The total base shear capacity of the 6 piles is tested to be around 300 kN and an additional 150 kN is provided by side friction and passive mobilization along the foundation beams. The system response diagrams clearly show the effects of the (more flexible) wooden piles with concrete extension. Moreover, the effect of the gap is clear. Larger deformations are required to mobilize vertical action at the foundation beams and therewith allow for redistribution of base shear from piles to foundation beams, which occurs at about 0.23 m for the analyzed system. When the cumulative lateral capacity of the piles and the side friction and passive resistance in the undamaged state are exceeded by the total base shear then lateral displacements may increase. The transient short duration character of induced earthquakes will cap the maximum possible lateral deformations of such a system, which could possibly be evaluated based on a Newmark type calculation or alternatively based on finite element calculations. Interestingly the associated vertical displacements that could develop after the onset of piles shear failure in this case still remain relatively small. Due to the system response and the capacity of the foundation beams no severe bearing capacity failure of the foundation occurs after piles failing in shear. The total system response in this sense is stable up to base shear levels exceeding the piles lateral capacity by 50%.

4 CONCLUSIONS

This paper describes simple foundation verification methods that may not be fully performance-based dynamic, but do bring added value compared to basic limit equilibrium verifications. The presented concepts can be used to evaluate performance targets of NPR 9998 and are integrated with the Dutch design codes for geotechnical structures and the new Dutch seismic design guideline NPR 9998.

The floating pile calculation concept integrated with the Dutch geotechnical design code and liquefaction triggering calculations provides a solid approach to evaluate the risks of liquefaction induced pile settlements. The approach may appear very simple, but given the difficulties and uncertainties associated with modelling liquefaction effects on piles in FEM simulations one may argue if the much simpler method presented in this paper results lower accuracy.

The foundation pushover framework presented in this paper evaluates overall response of a pile foundation system subject to base shear and integrates contributions of both piles and foundation beams. Mobilisation of load bearing mechanisms and occurrence of damage have been coupled to both horizontal and vertical ultimate resulting deformations. Various cases with different pile and soil configurations are evaluated. The contribution of the soil reactions on the foundation beams appears to be very significant for the overall foundation performance This effectively results in a lower lateral force demand to the piles and a ductile system response with minor contribution to structure collapse risk.

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