

**ASSESSMENT OF NEGATIVE FRICTION AND OF
NEUTRAL POINT IN PILES BY 3D NUMERICAL
ANALYSES**

**KAZIKLARDA OLUŞAN NEGATİF SÜRTÜNME VE
NÖTR NOKTANIN 3B SAYISAL ANALİZLERLE
DEĞERLENDİRİLMESİ**

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ABSTRACT

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Negative skin friction (NSF), caused by greater settlement of compressible soil relative to the pile, generates downward shaft shear and dragload. At pile-supported viaduct abutments, this vertical interaction may be accompanied by embankment-induced lateral soil movement, leading to pile bending and horizontal group displacement. This study investigates the combined axial and lateral pile response associated with this mechanism using a floating 5×5 pile group subjected to one-sided surcharge in PLAXIS 3D. Piles are represented by embedded beam elements, and NSF development is simulated through consolidation. Dragload, neutral-plane depth, bending moment and lateral displacement are evaluated for each pile from the same analyses. The parametric study examines surcharge magnitude, pile spacing and length, clay thickness, embankment distance to piles, interface strength and pile-head load, with a four-sided surcharge case analyzed as a symmetric reference. Results show pronounced front-to-back differences in dragload and neutral-plane position. Under one-sided loading, bending in the loading direction may govern the structural response while the group translates away from the fill. Thus, pile-supported abutments on soft ground should be assessed as three-dimensional soil–pile–cap systems rather than by considering NSF only as an axial load-transfer problem.

Keywords: Negative Skin Friction, Floating Pile Group, Viaduct Abutment, Asymmetric Surcharge, Neutral Plane, PLAXIS 3D

ÖZET

KAZIKLARDA OLUŞAN NEGATİF SÜRTÜNME VE NÖTR NOKTANIN 3B SAYISAL ANALİZLERLE DEĞERLENDİRİLMESİ

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Sıkışabilir zeminin kazığa göre daha fazla oturması sonucunda gelişen negatif çevre sürtünmesi (NÇS), kazık şaftı boyunca aşağı yönlü kayma gerilmeleri ve sürüklenme yükü oluşturur. Kazıklarla desteklenen viyadük kenar ayaklarında bu düşey etkileşime, yaklaşım dolgusunun neden olduğu yanal zemin hareketleri de eşlik edebilir ve bunun sonucunda kazıklarda eğilme ile kazık grubunda yatay yer değiştirme meydana gelebilir. Bu çalışmada, söz konusu mekanizmadan kaynaklanan birleşik eksenel ve yanal kazık davranışı, PLAXIS 3D ortamında tek taraflı sürşarj yüküne maruz bırakılan 5×5 düzenindeki yüzen kazık grubu kullanılarak incelenmiştir. Kazıklar gömülü kiriş elemanlarıyla temsil edilmiş ve NÇS gelişimi konsolidasyon analizleriyle modellenmiştir. Her bir kazığın sürüklenme yükü, nötr düzlem derinliği, eğilme momenti ve yanal yer değiştirmesi aynı analizler üzerinden değerlendirilmiştir. Parametrik çalışmada sürşarj büyüklüğü, kazık aralığı ve uzunluğu, kil tabakası kalınlığı, dolgunun kazıklara uzaklığı, arayüz dayanımı ve kazık başı yükünün etkileri araştırılmış; dört taraftan sürşarj uygulanan bir durum simetrik referans olarak analiz edilmiştir. Sonuçlar, sürüklenme yükü ve nötr düzlem konumunun kazık grubunun ön ve arka sıraları arasında belirgin farklılıklar gösterdiğini ortaya koymuştur. Tek taraflı yükleme altında, kazık grubu dolgudan uzaklaşacak şekilde yatay ötelenirken yükleme doğrultusundaki eğilme yapısal davranışı belirleyebilir. Bu nedenle, yumuşak zeminlerdeki kazıklı viyadük kenar ayakları, NÇS'nin yalnızca eksenel yük aktarımı problemi olarak ele alınması yerine, üç boyutlu zemin-kazık-başlık etkileşimi kapsamında değerlendirilmelidir.

Anahtar Kelimeler: Kazıklarda Negatif Çevre Sürtünmesi, Yüzen Kazık Grubu, Viyadük Kenar Ayağı, Asimetrik Yükleme, Nötr Düzlem, PLAXIS 3D

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SYMBOLS AND ABBREVIATIONS

Symbols

$\Delta s(z)$	Vertical settlement of soil at depth z
$\Delta p(z)$	Vertical displacement of pile at depth z
$\Delta r(z)$	Relative displacement between soil and pile
z	Depth below ground surface
z_n	Neutral plane depth
$\tau(z)$	Shaft shear stress at depth z
$N(z)$	Axial force in pile at depth z
D	Pile diameter
s	Center-to-center pile spacing
$\frac{s}{D}$	Normalized pile spacing ratio
σ'_v	Vertical effective stress
u	Excess pore water pressure
c_v	Coefficient of consolidation
t	Time
K_0	Coefficient of earth pressure at rest
Q	Axial load at the pile head
N_{max}	Maximum axial force along the pile
Q_{nsf}	Dragload due to negative skin friction, excluding pile self-weight
Q_s	Positive shaft resistance mobilized below the neutral plane
W_{pile}	Self-weight of the pile above the neutral plane
Q_p	Mobilized toe (base) resistance
M_2, M_3	Bending moments about the pile cross-section axes
u_y	Lateral displacement of the pile in the loading direction
R_{inter}	Interface strength reduction factor
q	Embankment surcharge intensity
d	Horizontal offset between embankment toe and pile cap
L	Pile length
H_{clay}	Thickness of the compressible clay layer
λ	Dimensionless pile-head load ratio governing the neutral-plane depth (Section 5.6)
R_s	Available shaft resistance over the full pile length (used in $\lambda = Q/R_s$)
γ'	Submerged (effective) unit weight

Abbreviations

NSF	Negative Skin Friction
FEM	Finite Element Method
3D FEA	Three-Dimensional Finite Element Analysis
PLAXIS 3D	Three-Dimensional Finite Element Software by Bentley
FS	Factor of Safety
OCR	Overconsolidation Ratio
RC	Reinforced Concrete

1. INTRODUCTION

1.1 Introduction

Pile foundations are extensively used in geotechnical engineering to support structures constructed on weak, compressible, or highly deformable soils where shallow foundations cannot satisfy bearing capacity or serviceability requirements. In many practical applications, such as embankments on soft ground, bridge approaches, port structures, industrial facilities, and reclaimed lands, piles are subjected not only to structural loads but also to additional actions arising from long-term ground deformations. One of the most critical and challenging phenomena associated with such conditions is negative skin friction (NSF), which develops when the surrounding soil settles more than the piles.

Negative skin friction induces downward-directed shear stresses along the pile shaft, generating an additional axial force that acts concurrently with the applied structural load. This additional force, commonly termed dragload, may significantly increase the axial demand on piles and alter the internal force distribution within the foundation system. If not properly accounted for in design, NSF can lead to excessive settlements, overstressing of piles, and, in extreme cases, structural distress. Consequently, the reliable assessment of NSF has long been recognized as a fundamental issue in the design of pile foundations constructed in compressible soils.

While the behavior of single piles under NSF has been widely investigated and is reasonably well understood, the response of pile groups subjected to negative skin friction remains considerably more complex and less documented. In pile groups, the interaction between adjacent piles, the stiffness of the pile cap, and the overlapping stress and displacement fields in the surrounding soil give rise to a highly three-dimensional response. These interactions significantly influence the magnitude and distribution of NSF, as well as the settlement behavior of individual piles within the group.

In particular, floating pile groups, where pile tips do not reach a firm bearing stratum, are especially sensitive to NSF effects. Floating piles rely predominantly on shaft resistance

to transfer loads to the surrounding soil. As a result, any settlement of the soil mass relative to the piles directly affects the mobilization of shaft resistance and the development of negative skin friction. In such systems, the absence of significant end bearing capacity amplifies the consequences of dragload and makes the accurate prediction of pile–soil interaction essential for safe and economical design.

A configuration in which these settlement-inducing mechanisms combine in a particularly demanding way is the pile-supported viaduct or bridge abutment. In this situation an approach embankment is placed on the soft ground on one side of the foundation only, while the piles simultaneously carry the vertical load transmitted from the superstructure. The embankment therefore acts as a one-sided surcharge: its vertical component drives consolidation settlement and hence negative skin friction, while the accompanying lateral movement of the soft soil away from the fill loads the piles in bending (Stewart et al. 1994b). Centrifuge testing has confirmed that such embankment-induced lateral soil movements can control the serviceability of piled abutments on soft clay (Stewart et al. 1994a).

The dragload and neutral plane position differ between the loaded (front) and far (back) sides of the group, significant bending moments develop in the direction of the embankment, and the group as a whole translates horizontally. This asymmetric configuration, examined here for a true floating group, is the central subject of the present thesis and is illustrated conceptually in Figure 1.1.

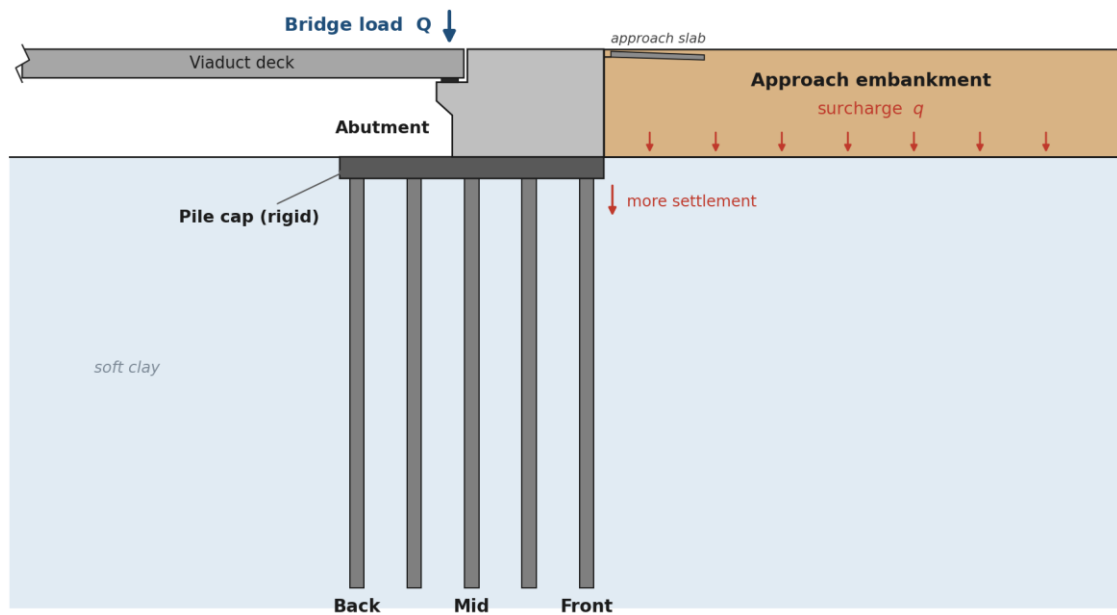


Figure 1.1. One-sided (asymmetric) surcharge at a viaduct abutment:

1.2 Objectives of the Study

The primary objectives of this thesis are:

1. To quantify the development of negative skin friction, the resulting dragload and the neutral plane position for each pile of a floating 5×5 group subjected to one-sided embankment surcharge, and to characterize the variation between the loaded (front) and far (back) sides of the group.
2. To determine the bending moments and lateral displacements induced by the one-sided loading, and to assess their significance relative to the axial drag.
3. To perform a parametric study examining the influence of surcharge magnitude (q), pile spacing (s), pile length (L), clay layer thickness (H_{clay}), embankment–cap offset distance (d), interface strength (R_{inter}) and pile head load (Q) on the group response.
4. To compare the asymmetric response with an equivalent symmetric (four-sided) surcharge case used as a control, thereby isolating the effects that are specific to one-sided loading.
5. To derive design-oriented insights for pile-supported viaduct abutments and similar foundations on compressible soils.

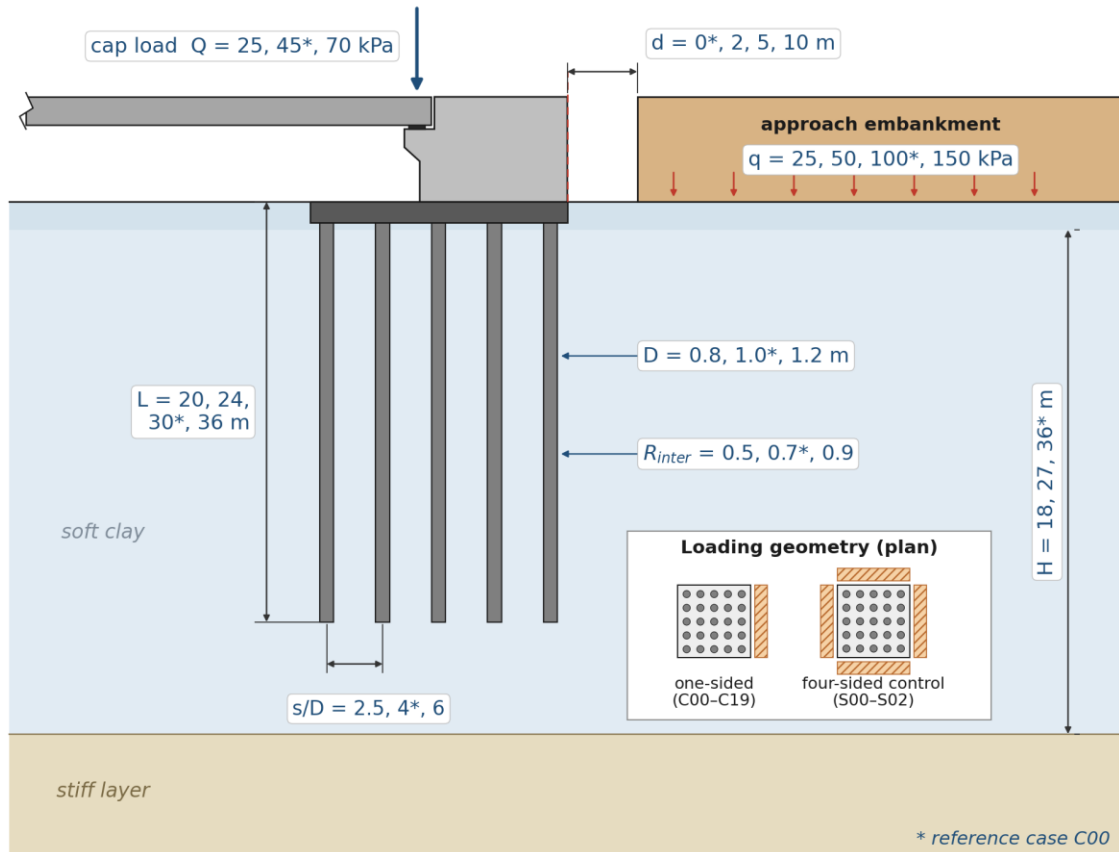


Figure 1.2. Schematic overview of the principal parametric variables and loading configurations considered in the study.

1.3 Scope and Structure of the Thesis

This study focuses on a floating pile group embedded in a deep, compressible clay deposit and loaded by a constant vertical bridge load together with an embankment surcharge placed on one side of the group, representative of a viaduct abutment. The analyses are performed with PLAXIS 3D software and include soil consolidation, pile–soil interface behavior and group interaction, as well as the flexural response and lateral displacement of the piles induced by the one-sided loading. A symmetric (four-sided) surcharge case is analyzed as a control. Seismic actions, battered piles, cyclic loading and structural non-linearity of the piles are outside the scope of this research.

The thesis is organized as follows:

- Chapter 1 introduces the problem, objectives, and scope of the study.
- Chapter 2 presents a comprehensive review of existing literature related to NSF, pile group behavior, and numerical modeling approaches.

- Chapter 3 outlines the theoretical background governing soil–pile interaction, negative skin friction, and neutral plane mechanics.
- Chapter 4 describes the numerical modeling framework and analysis methodology adopted in this study.
- Chapter 5 presents and discusses the results of the numerical analyses, beginning with the asymmetric reference case, followed by the parametric series and the symmetric control analyses.
- Chapter 6 summarizes the main findings and conclusions, and Section 6.1 outlines recommendations for future research.

2. LITERATURE REVIEW

The behavior of pile foundations is governed by the interaction between the pile and the surrounding soil through load transfer mechanisms mobilized along the pile shaft and, where applicable, at the pile base. Early studies on pile–soil interaction established that axial loads applied to piles are transferred progressively to the soil through skin friction, while pile tip resistance contributes depending on the stiffness and strength of the bearing stratum (Meyerhof 1976; Tomlinson 1957).

For floating piles, the contribution of end bearing is generally limited, and the majority of the applied load is transferred through shaft resistance. This makes the pile response highly sensitive to changes in soil stress state and deformation characteristics (Poulos and Davis, 1980). The mobilization of shaft resistance depends on the relative displacement between the pile and the soil, as well as the effective stress level acting at the pile–soil interface (Randolph and Wroth 1979).

In pile groups, soil–pile interaction becomes significantly more complex due to pile–pile interaction effects. Adjacent piles influence each other through overlapping stress and displacement fields, resulting in reduced lateral soil deformation and altered load transfer mechanisms compared to isolated piles (Poulos and Davis 1980; Randolph and Wroth 1979). This interaction leads to changes in group efficiency, settlement behavior, and axial force distribution within the group (Poulos and Davis 1980).

The stiffness of the pile cap plays a critical role in governing pile group behavior. A rigid pile cap enforces settlement compatibility among piles, redistributing axial loads and modifying the mobilization of shaft resistance along individual piles (Poulos and Davis 1980). These effects are particularly important in compressible soils, where group settlement and interaction dominate the overall response of the foundation system.

2.1 Negative Skin Friction in Floating Pile Groups

The mechanism of negative skin friction is fundamentally governed by the relative vertical displacement between the soil and the piles. When the soil settles more than the piles due to consolidation, surcharge loading, groundwater level reduction, or long-term creep, downward shear stresses are mobilized along the pile–soil interface. These stresses act in the same direction as the applied loads and increase the axial force within the piles.

In pile groups, the development of NSF is strongly influenced by several additional factors that are absent in single pile systems, including:

- pile–pile interaction effects,
- pile spacing and group geometry,
- stiffness and rigidity of the pile cap,
- non-uniform settlement patterns across the group,
- redistribution of dragload among individual piles.

Field observations and numerical studies have shown that negative skin friction is not uniformly distributed within a pile group. Interior piles are partially shielded from the settling soil by the surrounding perimeter piles, and therefore typically attract smaller dragloads than edge and corner piles (Comodromos and Bareka 2005; Huang et al. 2015; Lee et al. 2002). Moreover, the presence of a rigid pile cap tends to restrain differential pile settlement, further modifying the distribution of axial forces within the group. These aspects highlight the inadequacy of applying single-pile-based design approaches directly to pile group systems.

2.2 Mechanism of Negative Skin Friction

Negative skin friction (NSF) develops when the surrounding soil settles more than the pile, inducing downward-directed shear stresses along the pile shaft. This phenomenon was first systematically described through field observations and analytical interpretation by Endo et al. (1969) and Fellenius (1972), who introduced the concept of dragload and emphasized its significance in pile design.

The fundamental cause of NSF is the relative vertical displacement between the soil and the pile. When soil settlement exceeds pile settlement due to consolidation, surcharge loading, or groundwater level reduction, the direction of shear stress at the pile–soil interface reverses, acting downward and adding to the axial force in the pile (Fellenius 1984). This additional force must be resisted by the pile structural capacity and may significantly reduce the available safety margin. Several mechanisms have been identified as primary contributors to the development of NSF.

2.2.1 Consolidation of Compressible Soils

In soft clay deposits, NSF is commonly associated with primary and secondary consolidation. As excess pore water pressures dissipate, effective stresses increase, leading to vertical soil settlement relative to the pile (Bjerrum 1967; Indraratna et al. 1992). Creep and secondary compression may increase long term settlement, particularly in highly plastic or organic soils (Mesri and Castro 1987).

2.2.2 Surcharge Loading and Fill Placement

The placement of embankments or structural fills induces additional vertical stresses in the soil mass, resulting in settlement that may exceed pile deformation. Field studies have shown that NSF induced by fill loading can be comparable to or even exceed structural loads in extreme cases (Fellenius 2006; Wong and Teh 1995).

2.2.3 Groundwater Level Changes

Lowering of the groundwater table increases effective stress in the soil, triggering consolidation and settlement. This mechanism has been identified as a major cause of NSF in urban and industrial areas where groundwater extraction is common (Alonso et al. 1984).

2.3 Neutral Plane Concept

The neutral plane is a fundamental concept in the assessment of negative skin friction and was formally introduced through field observations and analytical interpretations by Endo

et al. (1969) and Fellenius (1972). It is defined as the depth at which the vertical displacement of the pile equals that of the surrounding soil. Above the neutral plane, the soil settles more than the pile, resulting in the mobilization of negative skin friction; below this depth, the pile settles more than the soil and positive skin friction develops, contributing to load resistance (Fellenius 1984).

The position of the neutral plane determines the extent of the pile shaft subjected to negative skin friction and thereby the magnitude of the dragload acting on the pile. Analytical and observational studies have shown that the maximum axial force in a pile typically occurs near the neutral plane, making its accurate determination critical for structural design (Alonso et al. 1984; Fellenius 2006).

Within the unified design framework, the neutral plane is determined jointly from force and settlement equilibrium along the pile, and the dragload is treated as a quantity affecting settlement and structural demand rather than geotechnical capacity (Fellenius 2004, 2006). Centrifuge testing has further shown that the neutral plane is load-dependent: applying axial load at the pile head raises it, whereas renewed surcharge drives it downward again (Leung et al. 2004).

In compressible soils, the depth of the neutral plane is influenced by several parameters, including soil compressibility, pile stiffness, pile length, and loading conditions (Indraratna et al. 1992; Wong and Teh 1995). Alonso et al. (1984) demonstrated that the neutral plane is not a fixed property but evolves as consolidation progresses, shifting downward with increasing soil settlement.

For pile groups, the neutral-plane depth is not necessarily the same for all piles. Pile–pile interaction and cap-induced load redistribution change both relative settlement and pile-head load from one position to another (Huang et al. 2015; Lee et al. 2002). Published studies therefore demonstrate a position-dependent neutral plane, but the center-to-edge ordering should not be treated as universal for all group geometries and loading arrangements.

Accordingly, the present study treats the neutral plane as a pile-specific quantity rather than assigning one depth to the entire group. A more consistent group-effect observation is the shielding of interior piles, which can reduce dragload relative to perimeter piles or to an isolated pile (Lee et al. 2002; Lee and Ng 2004).

The accurate determination of the neutral plane is essential, as it governs:

- the extent of the pile shaft subjected to negative skin friction,
- the magnitude of dragload acting on the piles,
- the distribution of axial forces along the pile length,
- the structural design requirements of the piles.

2.4 Analytical and Empirical Methods for Predicting Negative Skin Friction

Several analytical and empirical methods have been proposed to estimate negative skin friction on piles. These methods are primarily based on simplified representations of soil–pile interaction and were originally developed for single piles.

2.4.1 Effective Stress–Based Methods

Effective stress approaches relate NSF to the vertical effective stress acting on the pile shaft. The widely used β -method assumes that the unit negative skin friction is proportional to the effective vertical stress through a constant coefficient β (Burland 1973; Meyerhof 1976). Although this method is widely applied to clays, it does not account for time dependent settlement or group interaction effects.

The λ -method, proposed by Focht and Vijayvergiya (1972), relates the average unit shaft friction to the mean effective overburden stress and the undrained shear strength through an empirical coefficient λ . While practical for driven piles in clay, the method was developed for single-pile capacity and does not explicitly address pile group interaction (Wong and Teh 1995).

2.4.2 Adhesion-Based Methods

The α -method relates negative skin friction in cohesive soils to the undrained shear strength through an adhesion factor α (Fellenius 1984; Tomlinson 1957). Although widely used in practice, the α -method neglects settlement compatibility and consolidation effects, limiting its applicability to NSF problems dominated by time-dependent soil deformation.

2.4.3 Load–Transfer Methods

Load–transfer methods, such as t – z and q – z approaches, represent the pile–soil interface behavior using nonlinear shear stress–displacement relationships (Coyle and Reese 1966). These methods allow for displacement-controlled analysis and can capture the progressive mobilization of NSF. However, they are fundamentally one-dimensional and cannot represent three-dimensional group effects or stress overlap between adjacent piles (Poorooshasb et al. 1996).

Overall, while analytical methods provide valuable insights and are suitable for preliminary design, their limitations become pronounced when applied to pile groups in compressible soils.

2.5 Negative Skin Friction in Pile Groups

Negative skin friction in a pile group is not distributed uniformly among the individual piles. The response is affected by pile–pile interaction, overlapping soil-displacement fields and the compatibility imposed by the pile cap. Experimental and numerical studies generally show smaller dragloads on shielded interior piles and larger dragloads on perimeter and corner piles (Huang et al. 2015; Lee et al. 2002; Lee and Ng 2004).

Pile spacing controls the strength of this interaction. At close spacing, the soil enclosed by the group tends to move more collectively with the piles, which reduces the relative soil–pile displacement around the inner piles. As spacing increases, this shielding effect becomes weaker and the response approaches that of isolated piles. Huang et al. (2015)

reported that the group effect became small at spacings of about five pile diameters for the conditions examined in their tests.

The rigid cap provides an additional coupling mechanism because the pile heads cannot settle independently. Differences in shaft loading are therefore redistributed through the cap, and the resulting dragload and neutral-plane depth depend on the position of a pile within the group. For this reason, the present study evaluates the complete 5×5 group rather than extrapolating a single-pile response.

2.6 Three-Dimensional Numerical Studies on Negative Skin Friction

Three-dimensional numerical analysis is well suited to negative-skin-friction problems in pile groups because pile geometry, soil consolidation, pile–soil interaction and group effects can be represented within the same model. This is important when dragload and neutral-plane depth vary from one pile to another. For individual piles, axisymmetric finite element analyses have also been used to investigate NSF development and neutral-plane behavior (El-Mossallamy et al. 2013).

Numerical studies of pile groups have shown that three-dimensional models can reproduce the position-dependent shielding of interior piles and the associated redistribution of axial force (Comodromos and Bareka 2005; Lee et al. 2002; Lee and Ng 2004). In contrast, simplified single-pile or two-dimensional idealizations cannot directly resolve the response of every pile in the group.

Accordingly, a full three-dimensional model is used in this thesis. This is particularly necessary for the one-sided surcharge problem considered here, because the loading is asymmetric and produces both axial drag and lateral pile response.

2.7 Non-Uniform and One-Sided Surcharge Loading

Most experimental and numerical studies of NSF in groups assume that the surcharge surrounds the piles uniformly. Considerably fewer studies address non-uniform loading.

In the sand model tests of (Huang et al. 2015), surcharge applied on one side of the group only produced a distinctly ordered spatial response: the negative skin friction and the neutral plane depth varied systematically with pile position, and the piles under side loading developed a smaller dragload with a shallower neutral plane than under surrounding loading. The same study observed that group interaction effects essentially vanished once the pile spacing reached about five diameters.

These observations indicate that the loading geometry, and not only its magnitude, controls the distribution of dragload within a group. However, systematic three-dimensional numerical investigations of one-sided surcharge on floating pile groups, in which the axial, flexural and lateral components of the response are examined together, remain scarce.

2.8 Piled Bridge Abutments and Lateral Soil Movement

The specific situation of a pile-supported bridge abutment adjacent to an approach embankment couples negative skin friction with lateral soil–structure interaction. Stewart et al. (1994b) proposed a design framework for piled abutments on soft clay loaded by lateral soil movements generated by the approach fill, and their companion centrifuge test series clarified the influence of embankment geometry, pile arrangement and time-dependent consolidation on the response of the pile group Stewart et al. (1994a).

Numerical back-analyses of such systems initially relied on plane-strain idealizations in which a pile row is represented by an equivalent wall (Ellis and Springman 2001). While computationally attractive, this simplification cannot resolve the three-dimensional variation of forces between individual piles of a group, which motivates the use of full three-dimensional models in the present study.

In these settings the piles experience simultaneously an axial dragload from consolidation settlement and flexure from the lateral movement of the soft soil. The combined axial–flexural response of a true floating group under such one-sided loading has received little systematic attention, and it is this combination that the present study addresses.

2.9 Summary of Literature Gaps and Research Needs

The review of existing literature indicates that negative skin friction has been extensively investigated for single piles, both through analytical formulations and field observations. Foundational studies established the fundamental concepts of dragload and neutral plane and clarified the role of soil settlement in NSF development (Alonso et al. 1984; Fellenius 1972, 1984). Subsequent research expanded these concepts by incorporating consolidation effects and time-dependent soil behavior (Indraratna et al. 1992; Wong and Teh 1995).

Despite these advances, several important gaps remain, particularly about floating pile groups:

1. Dominance of single-pile-based approaches
Most analytical and empirical NSF prediction methods were developed for isolated piles. Their direct application to pile groups neglects pile–pile interaction, stress overlap, and group-induced settlement patterns, leading to questionable accuracy when used for group foundations (Poulos 1989; Randolph and Wroth 1979).
2. Limited understanding of dragload distribution within pile groups
Experimental and numerical studies have demonstrated that dragload is not uniformly distributed among piles within a group, with perimeter and corner piles attracting greater dragload than the shielded interior piles (Comodromos and Bareka 2005; Lee and Ng 2004). However, systematic evaluation of these differences under varying soil conditions and group configurations remains limited.
3. Inadequate characterization of neutral plane behavior in pile groups
While the neutral plane concept is well established for single piles, its interpretation for pile groups is far more complex. Existing studies suggest that the neutral plane may vary between individual piles and evolve during consolidation, yet comprehensive parametric investigations addressing this issue are scarce (Lee and Ng 2004).
4. Insufficient focus on floating pile groups
Many previous studies have focused on end-bearing piles or mixed foundation

systems. Floating pile groups, which rely predominantly on shaft resistance and are therefore more sensitive to NSF effects, have received comparatively less attention in literature.

5. Need for verified three-dimensional numerical frameworks
Although three-dimensional finite element analyses have been applied to dragload and group interaction (Comodromos and Bareka 2005; Lee et al. 2002), systematic parametric studies specifically addressing NSF and neutral-plane development in floating pile groups under one-sided surcharge remain limited.
6. Scarcity of studies on one-sided (asymmetric) surcharge
Bridge and viaduct abutments load the soft ground on one side of the foundation only, so that the piles experience axial drag, bending and lateral displacement simultaneously. Apart from a limited number of model tests (Huang et al. 2015) and abutment-specific studies (Stewart et al. 1994b), the combined response of true floating pile groups to one-sided surcharge has not been systematically examined by three-dimensional numerical analysis.

These gaps highlight the need for a comprehensive investigation of negative skin friction in floating pile groups using advanced three-dimensional numerical analysis. Such an approach enables explicit consideration of pile–pile interaction, soil consolidation, interface behavior, and group geometry, thereby providing a more realistic assessment of dragload development and neutral plane location.

Accordingly, the present study addresses these gaps by employing three-dimensional finite element analysis to investigate, within a single framework, the axial dragload, the neutral plane position, the bending moments and the lateral displacements of a floating pile group subjected to one-sided embankment surcharge representative of a viaduct abutment. A symmetric surcharge case is analyzed in parallel as a control, so that the effects specific to the asymmetric loading can be isolated.

3. THEORETICAL BACKGROUND

3.1 Load Transfer Mechanisms in Pile Foundations

The axial load applied to a pile is transferred to the surrounding soil through a combination of shaft resistance and base resistance. For floating piles, the contribution of base resistance is generally limited, and load transfer occurs predominantly through skin friction mobilized along the pile shaft (Poulos and Davis 1980; Randolph and Wroth 1979).

The mobilization of shaft resistance depends on the relative displacement between the pile and the soil, the effective stress acting normal to the pile surface, and the shear strength of the pile–soil interface (Randolph and Wroth 1979). As axial load increases, shear stresses develop progressively from the pile head downward, with the magnitude and distribution governed by soil stiffness, pile rigidity, and boundary conditions.

In pile groups, load transfer mechanisms are significantly influenced by pile–pile interaction. Overlapping stress fields around adjacent piles reduce lateral soil deformation and alter the mobilization of shaft resistance compared to isolated piles (Poulos 1989; Randolph and Wroth 1979). This interaction affects both the axial load distribution and settlement behavior within the group.

For floating pile groups embedded in compressible soils, the stiffness of the pile cap plays a crucial role. A rigid pile cap enforces settlement compatibility among piles, resulting in redistribution of axial loads and modification of shaft resistance mobilization along individual piles (Poulos 1989). Consequently, load transfer in pile groups must be evaluated as a three-dimensional problem rather than an assembly of independent piles.

3.2 Relative Displacement Theory and Negative Skin Friction

Negative skin friction is fundamentally governed by the relative vertical displacement between the pile and the surrounding soil. When soil settlement exceeds pile settlement,

downward shear stresses are mobilized along the pile shaft, acting in the same direction as the applied structural load (Fellenius 1972, 1984).

Let $\Delta s(z)$ denote the vertical settlement of the soil at depth z , and $\Delta p(z)$ denote the vertical displacement of the pile at the same depth. The relative displacement is defined by Equation (3.1). The direction and magnitude of shaft shear stress depend on its sign and magnitude:

$$\Delta r(z) = \Delta s(z) - \Delta p(z) \quad (3.1)$$

- If $\Delta r(z) > 0$, negative skin friction develops.
- If $\Delta r(z) < 0$, positive skin friction is mobilized.

This kinematic criterion is the theoretical basis used in the remainder of the chapter. The mechanisms that generate relative pile–soil movement were reviewed in Chapter 2; in the three-dimensional group model the same criterion is evaluated independently along each pile.

3.3 Development of Dragload and Axial Force Distribution

The downward shear stresses associated with negative skin friction generate an additional axial force within the pile, commonly referred to as dragload. The dragload accumulates along the pile shaft from the ground surface to the depth of the neutral plane, resulting in a maximum axial force near this location (Alonso et al. 1984; Fellenius 1972).

Three quantities are involved in this mechanism and are kept distinct throughout the present work, since the recent literature on neutral plane design treats them as separate concepts:

Negative skin friction denotes the downward shear stress acting on the pile shaft, expressed in units of stress.

The dragload is the additional axial force that this shear stress accumulates within the pile, expressed in units of force, and it is this quantity that is reported in kilonewtons throughout Chapter 5.

Downdrag refers to the settlement of the pile associated with the same mechanism, expressed as a displacement. The distinction matters in design because the dragload affects the structural demand on the section whereas downdrag governs serviceability, and because the two do not necessarily reach their critical values under the same conditions.

In a finite element model, the computed axial force also contains the self-weight of the pile. Since dragload is the force transferred to the pile by the settling soil, the pile self-weight must be removed before the NSF-induced force is compared between cases. For the reference geometry, the calculation is written explicitly below.

$$\begin{aligned}
 A &= \pi D^2 / 4 = \pi (1.0)^2 / 4 = 0.7854 \text{ m}^2 \\
 W_{\text{pile}} &= A \cdot \gamma_p \cdot z_n \\
 W_{\text{pile}} / z_n &= A \cdot \gamma_p = 0.7854 \times 25 = 19.6 \text{ kN/m} \\
 Q_{\text{nsf}} &= (N_{\text{max}} - Q) - W_{\text{pile}}
 \end{aligned}$$

where:

A = pile cross-sectional area (m^2)

D = pile diameter (m)

γ_p = unit weight of pile material (kN/m^3)

z_n = neutral-plane depth (m)

W_{pile} = pile self-weight above the neutral plane (kN)

N_{max} = maximum axial force along the pile (kN)

Q = axial load at the pile head (kN)

Q_{nsf} = dragload caused by negative skin friction, excluding pile self-weight (kN)

For the reference pile, self-weight therefore contributes 19.6 kN for every meter above the neutral plane. The self-weight-corrected Q_{nsf} is used for NSF comparisons, whereas N_{max} is retained for structural verification.

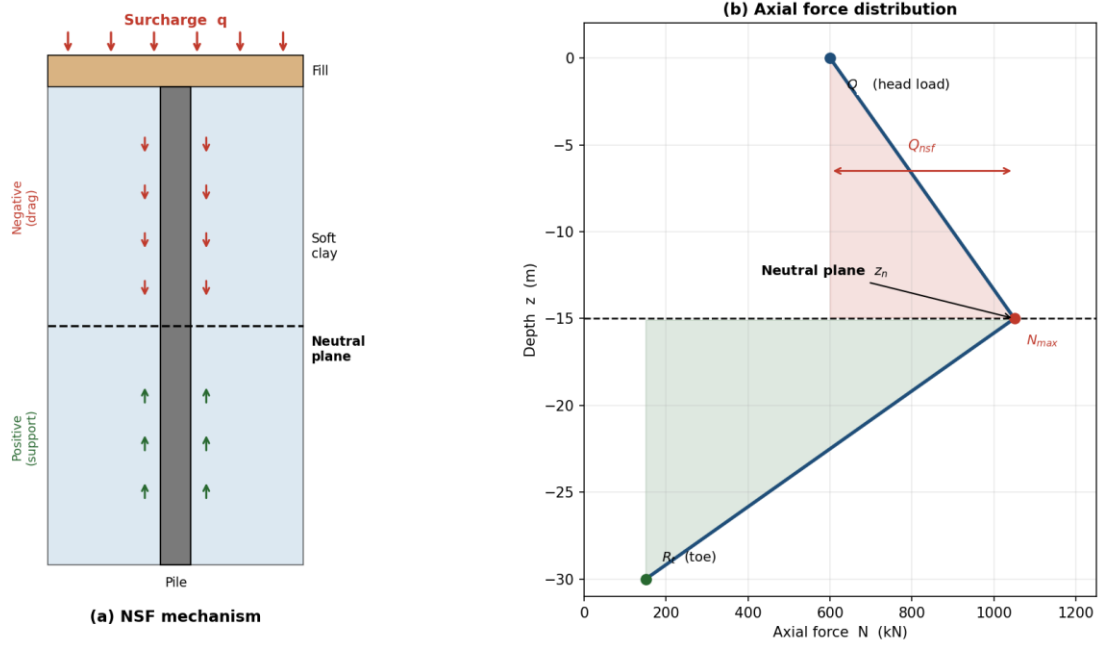


Figure 3.1. Mechanism of negative skin friction and corresponding axial force distribution along the pile.

The axial force distribution along the pile is governed by Equation (3.2):

$$Q_{nsf} = \pi D \int_0^{z_n} \tau_n(z) dz \quad (3.2)$$

where $\tau(z)$ is the shaft shear stress and D is the pile diameter. In the negative skin friction zone, $\tau(z)$ acts downward, increasing the axial force with depth.

For design calculations, the unit negative skin friction is most commonly expressed in terms of the vertical effective stress through the effective stress (β) approach:

$$\tau_n(z) = \beta \sigma'_v(z), \quad \beta = K \tan \delta' \quad (3.3)$$

where K is the coefficient of lateral earth pressure acting on the shaft and δ' is the effective interface friction angle. For soft, normally consolidated clays the coefficient β is commonly reported in the range of about 0.20–0.30 (Burland 1973; Fellenius 2004). The dragload then follows by integrating the unit negative skin friction over the shaft perimeter from the ground surface to the neutral plane:

$$Q_{nsf} = \pi D \int_0^{z_n} \tau_n(z) dz \quad (3.4)$$

This formulation is used in Chapter 5 as an independent hand-calculation check on the numerically computed dragloads: with the effective stress profile of the reference soil

column and an interface coefficient consistent with the adopted interface reduction factor, the β -method provides an order-of-magnitude verification of the finite element results for a single, unshielded pile.

Field measurements and numerical studies have shown that the maximum axial force may exceed the applied structural load by a substantial margin, particularly in floating piles embedded in highly compressible soils (Fellenius 2006; Indraratna et al. 1992). This additional force must be explicitly considered in structural design to prevent overstressing of the pile.

In pile groups, dragload distribution is non-uniform. Due to pile–pile interaction and confinement effects, perimeter and corner piles typically attract greater dragloads than the shielded interior piles (Comodromos and Bareka 2005; Huang et al. 2015; Lee et al. 2002). Moreover, pile cap rigidity can redistribute axial forces, further modifying the dragload profile within the group (Poulos 1989).

These observations underline the importance of evaluating dragload and axial force distribution using three-dimensional approaches that explicitly account for group effects.

3.4 Neutral Plane Determination: Theoretical Considerations

The determination of the neutral plane is central to the assessment of negative skin friction, as it defines the transition between negative and positive shaft resistance along the pile. The neutral plane is theoretically located at the depth where the relative vertical displacement between the soil and the pile becomes zero (Alonso et al. 1984; Fellenius 1972).

Using the relative-displacement framework introduced above, the neutral-plane depth is defined by Equation (3.5):

$$\Delta s(z_n) = \Delta p(z_n) \quad (3.5)$$

At this depth, the direction of shaft shear stress changes sign, and the axial force in the pile reaches its maximum value. Fellenius (1984) emphasized that the neutral plane should not be interpreted as a load transfer boundary but rather as a kinematic condition governed by settlement compatibility.

In the unified design framework, the neutral plane is obtained from the simultaneous satisfaction of force and settlement equilibrium (Fellenius 2004). The force condition states that, at the neutral plane, the downward-acting forces, consisting of the head load and accumulated negative skin friction, are balanced by the upward-acting resistances mobilized below it:

$$Q + Q_{\text{nsf}} = Q_p + Q_s \quad (3.6)$$

where Q_p is the mobilized toe resistance and Q_s the positive shaft resistance accumulated from the toe up to the neutral plane.

Graphically, the neutral plane lies at the intersection of the load distribution curve, drawn downward from Q with the negative friction added, and the resistance curve, drawn upward from the toe. The settlement condition requires, at the same depth, equality of pile and soil settlement. Because the mobilized toe resistance depends on the toe penetration, which itself depends on the settlement profile, the three quantities, neutral plane depth, dragload and toe resistance, are interdependent and must be solved consistently (Fellenius 2004).

Two refinements of the idealized picture deserve mention. First, the shaft shear does not reverse abruptly at a point: relative displacement passes through zero over a finite length, producing a transition zone within which the shear changes sign gradually and the axial force profile is rounded rather than sharply peaked (Fellenius 2004). Second, the neutral plane is load-dependent: increasing the head load raises it, while additional surcharge and continued consolidation push it downward, as demonstrated directly in centrifuge experiments (Leung et al. 2004). Both aspects are relevant to the interpretation of the numerical results in this study, in which head loads differ from pile to pile because of the rigid cap.

For the present study, the key implication is load dependence: changes in pile-head load or settlement alter the depth at which the kinematic and force-equilibrium conditions are satisfied. In a pile group this check must therefore be performed for each pile separately because cap load redistribution changes the head load and relative displacement from pile to pile.

3.5 Consolidation Theory and Settlement-Induced Negative Skin Friction

Settlement-induced negative skin friction is primarily associated with soil consolidation, particularly in soft and normally consolidated clay deposits. Consolidation occurs as excess pore water pressures generated by applied loads dissipate over time, leading to an increase in effective stress and corresponding soil settlement (Terzaghi 1943).

The one-dimensional consolidation process is governed by:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \quad (3.7)$$

where u is the excess pore water pressure and c_v is the coefficient of consolidation. As consolidation progresses, vertical soil settlement increases relative to pile displacement, promoting the development of NSF along the pile shaft (Bjerrum 1967; Indraratna et al. 1992).

The progress of consolidation in time is conveniently expressed through the dimensionless time factor:

$$T_v = \frac{c_v t}{H_{dr}^2} \quad (3.8)$$

where H_{dr} is the drainage path length, equal to the layer thickness for single (one-way) drainage and half of it for double drainage. The average degree of consolidation, and with it the developed fraction of the final settlement and of the negative skin friction, increases with T_v (Terzaghi 1943). In the numerical study, the surcharge is activated first and is followed directly by a single consolidation phase, which is continued until the maximum excess pore pressure falls below the adopted 1 kPa criterion. Creep and secondary compression, which can add to the long-term settlement of highly plastic soils (Mesri and Castro 1987), are not included, so the computed long-term dragloads should be regarded as lower-bound estimates in soils with pronounced secondary compression.

In the numerical model, consolidation is used to generate the time-dependent soil–pile relative displacement required by the preceding equations. The drainage boundaries, loading sequence and termination criterion used for this calculation are defined in Chapter 4.

3.6 Pile Group Interaction and Cap Stiffness Effects

For a pile group, the axial equilibrium described above is coupled through pile–pile interaction and the pile cap. These mechanisms alter the load received by each pile and the relative movement of the surrounding soil (Poulos 1989; Randolph and Wroth 1979).

A rigid pile cap enforces compatible pile-head displacements and redistributes structural load among the piles (Poulos 1989). This redistribution changes the neutral-plane position and dragload from one pile to another; its direction depends on loading geometry and pile position and is evaluated directly in Chapter 5.

Pile spacing controls the strength of group interaction. Shielding generally decreases as spacing increases, so the normalized spacing s/D is treated as an independent parameter in the numerical study (Huang et al. 2015; Lee et al. 2002).

These coupled effects motivate the full-group three-dimensional representation adopted in Chapter 4.

3.7 Response of Piles to Lateral Soil Movement

The theory presented so far concerns the axial response of piles to vertical soil settlement. When the surcharge is applied on one side of the foundation only, a second mechanism is activated: the soft soil beneath the embankment deforms not only vertically but also horizontally, displacing laterally away from the loaded area. Piles standing in this moving soil are loaded passively along their shafts. This differs fundamentally from active lateral loading applied at the pile head because the soil drives the pile rather than resisting it (Stewart et al. 1994b). The free-field lateral movement is largest near the surface and adjacent to the embankment, and decays with depth and with distance from the fill; it

develops partly rapidly, through undrained shear distortion during construction, and partly slowly, as consolidation proceeds (Stewart et al. 1994a).

The structural response of a pile to this movement can be described by beam bending. Denoting the pile deflection by $y_p(z)$, the governing relation is:

$$EI \frac{d^4 y_p}{dz^4} = q_{lat}(z) \quad (3.9)$$

where EI is the flexural rigidity of the pile and $q_{lat}(z)$ is the net lateral load per unit length transferred from the soil.

This load arises from the relative lateral displacement between the free-field soil movement $y_s(z)$ and the pile deflection $y_p(z)$: where the soil moves more than the pile, it presses against the shaft, and the transferable pressure is bounded by the ultimate lateral resistance of the soil flowing around the pile (Poulos 1989; Stewart et al. 1994b). A stiff pile that deflects little therefore attracts load over the full depth of moving soil, whereas a flexible pile that follows the soil attracts less, reflecting the opposite form of the relative displacement principle that governs negative skin friction in the axial direction.

Group and cap effects modify this picture in three ways. First, the rigid cap forces all pile heads to translate equally and restrains their rotation, so the piles bend in double curvature with a fixity moment at the head and a reversed moment at depth, near the base of the moving soil layer. Second, the piles closest to the embankment partially shield the rows behind them from the moving soil, so the lateral load, and with it the bending moment, decreases from the front row toward the back row. Third, the cap redistributes the vertical head loads: the asymmetric settlement field rotates the cap slightly and shifts head load toward the loaded side, so that front piles carry a larger share of the superstructure load than back piles. Through the load-dependence of the neutral plane discussed above, this redistribution links the lateral mechanism back to the axial one. It raises the neutral plane of the heavily loaded front piles and deepens that of the lightly loaded back piles, even though the front side experiences the larger soil settlement. Figure 3.2 illustrates this mechanism using a consistent direction convention: the embankment is on the right, and positive free-field soil movement and pile translation are directed away from it.

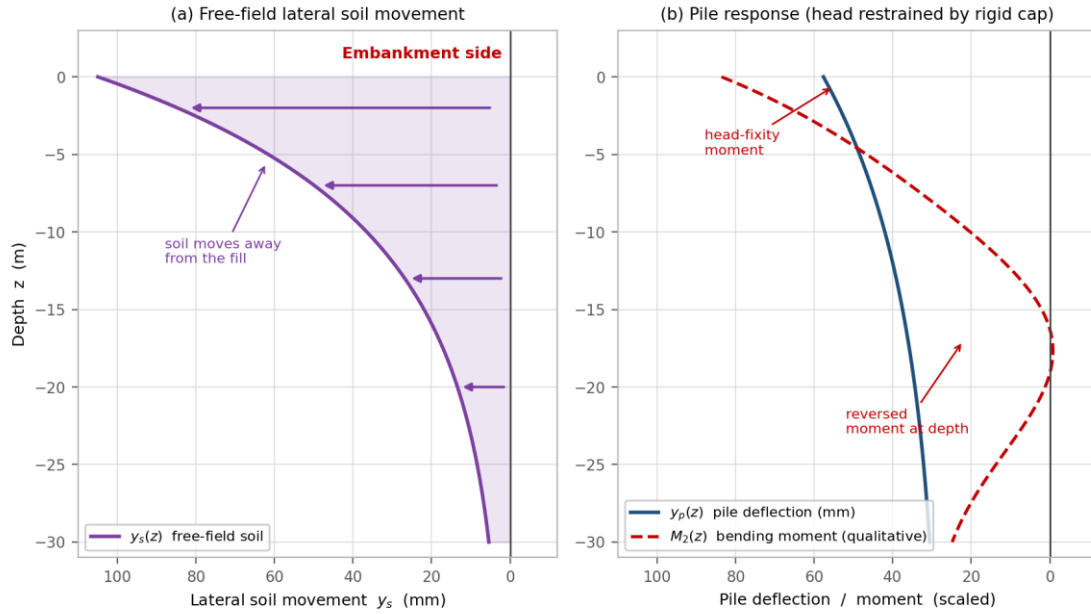


Figure 3.2. Conceptual illustration of lateral soil movement and pile deflection / moment (schematic, not based on numerical analysis results).

For design, the essential consequence is that under one-sided surcharge the pile cross-section must resist the axial force $N(z)$, which includes the dragload, simultaneously with the bending moment $M(z)$ generated by the lateral soil movement. The two demands peak at different depths, the axial force at the neutral plane and the bending moment near the head and within the upper part of the moving layer. Therefore, the governing section is not known in advance and the combined N – M demand must be checked over the full pile length. Classical NSF design, which considers the axial component alone, is therefore incomplete for abutment-type loading; quantifying the relative importance of the two components is one of the central aims of this thesis.

3.8 Implications for Numerical Modeling

The theoretical considerations presented in this chapter highlight the inherently three-dimensional and time-dependent nature of negative skin friction in floating pile groups. Accurate assessment of NSF and neutral plane behavior requires modeling approaches capable of representing:

- soil consolidation and time-dependent settlement,

- nonlinear soil constitutive behavior,
- pile–soil interface mechanics,
- pile–pile interaction and cap stiffness effects,
- spatial variation of neutral plane depth within pile groups.

Three-dimensional finite element analysis provides a robust framework for addressing these requirements (Comodromos and Bareka 2005; Lee et al. 2002). By explicitly modeling pile geometry, soil stratigraphy, interface behavior, and staged loading, numerical analysis enables direct evaluation of relative displacement, shaft shear stress distribution, and axial force development along individual piles.

For the asymmetric problem examined in this thesis, two further modeling requirements arise. First, the analysis must recover the flexural response of the piles, including bending moments, shear forces and lateral displacements, which excludes purely axial load-transfer idealizations of the pile–soil system. Second, one-sided loading destroys the symmetry planes that are commonly exploited to reduce model size in NSF analyses of pile groups, so the full group must be discretized rather than a quarter or half model. Both requirements are met economically by the embedded beam representation adopted in Chapter 4, which yields the complete internal force distribution of every pile within a full three-dimensional model (Engin et al. 2007; Sadek and Shahrour 2004).

The theoretical framework developed in this chapter forms the basis for the numerical modeling strategy adopted in this study. The subsequent chapter describes the implementation of these concepts using three-dimensional finite element analysis to investigate negative skin friction and neutral plane behavior in floating pile groups.

4. NUMERICAL MODELING AND ANALYSIS

METHODOLOGY (PLAXIS 3D)

4.1 General Modeling Approach

In this study, the behavior of floating pile groups subjected to negative skin friction is investigated using three-dimensional finite element analysis (3D FEA). Numerical modeling is performed to capture the complex interaction between soil consolidation, pile–soil interface behavior, and pile group effects that cannot be adequately represented by analytical methods (Comodromos and Bareka 2005; Lee et al. 2002).

Advances in numerical methods and computational power have enabled the use of three-dimensional finite element analysis (3D FEA) as a powerful tool for investigating pile–soil interaction problems. Modern geotechnical software, such as PLAXIS 3D software, allows for realistic modeling of soil constitutive behavior, pile–soil interface characteristics, consolidation processes, and staged construction sequences.

Three-dimensional numerical analysis provides the capability to:

- explicitly model pile groups and pile caps,
- simulate soil consolidation and settlement-induced NSF,
- capture pile–pile interaction effects,
- determine the distribution of dragload within the group,
- identify the neutral plane for individual piles.

These features make 3D FEA particularly suitable for studying NSF in floating pile groups, where spatial interaction and time-dependent behavior play a dominant role.

Three-dimensional analysis is adopted due to the inherently spatial nature of pile group behavior. Previous studies have demonstrated that two-dimensional or axisymmetric models are insufficient for representing pile–pile interaction, stress overlap, and differential dragload distribution among piles within a group (Lee et al. 2002; Lee and

Ng 2004). Accordingly, all analyses in this study are conducted using a full 3D numerical framework.

The numerical simulations are carried out using PLAXIS 3D software, which allows explicit modeling of pile groups, pile caps, nonlinear soil constitutive behavior, soil–pile interface elements, and time-dependent consolidation processes. These capabilities make PLAXIS 3D software particularly suitable for evaluating negative skin friction and neutral plane development in floating pile groups (Comodromos and Bareka 2005; Engin et al. 2007).

The overall numerical strategy consists of:

1. Establishing the initial in-situ stress state of the soil,
2. Modeling the pile group and pile cap system,
3. Simulating settlement-inducing mechanisms through consolidation,
4. Evaluating the development of negative skin friction and dragload,
5. Identifying the neutral plane for individual piles within the group.

4.2 Geometry of the Numerical Model

4.2.1 Model Domain and Extent

The reference numerical domain extends from -40 m to +40 m in both horizontal directions, giving a plan area of 80 m $\times$ 80 m, and reaches a depth of 50 m below ground level. The outer pile rows lie at ± 8 m from the group center, so the minimum horizontal clearance between the outer piles and the lateral model boundaries is 32 m, equivalent to 32 pile diameters for the reference $D = 1.0$ m pile.

The adequacy of this domain was assessed directly rather than solely from a geometric rule of thumb. The base of the model lies 20 m below the reference pile toes, and the computed displacement and excess pore-pressure fields were inspected to confirm that they decay to negligible levels before reaching the lateral and bottom boundaries. This a posteriori check indicates that the boundaries do not materially constrain either the embankment settlement trough or the lateral movement of the soft clay.

The overall three-dimensional numerical domain and the location of the surcharge relative to the pile cap are illustrated in Figure 4.1.

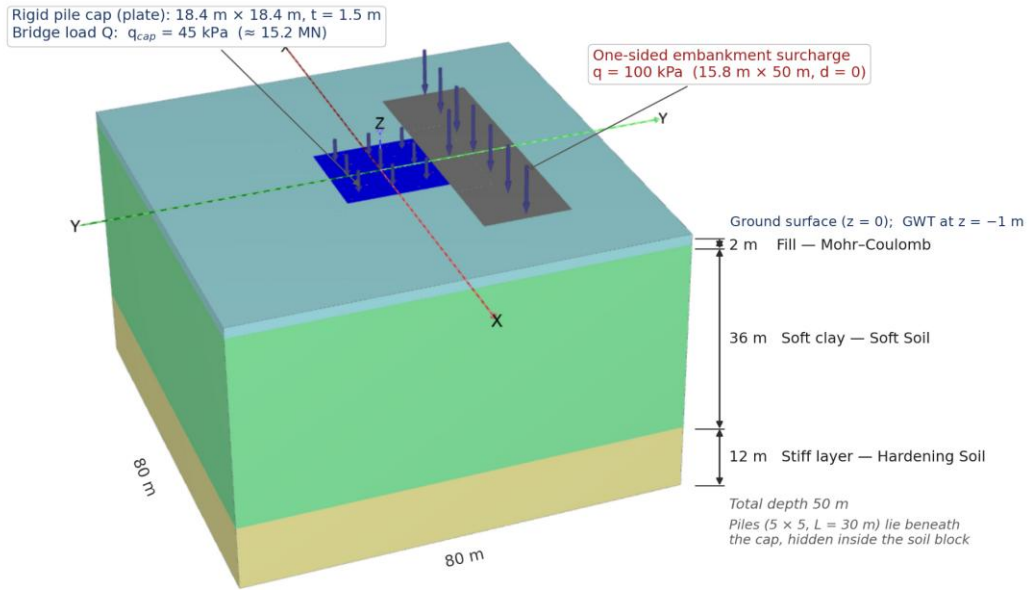


Figure 4.1. Overall three-dimensional view of the numerical model domain.

4.2.2 Pile Group Configuration

The pile group is modeled as a group of vertically aligned floating piles connected by a rigid pile cap. The number of piles and their arrangement are selected to represent typical engineering practice while allowing clear observation of pile group effects. Square pile group configurations are commonly adopted in the literature due to their symmetry and relevance in practice (Comodromos and Bareka 2005; Huang et al. 2015).

A 5×5 pile arrangement was selected because it allows the responses of corner piles, perimeter edge piles, intermediate interior piles, and the central pile to be evaluated separately. This configuration provides sufficient variation in pile position to investigate group shielding and the non-uniform effects of one-sided loading, while keeping the three-dimensional model size and computational requirements manageable.

Pile spacing is defined in terms of the pile diameter D , with center-to-center spacing expressed as s/D . This non-dimensional representation facilitates comparison with

previous studies and enables systematic parametric evaluation of group interaction effects (Lee et al. 2002; Randolph and Wroth 1979).

In the present study the reference configuration is a 5×5 group of twenty-five bored piles of diameter $D = 1.0$ m and length $L = 30$ m, arranged on a square grid at a center-to-center spacing of $s = 4.0$ m ($s/D = 4$). The pile toes terminate within the compressible clay so that the group is truly floating, and the heads are connected by a rigid cap of 1.5 m thickness. The pile head load transmitted from the superstructure is selected to remain well within the bearing capacity of the group, so that the analyses represent a serviceability condition. The plan layout of the group, the one-sided surcharge centered on the group in the transverse x -direction, and the row naming convention used throughout the thesis (front row on the loaded side, back row on the far side) are shown in Figure 4.2.

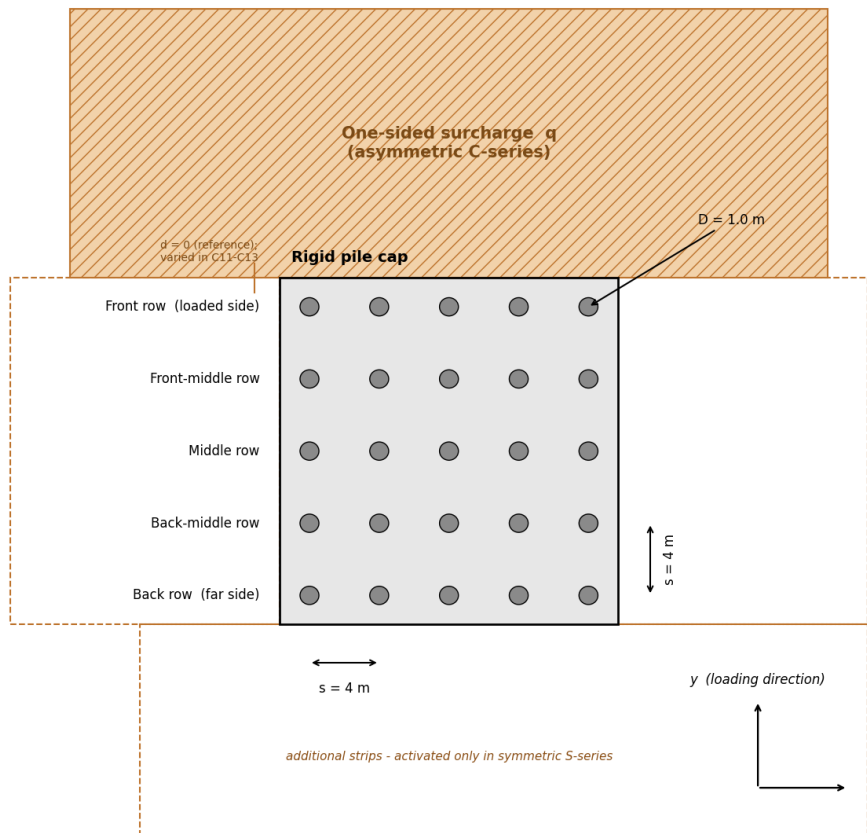


Figure 4.2. Reference pile-group arrangement

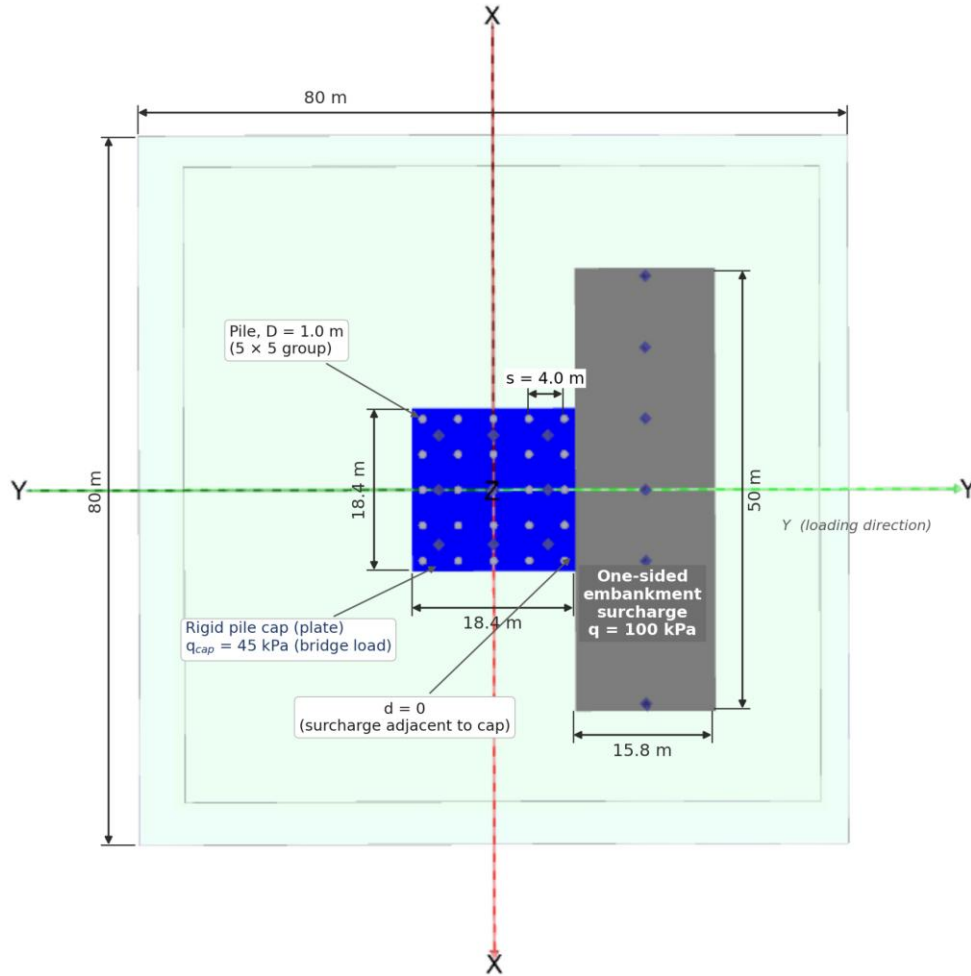


Figure 4.3. PLAXIS plan view of the reference model

4.3 Soil Stratigraphy and Material Properties

4.3.1 Soil Profile

The soil profile considered in the numerical model represents a compressible soil deposit typical of soft clay environments where negative skin friction is expected. The stratigraphy is assumed to be horizontally layered, consistent with many previous analytical and numerical studies (Indraratna et al. 1992; Wong and Teh 1995).

The thickness of the compressible soil layer is selected such that the piles remain floating, i.e., the pile tips do not reach a rigid bearing stratum. This configuration emphasizes the role of shaft resistance and settlement-induced NSF.

The reference profile consists of a 2 m thick surface fill underlain by a 36 m thick soft clay layer, below which a stiffer stratum extends to the base of the model at 50 m depth. With a pile length of 30 m, the pile toes remain 8 m above the base of the soft clay and 20 m above the bottom boundary, ensuring genuinely floating behavior in the reference case.

The adopted reference stratigraphy, groundwater level and floating pile toe position are shown in Figure 4.4.

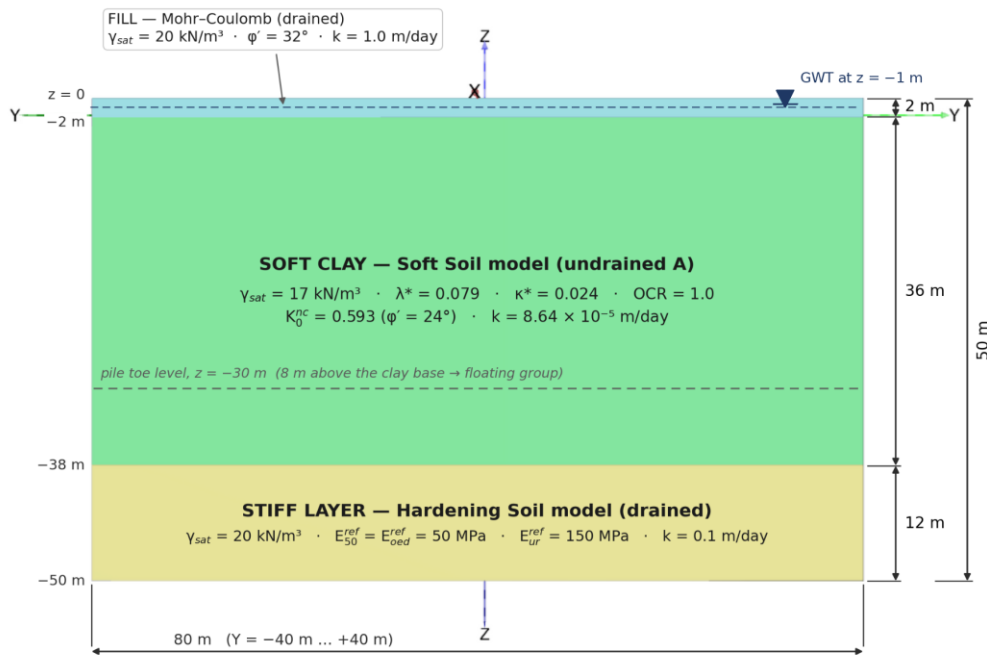


Figure 4.4. Reference soil profile and stratigraphy adopted in the numerical model, showing the fill, soft clay, stiff layer and the pile toe level of the floating group.

4.3.2 Soil Constitutive Model

Three constitutive models are used to represent the three soil layers in PLAXIS 3D. The 2 m surface fill is modeled with the Mohr–Coulomb model under drained conditions, the soft clay is modeled with the Soft Soil model using Undrained A behavior, and the stiff lower layer is modeled with the Hardening Soil model under drained conditions.

The Soft Soil model is assigned to the clay because consolidation and stress-dependent compressibility are the main mechanisms governing settlement and the development of negative skin friction in the analyses. The Hardening Soil model is used for the stiffer lower stratum, while the Mohr–Coulomb model provides a simple representation of the surface fill.

The study uses an idealized reference parameter set rather than a site-specific calibration. The adopted baseline parameters are listed in Table 4.1 and are kept unchanged unless a parameter is intentionally varied in the parametric study.

Table 4.1. Soil material parameters adopted in the numerical model.

Parameter (unit)	Fill	Soft clay	Stiff stratum
Constitutive model	Mohr–Coulomb	Soft Soil	Hardening Soil
Drainage type	Drained	Undrained (A)	Drained
Depth range (m)	0 – 2	2 – 38	38 – 50
Unsaturated unit weight γ_{unsat} (kN/m³)	19.0	16.0	19.0
Saturated unit weight γ_{sat} (kN/m³)	20.0	17.0	20.0
Initial void ratio e_0	0.500	1.200	0.500
Effective friction angle ϕ' (°)	32.0	24.0	32.0
Effective cohesion c' (kPa)	-	5	-
Reference stiffness E' or E_{50}^{ref} (kPa)	30 000	n/a	50 000
Oedometer stiffness $E_{\text{oed}}^{\text{ref}}$ (kPa)	n/a	n/a	50 000
Unloading–reloading stiffness $E_{\text{ur}}^{\text{ref}}$ (kPa)	n/a	n/a	150 000
Poisson ratio ν (or ν_{ur})	0.300	0.150*	0.200
Modified compression index λ^*	n/a	0.0791	n/a
Modified swelling index κ^*	n/a	0.0237	n/a
Coefficient of lateral earth pressure K_0^{nc}	0.4701	0.5933	0.4701
Overconsolidation ratio OCR	n/a	1.0	1.0
Interface strength factor R_{inter}	0.70	0.70	0.70
Permeability $k_x = k_y = k_z$ (m/day)	1.0	8.64×10^{-5}	0.1
Initial porosity n	0.3333	0.5455	0.3333
Soil classification (Standard)	Coarse	Fine	Coarse

Note: Table 4.1 lists the baseline material parameters used in the reference PLAXIS 3D model. Hydraulic parameters and drainage conditions are discussed separately in Section 4.3.3.

* ν_{ur} (unloading–reloading Poisson’s ratio); default value recommended in the PLAXIS Material Models Manual.

4.3.3 Hydraulic Properties

The hydraulic parameters govern the rate at which the excess pore pressures generated by the embankment dissipate and therefore the time required for negative skin friction to develop fully. The water table was placed at 1 meter below ground level, so the entire profile except the half of the surface fill lies below the phreatic surface and the submerged unit weights apply throughout. The permeability of the soft clay was taken as 8.64×10^{-5} m/day in all three directions, while the fill and the stiff stratum were assigned values four and three orders of magnitude larger respectively. The flow boundary conditions were set so that the four vertical faces of the model are closed while the upper and lower horizontal boundaries are open. Water expelled from the compressible layer therefore escapes upward through the fill and downward through the stiff stratum, both of which are far more permeable than the clay itself, and the drainage path is half the thickness of the clay, that is 18 m. Closing the vertical faces is consistent with the symmetry of the far field and prevents artificial lateral drainage toward the boundaries of the domain.

These values can be used to estimate the duration of consolidation independently of the finite element solution. Using the effective vertical stress at approximately the clay mid-depth after surcharge application gives the following sequence.

$$\begin{aligned}
 E_{oed} &\approx \sigma'_v / \lambda^* = 250 / 0.0791 \approx 3160 \text{ kPa} \\
 m_v &\approx 1 / E_{oed} = 3.16 \times 10^{-4} \text{ m}^2/\text{kN} \\
 c_v &= k / (m_v \cdot \gamma_w) = 8.64 \times 10^{-5} / (3.16 \times 10^{-4} \times 9.81) \approx 0.028 \text{ m}^2/\text{day} \\
 H_{dr} &= 36 / 2 = 18 \text{ m} \\
 t_{90} &= 0.848 \cdot H_{dr}^2 / c_v \approx 27 \text{ years} \\
 t_{99} &= 1.781 \cdot H_{dr}^2 / c_v \approx 57 \text{ years}
 \end{aligned}$$

where:

- E_{oed} = representative tangent oedometer modulus (kPa)
- σ'_v = effective vertical stress at the representative depth (kPa)
- m_v = coefficient of volume compressibility (m^2/kN)
- k = vertical permeability (m/day)
- γ_w = unit weight of water (kN/m^3)
- c_v = coefficient of consolidation (m^2/day)
- H_{dr} = drainage path length (m)
- t_{90}, t_{99} = estimated times to 90% and 99% consolidation

The analyses reported in Chapter 5 reached the prescribed pore-pressure criterion after approximately 65 years, which agrees with the closed-form estimate. This agreement

confirms that a pore-pressure termination criterion, rather than an arbitrary elapsed time, is appropriate for comparing the fully developed long-term response of the different scenarios.

Table 4.2. Hydraulic parameters and flow boundary conditions adopted in the analyses.

Parameter	Fill	Soft clay	Stiff layer
$k_x = k_y = k_z$ (m/day)	1.0	8.64×10^{-5}	0.1
Anisotropy k_h / k_v	1.0	1.0	1.0
Initial porosity n	0.3333	0.5455	0.3333
Groundwater table	At ground surface ($z = 0$ m)		
Vertical model boundaries	Closed (no flow)		
Upper and lower boundaries	Open (free drainage)		
Drainage path of the clay	18 m (two-way drainage)		
Coefficient of consolidation c_v	-	≈ 0.028 m ² /day (≈ 10 m ² /year)	-
Termination criterion	Maximum excess pore pressure < 1 kPa		

The duration of consolidation was not fixed in advance. After activation of the surcharge, a single consolidation phase was continued until the maximum excess pore pressure generated by the embankment fell below 1 kPa. All results reported in Chapter 5 correspond to this fully dissipated condition, so the scenarios are compared at an equivalent degree of consolidation rather than at an equivalent elapsed time.

4.4 Modeling of Piles and Structural Elements

4.4.1 Pile Representation

The piles are modeled as linear elastic structural elements embedded within the soil continuum. This approach has been widely adopted in three-dimensional numerical studies investigating negative skin friction and pile group behavior, as it provides an efficient and accurate representation of pile stiffness while maintaining computational stability (Engin et al. 2007; Sadek and Shahrour 2004).

Specifically, the piles are represented with the embedded beam formulation, in which each pile is a beam element inserted into the soil mesh and coupled to it through special line-to-volume interface elements along the shaft and a point-to-volume interface at the base (Sadek and Shahrour 2004). The formulation was extended for pile applications with elasto-plastic skin and foot interfaces, so that a limiting skin traction profile and an ultimate base resistance can be prescribed (Engin et al. 2007). Compared with a solid (volume element) representation of each pile, this approach reduces the mesh size by roughly an order of magnitude for a 25-pile group, which is essential for performing long-term coupled consolidation analyses at acceptable cost while still recovering axial force, shear and bending moment distributions for every pile.

The properties assigned to the structural elements are listed in Table 4.3. For the embedded beams, the limiting skin traction is defined as layer-dependent, so that the available shaft resistance follows the strength of the adjacent soil through the interface factor, and an ultimate base resistance $F_{\max}$ is prescribed at the toe consistent with the floating character of the piles.

Table 4.3. Properties of the structural elements (piles and pile cap).

Element	Property	Value
Pile (embedded beam)	Cross-section	Solid circular
	Diameter D	1.0 m (0.8 and 1.2 m in series C18–C19)
	Length L	30 m (20, 24 and 36 m in series C06–C08)
	Cross-sectional area A	0.7854 m ²
	Second moment of area $I_2 = I_3$	0.04909 m ⁴
	Material behavior	Linear elastic
	Elastic modulus E	30×10^6 kPa (C30)
	Unit weight γ	25.0 kN/m ³
	Axial skin resistance	Layer dependent
	Limiting base resistance $F_{\max}$	300 kN (2500 kN in C09–C10)
Pile cap (plate)	Thickness	1.5 m
	Material behavior	Linear elastic, isotropic
	Unit weight γ	25.0 kN/m ³
	Elastic modulus $E_1 = E_2$	30×10^6 kN/m ²
	Poisson ratio ν_{12}	0.20
	Shear moduli $G_{12} = G_{13} = G_{23}$	12.5×10^6 kN/m ²

The shaft resistance of the embedded beams was not prescribed as a fixed limiting traction but derived from the strength of the layer through which each segment of the pile passes. In this formulation the limiting shear that can be transferred along the shaft is computed from the local effective stress state together with the interface strength reduction factor, so that the resistance grows with depth as the effective overburden increases and responds to the additional stress imposed by the embankment. Two consequences that follow are important for the present study. First, the interface factor genuinely governs the shaft response, which makes the series C14–C15 a meaningful investigation rather than a formal one. Second, and more importantly for an asymmetric problem, limiting resistance is no longer identical for every pile of the group: the piles beneath the embankment experience a higher effective stress and therefore mobilize a larger shaft resistance than those on the far side. A single prescribed traction profile would have suppressed precisely the spatial variation that this study sets out to examine.

The base resistance, by contrast, was prescribed as a limiting value rather than computed from the bearing capacity of the underlying material. A value of 300 kN was adopted for the reference configuration, which is small relative to the shaft capacity and therefore preserves the intended floating character of the group. Where the geometry places the pile toe within the stiff stratum, as occurs once the clay layer is thinned in series C09–C10, retaining this value would have implied a toe condition inconsistent with the surrounding material; the limiting resistance was therefore raised to 2500 kN for those two analyses on the basis of a conventional bearing capacity estimate for the stiff layer. This series consequently reflects the combined influence of clay thickness and toe support rather than thickness alone and is interpreted accordingly in Chapter 5.

The elastic modulus and cross-sectional properties of the piles are selected to reflect typical reinforced concrete piles used in practice. Previous research has shown that, for NSF-related problems, pile material nonlinearity has a minor influence compared to soil deformation and interface behavior, provided that the piles remain within the elastic range (Fellenius 2006; Huang et al. 2015). Accordingly, piles are assumed to behave elastically throughout the analysis.

Pile length is chosen such that the piles remain floating within the compressible soil layer, with no significant contribution from end-bearing resistance. This is a deliberate modeling choice to investigate settlement-induced shaft loading in the floating group considered in this study.

A close-up PLAXIS view of the embedded-beam pile group, rigid pile cap and adjacent surcharge is given in Figure 4.5.

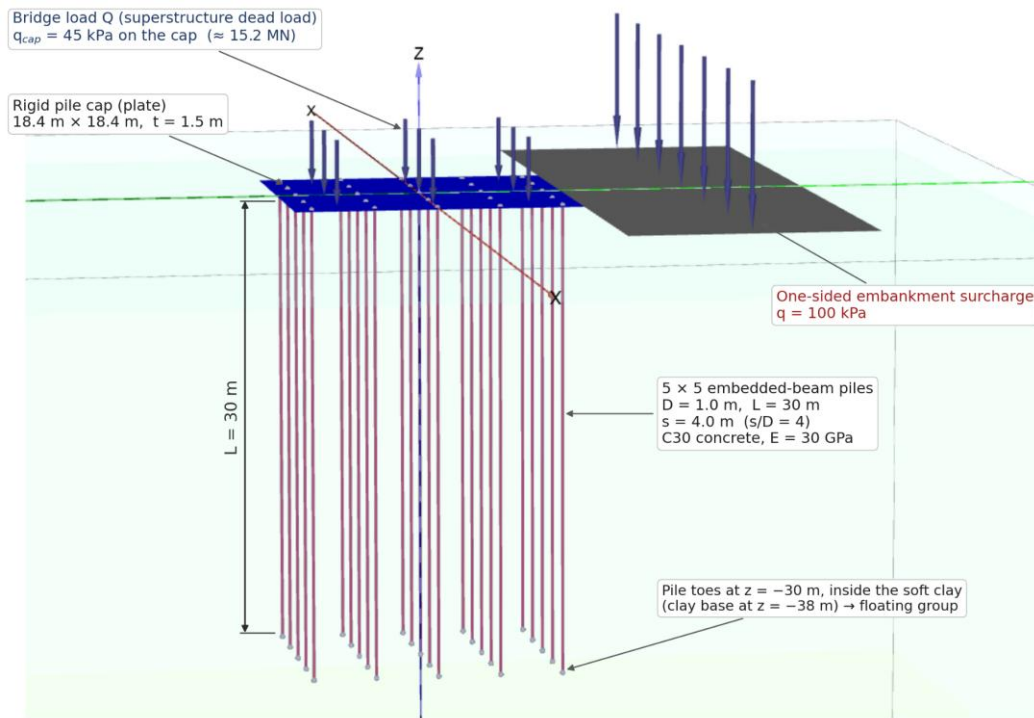


Figure 4.5. Three-dimensional PLAXIS representation of the rigid pile cap, 5×5 embedded-beam pile group and one-sided surcharge in the reference model.

4.4.2 Pile Cap Modeling

The pile cap is modeled as a rigid plate element connecting the pile heads. The rigid cap assumption enforces equal vertical displacement at the pile heads and ensures settlement compatibility within the group. This modeling approach has been shown to significantly affect dragload redistribution and neutral plane behavior in pile groups (Huang et al. 2015; Poulos 1989).

The plate properties adopted for the cap are $E_1 = E_2 = 30 \times 10^6 \text{ kN/m}^2$, $\nu_{12} = 0.20$ and $G_{12} = G_{13} = G_{23} = 12.5 \times 10^6 \text{ kN/m}^2$ at a thickness of 1.5 m. The resulting plate rigidity is checked as follows.

$$D_{\text{plate}} = E \cdot t^3 / [12(1 - \nu^2)]$$

$$D_{\text{plate}} = 30 \times 10^6 \times 1.5^3 / [12(1 - 0.20^2)] \approx 8.79 \times 10^6 \text{ kN} \cdot \text{m/m}$$

where:

D_{plate} = plate flexural rigidity per unit width

E = elastic modulus of the cap plate

t = plate thickness

ν = Poisson ratio

This rigidity is several orders of magnitude larger than the stiffness of the surrounding soft clay. The term rigid cap therefore refers to relative rigidity: cap bending and differential deformation are negligible compared with pile and soil deformation, while the plate remains a finite-stiffness structural element.

4.5 Soil–Pile Interface Modeling

The interaction between the piles and the surrounding soil is modeled using interface elements, which allow relative displacement and slip between the pile surface and the soil. Accurate representation of interface behavior is essential for simulating negative skin friction, as NSF develops directly from relative pile–soil movement (Alonso et al. 1984; Matyas and Santamarina 1994).

The interface strength parameters are defined as a fraction of the corresponding soil strength parameters using an interface reduction factor. This approach accounts for differences in roughness and bonding between the pile surface and the soil, and it has been widely used in numerical studies of pile foundations (Engin et al. 2007; Lee et al. 2002).

In the reference case the interface reduction factor is $R_{\text{inter}} = 0.7$, representing a bored pile in soft clay. The interface strength parameters are obtained from the soil parameters using the following relations.

$$\tan \delta' = R_{\text{inter}} \cdot \tan \phi'$$

$$c_a = R_{inter} \cdot c'$$

where:

R_{inter} = interface strength reduction factor

δ' = effective pile–soil interface friction angle

ϕ' = effective soil friction angle

c_a = interface adhesion

c' = effective soil cohesion

Once interface slip is mobilized, negative skin friction is governed by the limiting interface shear strength; consequently, R_{inter} is one of the influential parameters in the analysis. Its effect is examined explicitly in cases C14–C15, where $R_{inter} = 0.5$ and 0.9 .

Interface elements enable the numerical model to capture:

- reversal of shear stress direction along the pile shaft,
- progressive mobilization of negative and positive skin friction,
- realistic development of dragload during consolidation.

Without interface elements, relative displacement between the pile and soil would be constrained, leading to unrealistic stress transfer and inaccurate prediction of NSF (Engin et al. 2007).

4.6 Boundary Conditions and Initial Stress State

4.6.1 Boundary Conditions

Boundary conditions are selected to replicate an infinite soil domain while preventing rigid-body motion. The vertical boundaries of the model are constrained against horizontal displacement but are free to move vertically, allowing soil settlement to develop naturally.

The bottom boundary is fixed in all directions to prevent vertical and horizontal movement, representing a sufficiently deep soil layer below the zone of influence of the pile group. The adequacy of the chosen boundaries was assessed through the computed displacement and excess pore-pressure fields, as described in Section 4.2.1.

4.6.2 Initial Stress State

The initial in-situ stress state of the soil is established using the K_0 procedure, which defines the initial effective stresses based on the coefficient of earth pressure at rest. This approach is appropriate for normally consolidated and lightly overconsolidated soils and has been widely adopted in numerical studies involving consolidation and settlement-induced NSF (Lee and Ng 2004).

The coefficient of earth pressure at rest is selected based on soil type and stress history, following established empirical relationships. Establishing a realistic initial stress state is essential for accurate simulation of subsequent consolidation and settlement processes (Terzaghi 1943).

4.7 Consolidation and Loading Stages

Negative skin friction is induced in the numerical model through settlement-generating mechanisms rather than direct application of additional axial loads to the piles. This approach reflects the physical origin of NSF and is consistent with theoretical and experimental observations (Alonso et al. 1984; Fellenius 1972).

4.7.1 Consolidation Simulation

Consolidation is simulated by allowing excess pore water pressures generated by applied loading to dissipate over time. This time-dependent analysis captures the gradual development of soil settlement relative to pile displacement, which governs the mobilization of NSF (Indraratna et al. 1992).

Appropriate drainage conditions are defined at the ground surface, while lateral drainage is restricted to reflect typical field conditions. This setup ensures that consolidation occurs predominantly in the vertical direction.

In the calculation phases, the ground surface is defined as a draining boundary while the vertical model faces are closed to flow, so that dissipation occurs predominantly vertically through the clay. Following surcharge activation, one consolidation phase is computed

and continued until the maximum excess pore pressure falls below 1 kPa. By that point the excess pore pressures in the clay have largely dissipated and negative skin friction has approached its long-term value, consistent with the time-factor considerations of Section 3.5.

4.7.2 Loading Sequence

The numerical analysis is conducted in stages to replicate realistic construction and loading conditions:

1. Generation of initial stresses,
2. Installation of piles and pile cap,
3. Application of structural load to the pile cap (if applicable),
4. Application of surcharge load or equivalent mechanism inducing consolidation,
5. Time-dependent consolidation analysis.

Staged construction has been shown to significantly influence NSF development and neutral plane evolution, particularly in soft soils (Indraratna et al. 1992; Lee and Ng 2004).

In the reference (asymmetric) case, the embankment surcharge of $q = 100$ kPa is applied as a uniformly distributed load on the ground surface on one side of the group only, immediately adjacent to the pile cap (offset $d = 0$). After the surcharge is activated, the analysis proceeds directly to a single consolidation phase, which is continued until the maximum excess pore pressure anywhere in the mesh falls below 1 kPa. For the reference configuration this condition was reached after approximately 23 800 days. All results reported in Chapter 5 correspond to this fully dissipated condition. In the symmetric control case, the same surcharge intensity is applied on all four sides of the group, with all other parameters unchanged, so that any difference in response can be attributed solely to the loading geometry.

4.7.3 Evaluation of Output Parameters

The primary outputs extracted from the numerical analyses include:

- vertical displacement profiles of soil and piles,
- shaft shear stress distribution along individual piles,
- axial force distribution within piles,
- location of the neutral plane for each pile.

The neutral plane is identified as the depth at which the relative displacement between soil and pile changes sign, or equivalently where shaft shear stress transitions from negative to positive (Alonso et al. 1984; Fellenius 1984).

Because the one-sided surcharge also mobilizes a flexural response, the peak bending moments about the two cross-sectional axes (M_2 , in the loading direction, and M_3 , transverse to it), the shear forces and the lateral displacement u_y of each pile are additionally extracted. The increase in axial force from the pile head to the neutral plane is $N_{\max} - Q$, where Q is the axial force at the topmost node of the pile and $N_{\max}$ the maximum axial force along it, and the NSF dragload Q_{nsf} is obtained by subtracting the self-weight of the pile above the neutral plane as defined in Chapter 3. The toe resistance Q_p is read at the lowest node.

A brief clarification is necessary regarding the determination of the neutral plane. Strictly, it corresponds to the depth at which the pile and surrounding soil undergo equal vertical displacement and may also be identified from the reversal of shaft shear. In this study, the depth of maximum axial force was used because it could be extracted consistently for all twenty-five piles across the twenty-three principal analyses. Under fully mobilized negative skin friction, these indicators are expected to be close, as confirmed by representative shaft-shear checks. However, under very light surcharge, where little or no dragload developed, the maximum-axial-force depth should be regarded as a nominal value rather than a fully developed neutral plane.

4.8 Data Extraction and Post-Processing

For every calculation phase, the internal forces of all embedded beams, including the axial force N , shear forces and bending moments M_2 and M_3 , together with the pile displacements are exported for all beam nodes. With about 125 nodes per pile, a single

analysis of the 25-pile group yields on the order of three thousand output rows, which makes manual processing impractical and motivates an automated workflow.

The exported data are processed in a purpose-built spreadsheet template. Each output node is first assigned to its pile by matching the node coordinates to the known pile positions. For every pile, the template then identifies the head load Q as the axial force at the topmost node, the maximum axial force $N_{\max}$ and its depth (taken as the neutral plane depth z_n), the toe resistance Q_p at the lowest node, the axial-force increase $N_{\max} - Q$, and the peak magnitudes of M_2 , M_3 and the lateral displacement u_y . Because compressive forces are negative in the sign convention of the software, absolute magnitudes are reported throughout. The per-pile quantities are finally aggregated into row averages (front to back) and displayed as 5×5 spatial maps of the group, which allows the response to be examined both pile-by-pile and by position. The same template is applied to every scenario, ensuring that all cases are reduced in an identical and reproducible manner.

4.9 Numerical Analysis Framework

The parametric study is organized around a reference case, denoted C00, from which each parameter is varied in turn while all others are held at their reference values. The asymmetric series is labeled C00–C19 and the symmetric control series S00–S02, as summarized in Table 4.4. Each analysis follows the staged sequence described above.

Table 4.4. Analysis framework: asymmetric parametric series (C) and symmetric control series (S).

Series	q (kPa)	s/D	L (m)	H_{clay} (m)	d (m)	R_{inter}	q_{cap} (kPa)	D (m)
C00	100	4.0	30	36	0	0.7	45	1.0
C01	25	4.0	30	36	0	0.7	45	1.0
C02	50	4.0	30	36	0	0.7	45	1.0
C03	150	4.0	30	36	0	0.7	45	1.0
C04	100	2.5	30	36	0	0.7	45	1.0
C05	100	6.0	30	36	0	0.7	45	1.0
C06	100	4.0	20	36	0	0.7	45	1.0
C07	100	4.0	24	36	0	0.7	45	1.0
C08	100	4.0	36	36	0	0.7	45	1.0

Series	q (kPa)	s/D	L (m)	H _{clay} (m)	d (m)	R _{inter}	q _{cap} (kPa)	D (m)
C09*	100	4.0	30	18	0	0.7	45	1.0
C10*	100	4.0	30	27	0	0.7	45	1.0
C11	100	4.0	30	36	2	0.7	45	1.0
C12	100	4.0	30	36	5	0.7	45	1.0
C13	100	4.0	30	36	10	0.7	45	1.0
C14	100	4.0	30	36	0	0.5	45	1.0
C15	100	4.0	30	36	0	0.9	45	1.0
C16	100	4.0	30	36	0	0.7	25	1.0
C17	100	4.0	30	36	0	0.7	70	1.0
C18	100	5.0	30	36	0	0.7	45	0.8
C19	100	3.33	30	36	0	0.7	45	1.2
S00	100	4.0	30	36	0	0.7	45	1.0
S01	50	4.0	30	36	0	0.7	45	1.0
S02	150	4.0	30	36	0	0.7	45	1.0

* In cases C09 and C10, the pile toes penetrate the underlying stiff stratum; therefore, these two cases do not represent fully floating toe conditions.

For clarity, the analysis cases are grouped according to the parameter investigated in Table 4.5.

Table 4.5. Analysis groups used in the parametric study and the corresponding investigated effects.

Group	Cases	Investigated effect	Values examined
Reference	C00	Reference configuration	Baseline values in Table 4.4
Surcharge intensity	C01–C03	Surcharge load, q	25, 50, 100, 150 kPa
Pile spacing	C04–C05	Normalized pile spacing, s/D	2.5, 4.0, 6.0
Pile length	C06–C08	Pile length, L	20, 24, 30, 36 m
Clay thickness	C09–C10	Compressible clay thickness, H _{clay}	18, 27, 36m
Embankment offset	C11–C13	Clear distance from surcharge to cap, d	0, 2, 5, 10 m
Interface strength	C14–C15	Interface reduction factor, R _{inter}	0.5, 0.7, 0.9
Pile-head load	C16–C17	Cap surcharge / pile-head load, q _{cap}	25, 45, 70 kPa

Group	Cases	Investigated effect	Values examined
Pile diameter	C18–C19	Pile diameter, D	0.8, 1.0, 1.2 m
Symmetric control	S00–S02	Loading configuration and surcharge intensity	Four-sided; $q = 50, 100, 150$ kPa

In the diameter series C18–C19 the center-to-center spacing is kept at its reference value of 4.0 m rather than at a constant s/D ratio, so that the pile grid and the finite element mesh remain unchanged between the three diameters. The normalized spacing consequently varies from 5.0 to 3.33 across this series, and the results are therefore interpreted together with the dedicated spacing series C04–C05 in order to separate the influence of diameter from that of relative spacing. The quantity of primary interest in this series is the ratio of the computed bending moment to the flexural capacity of the section, since both the flexural stiffness and the moment capacity of a pile increase with diameter while the shaft area, and hence the dragload, increases only in proportion to the perimeter.

4.10 Mesh Sensitivity

To assess mesh dependence using the actual numerical outputs, the C00 reference case was rerun with one mesh coarser and one mesh finer than the discretization adopted for the parametric study. All material parameters, staged construction, loading and the final excess-pore-pressure criterion were kept unchanged. The adopted mesh therefore occupies the middle level of the comparison and remains the discretization used for the C- and S-series analyses reported in Chapter 5.

Table 4.6. Influence of mesh refinement on the computed response of the reference case.

Quantity	Coarser Mesh	Used Mesh	Finer Mesh	Change, Used→ Finer
Mean head load Q (kN)	1216	1220	1205	-1.2 %
Net dragload, front row (kN)	1001	952	934	-1.8 %
Net dragload, group mean (kN)	404	364	357	-1.9 %
Front-to-back dragload ratio	2.23	2.30	2.39	+3.8 %
z_n / L , front row	0.434	0.451	0.456	+1.0 %
z_n / L , group mean	0.489	0.497	0.505	+1.7 %
$M_{2,max}$ (kNm)	2190	2105	2019	-4.1 %
$u_{y,max}$ (mm)	99.1	98.8	98.1	-0.7 %
Mean toe resistance (kN)	303	301	303	+0.6 %

The comparison shows a clear tendency toward convergence. Relative to the coarser mesh, the adopted discretization reduces the group-mean net dragload from 404 to 364 kN and the peak bending moment about the loading axis from 2190 to 2105 kNm. Refining the adopted mesh further changes the front-row net dragload by only -1.8%, the group-mean net dragload by -1.9%, the front-row normalized neutral-plane depth by +1.0%, and the group-mean normalized neutral-plane depth by +1.7%. The maximum lateral displacement changes by only -0.7%. The largest remaining change is in the peak bending moment, which decreases by 4.1% from 2105 to 2019 kNm.

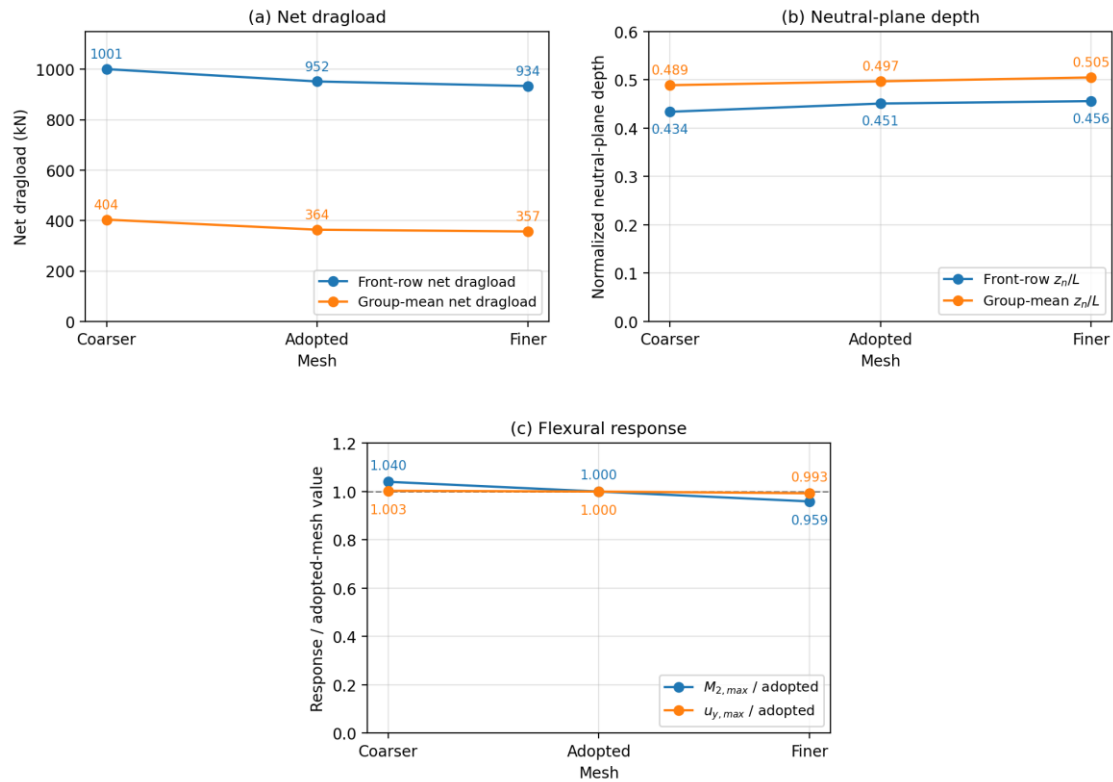


Figure 4.6. Effect of mesh refinement on the computed response of the reference case: (a) net dragload, (b) neutral plane depth, (c) flexural response.

The spatial pattern is also stable. The front-to-back net-dragload ratio increases from 2.23 on the coarser mesh to 2.30 on the adopted mesh and 2.39 on the finer mesh; the adopted-to-finer change is 3.8%. Mean toe resistance remains essentially unchanged at approximately 301-303 kN, and the mean head load varies by only 1.2% between the adopted and finer meshes. These results indicate that the quantities controlling the interpretation, neutral-plane depth, lateral displacement, bending response and front-to-back asymmetry, are substantially mesh-independent at the adopted discretization.

On this basis, the adopted mesh used throughout the parametric study is retained. The finer rerun does not materially alter any of the conclusions drawn from C00, while increasing the computational cost. For the adopted-to-finer refinement, all principal response measures vary by less than about 5%, and the axial dragload measures by about 2% or less. The coarser mesh, by contrast, gives a group-mean net dragload approximately 11% higher than the adopted mesh, demonstrating that the refinement level used in the main analyses is justified.

A comparable difficulty arises in the spacing series C04–C05, where altering the center-to-center distance necessarily alters the plan dimensions of the cap. Had the distributed load been held constant, the total force delivered to the group would have varied by a factor of approximately five between the closest and widest spacings, and the resulting differences would have reflected the level of head load rather than the spacing itself. The surface pressure was therefore scaled inversely with the cap area so that the vertical load carried by each pile remained essentially unchanged, and the self-weight of the cap was excluded from these analyses because it would otherwise have reintroduced the same imbalance. The embankment was shifted together with the edge of the cap so that the clear distance between the fill and the foundation remained zero in all three cases.

5. RESULTS AND DISCUSSION

This chapter first compares the 2D and 3D idealizations, then presents the asymmetric reference case C00, the parametric study and the symmetric control analyses. The results are then compared directly. All results reported here correspond to the end of consolidation: rather than stopping at a fixed time, each analysis was continued until the excess pore pressures had essentially dissipated, which for the reference case required approximately 23 800 days, or some 65 years. The values therefore represent the final long-term condition rather than an intermediate state. The row-averaged response of the group in the reference case is summarized in Table 5.1.

5.1 Comparison of 2D and 3D Idealizations

To clarify the added value of the full three-dimensional model, the reference case was also examined through a 2D plane-strain idealization and compared with the 3D 5×5 pile group on the same front-to-back basis. In the 2D model, one embedded-beam row is obtained for each longitudinal position, so the results are reported as a single value per row in kN/m; for visual comparison, these row forces may be spacing-normalized by the 4 m pile spacing, but this normalization does not create separate corner, edge, and center piles. In the 3D model, by contrast, all twenty-five piles are resolved individually.

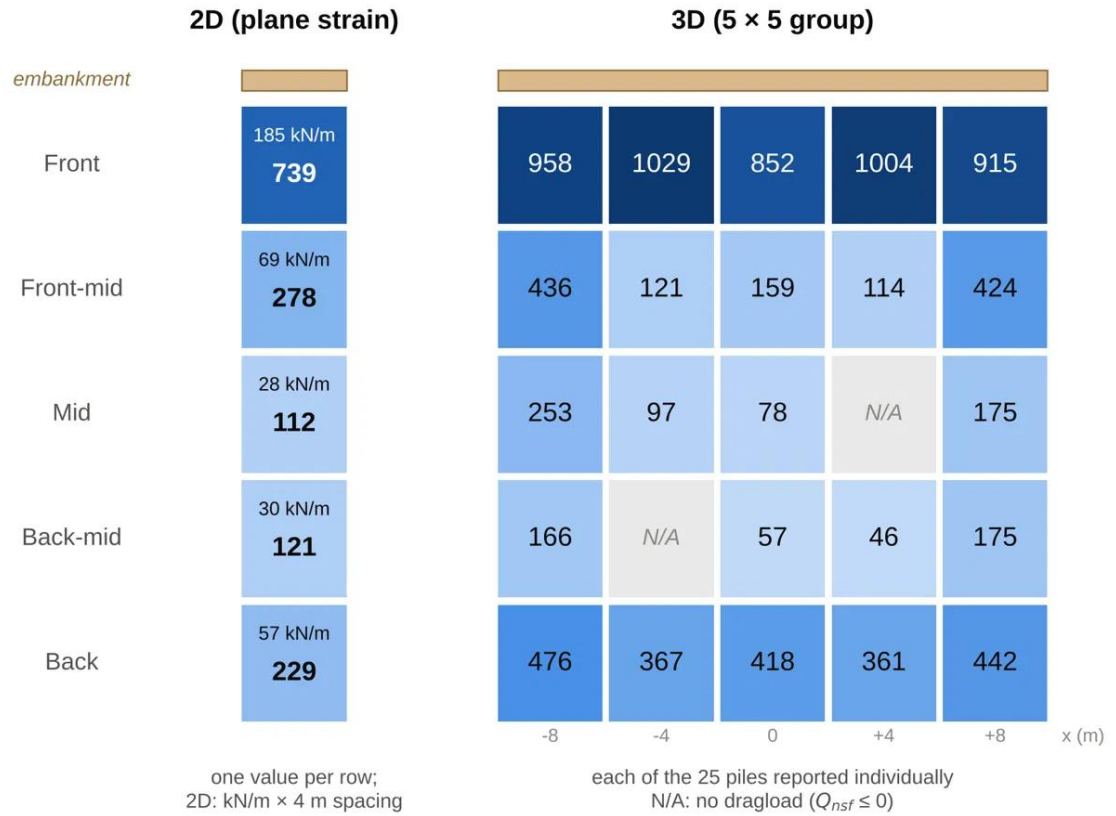


Figure 5.1. Comparison of the net dragload obtained from the 2D plane-strain idealization and from the 3D 5×5 pile-group model for the reference case.

Figure 5.1 shows that both idealizations identify the front row as the most heavily affected zone and both indicate a non-uniform response along the group length. However, the 2D model collapses each row into a single representative value and therefore cannot reproduce the strong transverse variability seen in the 3D group, where corner, edge and inner piles of the same row may differ markedly and some piles develop no net dragload at all.

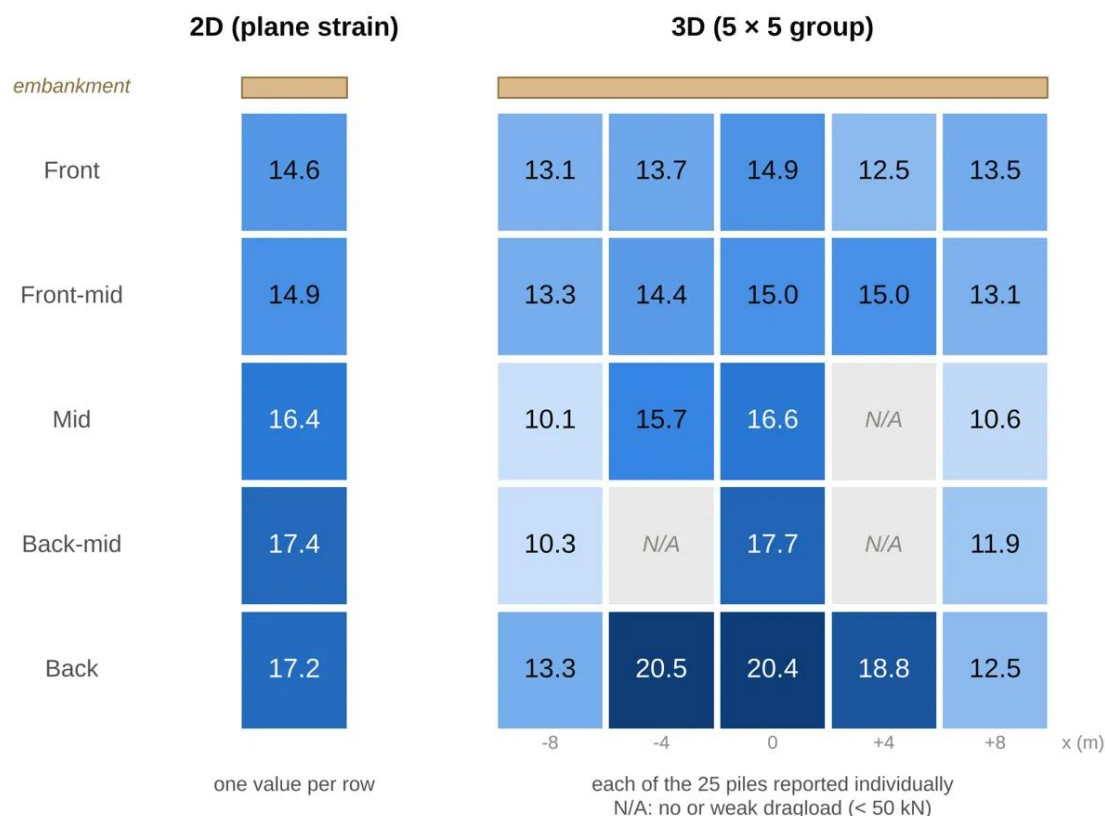


Figure 5.2. Comparison of the neutral-plane depth obtained from the 2D plane-strain idealization and from the 3D 5×5 pile-group model for the reference case.

The neutral-plane comparison leads to the same conclusion. The 2D idealization suggests a smooth row-by-row trend, whereas the 3D analysis reveals a pile-specific response with substantial scatter even within the same row. This difference is a direct consequence of pile–pile interaction, shielding, and cap-induced load redistribution, all of which are inherently three-dimensional effects. Accordingly, the 2D plane-strain model can be useful for preliminary interpretation of the overall front-to-back tendency, but it is not sufficient when the objective is to evaluate each pile separately or to quantify the combined axial and lateral response of the full group.

5.2 Asymmetric Reference Case (C00): Preliminary Results

The pile-row terminology used in Table 5.1 and in the subsequent front-to-back comparisons is shown explicitly in Figure 5.3(a); panels (b) and (c) place that nomenclature beside the corresponding dragload distributions.

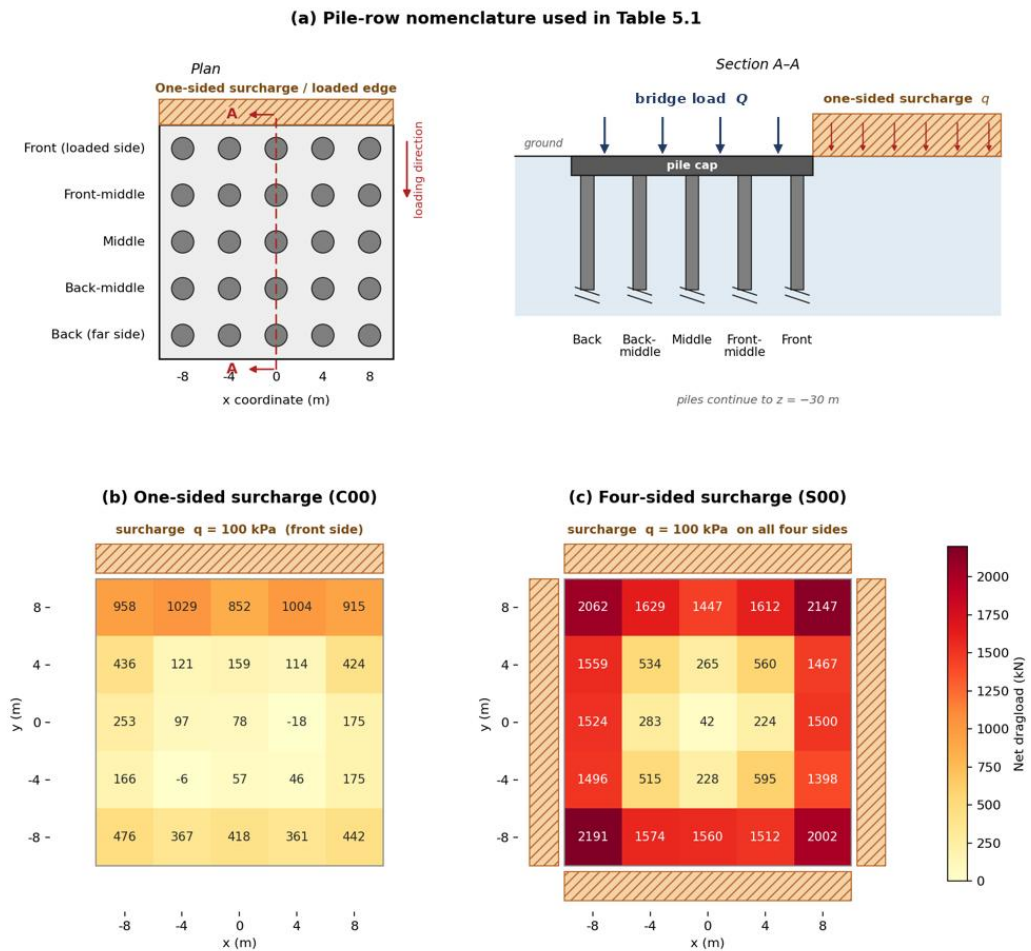


Figure 5.3. Pile-row nomenclature and net dragload distribution at the end of consolidation:

Table 5.1. Row-averaged response of the 5×5 floating pile group under one-sided surcharge (reference case C00) at the end of consolidation.

Row (position)	Q (kN)	$N_{\max} - Q$ (kN)	W_{pile} (kN)	Q_{nsf} (kN)	z_n (m)	$M2$ (kNm)	u_y (mm)
Front (loaded side)	974	1217	266	952	13.54	2015	95.4
Front-middle	1541	529	278	251	14.16	859	70.4
Middle	1482	395	279	117	14.19	446	53.2
Back-middle	1305	393	306	87	15.57	230	41.9
Back (far side)	798	749	336	413	17.11	441	34.8

Three features of the reference response deserve attention. First, the flexural response follows the loading directly and decays across the group: the mean bending moment in the loading direction falls from 2015 kNm in the front row to 441 kNm in the back row,

a ratio of about four and a half, while the lateral displacement decreases from 95 to 35 mm over the same distance. The peak loading-direction moment of 2105 kNm is sufficiently large that flexure must be assessed together with the axial response rather than ignored. A supplementary reinforced-concrete N–M interaction screening is presented in Appendix A; for the reference case the critical same-depth pair gives a screening index of about 0.67 for the representative 1.0 m pile section. This comparison is used only as a structural reasonableness check, because the numerical load program did not distinguish characteristic and design load combinations. Second, once the self-weight of the pile is removed the net dragload is distributed in a markedly uneven manner: the front row attracts 952 kN and the back row 413 kN, while the three interior rows carry between 87 and 251 kN. The piles beneath the embankment are exposed to a higher effective stress and therefore mobilize more shaft resistance, whereas the interior of the group is shielded by its own perimeter. Third, the neutral plane deepens steadily from the loaded side toward the far side, from 13.5 m to 17.1 m, that is from 0.45 to 0.57 of the pile length. This gradient mirrors the distribution of structural load enforced by the rigid cap: the interior rows carry the largest head load, close to 1500 kN, and the back row the smallest at 798 kN, and a pile carrying less head load requires a longer length of positive shaft resistance to reach equilibrium. The relationship between head load, neutral plane depth and dragload observed here within a single group is the same one that emerges across the parametric study, and it reproduces the load dependence of the neutral plane demonstrated experimentally by Leung et al. (2004).

5.3 Parametric Series (C01–C19)

The reference behavior described above was then examined as each parameter of the parametric study was varied in turn. The quantities reported for every case are the net dragload averaged over the group, the depth of the neutral plane normalized by the pile length, the largest bending moment developed in the loading direction, the peak lateral displacement and the mean toe resistance. Because the self-weight correction depends on the neutral plane depth, and because both the pile length and the diameter change within the program, all normalized quantities are computed with the geometry of the case to which they belong rather than with the reference values.

Table 5.2. Summary of the asymmetric parametric study at the end of consolidation.

Case	Varied value	Q_{nsf}	F/B	z_n/L	M2,max	$u_{y,max}$	Q_p
C01	$q = 25 \text{ kPa}$	-0	-	0.468 *	597	20.9	289
C02	$q = 50 \text{ kPa}$	111	-	0.507 *	1024	39.3	303
C00	$q = 100 \text{ kPa}$ (ref.)	364	2.30	0.497	2105	98.8	301
C03	$q = 150 \text{ kPa}$	562	1.81	0.497	3746	230.6	302
C11	$d = 2 \text{ m}$	304	3.22	0.522	1960	91.4	302
C12	$d = 5 \text{ m}$	219	4.78	0.532	1548	81.9	303
C13	$d = 10 \text{ m}$	114	11.29	0.530	1061	70.1	300
C06	$L = 20 \text{ m}$	183	2.25	0.344	2727	114.1	304
C07	$L = 24 \text{ m}$	264	2.57	0.417	2497	108.5	305
C08	$L = 36 \text{ m}$	479	2.59	0.572	2036	94.8	306
C09	$H_{clay} = 18 \text{ m}$	568	3.43	0.654	2019	91.2	619
C10	$H_{clay} = 27 \text{ m}$	608	3.00	0.764	2074	104.7	992
C14	$R_{inter} = 0.5$	279	1.43	0.351	2228	102.0	302
C15	$R_{inter} = 0.9$	403	3.59	0.568	2076	96.1	301
C16	cap load 25 kPa	394	2.44	0.553	2048	96.6	302
C17	cap load 70 kPa	329	2.38	0.430	2221	104.2	300
C18	$D = 0.8 \text{ m}$	326	1.77	0.411	1516	114.8	195
C19	$D = 1.2 \text{ m}$	356	3.39	0.553	2839	87.9	430
C04	$s/D = 2.5$	269	13.42	0.513	2782	101.9	300
C05	$s/D = 6$	263	4.25	0.423	2387	93.8	297

Figure 5.4 presents the outcome of the parametric study in a form that allows the two quantities of interest to be read together. In each panel the bars give the net dragload averaged over the group and the line gives the neutral plane depth normalized by the pile length, so that the axial magnitude and the position at which the axial force reverses can be compared directly.

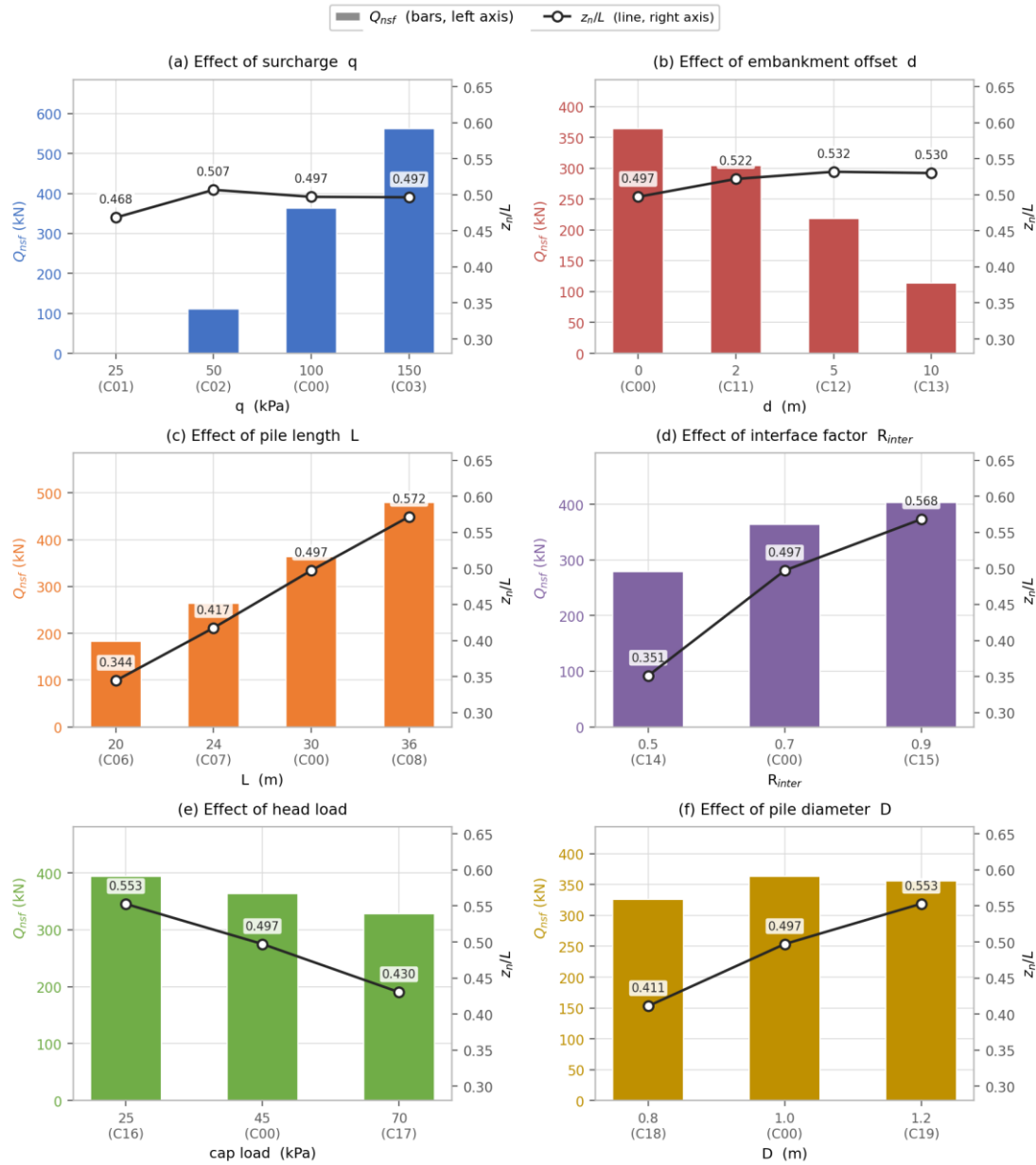


Figure 5.4. Influence of each parameter on the net dragload (bars, left axis) and on the normalized neutral plane depth (line, right axis). The reference case C00 appears in every panel and provides the common point of comparison.

5.3.1 Surcharge Intensity

The intensity of the embankment load emerges as the strongest single influence. Raising the surcharge from 25 to 150 kPa increases the mean net dragload from a value that is effectively zero to 562 kN, while the peak bending moment rises from 597 to 3746 kNm and the largest lateral displacement from 21 to 231 mm. The response is not merely proportional: between 25 and 50 kPa the increase in net dragload is relatively limited for

much of the group, whereas between 100 and 150 kPa every quantity grows sharply. The reason lies in the relative movement that drives the mechanism. At low surcharge the compressible layer settles only slightly more than the piles themselves, so the shaft shear reverses at shallow depth and much of the group develops no dragload at all; in the lightest case seventeen of the twenty-five piles show a net dragload of zero or below. As the load increases, the settlement of the surrounding soil outpaces that of the group over a growing length of shaft, and the mechanism engages fully.

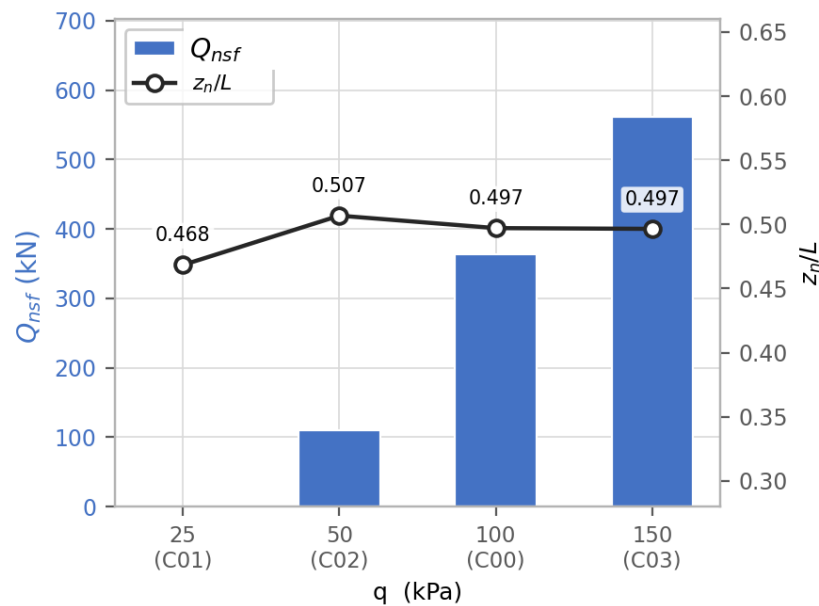


Figure 5.5. Influence of the embankment surcharge on the net dragload and the neutral plane depth.

5.3.2 Embankment Offset

The distance between the embankment and the foundation was found to be the most effective means of moderating the lateral effects. Moving the fill from the edge of the cap to ten meters away reduces the peak moment from 2105 to 1061 kNm and the lateral displacement from 99 to 70 mm, while the mean net dragload falls from 364 to 114 kN. The decline is smooth rather than abrupt, and no threshold distance emerges within the range examined at which the asymmetric effects can be regarded as negligible. It is worth noting that the neutral plane hardly moves across this series, remaining between 0.50 and 0.53 of the pile length: the offset alters the magnitude of the loading transmitted to the group without changing the depth at which the axial force reverses.

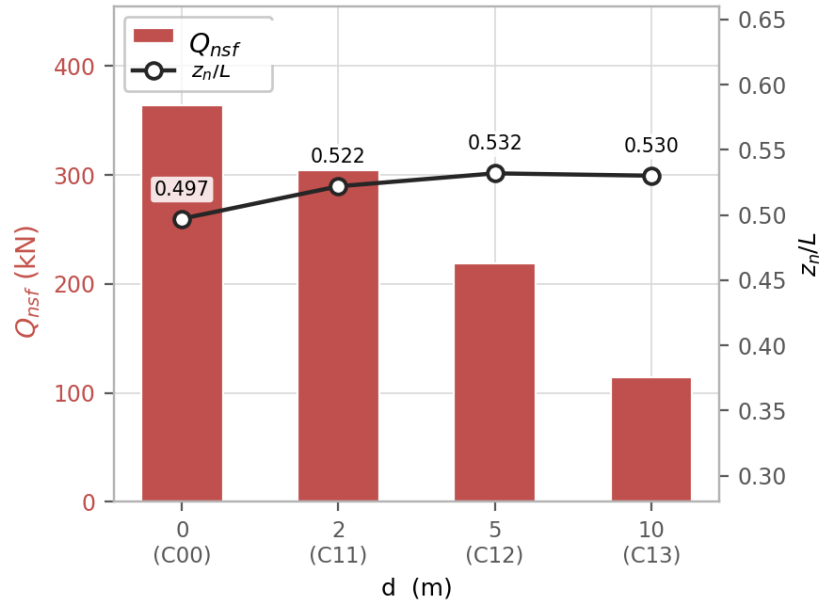


Figure 5.6. Influence of the clear distance between the embankment and the cap.

5.3.3 Pile Length

Pile length acts on the two aspects of the response in opposite directions. Lengthening the piles from 20 to 36 m raises the net dragload from 183 to 479 kN, since a longer shaft is exposed to the settling soil over a greater depth, and deepens the normalized neutral plane from 0.34 to 0.57. At the same time the peak bending moment falls from 2727 to 2036 kNm and the lateral displacement from 114 to 95 mm, because the longer piles are restrained more effectively by the deeper soil. Shortening the piles is therefore beneficial for the axial demand and detrimental for the flexural one, which is an inconvenient combination for design: the two criteria cannot be satisfied by the same choice.

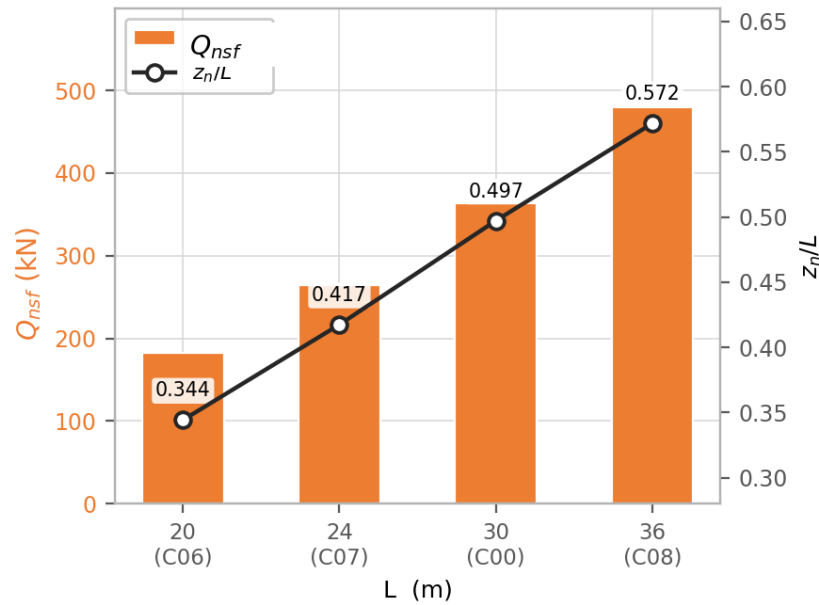


Figure 5.7. Influence of the pile length.

5.3.4 Clay Thickness and Toe Condition

Reducing the thickness of the compressible layer changes the character of the foundation rather than merely the magnitude of its response. When the clay is thinned so that the toe enters the stiff stratum, the mean toe resistance rises from 301 kN to 619 kN and then to 992 kN, and the normalized neutral plane descends from 0.50 to 0.65 and finally to 0.76. The group ceases to behave as a floating foundation and begins to transfer a significant proportion of its load through the base. Somewhat counter-intuitively the dragload increases rather than decreases, reaching 608 kN in the case with the intermediate clay thickness. The explanation lies in the restraint provided by the stiffer toe: the piles settle less than before, the relative movement between soil and shaft therefore grows, and negative skin friction develops over a longer portion of the shaft even though the compressible layer itself is thinner. As noted in Chapter 4, the limiting base resistance was adjusted in these two analyses to remain consistent with the material surrounding the toe, so the series reflects the combined effect of layer thickness and toe support.

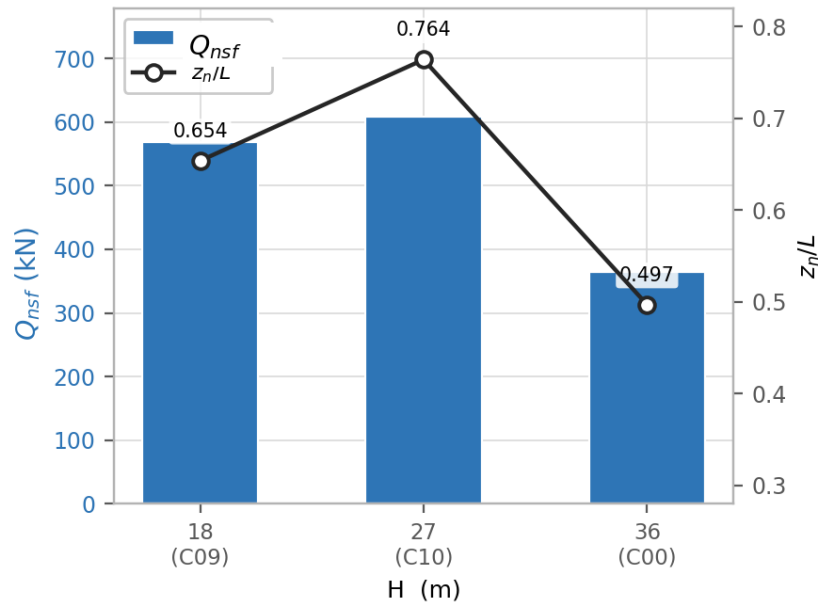


Figure 5.8. Influence of the clay thickness. In these analyses the pile toe enters the stiff stratum, so the limiting base resistance was also adjusted.

5.3.5 Interface Strength

The interface strength reduction factor exerts the influence that would be expected once the shaft resistance is derived from the properties of the surrounding layers. Increasing the factor from 0.5 to 0.9 raises the mean net dragload from 279 to 403 kN and deepens the normalized neutral plane from 0.35 to 0.57, while the bending response is almost unaffected, the peak moment changing by around seven percent. This separation is instructive: the interface governs how much shear the soil can transmit to the shaft and therefore controls the axial mechanism, whereas the flexural demand originates in the lateral movement of the soil mass and is largely indifferent to the roughness of the pile surface.

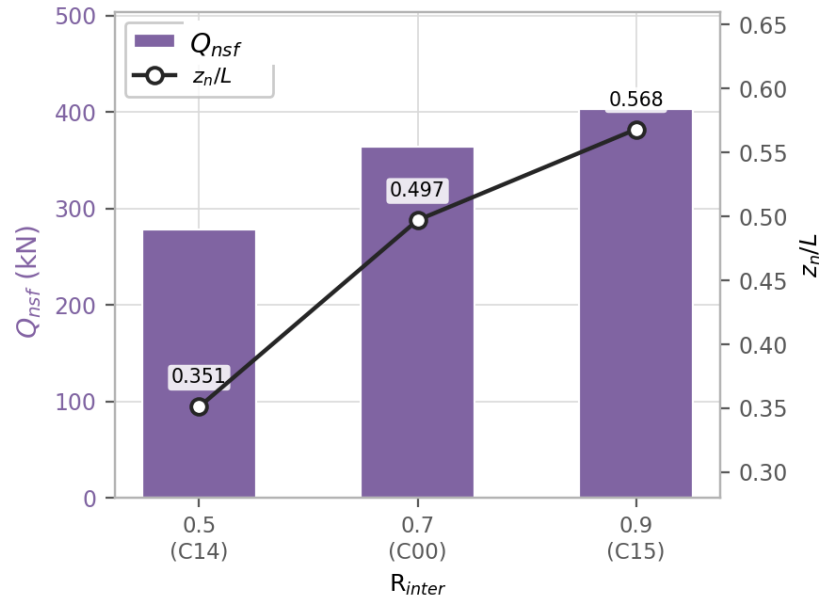


Figure 5.9. Influence of the interface strength reduction factor.

5.3.6 Head Load

The vertical load carried by the cap acts on the neutral plane in the manner predicted by equilibrium considerations. Increasing the load from 25 to 70 kPa raises the position of the neutral plane from 0.55 to 0.43 of the pile length and reduces the mean net dragload from 394 to 329 kN. A more heavily loaded pile requires a shorter length of positive shaft resistance to satisfy equilibrium, so the reversal occurs higher up and less negative friction accumulates above it. The same relationship was observed within the reference group itself, where the interior piles carry the largest share of the structural load and develop both the shallowest neutral plane and the smallest dragload. The consistency between the two observations, one across scenarios and one within a single group, supports the interpretation that the axial equilibrium of each pile is governed principally by the load it receives from the cap. This behavior reproduces the load dependence of the neutral plane demonstrated in centrifuge testing Leung et al. (2004).

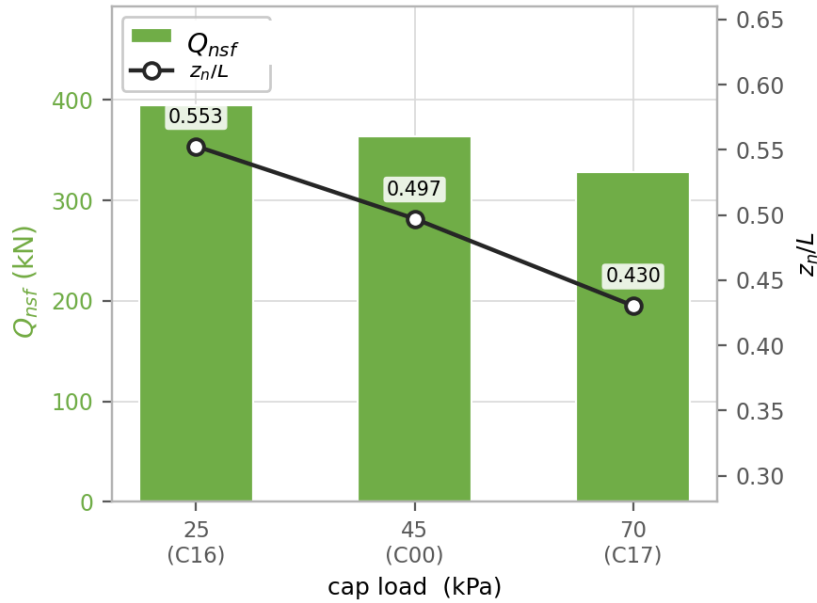


Figure 5.10. Influence of the vertical load transmitted through the cap.

5.3.7 Pile Diameter

The diameter series provides a clear stiffness trend. A larger pile attracts a greater bending moment, the peak increasing from 1516 kNm at 0.8 m diameter to 2839 kNm at 1.2 m, because the flexural stiffness of the section grows rapidly with diameter while the imposed soil movement remains unchanged. To place this trend on a consistent structural basis, the three diameter cases were examined using the reinforced-concrete N–M interaction screening documented in Appendix A. Comparable sections with nearly constant longitudinal reinforcement ratios were used: 19Φ32 for $D = 0.8$ m, 30Φ32 for $D = 1.0$ m and 43Φ32 for $D = 1.2$ m. Using axial force and resultant bending moment taken at the same depth, the maximum screening index decreases from approximately 0.92 to 0.67 and 0.54 as the diameter increases. This supports the relative conclusion that the larger sections provide greater structural reserve within the parametric comparison, without treating the screening as a code-based adequacy check. Lateral displacement follows the same trend, decreasing from 115 to 88 mm, whereas the axial dragload is comparatively insensitive to the diameter, the mean net value varying only between 326 and 364 kN across the three cases.

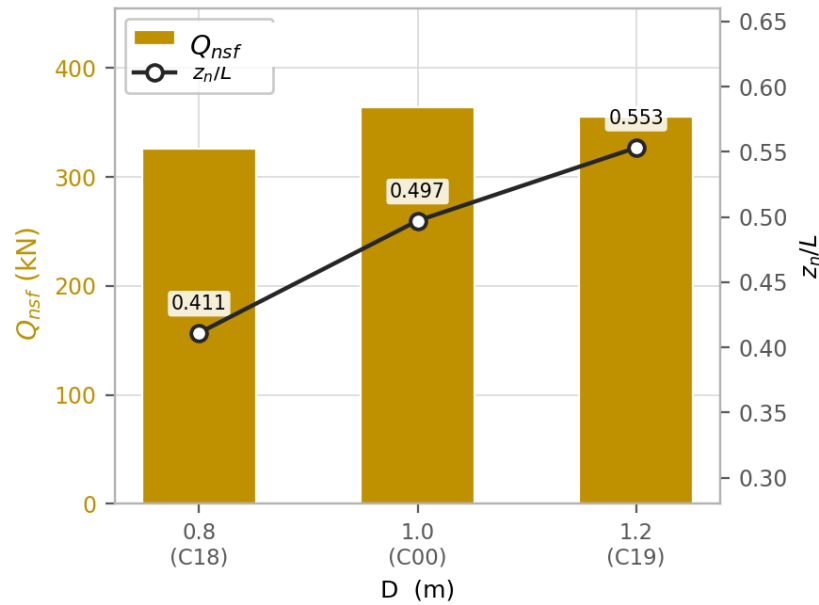


Figure 5.11. Influence of the pile diameter.

5.3.8 Pile Spacing

The spacing series must be interpreted with caution. Altering the center-to-center distance changes the plan area of the cap, and although the surface pressure was scaled so that the nominal load per pile remained constant, the axial force actually delivered to the piles still differed between the three cases by about sixteen percent because the contact between the cap and the settling soil scales with the area of the cap. The mean net dragload does not vary monotonically with spacing, standing at 269 kN for the closest arrangement, 364 kN for the reference and 263 kN for the widest, and the differences are of the same order as those produced by the residual variation in head load. The influence of spacing on the axial mechanism therefore cannot be isolated from the present results. The flexural response does show a systematic pattern, the peak moment being largest in the most closely spaced group at 2782 kNm, but the distribution of that moment within the group changes character in a way that is discussed in the following section.

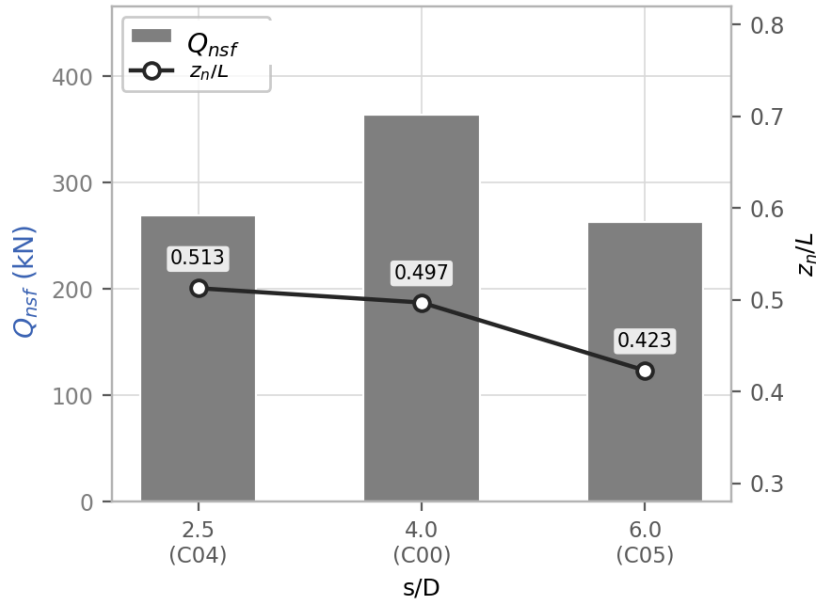


Figure 5.12. Influence of the pile spacing. The axial load per pile could not be held exactly constant across this series, so the trend should be read with caution.

5.4 Symmetric Control Case (S-Series)

The symmetric series was analyzed in order to establish which features of the response are specific to one-sided loading and which would arise under any surcharge of comparable intensity. In these analyses the same embankment pressure was applied on all four sides of the group, every other aspect of the model being left unchanged. The outcome was not the one anticipated at the outset, and it is worth setting out plainly.

It had been expected that symmetric loading would suppress the flexural response almost entirely, on the reasoning that the lateral pressures acting on opposite faces of the group would cancel. This expectation was not borne out. The largest resultant moment for an individual pile in the symmetric reference case is 3292 kNm, against an individual-pile maximum of 2183 kNm in the asymmetric case. These are peak values from individual piles, not the position-averaged values reported in Table 5.3; on that basis, the four-sided configuration produces the greater local flexural demand at the perimeter. Judged by peak values alone one would conclude, wrongly, that one-sided loading is the less severe of the two.

A further point of method arises before the two cases can be compared. In the asymmetric analyses the bending is overwhelmingly directed along the axis of loading, so the moment M_2 about that axis describes the response adequately: the transverse component M_3 adds only about five percent to the resultant. Under four-sided loading this is no longer true. Each pile is pushed outward along its own radial direction, so the two components are of comparable size and the resultant exceeds either of them by a considerable margin; for the corner piles M_2 and M_3 are 2283 and 2257 kNm respectively, giving a resultant of 3211 kNm. Reporting M_2 alone would therefore understate the symmetric case by some forty percent and distort the comparison. In what follows the resultant moment, defined as the square root of the sum of the squares of the two components, is used for both cases. The same reasoning applies in principle to the horizontal displacement; the component in the loading direction is retained here because it is the quantity of interest for an abutment, but it should be borne in mind that it underestimates the total movement of the perimeter piles in the symmetric case.

Table 5.3. Response of the reference group by pile position under one-sided (C00) and four-sided (S00) surcharge of the same intensity.

Position	n	M_{res} asym. (kNm)	M_{res} sym. (kNm)	u_y asym. (mm)	u_y sym. (mm)	Q_{nsf} asym./sym. (kN)
Corner	4	1347	3211	62.3	96.3	698 / 2101
Edge	12	987	2291	59.7	62.0	472 / 1523
Inner corner	4	559	1283	57.3	45.0	68 / 551
Inner edge	4	482	858	56.9	22.7	74 / 250
Center	1	435	187	56.6	1.7	78 / 42

Under four-sided loading the resultant moment and the lateral displacement decay in a regular fashion from the perimeter toward the interior, and the pile at the center of the group is left almost entirely undisturbed: it carries a resultant moment of 187 kNm and moves horizontally by 1.7 mm in the loading direction, which is to say that it is barely affected by a surcharge that has displaced the corner piles by some 96 mm. The pattern is concentric, and its physical meaning is that each pile is pushed outward by the fill on its own side while the group as a whole remains where it was.

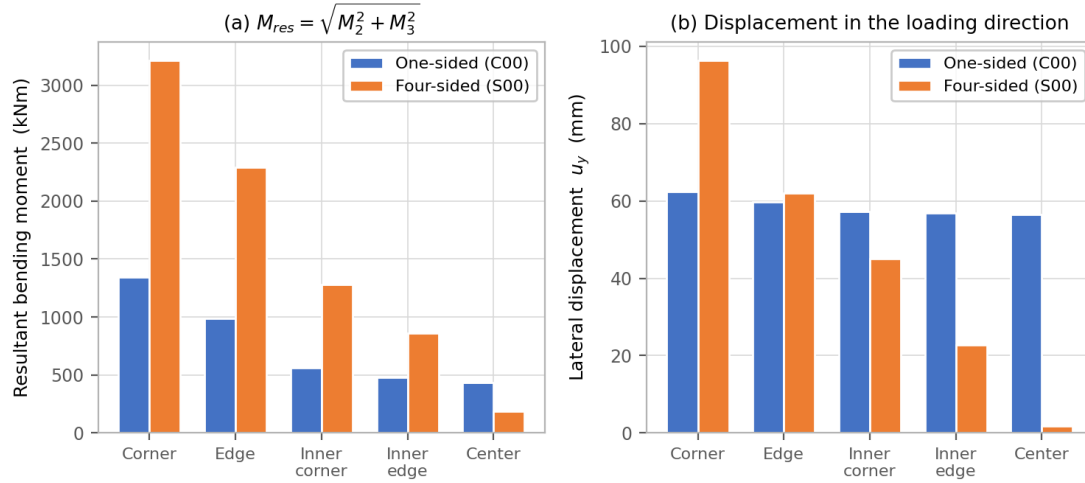


Figure 5.13. Comparison of one-sided and four-sided surcharge by pile position:

One-sided loading produces something different in kind. The resultant moment is smaller in absolute terms than in the symmetric case at every position around the perimeter, but it does not decay toward the interior: the center pile carries 435 kNm against 187 kNm in the symmetric case, and displaces by 57 mm against 1.7 mm. Every pile in the group, wherever it is situated, has moved away from the embankment by a broadly similar amount. The foundation has not merely deformed internally; it has translated bodily. This is the effect that the symmetric configuration does not reproduce, and it is the one that matters for a bridge abutment, since bearings and expansion joints respond to the movement of the structure as a whole rather than to distortion within the pile group.

The axial mechanism is likewise reorganized rather than merely rescaled. Under symmetric loading the dragload follows the concentric pattern of the settlement field, falling from 2101 kN at the corners to 42 kN at the center, a ratio of roughly fifty to one that indicates very pronounced shielding of the interior. This pronounced center-to-corner difference is a result of the present model and should not be treated as a general group-effect ratio without independent validation. Under one-sided loading the concentric pattern is replaced by a gradient across the group, the piles beneath the fill attracting rather more than twice the dragload of those on the far side.

A further feature of the symmetric case is worth recording because it illustrates the role of the cap. The structural load is distributed very unevenly among the piles, the corner piles receiving 417 kN and the central pile 2525 kN, which is the opposite of the distribution expected under a rigid cap carrying vertical load alone. When the dragload is added, however, the total axial force including pile self-weight is nearly the same in both, 2838 kN at the corner and 2799 kN at the center. The rigid cap has redistributed the structural load so as to compensate for the very different amounts of dragload attracted by piles in different positions. Neither component can be interpreted on its own; it is their sum that the cap holds in balance.

Increasing the symmetric surcharge from 50 to 150 kPa raises the mean net dragload from 680 to 1618 kN and the peak moment from 990 to 3922 kNm, so the concentric mechanism intensifies with load in the same way as its asymmetric counterpart without altering its character.

5.5 Group Effect and Shielding

The symmetric analyses provide the cleanest view of group behavior available in this study. Under four-sided loading the settlement field is concentric about the group, so any variation in dragload between piles reflects their position within the foundation rather than their distance from a one-sided source of loading. The asymmetric analyses superimpose the two effects and cannot separate them; the symmetric series therefore serves as the reference against which the group effect itself is quantified.

Piles were classified by position as corner, edge, inner corner, inner edge and center, which for a 5×5 arrangement gives 4, 12, 4, 4 and 1 piles respectively. The isolated-pile dragload used for normalization was computed using the β -method and neutral-plane force equilibrium described in Section 0, with the head load of the corresponding analysis, so that the comparison is made against a pile carrying the same structural load but surrounded on all sides by settling soil.

Figure 5.14 presents the result. The left panel gives the absolute dragloads for the three surcharge levels, and the right panel the same data normalized by the corner pile of each case.

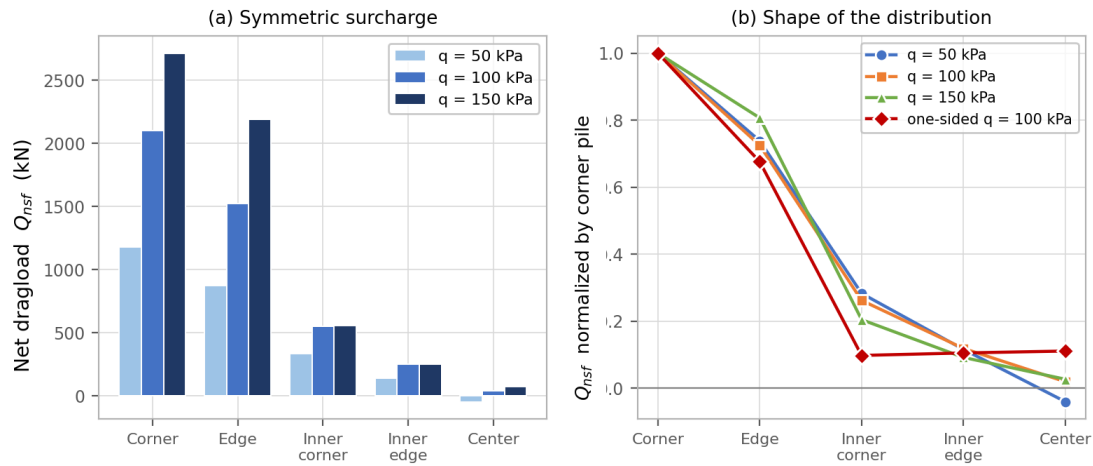


Figure 5.14. Distribution of net dragload by pile position

Two features stand out. The first is the severity of the reduction toward the interior. Under the reference surcharge the corner piles carry 2101 kN, the edge piles 1523 kN, the inner ring between 250 and 551 kN, and the central pile 42 kN. Expressed as a group factor relative to the corner, the values are 1.00, 0.72, 0.26, 0.12 and 0.02. The central pile of the group therefore attracts essentially no dragload at all, and the outer two rings carry almost the whole of it. The second feature is that the shape of this distribution is largely independent of the load level: the normalized curves for 50, 100 and 150 kPa lie close together in the right panel, although the absolute values differ by a factor of more than two. The group effect is thus a geometric property of the arrangement rather than a consequence of the loading intensity.

Table 5.4. Group factor by pile position under symmetric surcharge. Values are net dragloads normalized by the isolated-pile dragload computed for the same head load.

Position	n	S01 (50 kPa)	S00 (100 kPa)	S02 (150 kPa)	C00 (one-sided)
Corner	4	1.87	1.98	1.81	0.63
Edge	12	1.38	1.44	1.46	0.42
Inner corner	4	0.53	0.52	0.37	0.06
Inner edge	4	0.22	0.24	0.17	0.07
Center	1	-0.07	0.04	0.05	0.07
Group mean	25	0.86	0.88	0.83	0.33

Table 5.4 expresses the same result against the isolated-pile value rather than the corner pile. The perimeter piles exceed the isolated-pile dragload, the corner piles by a factor approaching two, while the interior piles fall far below it. At first sight the first of these is surprising, since shielding is usually thought of as a reduction. The explanation lies in the rigidity of the cap. Because the interior piles attract almost no dragload, equilibrium of the cap requires them to carry a larger share of the structural load, and the corner piles correspondingly less; in the reference symmetric case the central pile receives 2525 kN of head load against 417 kN at the corner. A pile with a small head load develops a deep neutral plane and accumulates negative friction over most of its length, which is exactly what the corner piles do. The group factor therefore measures the combined outcome of two mechanisms that act in the same direction at the perimeter and in opposite directions in the interior: the screening of the inner piles from the settling soil, and the redistribution of structural load by the cap in response to that screening.

The S00 reference case gives a group mean factor of 0.88; across the three symmetric cases the group mean factor ranges from 0.83 to 0.88. This indicates that the foundation as a whole attracts nearly as much dragload as twenty-five isolated piles would, even though it is distributed very unevenly among them. Under one-sided loading the mean falls to 0.33, but this figure is not comparable in kind: there the reduction reflects the fact that only part of the group lies beneath the fill, not that shielding is stronger.

The present analyses give a center-to-corner dragload ratio of approximately 0.02 under symmetric loading. Three features of this numerical configuration may contribute to the pronounced shielding. The compressible layer here is 36 m thick and the piles are floating within it, so the settlement available to be screened is large and the interior of the group can move with the soil rather than relative to it. The cap is modeled as fully rigid, which maximizes the redistribution of structural load described above; a flexible cap would permit differential settlement between piles and would reduce the effect. And the group is a full 5×5 arrangement, in which the central pile is screened on all sides by two complete rings, a shielding arrangement that cannot be assumed to develop to the same extent in smaller groups. The magnitude of the shielding reported here should therefore be regarded as specific to this configuration rather than as a general design value.

The influence of spacing could not be established from the present study. The three spacings examined give group mean factors of 0.24, 0.33 and 0.25 at s/D values of 2.5, 4 and 6, which is not a monotonic trend, and the differences are of the same order as the residual variation in head load between those analyses that was noted in Section 5.3.8. Since the literature indicates that group interaction diminishes with increasing spacing and becomes small beyond about five diameters (Huang et al. 2015), the absence of a clear trend here is more likely to reflect the limitations of the comparison than the absence of an effect. A dedicated analysis series in which the head load is applied directly at the pile heads, removing the cap from the load path, would be required to isolate it.

Two implications follow for design. The first is that the dragload cannot be assumed uniform across a large group: using a single value derived for an isolated pile would overestimate the demand on the interior piles by an order of magnitude while underestimating it at the corners. The second is that dragload and structural load are not independent quantities in a capped group. The corner piles of the symmetric reference case carry 417 kN of head load and 2101 kN of dragload, the central pile 2525 kN and 42 kN; the external force sums (head load + net dragload, excluding pile self-weight), 2518 and 2567 kN, differ by two percent. Because pile self-weight is the same for both positions, this near equality is also reflected in the total axial force. The cap therefore redistributes the structural load so as to balance the combined axial demand; neither component should be interpreted in isolation.

5.6 Neutral Plane Depth as a Function of the Load Ratio

The parametric results were examined for a single quantity that might govern the depth of the neutral plane across the whole study. The motivation comes from the equilibrium condition itself: the neutral plane forms where the head load together with the accumulated negative friction is balanced by the positive shaft resistance below, so its position should depend on the head load relative to the shaft resistance the pile can develop rather than on either quantity alone. This suggests a dimensionless load ratio.

Neutral-plane depth was related to the balance between the pile-head load and the shaft resistance available along the pile. The following dimensionless load ratio was defined for this comparison:

$$\lambda = \frac{Q}{R_s}, \quad R_s = \frac{1}{2}\pi D L^2 \beta \gamma' \quad (5.1)$$

in which Q is the axial load at the pile head, R_s the shaft resistance available over the full pile length, β the shaft-friction coefficient used in the simplified resistance estimate and γ' the submerged unit weight of the compressible layer. The factor of one half results from integrating a linearly increasing effective-stress profile over the pile length.

The normalized neutral-plane depths are plotted against the load ratio in Figure 5.15. The sixteen analyses, whose individual parameters differ widely, fall close to a single curve, although the interface-strength cases show more scatter than the other cases.

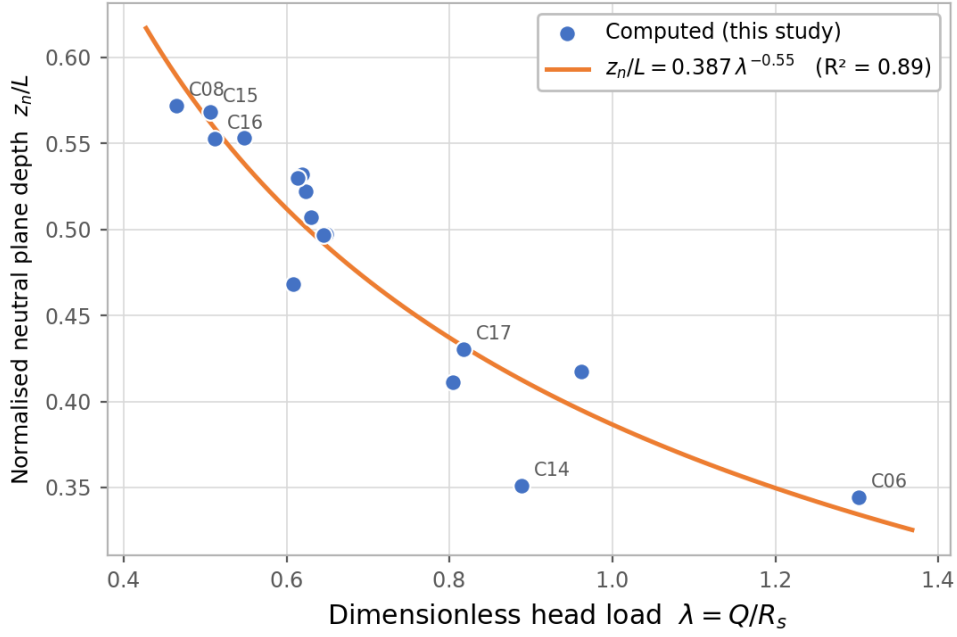


Figure 5.15. Normalized neutral plane depth as a function of the dimensionless head load ratio. Each point represents one analysis of the parametric study; the curve is the power law fitted to the sixteen points.

Sixteen analyses were used. The symmetric cases were excluded because their loading geometry differs from the one-sided cases. Cases C04–C05 were excluded because the pile-head load changed with spacing, and C09–C10 were excluded because the pile toes entered the stiff stratum and the group no longer behaved as a floating foundation. The load ratio was computed for each analysis and a power law was fitted in logarithmic form; the detailed data set, fitting procedure and residuals are provided below. The fitted curve is:

$$\frac{z_n}{L} = 0.387 \lambda^{-0.55} \quad (5.2)$$

Over the sixteen analyses the fitted curve reproduces the computed depths with $R^2 = 0.887$ and a mean absolute error of 4.2 per cent. The negative exponent expresses the behavior observed throughout the study and demonstrated experimentally by Leung et al. (2004) for single piles: a more heavily loaded pile reaches equilibrium over a shorter length of positive shaft resistance, so its neutral plane rises. What the present result adds is that the same dependence holds within a group and at full scale, and that it is captured by a ratio derived directly from the equilibrium condition rather than by the individual parameters.

The two analyses that depart most from the curve are those in which the interface strength was altered, which is expected, since a change in interface strength modifies the shaft resistance in a manner that the linear stress profile assumed in R_s does not fully represent.

This relationship should be read as a description of the analyses performed, not as a design rule. It was obtained for one soil profile, one group geometry and one cap condition, and the ratio λ spans only 0.47 to 1.30 across the sixteen cases; the curve should not be extrapolated beyond that range. Its value lies in what it shows about the mechanism: that the position of the neutral plane in a floating group is governed by the balance of head load against shaft resistance, in the manner the classical equilibrium argument predicts. The reported fit error is descriptive of the same calibration set; no cases were held back for independent validation.

5.6.1 Regression Data Set

Sixteen analyses were included. The symmetric cases S00–S02 were excluded because the loading geometry differs, and C04–C05 were excluded because the pile-head load was not constant in the spacing series. Cases C09–C10 were also excluded because the pile toes entered the stiff stratum and the group no longer behaved as a floating foundation. In these two cases, toe resistance increased from 301 kN in C00 to 619 and 992 kN, while the normalized neutral-plane depth increased from 0.50 to 0.65 and 0.76. These cases therefore combine changes in clay thickness and toe support. The group-mean neutral-plane depth was used for the fit. Table 5.5 lists the data.

Table 5.5. Data set used in the regression.

Case	q (kPa)	d (m)	L (m)	D (m)	R_{inter}	Q_{nsf} front (kN)	λ	$\frac{z_n}{L}$
C00	100	0	30	1.0	0.7	952	0.649	0.497
C01	25	0	30	1.0	0.7	316	0.609	0.468
C02	50	0	30	1.0	0.7	576	0.630	0.507
C03	150	0	30	1.0	0.7	1287	0.646	0.497
C06	100	0	20	1.0	0.7	430	1.303	0.344
C07	100	0	24	1.0	0.7	690	0.961	0.417
C08	100	0	36	1.0	0.7	1224	0.465	0.572
C11	100	2	30	1.0	0.7	910	0.624	0.522
C12	100	5	30	1.0	0.7	770	0.619	0.532
C13	100	10	30	1.0	0.7	604	0.614	0.530
C14	100	0	30	1.0	0.5	572	0.888	0.351

Case	q (kPa)	d (m)	L (m)	D (m)	R _{inter}	Q _{nsf} front (kN)	λ	$\frac{z_n}{L}$
C15	100	0	30	1.0	0.9	1226	0.506	0.568
C16	100	0	30	1.0	0.7	1040	0.512	0.553
C17	100	0	30	1.0	0.7	856	0.817	0.430
C18	100	0	30	0.8	0.7	755	0.804	0.411
C19	100	0	30	1.2	0.7	1092	0.548	0.553

5.6.2 Fitting Procedure

For each of the sixteen analyses, λ was calculated from Equation (5.1) and a power law was fitted by ordinary least squares in logarithmic coordinates. The fitted and adopted relationships are:

$$\begin{aligned} \text{initial fit: } z_n/L &= 0.383 \cdot \lambda^{-0.566} \quad (R^2 = 0.888) \\ \text{adopted fit: } z_n/L &= 0.387 \cdot \lambda^{-0.55} \quad (R^2 = 0.887) \\ 0.465 &\leq \lambda \leq 1.303 \end{aligned}$$

where:

λ = dimensionless pile-head load ratio defined in Equation (5.1)

z_n/L = normalized neutral-plane depth

The exponent was rounded to -0.55 because a value inferred from only sixteen points should not be reported with unnecessary precision; the leading coefficient was then re-fitted with the exponent fixed.

5.6.3 Residuals

Table 5.6 compares the computed and fitted normalized neutral-plane depths. The mean absolute error is 4.2 per cent and the largest single deviation is 17.5 per cent, in case C14 (interface factor 0.5). The residuals show no systematic trend with λ , which indicates that a single power law is an adequate description over the range examined; the larger deviation in C14 reflects the simplified linear stress profile assumed in R_s , as noted in Section 5.5.

Table 5.6. Computed and fitted normalized neutral-plane depths, with residuals.

Case	$\frac{z_n}{L}$ computed	$\frac{z_n}{L}$ predicted	Diff. (%)
C00	0.497	0.490	-1.3
C01	0.468	0.508	+8.5
C02	0.507	0.498	-1.6
C03	0.497	0.492	-1.0
C06	0.344	0.334	-2.9
C07	0.417	0.395	-5.4
C08	0.572	0.589	+3.1
C11	0.522	0.501	-4.0
C12	0.532	0.503	-5.4
C13	0.530	0.506	-4.6
C14	0.351	0.413	+17.5
C15	0.568	0.562	-1.1
C16	0.553	0.559	+1.1
C17	0.430	0.432	+0.4
C18	0.411	0.436	+6.0
C19	0.553	0.538	-2.7

5.7 Independent Analytical Verification of the Numerical Model

Because the numerical results underpin the conclusions of this study, an independent analytical check was carried out using conventional hand calculations rather than relying on the finite element solution alone. The purpose is verification rather than validation: the calculations test whether the stresses, settlements, shaft forces and neutral-plane position are mechanically consistent with the adopted input parameters and with the forces extracted from the numerical model. They do not replace field measurements, nor are they expected to reproduce three-dimensional pile-group shielding exactly. The calculations are set out step by step below so that they can be reproduced independently.

5.7.1 Verification Strategy and Limitations

The verification is organized at three levels. First, the soil and interface parameters used by the numerical model are reconstructed from the stated material data. Second, one-dimensional consolidation and effective-stress calculations are used to estimate free-field settlement, consolidation time, shaft resistance and neutral-plane depth. Third, direct force equilibrium is checked for the reference pile, and the section properties assigned to the embedded-beam elements are recalculated from geometry. Inherently three-

dimensional quantities, such as pile-to-pile load redistribution, shielding, spacing effects and cap interaction, cannot be reproduced exactly by a single-pile closed-form method. For these quantities, the hand solution is used as a reference or bounding check, while the remaining verification is based on equilibrium and consistent trends.

For the worked axial check, the actual pile-head force Q from the numerical output is used because the rigid cap redistributes the applied structural load unevenly among the twenty-five piles. A single-pile hand method cannot predict this cap redistribution. Therefore the comparison below verifies the shaft-transfer and neutral-plane calculation conditional on the pile-head force delivered by the cap; it does not constitute an independent check of cap load sharing.

As a separate post-processing consistency check, the pile-by-pile master dataset was re-aggregated by row for C00. The front, middle and back rows give mean neutral-plane depths of 13.54, 14.19 and 17.11 m respectively, with corresponding mean net dragloads of 952, 117 and 413 kN. These values reproduce the row-averaged results reported in Table 5.1 and confirm that the spreadsheet aggregation preserves the pile-level output without transcription error.

5.7.2 Recovery of the Strength Parameters

The effective friction angles are not stated explicitly in the material sets, but PLAXIS generates the coefficient of lateral earth pressure from the relation proposed by Jaky (1944), so they can be recovered by inversion:

$$K_0 = 1 - \sin\phi' \quad \Rightarrow \quad \phi' = \arcsin(1 - K_0) \quad (5.3)$$

Substitution of the PLAXIS K_0 values into Equation (5.3) gives:

$$\begin{aligned} \phi'_{\text{clay}} &= \arcsin(1 - 0.5933) = 24.0^\circ \\ \phi'_{\text{fill}} &= \arcsin(1 - 0.4701) = 32.0^\circ \\ \tan\delta &= R_{\text{inter}} \tan\phi' \end{aligned} \quad (5.4)$$

Using $R_{inter} = 0.70$ in Equation (5.4):

$$\begin{aligned}\tan \delta_{clay} &= 0.70 \times 0.4452 = 0.3116 \rightarrow \delta_{clay} = 17.3^\circ \\ \tan \delta_{fill} &= 0.70 \times 0.6249 = 0.4374 \rightarrow \delta_{fill} = 23.6^\circ \\ \beta &= K_0 \tan \delta\end{aligned}\tag{5.5}$$

Equation (5.5) then gives the shaft-friction coefficients:

$$\begin{aligned}\beta_{clay} &= 0.5933 \times 0.3116 = 0.1849 \\ \beta_{fill} &= 0.4701 \times 0.4374 = 0.2056\end{aligned}$$

where:

K_0 = coefficient of earth pressure at rest

φ' = effective soil friction angle

δ = pile–soil interface friction angle

R_{inter} = interface strength reduction factor

β = effective-stress shaft-friction coefficient

The value obtained for the clay lies within the range of 0.15–0.35 associated with soft clay deposits by Fellenius (2006), indicating that the reconstructed interface parameters are internally consistent.

5.7.3 Effective Stress Profile

Since the water table is at ground level, submerged unit weights apply from the surface downward:

$$\gamma' = \gamma_{sat} - \gamma_w\tag{5.6}$$

Equation (5.6) gives the submerged unit weights used in the hand calculation:

$$\begin{aligned}\gamma'_{fill} &= 20.00 - 9.81 = 10.19 \text{ kN/m}^3 \\ \gamma'_{clay} &= 17.00 - 9.81 = 7.19 \text{ kN/m}^3\end{aligned}$$

where:

γ' = submerged (effective) unit weight

γ_{sat} = saturated unit weight

γ_w = unit weight of water

The initial effective vertical stress at depth z within the clay is therefore obtained from Equation (5.7).

$$\sigma'_{v0}(z) = \gamma'_f h_f + \gamma'_c (z - h_f) = 20.38 + 7.19(z - 2) \quad (5.7)$$

With $h_f = 2$ m, representative values from Equation (5.7) are:

$$\begin{aligned}\sigma'_{v0}(z = 2 \text{ m}) &= 2(10.19) = 20.4 \text{ kPa} \\ \sigma'_{v0}(z = 20 \text{ m}) &= 2(10.19) + 18(7.19) = 149.8 \text{ kPa} \\ \sigma'_{v0}(z = 38 \text{ m}) &= 2(10.19) + 36(7.19) = 279.2 \text{ kPa} \\ \sigma'_v &= \sigma'_{v0} + q, \quad q = 100 \text{ kPa}\end{aligned}$$

where:

σ'_{v0} = initial effective vertical stress before surcharge

σ'_v = effective vertical stress after consolidation under surcharge

h_f = fill thickness

q = applied embankment surcharge

5.7.4 Free-Field Consolidation Settlement

The Soft Soil model follows a logarithmic compression law, so for a normally consolidated layer the vertical strain produced by a stress increment $\Delta\sigma$ is

$$\varepsilon_v = \lambda \ln \left(\frac{\sigma'_{v0} + \Delta\sigma}{\sigma'_{v0}} \right) \quad (5.8)$$

and the settlement of the layer is obtained by summation over sublayers,

$$s = \sum_i \varepsilon_{v,i} h_i \quad (5.9)$$

The 36 m clay layer was divided into twelve 3 m sublayers. For the first sublayer, whose mid-depth is 3.5 m, the calculation is:

$$\begin{aligned}\sigma'_{v0} &= 20.38 + 7.19 \times 1.5 = 31.2 \text{ kPa} \\ \sigma'_{v1} &= \sigma'_{v0} + q = 31.2 + 100 = 131.2 \text{ kPa} \\ \varepsilon_v &= 0.0791 \cdot \ln(131.2 / 31.2) \\ \varepsilon_v &= 0.0791 \times 1.4372 = 0.1137 \\ s_1 &= \varepsilon_v \cdot \Delta H = 0.1137 \times 3.00 = 0.341 \text{ m} \\ \Sigma s_{\text{clay}} &= 1.7965 \text{ m} \approx 1797 \text{ mm}\end{aligned}$$

where:

σ'_{v0} = initial effective vertical stress at the sublayer midpoint

σ'_{v1} = final effective vertical stress after surcharge

ε_v = vertical strain of the sublayer

ΔH = sublayer thickness

s_1 = settlement contribution of the first sublayer

Σs_{clay} = sum of the settlements of all twelve clay sublayers

The same procedure was applied to all twelve sublayers. This explicit summation provides the hand-calculated counterpart of the Soft Soil compression mechanism used in the finite element model.

$$s_{\text{total}} = 1797 + 5 + 13 \approx 1815 \text{ mm} \quad (5.10)$$

Almost the whole of this movement originates in the soft clay, which confirms that a relative displacement of the order of meters is available to drive negative skin friction along the pile shafts. It should be noted that this is the free-field value; the piled area settles far less, and it is precisely the difference between the two that generates the dragload.

5.7.5 Consolidation Time Check

The hydraulic parameters can also be checked independently through one-dimensional consolidation theory. At approximately the mid-depth of the clay after application of the reference surcharge, $\sigma'_v \approx 250 \text{ kPa}$. For the Soft Soil model the tangent oedometer modulus is approximated by σ'_v/λ^* , giving

$$E_{\text{oed,tan}} = \frac{\sigma'_v}{\lambda^*} = \frac{250}{0.0791} = 3161 \text{ kPa} \quad (5.11)$$

The corresponding coefficient of volume compressibility is

$$m_v = \frac{1}{E_{\text{oed,tan}}} = 3.16 \times 10^{-4} \text{ kPa}^{-1} \quad (5.12)$$

With the adopted vertical permeability and unit weight of water, the coefficient of consolidation is obtained from Equation (5.13).

$$c_v = \frac{k_v}{m_v \gamma_w} = \frac{k_v E_{\text{oed,tan}}}{\gamma_w} = 0.0278 \text{ m}^2/\text{d} \quad (5.13)$$

For two-way drainage, the drainage path is one-half of the 36 m clay thickness:

$$H_{\text{dr}} = H_{\text{clay}} / 2 = 36 / 2 = 18 \text{ m}$$

where:

H_{dr} = drainage path length

H_{clay} = thickness of the compressible clay layer

The corresponding time factor relation is given by Equation (5.14).

$$T_v = \frac{c_v t}{H_{\text{dr}}^2} \quad (5.14)$$

Using $T_v = 0.848$ for $U = 90\%$ and $T_v \approx 1.781$ for $U = 99\%$ gives

$$t_{90} = \frac{0.848 \times 18^2}{0.0278} = 9.88 \times 10^3 \text{ d} \approx 27.1 \text{ yr} \quad (5.15)$$

$$t_{99} = \frac{1.781 \times 18^2}{0.0278} = 2.08 \times 10^4 \text{ d} \approx 56.9 \text{ yr} \quad (5.16)$$

The reference PLAXIS analysis reached the adopted maximum excess-pore-pressure criterion of 1 kPa after approximately 23 800 days, or 65 years. The difference from the one-dimensional t_{99} estimate is about fourteen percent, which is reasonable because the coupled finite element model uses a stress-dependent constitutive law and a spatially varying drainage and stress field rather than a single representative modulus.

5.7.6 Negative and Positive Shaft Resistance by the Effective Stress Method

In the β -method the limiting shaft shear at any depth is proportional to the effective vertical stress. The force transferred between soil and pile per unit pile length is

$$t(z) = \beta \sigma'_v(z) \pi D \quad (5.17)$$

For the reference pile, Equation (5.17) gives the following representative shaft-force gradients:

$$dQ_s/dz \mid z = 0 \text{ m} \approx 64.6 \text{ kN/m}$$

$$dQ_s/dz \mid z = 10 \text{ m} \approx 103.3 \text{ kN/m}$$

$$dQ_s/dz \mid z = 30 \text{ m} \approx 186.9 \text{ kN/m}$$

where:

dQ_s/dz = shaft force transferred between soil and pile per unit pile length

The soil-induced downward shaft force accumulated above the neutral plane is denoted Q_{nsf} , while the positive shaft resistance mobilized below the neutral plane is denoted Q_s .

Using the shaft-force gradient from Equation (5.17), these quantities are defined by:

$$Q_{\text{nsf}}(z_n) = \int_0^{z_n} t(z) dz \quad (5.18)$$

The corresponding positive shaft resistance is evaluated over the portion of shaft below the neutral plane.

$$Q_s(z_n) = \int_{z_n}^L t(z) dz \quad (5.19)$$

5.7.7 Worked Neutral-Plane Calculation for the C00 Front-Row Pile

The neutral plane was recalculated using the force-equilibrium construction of the unified pile-design method described by Fellenius (2004, 2006). In this method the neutral plane is located where the downward load-transfer curve, obtained from the sustained pile-head load plus accumulated negative skin friction, intersects the upward resistance curve formed by positive shaft resistance and toe resistance. In the complete unified analysis this force-equilibrium depth also coincides with settlement equilibrium. The present hand check evaluates the force-equilibrium part explicitly. Because the PLAXIS embedded-beam model specifies a limiting toe resistance rather than an independently measured q - z relation, the prescribed $Q_p = 300$ kN toe term is retained in the analytical calculation. Thus, the calculation provides a complete force-equilibrium check of the adopted model inputs rather than an independent prediction of toe mobilization.

For the C00 front row, the row-average pile-head force from PLAXIS is used because the rigid cap redistributes the applied structural load among the twenty-five piles. The hand calculation otherwise treats the pile as an isolated pile subjected to the full surcharge stress, providing a conservative reference for the one-sided group model. The input values are:

$$\begin{aligned} Q &= 974 \text{ kN} \\ q &= 100 \text{ kPa} \\ D &= 1.0 \text{ m}, \quad L = 30 \text{ m}, \quad h_f = 2.0 \text{ m} \\ \beta_f &= 0.2056, \quad \beta_c = 0.1849 \\ \gamma'_f &= 10.19 \text{ kN/m}^3, \quad \gamma'_c = 7.19 \text{ kN/m}^3 \\ \gamma_p &= 25 \text{ kN/m}^3, \quad Q_p = 300 \text{ kN} \\ z_{n,\text{PLAXIS}} &= 13.54 \text{ m}, \quad Q_{p,\text{PLAXIS}} \approx 301 \text{ kN} \end{aligned}$$

where:

Q = sustained row-average pile-head axial load

q = embankment surcharge used in the isolated-pile hand check

D = pile diameter; L = pile length; hf = surface-fill thickness

β_f, β_c = effective-stress shaft-friction coefficients for fill and clay

γ'_f, γ'_c = submerged unit weights of fill and clay

γ_p = pile unit weight

Q_p = toe-resistance term used in the hand calculation

$z_{n,PLAXIS}$ = neutral-plane depth extracted from PLAXIS

Step 1 – Pile geometry and self-weight.

The pile area A , shaft perimeter u , and self-weight per unit length w_p follow directly from the circular geometry:

$$\begin{aligned} A &= \pi D^2 / 4, \quad u = \pi D, \quad w_p = A \gamma_p & (5.20) \\ A &= \pi (1.0)^2 / 4 = 0.7854 \, m^2 \\ u &= \pi (1.0) = 3.1416 \, m \\ w_p &= 0.7854 \times 25 = 19.635 \, kN/m \\ W_{p,total} &= w_p L = 19.635 \times 30 = 589.05 \, kN \end{aligned}$$

Step 2 – Define the load-transfer and resistance curves.

Using the β -method of Section 5.7.2, the accumulated negative shaft force above a trial depth z and the positive shaft resistance below it are:

$$Q_{nsf}(z) = \int_0^z u \beta(z) \sigma'_v(z) dz, \quad Q_s(z) = \int^z u \beta(z) \sigma'_v(z) dz \quad (5.21)$$

At the neutral plane, the axial force evaluated downward from the pile head must equal that evaluated upward from the toe. Including pile self-weight consistently on the two free bodies gives the literature force-equilibrium condition:

$$N \downarrow (z) = Q + w_p z + Q_{nsf}(z) = Q_p + Q_s(z) - w_p (L - z) = N \uparrow (z) \quad (5.22)$$

The same relation may be written for the whole pile as $Q + W_{p,total} + Q_{nsf}(z_n) = Q_p + Q_s(z_n)$.

Step 3 – Evaluate the shaft-force integrals for the reference profile.

Because the neutral plane lies below the 2 m fill layer, the negative shaft force is separated into fill and clay contributions. For the fill, the full 2 m length is above the neutral plane:

$$Q_{\text{nsf},f} = u\beta_f[q h_f + 0.5\gamma'_f h_f^2] = 142.35 \text{ kN} \quad (5.23)$$

$$Q_{\text{nsf},f} = 3.1416 \times 0.2056 \times [100(2) + 0.5(10.19)(2^2)] = 142.35 \text{ kN}$$

For the clay, let $x = z_n - h_f = z_n - 2$ and define $a = q + \gamma'_f h_f = 100 + 10.19(2) = 120.38$ kPa. The negative shaft force accumulated in the clay above the neutral plane is:

$$Q_{\text{nsf},c}(x) = u\beta_c[a x + 0.5\gamma'_c x^2] \quad (5.24)$$

The clay extends 28 m below the fill. The positive shaft resistance available between the neutral plane and the pile toe is therefore:

$$Q_s(x) = u\beta_c[a(28 - x) + 0.5\gamma'_c(28^2 - x^2)] \quad (5.25)$$

Step 4 – Solve for the neutral-plane depth.

Substituting Equations (5.23)–(5.25) into the whole-pile form of Equation (5.22) gives a quadratic equation in x :

$$4.1765x^2 + 139.8528x - 2189.7444 = 0 \quad (5.26)$$

$$x_1 = 11.623 \text{ m}$$

$$x_2 = -45.108 \text{ m} \rightarrow \text{discarded as non – physical}$$

$$z_n = h_f + x_1 = 2.000 + 11.623 = 13.623 \text{ m}$$

$$z_n / L = 13.623 / 30 = 0.454$$

The intersection of the two curves can be checked directly around the root:

$$z = 13.50 \text{ m}: N \downarrow = 2461.7 \text{ kN}, N \uparrow = 2490.8 \text{ kN}, \text{ residual} = -29.1 \text{ kN}$$

$$z = 13.62 \text{ m}: N \downarrow = 2478.3 \text{ kN}, N \uparrow = 2479.0 \text{ kN}, \text{ residual} = -0.7 \text{ kN}$$

$$z = 13.623 \text{ m}: N \downarrow = 2478.7 \text{ kN}, N \uparrow = 2478.7 \text{ kN}, \text{ residual} \approx 0 \text{ kN}$$

$$z = 14.00 \text{ m}: N \downarrow = 2531.1 \text{ kN}, N \uparrow = 2441.2 \text{ kN}, \text{ residual} = +89.9 \text{ kN}$$

Step 5 – Forces at the calculated neutral plane.

$$Q_{\text{nsf},c} = 1094.87 \text{ kN}$$

$$Q_{\text{nsf}} = Q_{\text{nsf},f} + Q_{\text{nsf},c} = 142.35 + 1094.87 = 1237.22 \text{ kN}$$

$$Q_s = 2500.27 \text{ kN}$$

$$W_p(z_n) = w_p z_n = 19.635 \times 13.623 = 267.49 \text{ kN}$$

$$W_{p,\text{below}} = w_p(L - z_n) = 19.635 \times 16.377 = 321.56 \text{ kN}$$

The whole-pile equilibrium closes as follows:

$$\text{Downward} = Q + W_{p,\text{total}} + Q_{\text{nsf}} = 974 + 589.05 + 1237.22 = 2800.27 \text{ kN}$$

$$\text{Upward} = Q_p + Q_s = 300 + 2500.27 = 2800.27 \text{ kN}$$

The maximum axial force occurs at the neutral plane and may be calculated from either free body:

$$N_{\max, \text{hand}} = Q + W_p(z_n) + Q_{\text{nsf}} = 974 + 267.49 + 1237.22 = 2478.7 \text{ kN} \quad (5.27)$$

$$\text{Check from below: } Q_p + Q_s - W_{p, \text{below}} = 300 + 2500.27 - 321.56 = 2478.7 \text{ kN}$$

where:

Q_{nsf} = accumulated negative shaft force (dragload) above the neutral plane

Q_s = positive shaft resistance mobilized below the neutral plane

Q_p = toe resistance

$N \downarrow, N \uparrow$ = axial force evaluated from the pile-head and pile-toe sides, respectively

$W_p(z_n)$ = pile self-weight above the neutral plane

$W_{p, \text{below}}$ = pile self-weight below the neutral plane

$N_{\max, \text{hand}}$ = hand-calculated maximum axial force at the neutral plane

Step 6 – Comparison with the numerical result.

$$z_{n, \text{hand}} = 13.623 \text{ m}$$

$$z_{n, \text{PLAXIS}} = 13.54 \text{ m}$$

difference in neutral-plane depth $\approx 0.6\%$

$$Q_{\text{nsf}, \text{hand}} = 1237 \text{ kN}$$

$$Q_{\text{nsf}, \text{PLAXIS}} (\text{front} - \text{row mean}) \approx 952 \text{ kN}$$

$$N_{\max, \text{hand}} = 2479 \text{ kN}$$

$$N_{\max, \text{PLAXIS}} (\text{front} - \text{row mean}) \approx 2191 \text{ kN}$$

The neutral-plane depth is reproduced closely even though the isolated-pile hand solution gives larger dragload and maximum axial force. The difference in force magnitude is expected because the hand solution applies the full surcharge stress around an isolated pile and assumes fully mobilized shaft transfer, whereas the three-dimensional model contains one-sided loading, pile-group shielding and rigid-cap load redistribution. The numerical front-row dragload is about 23% lower than the isolated-pile estimate, while the neutral-plane depth differs by only about 0.6%. This is consistent with the neutral plane being governed primarily by the balance between downward and upward load-transfer curves, not by dragload magnitude alone.

5.8 Structural and Parametric Checks

5.8.1 Pile Section Properties and Structural Screening

The structural properties assigned to the embedded beam can be checked directly from the circular section geometry. For the reference diameter $D = 1.0$ m,

$$A = \frac{\pi D^2}{4} = 0.7854 \text{ m}^2, \quad I = \frac{\pi D^4}{64} = 0.04909 \text{ m}^4 \quad (5.28)$$

$$EA = E A = 23.56 \times 10^6 \text{ kN}, \quad EI = E I = 1.473 \times 10^6 \text{ kN m}^2 \quad (5.29)$$

Applying the same circular-section relations to the diameter sensitivity cases gives:

$$D = 0.8 \text{ m}: A = 0.5027 \text{ m}^2, I = 0.02011 \text{ m}^4$$

$$D = 1.2 \text{ m}: A = 1.1310 \text{ m}^2, I = 0.10179 \text{ m}^4$$

where:

A = pile cross-sectional area

I = second moment of area of the circular pile section

The D^4 dependence of I independently confirms the stiffness scaling used to interpret the diameter series.

A section-level N–M interaction screening was performed using the reinforced-concrete pile section supplied for this study. The reference section has $D = 1.0$ m, C30 concrete, B420c reinforcement, 30 Φ 32 longitudinal bars and a nominal cover of 75 mm. For each PLAXIS node the axial compression and the resultant moment were paired at the same elevation, so independent maxima were not combined. The complete calculation, including the comparable 0.8 m and 1.2 m sections used in the diameter series, is documented in Appendix A. Because the numerical loading program was applied directly in PLAXIS without a separate classification into characteristic and design combinations, no additional action factor is introduced in this thesis. The resulting interaction index is therefore used only to compare the relative structural demand and to identify where sectional non-linearity may become relevant; it is not interpreted as a formal code-compliance result. A complementary shear and transverse-reinforcement screening based on the resultant PLAXIS shear-force components is presented in Appendix B.

5.8.2 Parametric Hand-Calculation Checks and Applicability

The same analytical framework provides a reproducible check on the principal parametric series. For the surcharge series C01–C03, q is replaced directly in the effective-stress profile. For C06–C08 the pile length L and the integration limits are changed. For C14–C15 the interface angle δ and β are recalculated from the relevant R_{inter} . For C16–C17 the actual pile-head force Q is changed in the force-equilibrium equation. For C18–C19 the diameter changes the perimeter, area, self-weight and flexural rigidity simultaneously. In each of these cases the analytical solution should reproduce the direction and approximate scale of the numerical response even though the three-dimensional group solution is expected to give a smaller dragload on shielded piles.

Two series require a different interpretation. The offset series C11–C13 cannot be checked by simply applying the full surface surcharge at the pile axis; an independent hand calculation should first determine the depth-dependent stress increment at the pile from an appropriate influence solution, and then use that stress increment in the effective-stress and force-equilibrium framework of equation (5.18) through equation (5.23). Likewise, the spacing series C04–C05 is fundamentally a group-interaction problem and has no unique single-pile closed-form equivalent; it is therefore checked by pile-by-pile axial equilibrium and by comparison with the isolated-pile bound rather than by expecting a direct match.

The clay-thickness cases C09–C10 also require caution because the pile toe enters the stiff layer and the limiting base resistance is changed at the same time. The 300 kN value used in the reference case is a prescribed numerical upper bound chosen to preserve floating-pile behavior, not a calculated ultimate bearing resistance. The 2500 kN value used in C09–C10 is independently checked by the conventional pile-base bearing-capacity calculation presented below; the series is therefore interpreted as the combined effect of clay thickness and the associated change in toe support rather than as a pure thickness effect.

Bearing-Capacity Check for the Stiff-Stratum Toe Limit

The embedded-beam base-resistance limit was increased from 300 kN to 2500 kN in cases C09–C10 because the pile toe enters the stiff stratum. The adopted value is checked below against a conventional effective-stress bearing-capacity estimate. To retain a conservative and transparent comparison, only the N_q contribution is used:

$$N_q = \exp(\pi \tan \phi') \tan^2 \left(45^\circ + \frac{\phi'}{2} \right) \quad (5.30)$$

$$Q_b = A_b \sigma'_{v,toe} N_q, \quad A_b = \frac{\pi D^2}{4} \quad (5.31)$$

For the stiff stratum, $\phi' = 32^\circ$ and Equation (5.30) gives $N_q \approx 23.18$. The base-resistance check is then:

$$\begin{aligned} A_b &= \pi D^2 / 4 = \pi (1.0)^2 / 4 = 0.7854 \text{ m}^2 \\ Q_{b,C09} &= 0.7854 \times 252 \times 23.18 \approx 4.58 \text{ MN} \\ Q_{b,C10} &= 0.7854 \times 225 \times 23.18 \approx 4.09 \text{ MN} \\ F_{max} &= 2.50 \text{ MN} < Q_{b,C09} \text{ and } Q_{b,C10} \end{aligned}$$

where:

A_b = pile base area

$\sigma'_{v,toe}$ = effective overburden stress at pile toe

N_q = bearing-capacity factor

Q_b = conservative ultimate base resistance from the N_q term

F_{max} = base-resistance limit adopted in the embedded-beam model

The adopted 2.50 MN model limit is therefore below both conventional bearing-capacity estimates and is retained as a conservative numerical cap rather than as the calculated ultimate bearing resistance.

This check confirms that the 2500 kN limit is compatible with the strength of the stiff material while remaining conservative. Because C09–C10 simultaneously change the clay thickness and the toe condition, their results are interpreted as a combined layer-thickness/toe-support effect, as stated in Chapters 4 and 5.

5.9 Cross-Case Neutral-Plane and Lateral Equilibrium Checks

The raw structural-force exports were checked systematically for all 23 principal analyses in the numerical study: the twenty asymmetric cases C00–C19 and the three symmetric control cases S00–S02. With 25 piles in each model, the verification therefore covers 575

individual pile profiles. For each pile, coincident element-end records at the same elevation were consolidated, the profile was ordered along the shaft, and two independent numerical-consistency checks were applied. The first compared the depth of maximum axial compression with the sustained reversal of shaft shear, while the second checked the beam-equilibrium relation between the gradient of bending moment and the corresponding shear force. This exercise is separate from the closed-form hand calculation above: it verifies the internal consistency of the exported finite-element response over the complete analysis set rather than reproducing the three-dimensional problem analytically.

$$dM_2/dZ \approx -Q_{13} \quad (5.32)$$

$$dM_3/dZ \approx Q_{12} \quad (5.33)$$

For the lateral check, numerical derivatives of the nodal moment profiles were evaluated by finite differences with respect to the PLAXIS global vertical coordinate Z (positive upward) and compared with the shear-force output according to (5.32) and (5.33). Across all 575 pile profiles from the 23 principal analyses, the median correlation coefficient was 0.993 for the M_2 – Q_{13} relation and 0.991 for the M_3 – Q_{12} relation. The mean absolute discrepancy, normalized by the peak shear force of each pile, was 2.18% and 2.30% respectively; the corresponding 90th-percentile values were 4.15% and 4.37%. The symmetric S-series gives similarly strong equilibrium, with median correlations of approximately 0.993 on both axes. The close agreement confirms that the bending-moment and shear-force distributions used in the interpretation satisfy beam equilibrium over both asymmetric and symmetric loading configurations to within the discretization error of the exported nodal results. For the C00 front-corner pile at $X = 8$ m, $Y = 8$ m, the correlation for the principal M_2 – Q_{13} pair is 0.999 with a normalized mean absolute discrepancy below 1%.

The neutral-plane consistency check was restricted to piles for which the post-processed net dragload was positive, because a physical negative-to-positive shaft-shear reversal is not expected to define a meaningful dragload neutral plane when the NSF mechanism is not mobilized. Of the 575 pile profiles, 504 satisfied this criterion. Across these developed profiles, the median absolute difference between the depth of maximum axial force and the sustained shaft-shear reversal was 0.55 m, equivalent to 1.90% of the pile length; the

90th percentile was 3.15 m (10.91% of pile length). The symmetric controls show substantially tighter agreement than the asymmetric series: the median difference is 0.18 m for S00–S02 taken together, compared with 0.69 m for C00–C19. The individual symmetric medians are 0.23 m for S00, 0.31 m for S01 and 0.14 m for S02. This is mechanically reasonable because the concentric loading field in the symmetric models produces a more regular axial-transfer pattern, whereas the one-sided cases contain a wider transition zone and stronger pile-to-pile redistribution. For the C00 front-corner pile, the maximum axial force occurs at 13.46 m depth and the sustained shaft-shear reversal occurs at approximately 12.91 m, a difference of 0.54 m. The larger discrepancies remain concentrated in weakly mobilized cases and in profiles containing broader transition zones. The case-by-case statistics are reported below in Table 5.7.

5.9.1 Case-by-Case Consistency Summary

The case-by-case summary below reports the raw-output consistency checks performed on every pile in the complete analysis set, comprising C00–C19 and S00–S02. “Developed piles” are those for which the net dragload after removal of pile self-weight is positive. Δz is the absolute difference between the depth of maximum axial compression and the sustained negative-to-positive shaft-shear reversal. The moment–shear columns report the median correlation coefficient and the mean normalized absolute error for the two local bending axes.

Table 5.7. Case-by-case summary of neutral-plane and moment–shear consistency checks for C00–C19 and S00–S02.

Case	Developed piles	Median Δz (m)	Median $\Delta z/L$ (%)	$r(M_2, Q_{13})$	NMAE ₂ (%)	$r(M_3, Q_{12})$	NMAE ₃ (%)
C00	23	0.45	1.5	0.992	2.05	0.993	2.05
C01	8	2.15	7.2	0.992	1.44	0.992	1.64
C02	13	2.02	6.7	0.992	1.69	0.992	1.66
C03	25	0.36	1.2	0.993	2.09	0.991	2.45
C04	20	0.76	2.5	0.991	2.39	0.991	2.67
C05	20	1.57	5.2	0.996	2.34	0.992	1.85
C06	25	1.42	7.1	0.985	3.32	0.975	4.00
C07	25	0.96	4.0	0.995	2.46	0.995	2.68
C08	23	1.16	3.2	0.986	2.07	0.982	2.09
C09	25	0.34	1.1	0.995	1.87	0.976	2.60
C10	25	1.87	6.2	0.994	1.50	0.974	1.78

Case	Developed piles	Median Δz (m)	Median $\Delta z/L$ (%)	$r(M_2, Q_{13})$	NMAE ₂ (%)	$r(M_3, Q_{12})$	NMAE ₃ (%)
C11	20	0.54	1.8	0.993	2.15	0.994	2.05
C12	19	0.54	1.8	0.995	1.96	0.992	1.78
C13	13	1.73	5.8	0.994	2.09	0.990	2.09
C14	25	0.87	2.9	0.995	2.01	0.994	2.41
C15	25	1.10	3.7	0.990	1.82	0.993	1.96
C16	24	0.74	2.5	0.993	1.91	0.993	1.99
C17	25	1.23	4.1	0.993	2.08	0.993	2.28
C18	25	0.24	0.8	0.990	2.15	0.984	2.42
C19	22	0.71	2.4	0.997	1.70	0.997	1.67
S00	25	0.23	0.8	0.993	2.99	0.994	2.96
S01	24	0.31	1.0	0.991	2.82	0.992	2.70
S02	25	0.14	0.5	0.993	3.12	0.992	3.08

The lower developed-pile counts in C01 and C02 are consistent with the weak-NSF behavior already identified in the surcharge series. The moment–shear equilibrium check remains strong in these cases because it tests structural internal-force consistency independently of whether negative skin friction is mobilized. The symmetric cases provide an additional and particularly clean check: S00 and S02 develop positive net dragload in all 25 piles, while the center pile of S01 remains slightly below the adopted net-dragload threshold after the pile self-weight correction. Nevertheless, the symmetric series gives median moment–shear correlations of about 0.99 and a median neutral-plane separation below 0.31 m in every case. The shortest-pile case C06 remains the largest average normalized moment–shear discrepancy in the program, but its mean is still only approximately 3–4% and the median correlation remains above 0.97 on both axes.

Taken together with the analytical calculations, these cross-case checks provide an additional verification layer over the complete 23-analysis: the neutral-plane extraction is consistent with the sign reversal of shaft shear where NSF is developed, and the lateral internal-force output satisfies the differential beam-equilibrium relations for both one-sided and four-sided surcharge configurations.

5.10 Summary of Analytical and Numerical Checks

Table 5.8 summarizes the principal independent checks for the reference case. The front-row numerical response is used for the axial comparison because it is the least shielded part of the one-sided group and is therefore the closest counterpart to the isolated-pile analytical model.

Table 5.8. Comparison between the independent analytical checks and the PLAXIS results for the C00 front row.

Quantity	Hand calculation	PLAXIS, front row	Difference / interpretation
Neutral plane depth z_n / L	0.454	0.451	$\approx 0.6 \%$
Net dragload Q_{nsf} (kN)	1237	952	PLAXIS $\approx 23 \%$ below isolated-pile estimate
Pile weight above neutral plane (kN)	267	266	$< 1 \%$
Maximum axial force N_{max} (kN)	2478	≈ 2191	PLAXIS $\approx 12 \%$ below isolated-pile estimate
Free-field settlement (mm)	1797	-	Mobilization check; not a piled-area settlement
Long-term consolidation time	56.9 years ($U = 99 \%$)	≈ 65 years ($u_{ex} < 1$ kPa)	$\approx 14 \%$

The neutral-plane comparison is the strongest of the independent checks. The principal values are:

$$(z_n/L)_{hand} = 0.454$$

$$(z_n/L)_{PLAXIS} = 0.451$$

$$\text{difference} \approx 0.6\%$$

$$W_p, hand(z_n) = 267 \text{ kN}$$

$$W_p, PLAXIS(z_n) = 266 \text{ kN}$$

where:

z_n/L = neutral-plane depth normalized by pile length

$W_p(z_n)$ = pile self-weight accumulated above the neutral plane

The close agreement in both normalized neutral-plane depth and pile-weight correction indicates that the depth coordinate, section weight and axial-force bookkeeping used in post-processing are internally consistent.

The hand-calculated net dragload is deliberately more conservative than the finite element value: 1237 kN for the isolated-pile effective-stress solution against 952 kN for the front

row and only 364 kN for the group mean. The difference is not treated as an error because the analytical method assumes uniform settlement around a single pile, whereas the finite element model includes one-sided loading, pile-group shielding, cap load redistribution and lateral soil movement. The important result is that the numerical solution falls below the isolated-pile bound and decreases further toward the protected interior of the group, which is mechanically consistent with the expected group effect.

The settlement and time calculations provide complementary checks on the consolidation mechanism. The free-field settlement of approximately 1797 mm is not expected to equal settlement beneath the pile cap, but it confirms that the relative movement required to mobilize shaft friction is available by a wide margin. The estimated t_{99} of 56.9 years compared with approximately 65 years in the coupled analysis confirms that the reported numerical results represent a genuinely long-term condition. Taken together, the stress, settlement, time, force-equilibrium and section-property checks provide independent support for the numerical model while making explicit which three-dimensional group effects cannot be reproduced by hand calculation alone.

6. CONCLUSION

This study investigated negative skin friction, neutral-plane development, and lateral response in a 5×5 floating pile group subjected to one-sided embankment surcharge. Twenty-three principal PLAXIS 3D analyses, together with two mesh-sensitivity reruns, were used to examine the response after consolidation. The results show that negative skin friction in a pile group should be treated as a coupled three-dimensional soil–pile interaction problem rather than as an axial dragload problem alone.

The supplementary 2D–3D comparison reinforces this interpretation. The plane-strain idealization reproduces only the broad longitudinal tendency of the response, whereas the full 3D model resolves the pile-to-pile variability, the shielding pattern, and the undeveloped or weak-dragload piles within the same group. For this reason, a 2D model may be acceptable for preliminary trend assessment, but a 3D representation is required when the design question depends on the response of individual piles.

One-sided and symmetric surcharge loading produced fundamentally different group responses. Under one-sided loading, the whole pile group translated away from the embankment and significant bending developed throughout the group. Under symmetric loading, the perimeter piles moved outward while the center of the group remained almost stationary. This shows that peak force values alone are not sufficient; the displacement pattern and the direction of loading are also important, especially for pile-supported bridge abutments.

Pile position and cap load redistribution strongly affected the axial response. The piles nearest the surcharge generally developed the largest dragloads, while the interior piles were protected by group shielding. The neutral plane was also closely related to the structural load carried by each pile: piles receiving a larger head load developed a shallower neutral plane and generally accumulated less negative skin friction. The rigid cap therefore redistributed structural load and dragload simultaneously, and these two components should be evaluated together.

The parametric analyses identified different controls on the axial and lateral mechanisms. Interface strength mainly controlled dragload and neutral-plane depth, whereas embankment offset had a strong influence on bending and lateral displacement. Increasing pile diameter reduced lateral displacement and increased the relative reinforced-concrete section reserve, even though the absolute bending moment increased. When the pile toe entered the stiff stratum, the neutral plane moved downward and dragload increased because the pile settlement was more strongly restrained. At very low surcharge, negative skin friction was not fully mobilized in many piles. The spacing series did not isolate a clear spacing effect because changes in cap geometry also changed the actual pile-head load distribution.

The fitted correlation summarizes the numerical trend for group-mean neutral-plane depth within the investigated parameter range. It was calibrated using one idealized soil profile and one pile-group configuration, without a separate validation data set. It is therefore reported as a study-specific summary of the analyses rather than as a general design equation.

Independent checks support the internal consistency of the numerical results. The analytical neutral-plane depth for the reference front row differs from the PLAXIS result by about 0.6%, and the settlement and consolidation-time calculations confirm the long-term deformation mechanism. Mesh refinement produced only small changes in the principal response measures. In addition, the raw-output verification of all 575 pile profiles gave a median moment-gradient/shear-force correlation of about 0.99 and close agreement between the depth of maximum axial force and the sustained shaft-shear reversal. The supplementary N–M and shear screenings also confirm that the lateral internal forces are large enough to warrant consideration, although they are not presented as code-based structural design checks.

Overall, the study shows that one-sided surcharge can produce strongly nonuniform dragload, pile-group translation, and substantial bending at the same time. For this type of foundation, isolated-pile dragload calculations alone cannot describe the complete response. A three-dimensional assessment that considers pile position, cap load

redistribution, neutral-plane development, axial force, and lateral deformation together provides a more realistic basis for evaluating floating pile groups on consolidating soft ground.

6.1 Future Study and Recommendations

Several limitations of the present work point directly to the investigations that would extend it most usefully.

The piles were treated as linear elastic throughout. The N–M screening presented in Appendix A removes the need for an arbitrary single moment-capacity threshold, but it also indicates that sectional non-linearity may become relevant in the higher-demand analyses. The numerical loading scheme was not organized into code-specific characteristic and design combinations. The screening result is therefore not interpreted as a formal capacity failure. A useful extension would be to repeat selected cases with a non-linear reinforced-concrete section or fiber model and explicitly defined structural load combinations. This would quantify the effects of cracking, stiffness reduction and moment redistribution on the lateral response.

The spacing series could not be interpreted. Changing the center-to-center distance changes the plan area of the cap, and although the surface pressure was scaled so that the nominal load per pile was preserved, the contact between the cap and the settling soil scales with the area and reintroduced a difference of some sixteen percent in the axial force actually delivered. A cleaner formulation would apply the structural load as concentrated forces at the pile heads, removing the cap from the load path altogether, at the cost of abandoning the redistribution behavior that proved so informative elsewhere. Both formulations have merit and the comparison between them would itself be instructive.

The shielding obtained under symmetric loading, expressed as a ratio of roughly one to fifty between the central and corner piles, is very pronounced in the present numerical configuration and has not been independently validated against a closely matching 5×5

group. The compressible layer thickness and the rigid-cap assumption may contribute to this result, but their separate contributions were not established. A study varying cap stiffness systematically, from a flexible slab to the rigid plate assumed in this work, would show how much of the redistribution observed is a property of the soil and how much an artifact of the idealization.

The limiting base resistance was prescribed rather than derived from the properties of the material surrounding the toe. This was a deliberate simplification, adopted to preserve the floating character of the group, and it was adjusted where the geometry placed the toe in the stiff stratum. Deriving the base resistance consistently from the bearing capacity of the underlying layer would remove the one remaining parameter in the model that rests on judgment rather than on the constitutive description of the ground.

Only a single soil profile was considered, and the conclusions are strictly applicable to a normally consolidated deposit of the thickness and compressibility adopted here. Overconsolidated deposits, layered profiles containing drainage horizons, and creep-susceptible clays would each modify the development of negative skin friction in ways that the present study cannot anticipate. Where creep is significant the dragload would continue to grow after primary consolidation is complete, and the sixty-five-year condition reported here would no longer represent a final state.

Lastly, the study is entirely numerical. The mechanisms it describes are consistent with published experimental work and with an independent closed-form check, but they have not been compared against measurements from an instrumented abutment. Field monitoring of a pile-supported abutment on soft ground, recording axial force along the shaft together with the lateral movement of the cap, would provide the independent validation that no amount of internal consistency checking can substitute for.

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APPENDIX A. Reinforced-Concrete N–M Interaction Screening

This appendix presents a reproducible section-level N–M interaction screening for the pile actions obtained from the numerical analyses. The purpose is to check the order of magnitude and the relative trend of the structural demand, not to perform a code-compliance design. The surcharge and pile-head loads were applied directly in PLAXIS as analysis inputs and were not separated into characteristic and design combinations; accordingly, no additional action factor is applied here. The direct finite-element actions are compared with a representative reinforced-concrete interaction envelope only as a structural screening measure.

A.1 Reference reinforced-concrete section

The supplied section workbook defines the reference pile as a circular $D = 1.0$ m reinforced-concrete section with C30 concrete, B420c longitudinal reinforcement, $30\Phi 32$ bars and a nominal cover of 75 mm. Using Equation (A.1):

$$\begin{aligned}\rho_s &= A_s / A_g \times 100 \\ \rho_s &= 24\,127 / 785\,398 \times 100 = 3.072\%\end{aligned}$$

where:

A_s = total longitudinal reinforcement area (mm²)

A_g = gross pile cross-sectional area (mm²)

ρ_s = longitudinal reinforcement ratio

$$\rho_s = \frac{A_s}{A_g} \quad (\text{A.1})$$

For the diameter sensitivity cases, geometrically comparable reinforcement arrangements were assigned so that the reinforcement ratio remained close to the reference value:

$$D = 0.8 \text{ m}, 19\Phi 32: \rho_s = 3.040\%$$

$$D = 1.0 \text{ m}, 30\Phi 32: \rho_s = 3.072\%$$

$$D = 1.2 \text{ m}, 43\Phi 32: \rho_s = 3.058\%$$

With the same material strengths and approximately geometrically similar reinforcement ratios, the interaction envelope can be extended from the 1.0 m calculation by the similitude relations in Equations (A.2) and (A.3).

$$N_R(D) = N_R(1.0) (D/1.0)^2 \quad (\text{A.2})$$

$$M_R(D) = M_R(1.0) (D/1.0)^3 \quad (\text{A.3})$$

A.2 Finite-element demand pair

The interaction check uses coincident axial force and bending moment at the same depth along the embedded beam. Compression is taken as positive for the screening, while the two bending components are combined as the resultant moment given by equation (A.4). This avoids the unconservative or overly conservative practice of combining an axial-force maximum and a moment maximum that occur at different elevations.

$$M_{res}(z) = (M_2^2 + M_3^2)^{1/2} \quad (\text{A.4})$$

At each depth the relative demand is expressed by the screening index in equation (A.5). The maximum value along all pile profiles in a scenario is used only to compare the scenarios on a common basis.

$$\eta(z) = \frac{M_{res}(z)}{M_R[N(z)]} \quad (\text{A.5})$$

A.3 Diameter-series screening results

The reinforcement and critical same-depth demand pairs used for the diameter comparison are summarized in Table A.1. The corresponding interaction envelopes and critical demand points are shown in Figure A.1.

Table A.1. Reinforcement and N–M screening results for the pile-diameter series.

D (m)	Longitudinal bars	ρ_s (%)	Critical N (kN)	Critical Mres (kNm)	MR at same N (kNm)	η_{max}
0.8	19Φ32	3.040	1445	1544	1680	0.92
1.0	30Φ32	3.072	1627	2151	3196	0.67
1.2	43Φ32	3.058	1862	2944	5434	0.54

The maximum interaction screening index decreases from approximately 0.92 to 0.67 and 0.54 as the pile diameter increases from 0.8 to 1.0 and 1.2 m. The result is consistent with the observed reduction in lateral displacement across the same series and supports the relative conclusion that the larger sections provide greater structural reserve despite

attracting larger absolute bending moments. No “adequate/inadequate” classification is assigned, because the numerical actions are not code-specific design combinations and the pile material model in PLAXIS remains linear elastic.

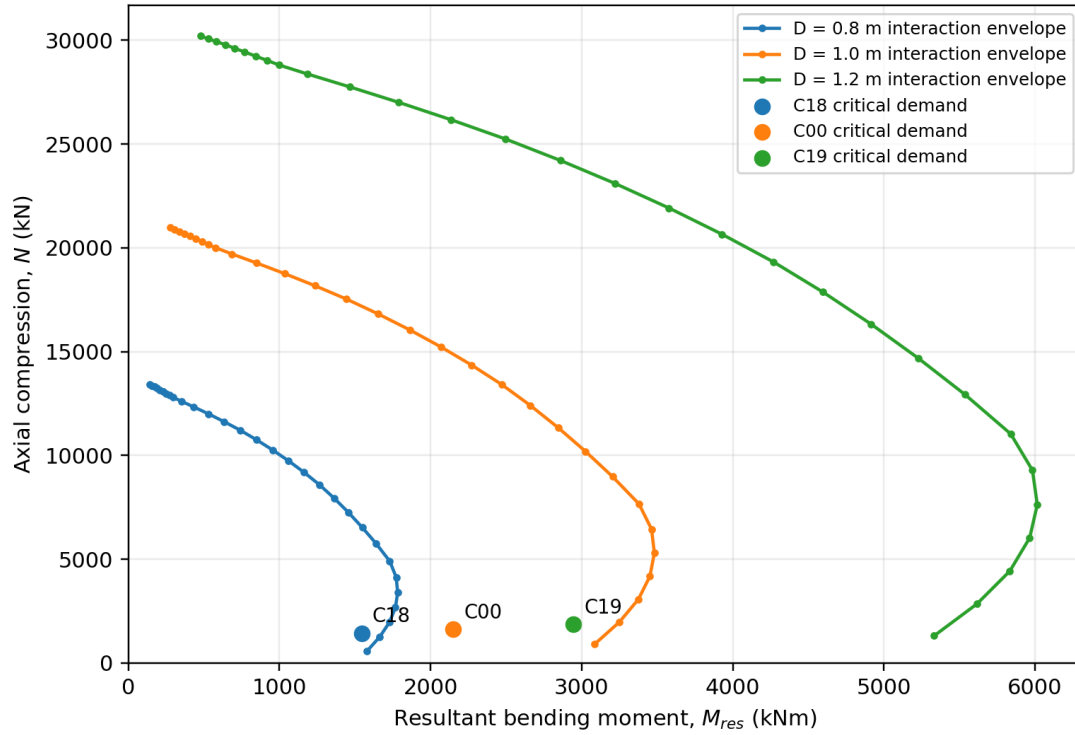


Figure A.1. Reinforced-concrete N–M interaction envelopes used for the diameter-series screening and the corresponding critical same-depth demand points.

A.4 Scope of interpretation

The same N–M screening procedure can be applied to all numerical cases, and the complete case-by-case calculations are retained in the accompanying spreadsheet. Across the wider numerical study the screening index varies substantially, and the more severe cases indicate that reinforced-concrete sectional non-linearity may become relevant. In the present thesis this observation is not interpreted as a formal adequacy judgment. A complementary, deliberately conservative shear and transverse-reinforcement screening is provided in Appendix B. A complete structural design would additionally require code-specific load combinations and partial factors, minimum and maximum transverse-reinforcement detailing, confinement and ductility requirements, crack-control checks and a non-linear sectional stiffness model; these remain outside the scope of the geotechnical parametric study.

APPENDIX B. Indicative Shear and Transverse-Reinforcement Screening

This appendix provides a supplementary shear screening for the embedded-pile actions obtained from PLAXIS 3D. Its purpose is to demonstrate that the transverse force components were also checked systematically and to estimate the order of magnitude of the associated transverse-reinforcement demand. As in Appendix A, the numerical study did not distinguish characteristic and design load combinations; therefore the direct finite-element actions are used without an additional action factor. The calculation is intentionally conservative and is not presented as a code-compliance design.

B.1 Finite-element shear resultant

The two local shear-force components reported for each embedded-beam node are combined into the resultant shear defined by equation (B.1). The scenario-level quantity used for screening is the largest resultant value over all pile nodes, as defined by equation (B.2). This procedure is applied to all C00–C19 and S00–S02 analyses.

$$V_{res}(z) = \sqrt{Q_{12}(z)^2 + Q_{13}(z)^2} \quad (B.1)$$

$$V_{res,max} = \max_z [V_{res}(z)] \quad (B.2)$$

B.2 Conservative transverse-reinforcement demand

Because this thesis is not a structural code-design exercise, no concrete shear-resistance term, axial-compression enhancement, strut-angle optimization or partial-factor combination is introduced. Instead, the complete resultant shear is assigned to an idealized 45° truss-type transverse-reinforcement mechanism. This deliberately neglects the concrete contribution and therefore provides an upper-bound steel-demand indicator rather than a pass/fail resistance check. For consistent comparison between pile diameters, an effective depth of 0.85D and a lever arm of 0.9 times the effective depth are adopted, as expressed by equation (B.3). The reinforcement design strength is taken as 365 MPa, consistent with the B420c basis of the supplied reinforced-concrete section workbook.

$$d_{eff} = 0.85D, \quad z = 0.9d_{eff} = 0.765D \quad (B.3)$$

With the above screening assumptions, the transverse-reinforcement intensity required to equilibrate the finite-element shear force is calculated from equation (B.4).

$$\left(\frac{A_{sw}}{s}\right)_{req} = \frac{V_{res,max}}{z f_{yd}} \quad (B.4)$$

For the reference case C00, the input values for the worked transverse-reinforcement screening are:

$$V_{max} = 891.8 \text{ kN at approximately } 1.97 \text{ m below the pile head}$$

$$D = 1.0 \text{ m}$$

$$d = 0.85D = 850 \text{ mm}$$

$$z = 0.90d = 765 \text{ mm}$$

where:

V_{max} = maximum resultant pile shear force

d = adopted effective section depth

z = adopted internal lever arm

Substitution of these values into Equation (B.4) gives the worked result reported in Equation (B.5).

$$\left(\frac{A_{sw}}{s}\right)_{C00} = \frac{891.8 \times 10^3}{765 \times 365} = 3.19 \text{ mm}^2/\text{mm} = 3194 \text{ mm}^2/\text{m} \quad (B.5)$$

B.3 Cross-case screening results

The same calculation was performed for all 23 principal analyses. The results are summarized in Table B.1. The table reports the maximum resultant shear, its depth below the pile head and the corresponding conservative transverse-steel demand per unit pile length. The full Q12–Q13 components and critical coordinates are retained in the accompanying calculation workbook.

Table B.1. Resultant shear and conservative transverse-reinforcement screening for the 23 principal analyses.

Case	D (m)	Resultant shear (kN)	Depth below pile head (m)	Required transverse steel (mm ² /m)
C00	1.0	891.8	1.97	3194
C01	1.0	264.2	1.97	946
C02	1.0	482.9	1.97	1730
C03	1.0	1191.0	1.97	4265
C04	1.0	963.7	2.00	3451
C05	1.0	1112.9	0.00	3986
C06	1.0	1075.7	1.97	3852
C07	1.0	1020.0	1.97	3653
C08	1.0	916.7	1.97	3283
C09	1.0	685.7	1.84	2456

Case	D (m)	Resultant shear (kN)	Depth below pile head (m)	Required transverse steel (mm ² /m)
C10	1.0	815.2	1.97	2920
C11	1.0	530.7	1.97	1901
C12	1.0	393.5	1.97	1409
C13	1.0	250.0	1.97	895
C14	1.0	939.1	1.97	3363
C15	1.0	876.3	1.97	3138
C16	1.0	877.8	1.97	3144
C17	1.0	925.6	1.97	3315
C18	0.8	810.1	1.97	3626
C19	1.2	912.5	1.97	2723
S00	1.0	800.7	4.77	2868
S01	1.0	334.7	4.77	1199
S02	1.0	1289.9	4.77	4620

The reference case corresponds to a conservative transverse-steel demand of approximately 3194 mm²/m. Across the full analysis the calculated screening demand ranges from about 895 to 4620 mm²/m. These values should not be read as final detailing requirements: the screening deliberately neglects the concrete contribution and does not apply code-specific limits on spacing, minimum reinforcement, confinement, ductility or shear-compression interaction. Its purpose is to demonstrate that the PLAXIS shear actions have been independently extracted and translated into a consistent structural-demand measure without expanding the thesis into a full reinforced-concrete design.

B.4 Scope of interpretation

The shear screening is therefore complementary to the N–M interaction assessment in Appendix A. Appendix A checks the coincident axial-force and bending-moment demand, whereas the present appendix checks the order of magnitude of the transverse-force demand using the same direct numerical actions. Neither appendix issues a formal structural adequacy statement because the applied numerical loads were not organized as code-specific characteristic and design combinations. A project-level pile design would require explicit load combinations and partial factors together with the applicable concrete shear model, minimum and maximum transverse reinforcement, confinement detailing and construction requirements.