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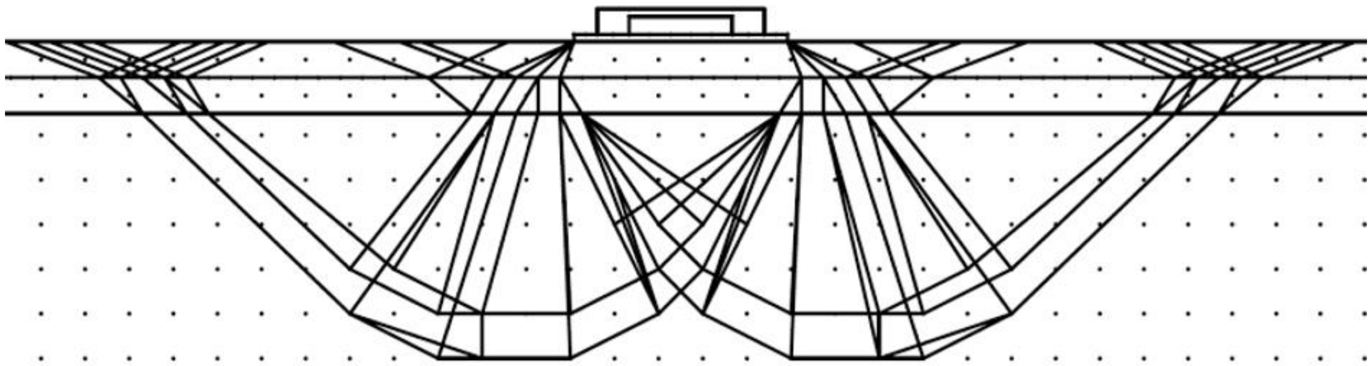

Estimation of collapse load on a granular working platform using limit analysis

A parametric study on a layered soil model

Master's Thesis in the Master's Programme Infrastructure and Environmental Engineering

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Master's Thesis BOMX02-16-41
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Cover: Failure mechanism in soil, derived from the numerical software LimitState:GEO.

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ABSTRACT

A major hazard related to piling is loss of rig stability due to insufficient bearing capacity of the ground, potentially leading to overturning of the piling rig. To facilitate a safe piling procedure a working platform of granular material may be constructed prior to piling, however there are no Swedish standards regarding this design. The required thickness of the working platform may be determined empirically, with well-established bearing capacity equations, or with numerical analysis. In this study the numerical geotechnical stability software LimitState:GEO was used. In the software the ultimate limit state for a plane strain, multiple soil layer problem, was modelled with the limit analysis approach. A sensitivity analysis, with the primary purpose to identify those input parameters with significant impact on the result, was conducted for a symmetrical loading condition. The sensitivity analysis was performed by changing one input parameter at a time, keeping other parameters constant. Furthermore, an unsymmetrical loading condition was modelled, in which a progressive shift of track bearing pressures from an equal load distribution over both tracks, to loading entirely on one track, was analysed. From the sensitivity analysis it was shown that the shear strength of the subgrade clay and dry crust clay, as well as the thickness of the dry crust clay, had a significant effect on the result. The unit weight and internal friction angle of the platform material had minor effect on the result. The result from the unsymmetrical loading condition was shown to be strictly dependent on the software feature "delineation", permitting rotational failure in soil.

Key words: LimitState:GEO, bearing capacity, working platform, limit analysis, pile driving rig, layered soil.

Estimering av brottgränslast för en arbetsbädd av krossmaterial
En parametrisk studie av en jordmodell med flera lager

Examensarbete inom masterprogrammet Infrastructure and Environmental Engineering

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SAMMANFATTNING

Vid pålningsarbeten utgör förlorad stabilitet av pålkranen en stor fara. Denna instabilitet kan orsakas av otillräcklig bärighetsförmåga i marken, och kan leda till att pålkranen välter. Genom att anlägga en arbetsbädd av krossmaterial förbättras förutsättningarna för ett säkert pålningsarbete. I Sverige finns ingen standard för hur en sådan arbetsbädd bör utformas. Arbetsbäddens tjocklek kan bestämmas empiriskt, med väletablerade analytiska bärighetsmetoder, eller genom numeriska beräkningsprogram. I detta examensarbete användes beräkningsprogrammet LimitState:GEO, vilket behandlar tvådimensionella, geotekniska stabilitetsproblem. Programmet användes för att beräkna brottgränstillståndet för en modell bestående av tre jordlager. En sensitivitetsanalys genomfördes för ett symmetriskt belastningsfall, och syftade till att analysera respektive parameters inverkan på resultatet. I analysen varierades en parameter i taget och resterande parametrar behölls konstanta. Ett osymmetriskt belastningsfall modellerades genom en gradvis överföring av pålkranens marktryck, från belastning av bägge larvfötter till enbart belastning av en larvfot. Resultatet från sensitivitetsanalysen visade att de tre parametrarna med signifikant påverkan på resultat var; leran och torrskorpans skjuvhållfasthet, samt tjockleken på torrskorpan. Även tunghet och friktionsvinkeln för krossmaterialet inkluderades i analysen, emellertid påverkade inte dessa parametrar resultatet i samma utsträckning. Analysen av det osymmetriska belastningsfallet var i stor utsträckning beroende av det tillvägagångssätt som användes för att tillåta rotationsbrott i jordmodellen.

Nyckelord: LimitState:GEO, bärförmåga, arbetsbädd, brottgränstillstånd, pålkran, jordmodell flera lager.

Contents

Preface

This master thesis was carried out from January 2016 to June 2016, at the Department of Civil and Environmental Engineering, division of GeoEngineering, Chalmers University of Technology, Sweden. The topic of this master thesis was initiated by Patrik Andersson at Skanska Grundläggning, to whom we would like to express our sincere thanks and appreciation to. We are also grateful to the employees at the geotechnical department at Skanska Teknik, for their useful help and input to this thesis. Our supervisor at Chalmers, Mats Karlsson, is highly appreciated for his help and advice regarding the planning, execution and completion of this master thesis. We would also like to show our appreciation to the FPS and Junttan, for their co-operation and involvement in our work. Finally, we would like to thank the support team at LimitState Ltd for assistance and advice regarding the numerical modelling.

Göteborg June 2016

Axel Grahnström

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Notations

Roman upper case letters

B	Foundation width
D	Platform thickness
D_f	Depth of foundation
E_d	Effect of actions
F_C	Factor of safety, undrained analysis
F_{cs}	Shape factor, cohesion
$F_{C\phi}$	Factor of safety, combined or drained analysis
F_{qs}	Shape factor, load
$F_{\gamma s}$	Shape factor, unit weight
G_{kj}	Permanent load, geotechnical actions
H	Thickness of layer
K_s	Punching shear coefficient
L	Foundation length
N_c	Bearing capacity factor, cohesion
N_q	Bearing capacity factor, load
N_γ	Bearing capacity factor, unit weight
Q_d	Specified design load
Q_{kj}	Variable load, geotechnical actions
Q_{ULS}	Analytical collapse load
R_d	Design bearing resistance
W	Platform width
X_d	Dimensional value
X_k	Characteristic value
$\bar{X}$	Derived value
Y'	Failure criterion

Roman lower case letters

c_u	Undrained shear strength
c'	Drained cohesion intercept
h_{DC}	Thickness, dry crust
h_{WP}	Thickness, working platform
n	Set of nodes
q_u	Ultimate bearing capacity

q_1	Ultimate bearing capacity upper layer
q_2	Ultimate bearing capacity lower layer
q_d	Design load
q_k	Characteristic load
s_c	Shape factor, bearing capacity cohesive subgrade
s_p	Shape factor, punching shear resistance
s_γ	Shape factor, bearing capacity platform material or granular subgrade
u_w	Pore water pressure

Greek letters

α	Load spread angle
γ	Unit weight
γ_{cu}	Partial factor, undrained shear strength
$\gamma_{c'}$	Partial factor, effective cohesion
γ_d	Partial factor on load
γ_G	Partial factor, favourable conditions
γ_M	Partial factor, soil parameter
γ_Q	Partial factor, unfavourable conditions
γ_{sat}	Saturated unit weight
γ_γ	Partial factor, density
$\gamma_{\phi'}$	Partial factor, angle of shearing resistance
ε_x	Normal strain, x-direction
ε_z	Normal strain, z-direction
η	Conversion factor
θ	Inclination
σ	Normal stress
σ'	Effective stress
σ_1'	Effective major principal stress
σ_3'.	Effective minor principal stress
σ_C	Limiting compressive stress
σ_T	Limiting tensile stress
τ	Shear stress
ϕ'	Internal angle of shearing resistance

Abbreviations

AMA 13	Work and material descriptions for construction works
BR470	Working platform design proposed by Federation of Piling Specialists
CEN	European Committee for Standardization
DA	Design approach in Eurocode
DLO	Discontinuity Layout Optimization
EC	Eurocode
EQU	Equilibrium of structure
FPS	Federation of Piling Specialists
GEO	Ground resistance
HYD	Hydraulic heave
LSG	LimitState:GEO
PPL	Uplift from water pressure
PSCF	Punching shear coefficient factor
SC	Safety class
TK GEO	Technical requirements on geotechnical structures from the Swedish Road Administration
ULS	Ultimate limit state

1 Introduction

Construction of buildings, roads and railways requires some sort of foundation, generally categorised into deep or shallow foundations (Knappett & Craig, 2012). The extent of the foundation required is basically determined by the ground conditions and magnitude of load. Piling with precast concrete piles is a deep foundation method, where loads are transmitted in the ground through piles driven into the ground with the use of a pile driving rig. The rigs are commonly tracked and have a weight around 30-100 tonnes and a height around 20 to 30 meters, generating a high centre of gravity (Junttan, 2016).

A major hazard related to piling is loss of rig stability due to insufficient bearing capacity of the ground, potentially leading to overturning of the rig. In Sweden there is about one overturning accident a year[1], associated with severe economic, health, and environmental risks. Installation of piles includes different rig configurations, loading, and operation conditions, giving rise to various magnitude and distribution in track bearing pressure (BSI, 2014). To prevent instability of a pile driving rig the ground must be assured to have sufficiently bearing capacity to withstand the load from the pile driving rig without total or excessive ground failure.

To facilitate a safe piling procedure a working platform of granular material may be constructed prior to piling. There are no Swedish standards regarding the design of working platforms, and generally experience or analytical bearing capacity calculations are used as a basis for the design[1]. In Great Britain a guidance paper describing an analytical design approach is proposed for working platform design (BRE, 2004). However, the approach is designed for British soil conditions and cannot directly be applied in Sweden. A typical soil condition in the Gothenburg region in Sweden implies a two layered soil model where surficial clay crust overlying clay with low shear strength.

LimitState:GEO is a numerical geotechnical stability software, modelling total failure for plane strain problems with the limit analysis approach (LSG, 2015). The software may be used to analyse various types of geotechnical stability problems, including bearing capacity analyses. The software outputs an adequacy factor, that the applied load or material parameters should either be multiplied or divided with in order to obtain the collapse load of the system.

1.1 Background

Skanska is one of the leading construction companies in Sweden and one of their objectives is to have zero work related accidents (Skanska, 2016). Though, accidents occur, and in 2011 and 2012, two of Skanska Grundläggning pile driving rigs overturned at two different work sites in Gothenburg[1]. Fortunately, there were no health related injuries associated to these two accidents, though they implied major economic losses. At both sites working platforms of granular material were installed, and load spreading timber mats were used. The underlying ground conditions were typical for the Gothenburg region with weak clay overlaid by surficial dry crust clay.

[1] Patrik Andersson District Manager Skanska Grundläggning, interview March 14, 2016.

Thorough investigations of the accidents were made by both the geotechnical department of Skanska and by the contractor division Skanska Grundläggning. In both cases the pile driving rig was found to be in a position causing excessive loading on the ground, and the specific ground failure was caused by insufficient bearing capacity of the soil and working platform[2]. The investigation included numerical analysis with the geotechnical finite element software Plaxis, however the ability of the model was not satisfying. An effort was made to design working platform for Swedish ground conditions with a modified version of the analytical design method proposed by Meyerhof (1974). In 2015 a master thesis was written at Chalmers for Skanska, with the main purpose to develop and perform a field study procedure that could verify the modified design method (Dahlgren & Nyman, 2015). It was concluded that from the particular study no verification of the modified method could be done.

In 2016 this particular master thesis was initiated by Skanska Grundläggning as they want to move further with the design of safe working platform and develop a straight forward design method that takes Swedish soil conditions into considerations.

1.2 Aim and objectives

The main objective of this study is to model a working platform in the numerical software LimitState:GEO, and to perform a sensitivity analysis with the primary purpose to identify input parameters with significant impact on the result. Furthermore, the study aims to review fundamental theory regarding bearing capacity problems and available methods for design of working platforms. Following research questions are posed, and are to be determined from the numerical modelling:

- In what way does the thickness of the granular platform influence the output adequacy factor on load?
- What is the effect on the output adequacy factor of using load spreading timber mats in the model?
- Does the numerical model enable design of a working platform in accordance with Eurocode 7?
- Is it possible model a rotational failure mechanism by considering an unsymmetrical loading condition?

1.3 Methodology

Reference material concerning bearing capacity and working platforms comprise of geotechnical; handbooks, reports and articles. The modelling in LimitState:GEO was conducted in accordance with specified instructions in the software manual. Input values for each soil parameter used in the numerical modelling and analytical calculations were concluded by the authors, in accordance with studied literature concerning ground conditions in Gothenburg. In order to identify those parameters significantly affecting the result of the analyses a sensitivity analysis was conducted, where one parameter at a time was changed. The range of values was determined by the authors with respect to values specified in literature. Track bearing pressures were obtained from an instruction manual for a certain pile driving rig. A thorough review of the performed analyses can be found in section 5.

[2] Peter Claesson Technical Expert Skanska Teknik, interview April 21, 2016.

1.4 Scope

In this report working platforms are regarded as firm non-reinforced ground supporting constructions of granular media, employed by tracked pile driving rigs. Furthermore, this report addresses overturning of pile driving rigs initiated by insufficient bearing capacity of working platform and underlying soil. The material parameters used in the analyses are not associated with any particular project. The clay's shear strength is treated as constant with depth. The track bearing pressures included in the analyses are only valid for certain loading and operational conditions of a certain pile driving rig. The pile driving rig is assumed to be positioned in a central position over the timber mat. The surface of the working platform is assumed to be levelled.

Working platforms may be designed by observational, empirical, analytical and numerical methods. In this report the two latter approaches are covered. The numerical method treats a short term analysis of a three-layered soil system, where the granular working platform is positioned on surficial clay crust underlain by weak clay. Load from both tracks, and a load spreading timber mat are included in the model. The analytical approach treats a two-layered soil system with granular working platform overlaying weak clay. In these analyses no timber mat is included, and only one load is considered. The software LimitState:GEO is restrained to model limit states of geotechnical problems and omits deformations, consequently the collapse state is studied solely. As the software models in plane strain any three-dimensional effects are neglected.

2 Important soil properties for evaluation of a soil's bearing capacity

The required thickness of a granular working platform may be determined by considering the system as a bearing capacity problem. Bearing capacity of soils is strictly related to the material's shear strength. An outline of soil's shearing resistance is presented in section 2.2 and some well-established analytical bearing capacity methods are presented in section 2.3.

2.1 Formation of surficial dry crust clay

The formation of soil from geological material such as rock includes weathering, transport and deposition, all having significant effect on soil properties (Knappett & Craig, 2012). Weathering includes the in-situ processes of disintegration and decomposition of geological material, where disintegration affects the structure of the material, and decomposition modifies the chemical properties of the minerals (Kenney, 1975). Weathered material is transported by natural forces including glacial movements, water and wind. The type of transportation governs the grain size and distribution in the soil. Eventually the transported matter deposits and the conditions under which the deposition takes place affect the particle arrangement.

According to the Unified Soil Classification System, USCS, which was developed from a concept presented by Casagrande in the 1940s, soils are divided into two categories based on the grain size; fine grained soils and coarse grained soils (ASTM International, 2011). In addition, there is one category covering highly organic soils. According to this system a coarse grained soil contains less than 50% of fractions finer than 0.06 mm, and a fine grained soil contains more than 50% of fractions finer than 0.06 mm. Clay is a soil type comprised of ≥40% fine grained material, and where at least 40% out of this material comprises of clay particles having a size <0.002 mm (SGI, 2016). Clay particles are thin and shaped like a sheet whereas sand and silt particles are rounded (Knappett & Craig, 2012). Soils are a particulate material with void space filled with water and/or air, the amount of the pore space filled with water is defined by the saturation ratio. Soils below the water table are considered to be fully saturated. In clay the pores are partly or completely filled with water and from the material structure water is easily retained causing a low permeability.

During the latest glacial period Sweden was fully covered with glacial ice (SGI, 2016). At the melting of the glacier fine fragments like clay particles was deposited as sediment in seas or lakes. In some areas in Gothenburg the clay has been proven to have greatness up to 100 meters (SGI, 2007). The shear strength of the clay in Gothenburg are around 8-16 kPa and generally increase linearly with depth (SGI, 2007). Furthermore, the clay is normal or slightly over consolidated, implying that the current effective stress level represents the highest level of stress that the soil has ever been exposed to. The relationship between undisturbed and remoulded shear strength is referred to as sensitivity and may vary significantly for different clays (SGI, 2008). Low and medium sensitive clays have a sensitivity ratio of <8, and between 8 and 30 respectively.

As a result from the upper part of a clay being exposed to physical and chemical stresses, a surficial clay crust, also called “dry crust clay”, with diverse structure and chemical properties than the original material may be developed (Lutenegger, 1995). The dry crust clay is characterised by a large variability of intrinsic properties. Even in a small sample of dry crust material it is likely to observe large variations in shear strength, water content and stress history. Raymond (1972) studied settlement of embankments on clay, and found that one of the major uncertainties in the analysis was associated to assessment of the dry crust’s properties. There are several well established methods for analysing shear strength of clay, however, these methods are ordinarily not applied on surficial clay crust (SGI, 2007).

In a study at Chalmers University of Technology in 1984, the geotechnical properties of dry crust samples were analysed, showing that the crust is characterised by high shear strength and lower water content than in the parent material (Ringsten, 1984). It was shown that there is a distinct fissures network in a dry crust, providing a high hydraulic conductivity at dry conditions. However, the crust is sensitive to seasonal changes and during rain events the fissure system diminishes due to swelling of the clay, decreasing the hydraulic conductivity by as much as 75 percent. The depth of the dry crust varies and the defining parameter of the depth is climate conditions, and seasonal variations of the ground water level (Lutenegger, 1995). In Gothenburg the dry crust is around zero to two meters deep and has generally the same unit weight, but significantly higher shear strength than the underlying clay (SGI, 2007).

It has been shown that the dry crust has important implications in geotechnical design (Lutenegger, 1995). Stability of slopes and embankments, along with bearing capacity, may be influenced to a certain extent by the geotechnical properties of the dry crust. However, there are disagreements of how to utilize the strength properties of the dry crust. The Swedish Geotechnical Commission stated that the bearing capacity of embankments could be considerable higher due to a weathered dry crust; however, utilizing the full thickness of the dry crust in design could lead to ground failures (Flodin & Broms, 1981). Caldenius (1925) purposed that when analysing bearing capacity of embankments, the dry crust should preferable be divided into three parts with respect to the shear strength properties. Helenelund (1953) studied the stability and failure of soil in railway embankments, and suggested that half the thickness of the dry crust should be included in the bearing capacity calculations. In a method description of slope stability analysis in Gothenburg, the undrained shear strength of surficial clay was restricted to 30 kPa (SGI, 2000).

2.2 Resistance of soil to failure in shear

Soils often fail in shear, and the design calculations of several of geotechnical constructions such as; slopes, retaining walls and bearing capacity of shallow foundations, are dependent on the shear strength of the soil (Knappett & Craig, 2012). For a small element of soil, it is clear that pressure from the overlaying and surrounding soil will act upon it, generating normal stresses, σ, working orthogonal to the soil surface. External loading of the soil induces shear stresses, τ, working parallel to the surface. Shear failure of a soil is reached when the applied shear stress equals or exceeds the shear strength of the soil. As depicted in Figure 2.1, the three dimensional state of stress may be simplified into two dimensions, a useful interpretation to facilitate analysis. In two dimensions there are both vertical and horizontal normal and shear stresses acting on the element.

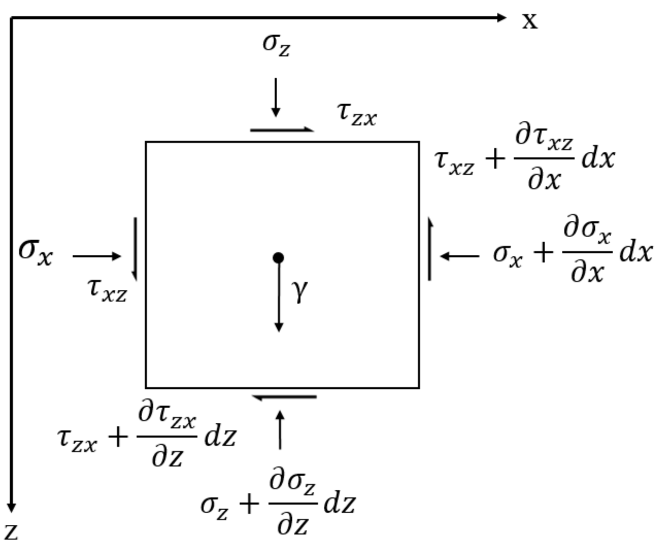


Figure 2.1 Normal and shear stresses acting on a two dimensional soil element (Knappett & Craig, 2012).

The magnitude of normal and shear stresses are not constant and the rate of change in two dimensions are described by $\partial\sigma_x/\partial x$, $\partial\sigma_z/\partial z$ and $\partial\tau_{xz}/\partial x$, $\partial\tau_{xz}/\partial z$ respectively (Knappett & Craig, 2012). The equations of equilibrium are derived by considering each element in the soil to be in static equilibrium, see equation 2.1 and 2.2. Displacements in soil induced by applied loading generate strains, expressed as $\varepsilon_x=\partial u/\partial x$ in the x-direction, $\varepsilon_z = \partial w/\partial z$ in z- direction, and as $\gamma_{xz}=\partial u/\partial z + \partial w/\partial x$ for shear strain. By considering soil masses as continuous media the equations of compatibility are given, see equation 2.3.

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xz}}{\partial z} = 0 \qquad (2.1)$$

$$\frac{\partial \tau_{xz}}{\partial x} + \frac{\partial \sigma_z}{\partial z} - \gamma = 0 \qquad (2.2)$$

$$\frac{\partial^2 \varepsilon_x}{\partial z^2} + \frac{\partial^2 \varepsilon_z}{\partial x^2} - \frac{\partial \gamma_{xz}}{\partial x \partial z} = 0 \qquad (2.3)$$

Soils are anisotropic material, exhibiting non-linear stress-strain behaviour (Knappett & Craig, 2012). In order to facilitate analysis, the relationship between stress and strain may be described by idealised constitutive models, also termed material models see Figure 2.2. In geotechnical problems dealing with stress and deformations below the failure criterion, Y', a model based on Hooke's linear relationship between stress and strain is commonly used. Below the yield point the soil exhibits a linear elastic behaviour, and when reaching the yield point unrestricted plastic strain occurs, representing continual deformation at constant stress. In analysis explicitly concerning failure of soil, a rigid-perfectly plastic material model may be implied. This model considers the material as rigid when the inherent stress level is below the yield stress, and as perfectly plastic when the yield stress level is reached.

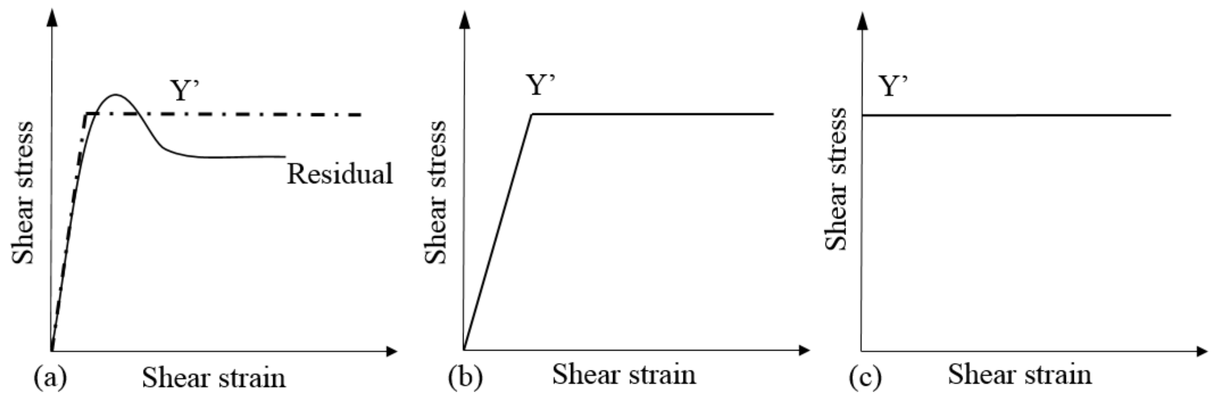


Figure 2.2 (a) Actual relationship between stress and strain derived from simple shear test or a triaxial compression test (Chen, 1975), (b) elastic-perfectly plastic material model (Knappett & Craig, 2012), (c) rigid-perfectly plastic material model.

The plasticity theory described by Hill (1951), considers non-linear and irreversible behaviour of material and comprise of a; yield function, hardening law and flow rule. The yield function is characterized by a yielding criterion based on principal stresses, for instance covered by the Mohr-Coulomb criterion (Knappett & Craig, 2012). The hardening law describes the strengthening of material as a relationship between yield stress and plastic strain. Furthermore, the relative magnitude of plastic strain arising for a certain state of stress is specified by the flow rule. The plasticity theory has been widely applied in stability applications such as bearing capacity and forms a basis of the limit analysis method described in section 2.3.2.

The principle of effective stresses, σ', was presented by Terzaghi in 1923, and applies to fully saturated soils (Terzaghi, 1943). The principle implies that stresses are transmitted between the contact points of the soil particles. The magnitude of effective stresses cannot be directly measured, but are obtained from the difference between normal stresses, σ and pore water pressure, u_w. The state where no shear stress is acting on a face is called effective principal stress, and the corresponding plane is called a principal plane (Knappett & Craig, 2012). In a two dimensional analysis, the effective major principal stress is denoted σ_1', and the effective minor principal stress σ_3'. The stress state of a soil may be displayed in a Mohr Circle, in a two-dimensional diagram, where values of shear stress, τ, is plotted against the effective normal stresses σ_1' and σ_3'. Drawing the circles diameter with an inclination of 2θ to the horizontal axis, the stress state on a plane with the inclination of θ to the minor principal stress is obtained.

In 1773 Coulomb proposed that shear strength of soil is dependent on the inherent resistance between soil particles, which are mainly controlled by frictional forces (Knappett & Craig, 2012). In addition to frictional forces other interlocking forces, such as chemical cementation, where soil grains are bound together typically by calcium, and cohesion, where atoms at the surface of a soil particle share electrons, contribute to the shear strength. Coulomb expressed the shear strength in terms of cohesion and friction and by considering the principals of effective stresses, the shear strength parameters are denoted c' and ϕ', see equation 2.4.

$$\tau_f = c' + \sigma_f' * tan\phi' \quad (2.4)$$

The Mohr-Coulomb criterion is a constitutive model describing strength behaviour of a soil, combining the stress state plotted in Mohr circle and the shear strength equation by Coulomb, plotted as a failure envelope, see Figure 2.3 (Knappett & Craig, 2012). For a critical combination of shear stress and effective normal stress the failure envelope tangents the circle, implying failure of soil.

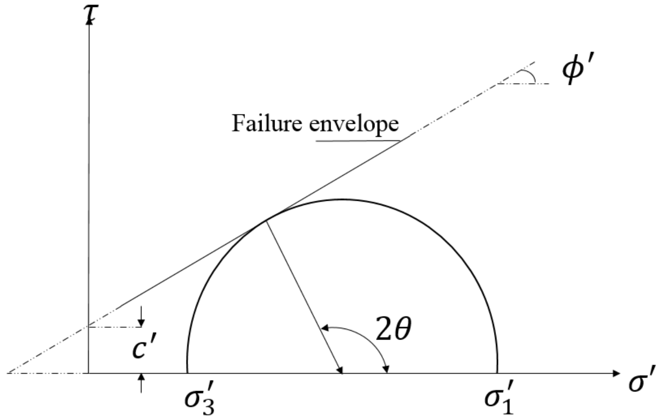


Figure 2.3 Mohr-Coulomb failure criterion (Knappett & Craig, 2012).

Terzaghis effective stress principle, relying on soil skeleton as the determining resistant of shear stresses, implies that pore water pressure has a certain impact on the shear strength of a soil. The pore water pressure is governed by the applied loading and consolidation properties of the soil (Knappett & Craig, 2012). Applied load on a fine grained soil gives rise to excess pore pressure, and thus a decrease in effective stresses and loss in frictional forces. Accordingly, for short time analysis the undrained shear strength is dimensional. For coarse grained soils water is allowed to drain and no excess pore water pressure is induced, implying use of drained shear strength. However, for momentary loading of coarse grained soils, induced for instance by earthquakes, excess pore water pressure is built up as a result from the rapid loading. Accounting for these types of scenarios an undrained analysis is appropriate.

2.2.1 Shear failure modes in soil with reference to a strip footing

Bearing capacity of soils is strictly related to the material's shear strength (Knappett & Craig, 2012). The ultimate bearing capacity of a foundation may be described as the maximum pressure which the soil is able to support without causing collapse or instability of the whole structure. For a strip footing shear failure occurs beneath, or adjacent to the footing when the bearing capacity of the supporting soil is exceeded. There are three identified modes of shear failure; general, local and punching, the occurrence depends on the shear strength and stiffness of the soil.

When a general shear failure occurs a continuous failure surface develops from the side of the footing to the surface of the ground (Knappett & Craig, 2012). A general failure is common when the foundation rests on a dense coarse grained material or a stiff fine grained soil. When the ground fails, heaving adjacent to the foundation occurs. Beneath the foundation an active wedge is developed and passive wedges are resisting the failure on each side of the foundation, see Figure 2.4 and Figure 2.7 in section 2.3. In an ideal case the failure surface occurs at both edges of the foundation, however a soil is seldom unison and the foundation is not completely horizontal. Hence the failure generally only occurs on one side of the foundation.

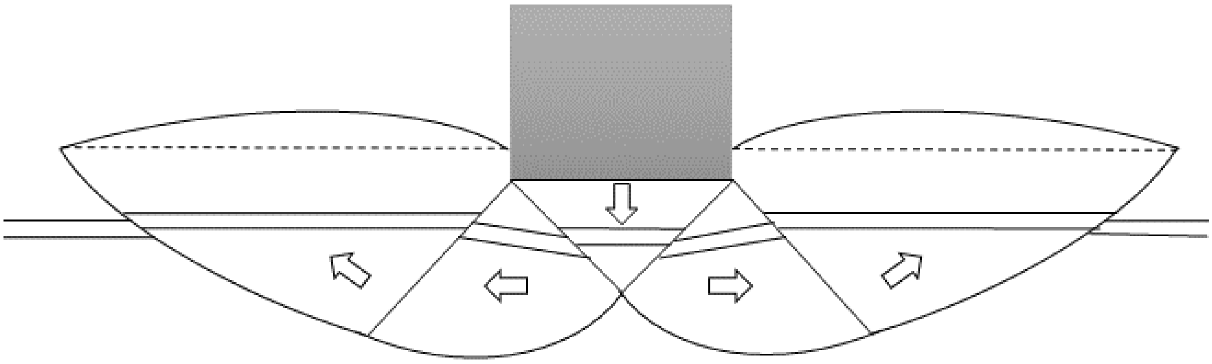


Figure 2.4 General shear failure in soil (TWf, 2015a).

A local shear failure occurs if the soil beneath the foundation is a relatively weak fine- or coarse grained soil with moderate compressibility. An increase in load will form a failure surface that occurs outwards from the foundation, see Figure 2.5 (Das, 2011). Compared to a general shear failure the ultimate bearing capacity for a local shear failure is not as well defined, and there is less heaving adjacent to the foundation.

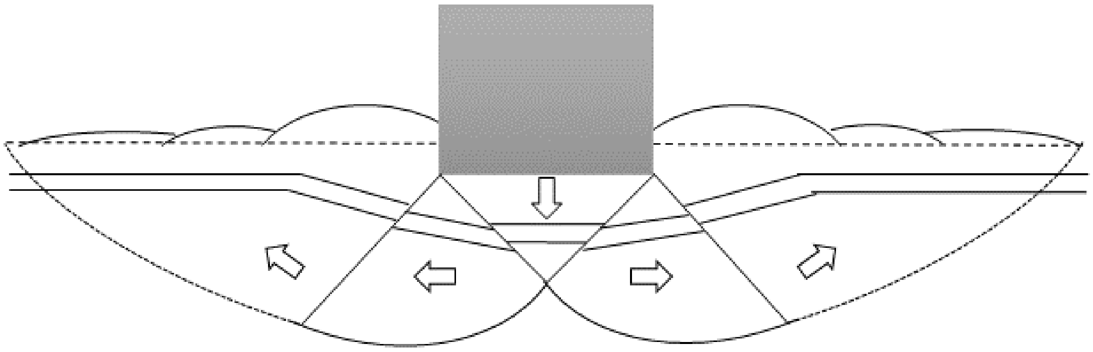


Figure 2.5 Local shear failure in soil (TWf, 2015a).

For a foundation positioned on a weak soil with high compressibility, a punching shear failure occurs when the load exceeds the bearing capacity (Das, 2011). This type of shear failure will not be extended to the ground surface since no shear plane develops, see Figure 2.6. A punching shear failure might also occur in a soil with low compressibility if the foundation is placed deep below the surface level.

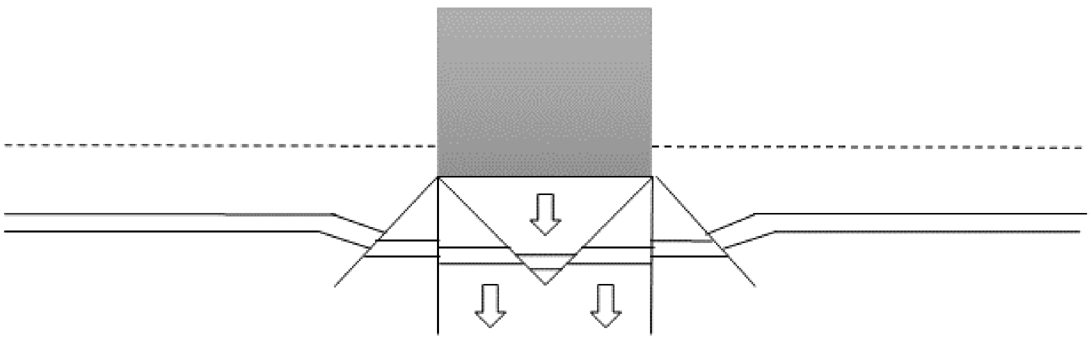


Figure 2.6 Punching shear failure in soil (TWf, 2015a).

2.3 Bearing capacity of soil - analytical design methods

In the design of working platforms the required thickness of the platform may be determined by considering the system as a bearing capacity problem. A soil stratum with an applied load from a pile driving rig may be considered to be influenced in similar means as from an incrementally loaded rigid footing. Considering the footing, stresses in the soil are initially elastic and as the yield stress is reached, plastic yielding occurs (Chen, 1975). The yielding occurs primarily at the corners of the footing, and with increased loading, plastic zones in the soil spread down in the stratum and towards the centre of the footing. Still, the plastic regions are enclosed by elastic regions sustaining the load. As a consequence of additional loading the plastic yielding spreads in the soil, and eventually the maximum bearing capacity of the soil is reached. At this stage impending plastic flow occur, and soil that is still elastic at this stage have insignificantly effect in sustaining the load. Beyond the maximum bearing capacity increased loading will generate a collapse of the footing.

In the field of geotechnics there are several analytical design models for the estimation of a soil's bearing capacity. Prandtl (1920) proposed a method applicable on shallow foundations in which soil is subdivided into solid wedges that at collapse state displace as rigid objects in the soil. Prandtl treated the soil as homogeneous, isotropic and weightless and the strength of the soil was defined in accordance with Coulomb's theory. Furthermore, the stress-strain relationship was expressed with a rigid-perfectly plastic material model and a frictionless boundary was assigned between the foundation and the soil stratum, permitting no shear stresses in this region. The failure mechanism incorporates a; active wedge beneath the footing, passive wedges next to the footing and plastic zones connecting the two wedges, see Figure 2.7. In addition to the early studies by Prandtl, there are several well-established solutions of bearing capacity problems including; Limit Equilibrium, Limit analysis and Slip line analysis. The fundamental theory of each approach along with important findings is described in section 0, 2.3.2 and 2.3.3

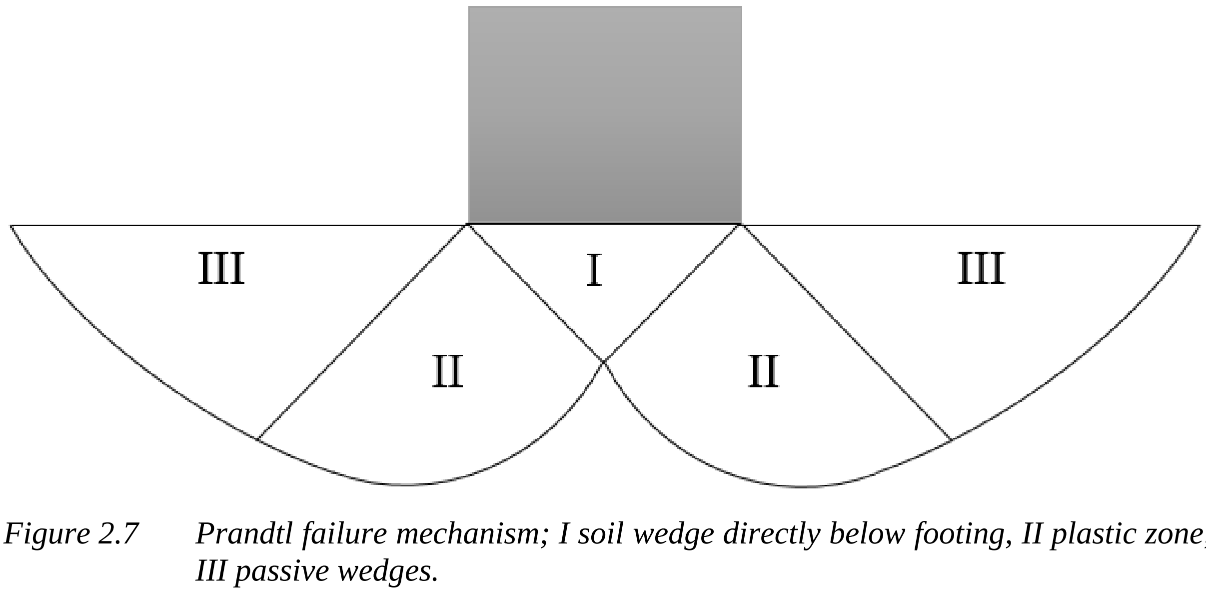


Figure 2.7 Prandtl failure mechanism; I soil wedge directly below footing, II plastic zone, III passive wedges.

2.3.1 Limit equilibrium

In order obtain an approximate solution for stability problems of soils, including bearing capacity and slope stability analysis, the well-established limit equilibrium method may be applied. In limit equilibrium methods failure in soil is assumed to induce a failure surface of simple shape, for instance planar or circular. The solution is achieved when the most critical position of the chosen failure surface is identified.

Terzaghi (1943) presented a comprehensive limit equilibrium theory of bearing capacity of soils incorporating equations for general and local shear failure, for three different geometrical shapes of a footing; strip, square and circular. Terzaghi's theory was extended from Prandtl's method, and incorporates self-weight of the material in the plastic zone, and roughness in the interface between soil and footing. The purposed equations for the ultimate bearing capacity, q_u, of general shear failure for a strip footing, can be seen in equation 2.5. The three different terms concern cohesion of soil, overburden pressure from self-weight of soil, and width of foundation respectively. N_c, N_q and N_γ are bearing capacity factors related to the angle of internal friction, ϕ and B represents the width of foundation.

$$q_u = c'N_c + \sigma_z' N_q + 0.5\gamma' B N_\gamma \qquad (2.5)$$

The work of Terzaghi have been continuously developed and applied to estimate ultimate bearing capacity of a foundation on a strong layer of soil, overlying a weaker one (Terzaghi & Peck, 1948). The strength of the upper layer was incorporated in the analysis by considering a load spread through it. The assumed load distribution adopted in the study was defined as the load spread angle, α, analogous to a vertical distance of two units for each horizontal distance unit, generating tan α = 0.5.

Meyerhof (1974) established an ultimate bearing capacity method of a footing on a two layered soil stratum for sand overlaying clay. In the analysis, resistance generated from a punching shear failure within a strong upper layer was considered, rather than accounting for a load spread through this layer. In the underlying soil a general shear failure is anticipated, however, an extensive upper layer may restrict the failure to this layer solely. The method was further developed by Meyerhof and Hanna (1978) by taking inclined loads into account. In 1979 the authors extended the study to a three layered soil model with special reference to layered sand (Meyerhof & Hanna). In the analysis two layers of strong sand on top of a weak subgrade was included. The performed experimentally studies were in 1980 accompanied by design charts, covering additional shape and depth factors to be included in Terzaghi's bearing capacity equations (Meyerhof & Hanna).

2.3.2 Limit analysis

The limit analysis method may be used to estimate the collapse load of a soil by using the plasticity theory (Chen, 1975). The limit analysis method can be used when estimating the collapse load, without taking eventual deformations into consideration (Aysen, 2002). The limit analysis theory is straightforward and convenient when calculating the ultimate load. Solutions by the limit analysis method require the stress equilibrium equations, stress-strain relations, and the compatibility equations to be complied.

The true ultimate load for a soil could be computed by the limit analysis, which consists of the lower- and upper bound-limit theorem (Chen & Davidson, 1972). A load that fulfils the criterions of the lower-bound theorem is considered statically admissible (Chen, 1975). For loads that satisfies the upper-bound theorem criterions the failure of the soil has already occurred for the solution (Aysen, 2002). Hence by evaluating the results from the two theorems the collapse load for a soil could be restricted from the lower- and upper-load (Makrodimopoulos & Martin, 2007).

Numerical implementation of the limit analysis involves discretization of soil by nodes, generating small elements (Lyamin, et al., 2007). This is in particular represented in the software LimitState:GEO.

2.3.3 Slip line analysis

Considering the footing described in section 2.3, stresses in the soil can be analysed by the slip line method, introduced by Kötter (1903). At the moment of impending plastic flow in the soil the yielding condition, described by the Coulomb criterion, and equilibrium conditions, described in section 2.2, are satisfied (Chen, 1975). In slip line analysis a set of differential equations of plastic equilibrium is established from combination these two conditions accompanied with initial stress conditions in the soil. To facilitate the solvent of the differential equations these are expressed as curvilinear coordinates that at every point of the yielded soil has the same direction as slip lines, also named failure surfaces.

Solving the differential equations analytically was firstly done by Prandtl (1921), though this analysis was carried through regarding the soil as weightless, an idealisation crucial to find a closed-form solution. Thus, slip-line solutions may be exact solutions, however several approximate methods have been developed and both upper-bound and lower-bound solutions may be obtained (Chen, 1975). Sokolovski (1965) described a numerical solution, and De Jong (1959) presented a graphical method of solution.

3 Software outline – LimitState:GEO

LimitState:GEO, hereinafter referred to as LSG, is a numerical geotechnical stability software modelling ultimate limit state, ULS, for plane strain problems with the limit analysis approach (LSG, 2015). It was industrialised in 2008 and developed by Smith and Gilbert at the University of Sheffield, England. The software may be used for analysis of various stability problems in soils, including slope stability and bearing capacity. The software provides an upper bound solution by the use of a numerical analysis method named Discontinuity Layout Optimization, DLO. In contrary to finite element methods, utilising incremental load to collapse state, LSG goes directly to the collapse state, omitting deformations and stressed prior to collapse state.

3.1 Discontinuity Layout Optimization

The objective of DLO is to identify the most critical translational sliding block failure mechanism from a setup of nodes (LSG, 2015). When soils fail in shear the failure occurs along slip lines, also called *discontinuity* lines, formed from interconnection of nodes. The potential slip lines identified in the analysis, between which relative shear displacement occurs, are each assigned a variable. The discontinuity lines confine blocks of soil, deforming equivalently (Smith & Gilbert, 2007). This compatibility is checked at each node in the system by setting up linear equations. In LSG a *layout* of potential slip lines is created by a set-up of nodes and in order to identify the critical mechanism involving the lowest energy dissipation a linear objective function is setup, comprising the variables associated with each discontinuity line. The defined linear optimization problem is eventually solved by the linear programming solver Mosek, utilising a mathematical *optimization* technique.

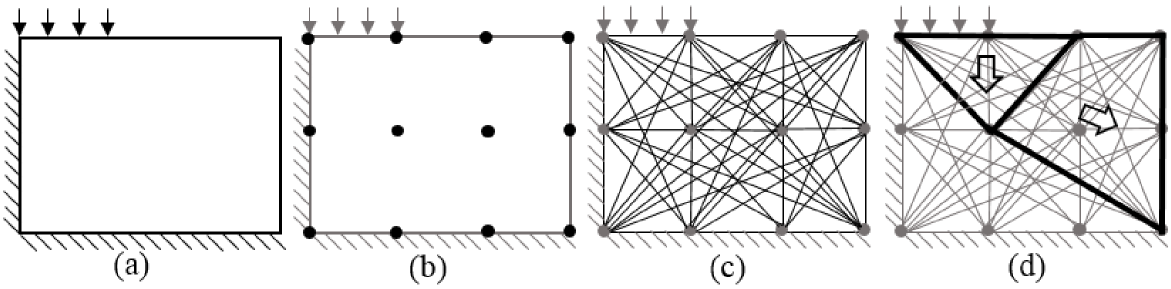


Figure 3.1 *(a) Defined problem domain, (b) evenly distributed nodes in the problem domain, (c) interconnection of nodes with potential discontinuity lines (d) identified critical discontinuities resulting in the critical failure mechanism.*

Nodes are evenly spaced in a rectangular grid within solids, and along boundary lines (LSG, 2015). The higher amount of nodes used in the model, the more discontinuity lines are generated, providing a higher accuracy of the model. The number of slip lines created from the set of nodes, *n*, can be described as $n(n-1)/2$ and the number of possible slip line mechanism topologies as $2^{n(n-1)/}2$. The basic values of number of nodes set in the software are 250, 500, 1000 and 2000, referred to as coarse, medium, fine and very fine nodal density. Setting a fine nodal density in a certain domain generates a specific distance between each node. Enlarging the domain distributes the nodes more distant, thus generating a decrease in accuracy. The same nodal spacing in domains of different sizes may be achieved by altering the scale factor, editable using the "custom setting". Slip lines are not permitted to cross boundary objects, affecting the consistency of the provided solution. Considering this, boundaries are assigned a higher nodal density by default.

The accuracy of the numerical model depends on uncertainties and accuracy of the; theoretical model that is the basis of the numerical analysis and accuracy of the numerical method itself (LimitState:GEO, 2016). The theoretical model in the software is based upon the theory of soil plasticity which has been widely used in geotechnical design. The accuracy of the numerical model itself is based on several tests, where solutions from LSG have been compared against other known limit analysis solutions. In the validation of six different vertically loaded footings on two layered cohesive soil, the discrepancy from the benchmark solution varies between 1.59 to 4.78 percent.

3.2 Generic principles of modelling in LimitState:GEO

There are various types of projects that may be analysed in LSG including; vertically, laterally and eccentrically loaded footings, gravity, stem, gabion, and sheet pile walls, as well as pipelines and slope stability problems (LSG, 2015). In each geotechnical project fundamental settings concerning; geometry, materials, loads, scenarios and analysis must be set. Each of the listed categories is described in the section 3.2.1 to 3.2.5.

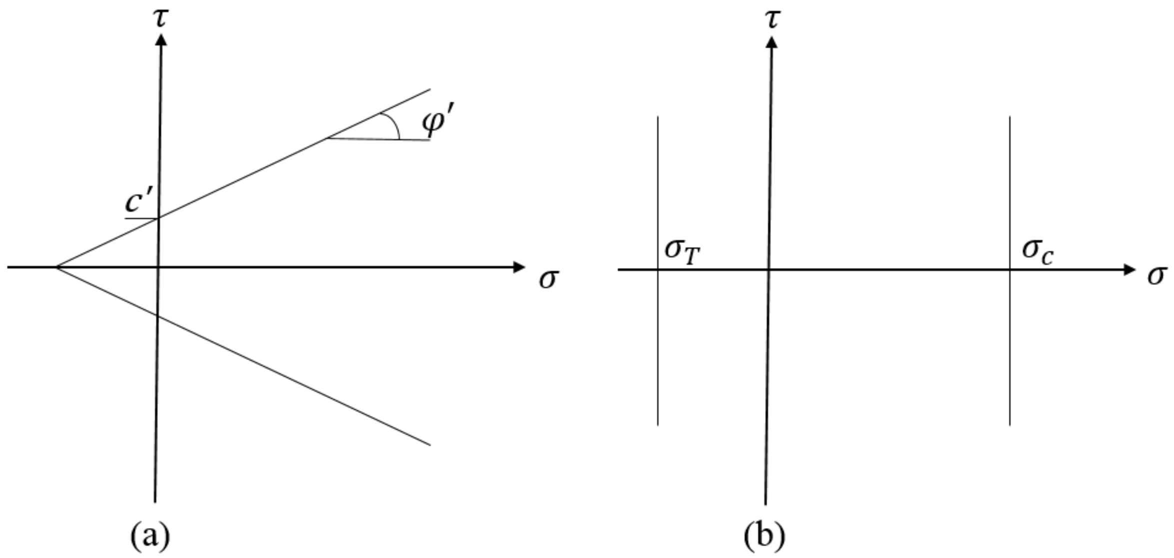
3.2.1 Geometry

Material included in the model is represented by solids, or boundaries generated around the solids (LSG, 2015). Solids are two-dimensional polygons, conventionally used for representing layer of soils. The extent of the model is specified by external boundaries, which in order to provide a correct failure mechanism must contain the collapse mechanism at all times. The support type of the external boundaries may be set to; free, fixed or symmetry. Free boundaries may be used for boundaries where displacement of soil is allowed in any direction. This setting is regularly used at the topmost horizontal boundary of a soil model. Fixed boundaries imply that only parallel displacements to the boundary are permitted. Setting the support type to symmetry may be employed for geotechnical problems associated with loading and geometry conditions truly symmetrical. The major advantage associated with symmetrically modelling is increase of computational efficiency.

3.2.2 Materials

Materials with different drainage behaviour, shear strength parameters and unit weight may be assigned to solids and boundaries in the model (LSG, 2015). To describe the stress-strain behaviour of a material in the model it is assigned with one, or a combination of material models, also called constitutive model. As the software exclusively deals with ULS, parameters required in the material models are solely those defining the yield surface. In the software there are three different constitutive models; Mohr Coulomb, Cutoff Material and Rigid. In addition, there is a special material model named Engineered Element, particularly used for modelling of geotechnical elements such as sheet pile walls and geotextiles.

The Mohr-Coulomb failure criterion is widely used in geotechnical design, and the fundamentals of it are described in section 2.2. The yield surface of a Mohr Coulomb material in LSG is depicted as a linear relationship between shear stress, τ, and normal stress, σ, and are defined in terms of effective friction angle, ϕ', and effective cohesion intercept, c', see Figure 3.1.



3.1 (a) Mohr Coulomb yield surface, (b) tension cutoff and crushing yield surface.

Modelling cohesive material with Mohr-Coulomb failure criterion may result in assigning the material an excessive tensile strength (LSG, 2015). Using the model Material Cutoff, a condition for at which normal stress-state the tensile failure occurs is set. This is certainly applicable for slope stability analysis, where tension cracks may be formed immediate to the top of the slope. Furthermore, the material model may be used to limit compressive stresses, usually to characterise crushing of material. The yield surface of the material model is depicted in Figure 3.1, where the two vertical lines intersecting the normal stress axis represents the limiting tensile stress, $\sigma_{T,}$ and a limiting compressive stress, σ_C. The Cutoff material model may be used solitary, or in combination with other material models like Mohr Coulomb.

The material model Rigid is used to model completely rigid material that is not a subject to failure, and may be applied to different geotechnical structures such as concrete retaining walls and footings (LSG, 2015). No properties but unit weight are specified for materials assigned with the rigid material model. The model cannot be used for materials assigned to boundaries but solely for material assigned to solids. Assigning an object this model implies that no nodes and consequently no slip lines will be generated in the material, resulting in higher solving efficiency.

The material model Engineered Element may be used to model geotechnical elements including; sheet pile walls, soil nails and geotextiles (LSG, 2015). The properties of an Engineered Element include material strength, such as rupture strength and plastic moment, as well as interaction with adjacent soil, represented by pull out and lateral factors. The generated material consists of a top and bottom interface, with the Engineered Element itself placed in between. The interfaces can be model by using any other material model such as Mohr Coulomb or Cutoff, and the Engineered Element itself is considered as rigid. However, at each vertex of the element it may break, bend or fail in compression.

In addition to the above mentioned material model a material may also be defined as a function of another material, referred to as a derived material. Generally, the inherent strength of the derived material is a portion of a parent material, obtained through the use of multipliers. By default, no material is assigned to boundaries. However, these may be assigned with any material, typically a derived material or the same material as in any adjacent solid. Furthermore, a frictionless soil material may be assigned to boundaries when modelling smooth interfaces.

3.2.3 Loads

In LSG loads can be either applied to the system, specified on boundaries, or arises from the self-weight of each component in the model (LSG, 2015). The applied loads may either be applied as vertical line loads [kN/m], typically used to represent loading on a structural element, or surcharge pressures [kN/m^2], representing distributed surface loads. However, lines loads are transformed into equivalent pressures over the width of the relevant boundary, thus all loads are in the software internally characterised as stresses. Loads may be set to vertical or inclined direction, and regardless of the failure mechanism the load is at all time fixed in the direction initially specified.

The use of LSG in accordance with Eurocode 7, hereinafter referred to as EC7, implies that loads shall be specified with respect to type of loading, and loading effect on the system (CEN, 2010). Each specification is associated with a partial factor. Favourable and unfavourable conditions represent whether a load or self-weight adds energy into the system or dissipates energy (LSG, 2015). The software automatically controls the specification of unfavourable and favourable conditions, and provides information whether the loads have been accurately specified. Furthermore, loads may be; Permanent, Variable or Accidental. Loads where the variation of magnitude is insignificant over time are considered to be Permanent, G, whereas loads varying in magnitude over time are defined as Variable, Q (CEN, 2006). Accidental loads are associated with instant loading of significant size.

3.2.4 Scenarios

The software is designed to correspond with different design codes, such as the partial factor approach used in ULS-design in EC7, see section 4.2.1. In LSG partial factors are used as multipliers on loads or as divisors material properties. Furthermore, a unity scenario comprising no partial factors on any property is available. The type of scenario providing the basis of the analysis is specified in the scenario manager. The manager obliges multiple scenarios to be treated in each analyse, providing separate adequacy factors in the output. For each scenario short term or long term analysis may be defined.

3.2.5 Analysis

The software provides a direct method proceeding straight to the collapse state. To adjust the load or strength parameters, and thereby drive the system to collapse, an adequacy factor is used (LSG, 2015). The adequacy factor, a, is given by Q_{ULS}/Q_d, where Q_{ULS} is the analytical collapse load and Q_d is the specified design load. The factor is displayed subsequently to the analysis. In LSG there are two approaches to make a system collapse; increase the existing load, or decrease the strength of the soil. In the software these approaches are named Factor Load(s) Analysis, and Factor Strength(s) Analysis respectively. The Factor Load(s) Analysis answers the question of “with what factor must the load be multiplied with, in order to drive the system to collapse?”. The Factor Strength(s) Analysis answers the question “with what factor must the strength of the soil be divided by, in order to drive the system to collapse?”.

Using the Factor Load(s) Analysis, loads in a system are increased until the model reaches the collapse state (LimitState:GEO, 2016). The increase may be restricted to either applied loads or to any self-weight of a material, or a combination of both. The adequacy property is to be set to true for the load or loads to be increased. In a multiple load system, it is possible to choose only to analyse the increase of one load, the magnitude of other loads is then unaltered. Subsequently to the analysis magnified soil displacement associated with the collapse are visualised, as well as bar charts displaying normal and shear stresses along discontinuity lines. Erratic stress distributions may be displayed in boundaries where the yielding criterion is not reached.

The DLO procedure generates solutions including translational mechanisms exclusively (LSG, 2015). The feature "model rotations" may be set to incorporate rotational mechanisms in the model. This implies that pre-defined solids like footings are allowed to rotate along the boundaries of the solid, incorporating rotational elements between every node, see Figure 3.2. Assuming the rigid solid is to rotate into a deformable body, such as underlying soil, the interface between these objects is modelled as translational motions rather than rotational motions, generating a transverse displacement of material. To increase the accuracy of the model, optimizing the nodal layout by distributing additional nodes to boundaries is a convenient measure. This approach is suitable for projects involving some degree of rotational failure.

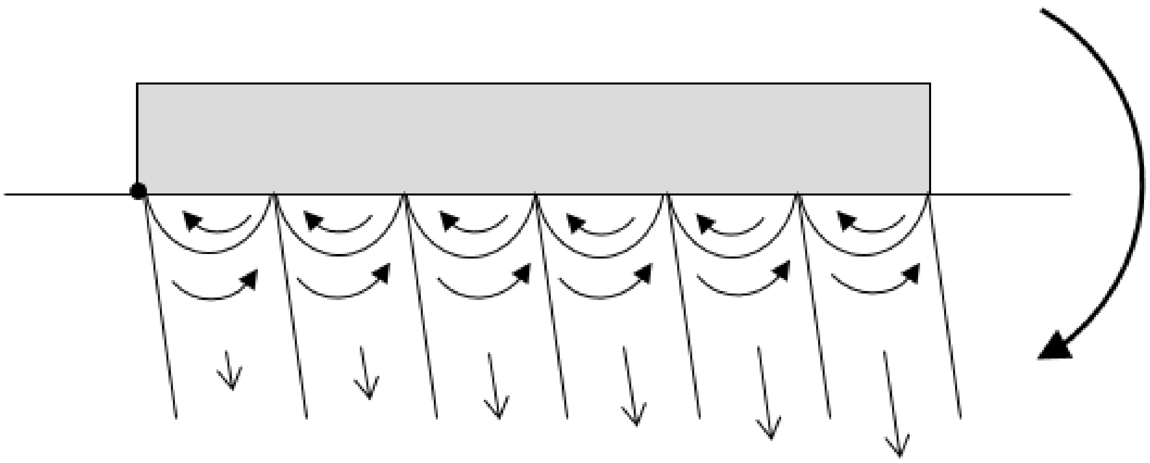


Figure 3.2 Rotation of a pre-defined footing with the setting model rotations (LSG, 2015).

In geotechnical problems involving significant rotational mechanisms, like eccentrically loaded footings, the rotations may be taken into account by delineation of a soil surface. The basis of the delineation approach is to incorporate additional lines in the model, situated around a rotating block below the foundation, see Figure 3.3. The lines permit material within the boundaries to rotate as firm blocks. By Limit State Ltd it is suggested to perform calibration test for geotechnical problems where significant rotational displacements are anticipated.

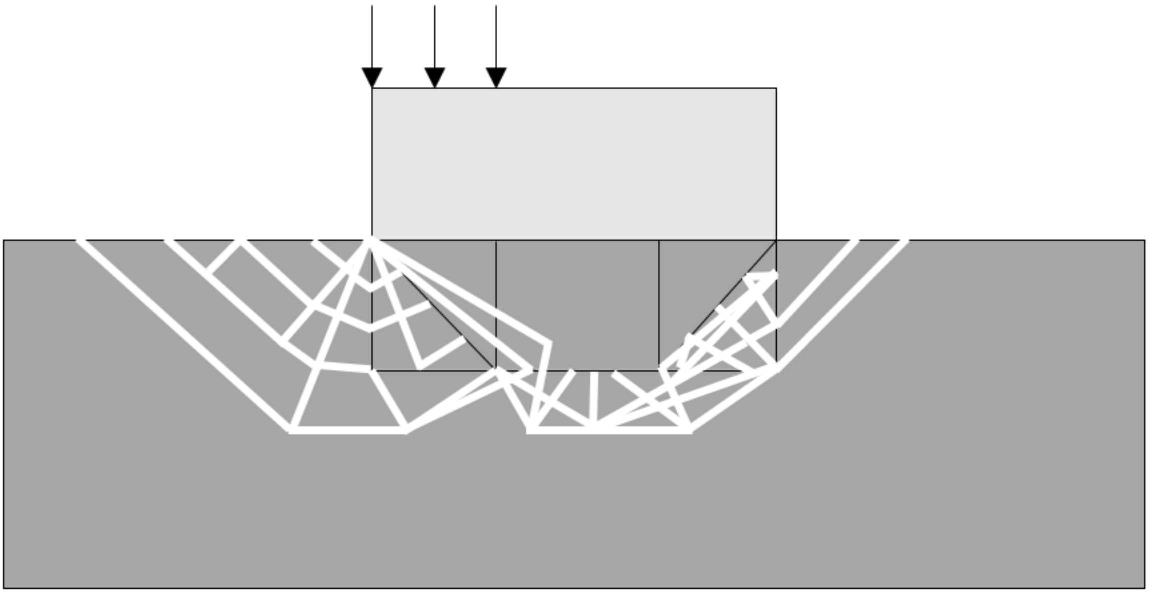


Figure 3.3 Delineation eccentrically loaded footing (LSG, 2015).

4 Working platform design

In Sweden there are no regulations or standards concerning the design of working platforms for pile driving rigs. Generally, experience or analytical bearing capacity calculations are used as a basis for the design. The lack of regulations may be prominent in the tender process, where contractors may restrain safety by accounting for no working platform or one of minimum extent[3]. There are several international associations working for safe design of working platforms, including the; European Federation of Foundation Contractors, British Federation of Piling Specialists, and Temporary Works forum. In 2004 the technical committee of the Federation of Piling Specialists, FPS, produced a guidance paper with the primary purpose of promoting safety in the design, construction and operation of ground supported working platforms (BRE, 2004). An outline of the guidance paper is found in section 4.4.

The work regarding design of safe working platforms are continuing, and in 2016 a guide regarding the overall design of working platforms is due (TWf, 2015b). A draft of the report was launched in 2015, and in addition to the guidelines regarding overall design, an outline regarding established analytical design methods, covering platform and foundation mechanisms are included (TWf, 2015a). Furthermore, the draft report includes a section about implementation of standards like EC7, in the design of working platforms. A review of design of working platform in accordance with EC7 can be found in 4.2. In the Netherlands guideline for the design of working platforms is under development by a committee under the institute called SBRCURnet in Delft[4].

4.1 Construction of a working platform for pile driving rigs

A working platform is generally considered as a temporary construction, thus it will not necessarily be a part of the final construction. As accidents occur it is essential to plan, design and maintain the working platform accurately. Skanska has developed a working process chart including description of six different activities conducted prior, and during use of any temporary construction (Skanska, 2013). Concerning working platforms, an additional category emphasising procedures to be carried out at the installation have been added by the authors, see Figure 4.1. The project manager at any of Skanska's construction project is to appoint a coordinator responsible for management, planning and monitoring of any temporary construction

Included in the planning process is a risk assessment, where the outcome is decisive for assigning the construction in one out of three safety classes, described in section 4.2.2. Furthermore, the engineer responsible for the design is to be determined. In the design process the maximum allowable loading conditions are to be established and certain ground conditions and platform material specified. Prior to the installation of the working platform blue prints are to be certified. In addition, a work description listing all stages of the installation, and use of the working platform including maintenance and monitoring, should be specified.

[3] Patrik Andersson District Manager Skanska Grundläggning, interview March 14, 2016.

[4] Henk de Koning, Nederlandse Vereniging Aannemers Funderingswerken, E-mail April 14

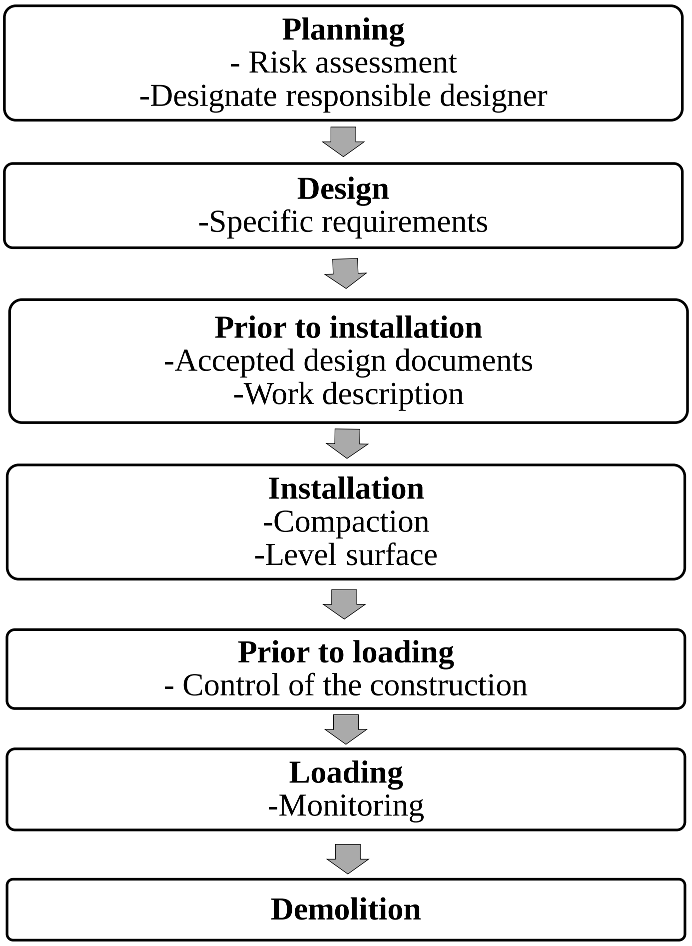


Figure 4.1 A process chart including activities that are carried out by Skanska for every granular working platform.

In the blue prints the type of granular material used in the working platform should be specified. The granular material is commonly crushed rock containing various fractions, generally0-90 mm or 0-150 mm[5]. If no measurements have been conducted on the material, empirical guideline values of the intrinsic material properties shall be used (Swedish Road Administration, 2013). These values are listed in TK GEO 13, a document produced by the Swedish Transport Administration, comprising technical requirements for design of geotechnical constructions for road and railways. The empirical value of the frictional angle and unit weight of subbase aggregates are 45 ° and 22kN/m^3 respectively. However, without extensive testing the material properties are not absolute and both frictional angle and unit weight may vary. Furthermore, terracing and levelling off the ground surface prior to installation of the granular material may require that existing ground materials are to be excavated. The granular material is normally placed upon a geotextile preventing the granular material to intrude into the underlying soil.

[5] Patrik Andersson District Manager Skanska Grundläggning, interview March 14, 2016.

At the installation of the working platform compaction and level surface are key factors. Compaction of the granular material is to be performed in accordance with the Swedish reference paper AMA 13, covering work and material descriptions for construction works. To the extent possible, the surface shall be even and level. Prior to the loading the execution of the working platform shall be assessed by the coordinator and/or responsible designer. During operation the driver shall experimentally control local bearing capacity of the ground by placing the mast foot on the ground and moving the centre of gravity of the rig forward. Unless the working platform can be efficiently included in the final construction, it shall be demolished after the piling.

One measure to reduce the track bearing pressure from a heavy machine is to transmit the load with the use of pads under outriggers, or timber mats (Plant Safety Group, 2014). For a pile driving rig timber mats may be used, typically placed tightly together and orthogonal to the moving direction of the rig (Skanska, 2015). The dimensions; length, width, height, varies among manufacturers but are generally around 6.0, 0.9, 0.2 meters respectively. The mats are not anticipated to be a subject to large or excessive deformations[6]. The mats are made out of hardwood logs, generally categorized in strength class C14, and are attached to each other with long bolts. Material in this category generally have the following properties; shear strength 3 MPa, density 3.1 kN/m^3, tension perpendicular to fibres 2 MPa, compression perpendicular to fibres 0.4 MPa (TräGuiden, 2003; VTT, 2005).

4.2 Design of working platform in accordance with Eurocode 7

In 1975 the European Commission introduced a program with the intention to develop European uniform structural design rules (European Commission, 2016). The work resulted in 10 different European Standards, EN 1990-1999, known as Eurocodes, each one focusing on different construction topics. The standards were completed between 2002 and 2007, and became obligatory in all design of supporting structures in Sweden between 2008 and 2011 (SIS, 2016). A unified European design approach for geotechnical structures are found in Eurocode 7, Swedish denomination SS-EN 1997, consisting of two different parts (European Commission, 2016). The first part ”General rules”, encompasses specification and requirements for analytical, empirical, numerical and observational design methods (CEN, 2006). The second part, “Ground investigation and testing”, includes specifications for planning and reporting of ground investigations as well as general requirements concerning; ground investigation reports, field, and laboratory testing procedures.

In EC7 there are undetermined parameters, these are subjected to national choices and displayed in national annexes. To facilitate the introduction of EC7 in Sweden the Swedish Implementation Commission for European Standards in Geotechnics, IEG, has published several application documents (IEG, 2008b). Design of geotechnical structures in accordance with EC7 with the use of computational analysis is generally carried out with the partial factor method. However, in numerical analysis unity for all partial factors is used and instead an overall factor of safety is applied. These two approaches are described in section 4.2.1 and section 4.2.2 respectively.

[6] Patrik Andersson District Manager Skanska Grundläggning, interview March 14, 2016.

4.2.1 Design with the partial factor approach

One of the suggested design methods in EC7 is the partial factor approach where partial factors are assigned to input parameters such as loads, referred to as actions, and/or soil strength parameters (CEN, 2010). By the use of partial factors, a design value is generated, and in the design it should be verified that the design value of the effect of actions, E_d, should be less or equal to the design resistance, R_d. This design procedure may be applied in the design of a working platform, and a review of the design procedure suggested in the Swedish Application document concerning overall stability of ground, can be seen in Figure 4.2 (IEG, 2008b).

Determination of relevant ULS. In EC7 there are five different ULS that, if relevant for the certain geotechnical project, should be accounted for in the design; loss of equilibrium of structure or ground, considering it as a rigid body, EQU, failure of a structure providing resistance, STR, failure of ground providing resistance, GEO, loss of equilibrium caused by uplift from water pressure, PPL, and hydraulic heave HYD.

Determination of design approach, DA. The use of partial factors in EC7 is governed by three different design approaches. DA1 includes two combinations, where partial factors are applied either to actions or to ground parameters. In DA2, partial factors are applied to actions and resistance. In DA3 the partial factors are applied to actions and ground parameters. In Sweden DA3 should be used for all geotechnical projects aside from piles where DA2 is used (Boverket, 2015)

Determination of safety class, SC. Geotechnical structures should be assigned with one of three safety classes, SC, defined in VVFS 2004:43. The classes represents the assessed risk of personal injury associated with the geotechnical project (Trafikverket, 2013). SC1 is associated with minor risk, SC2 some risk and SC3 major risk of personal injury. The SC governs the application of partial factor on loads, however material properties are unaffected.

Determination of dimensional actions, taking SC into consideration. Geotechnical projects may involve both geotechnical and structural actions. However, in overall stability analysis of ground, actions on soil are regarded as geotechnical (CEN, 2010). Define whether actions are permanent or variable, and if actions contribute, or aid to resist collapse, referred to as unfavourable and favourable actions respectively.

Determination of dimensional values for material properties. Material properties are generally obtained from geotechnical surveys, or alternatively from empirical relations, and to obtain dimensional values measured data needs to be converted (IEG, 2008b). The derived values, $\bar{X}$, from a survey are initially adjusted to eliminate biases, and from a compilation of the derived values a certain value should be chosen, referred to as the selected value. A characteristic value, X_k, is obtained by the use of a conversion factor, η, accounting for uncertainties related to ground properties and the certain construction. No conversion factor should be applied to empirically estimated values or soil layer thicknesses. The dimensional value, X_d, is obtained from an application of the partial factor to the characteristic value.

Figure 4.2 A review of the design procedure suggested in the Swedish Application Document concerning overall stability of ground.

4.2.2 Design with the unity approach

According to the Application Document for slopes and embankments, the unity approach with an overall factor of safety shall be used in numerical modelling (IEG, 2008b). It is also suggested that a thorough sensitivity analysis is carried out. To ensure overall stability of soils a safety factor in accordance with the Swedish report TK GEO 13 should be stated, see Table 4.1.

Table 4.1 Safety factors according to TK GEO 13

Safety Class	**Undrained analysis, F_C**	**Combined or drained analysis, $F_{C\phi}$**
1	1.35	1.20
2*	1.50	1.30
3	1.65	1.40

*Applicable values for geotechnical structures on clays with a sensitivity ratio less than 30.

4.3 Track bearing pressures for pile driving rigs

In the European Standard BS EN 16228-1:2014, the European Committee for Standardization, CEN, has specified safety requirements regarding design, construction, operation and maintenance along with hazard identification for drilling and foundation equipment (BSI, 2014). One of the identified hazards is loss of rig stability, potentially leading to overturning. Rig stability calculations shall ensure that the rig is stable for the most unfavourable combinations of operating and loading conditions, including maximum allowed slope of ground surface. The calculations are carried out for ideal ground conditions allowing no ground deformations or failure. However, rig instability may be caused without explicitly exceeding the inherent rig stability, typically when track bearing pressures exceeds the bearing capacity of the ground, leading to soil displacements. Thus, the different magnitude and distribution in ground pressure arising during pile operations must be taken into consideration in the design of working platforms.

As well as for rig stability calculations, estimation of ground pressures should take the most unfavourable combinations of operating and loading conditions into consideration (BSI, 2014). The basis of ground pressure calculations is to take the weight force of the various components of a rig, and eccentricity from the centre of rotation for each of the components, into consideration. From this a resultant load is obtained, and depending on loading and operating position the load may be rectangular or triangularly distributed over the tracks. The resultant load shall be distributed into a single load [N] for each track. Subsequently the ground pressure [N/m^2] is determined by taking into account the area over which the force acting on.

4.3.1 Track bearing pressures Junttan pile driving rig PM 23LC

Track bearing pressures are specific for each pile driving rig, and are only valid for certain operation and loading conditions. During piling operation, the centre of gravity for the rig alters as the components moves, generating different track bearing pressures. The track bearing pressures associated with the tracked pile driving rig PM 23LC, fabricated by Junttan, have by the manufacturer been estimated in accordance with the standard BS EN 16228-1:2014. The characteristics of the machine for a certain rig assembly, accounted for no pile, may be seen in *Table 4.2*

Table 4.2 Characteristic machine properties.

Machine attribute	**Property**
Machine	PM 23LC
Serial number	1260
Hammer	HHK 4HD
Hammer mass	7 300 kg
Counterweight	7 400 kg
Total mass	60 100 kg
Boom inclination	12.7 degrees
Leader inclination	18.43 degrees (back)
Mast assembly	6.0 + 6.6 + 5.0 + 6.0 m
Length track, l_t	4.9 m
Width track, w_t	0.9 m
Distance between tracks, d_t	4.7 m

In Figure 4.3 the magnitude and distribution of load on each track at three different operations can be seen. The magnitude in track bearing pressures obviously varies with mast orientation. The associated track bearing pressure may be seen in Table 4.3.

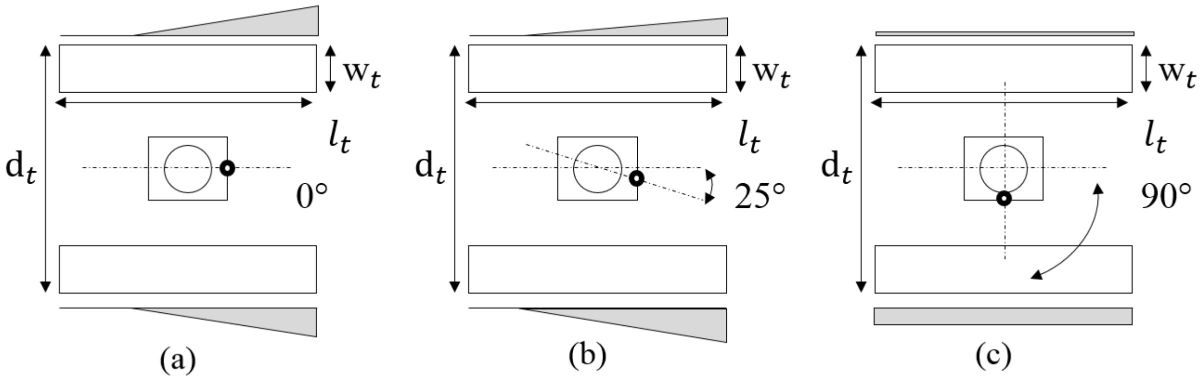


Figure 4.3 Magnitude and distribution of track bearing pressure, (a) no mast rotation, (b) 25 degrees mast rotation, (c) 90 degrees mast rotation.

Table 4.3 Associated track bearing pressures under right and left track for a mast rotation of 0, 25 and 90 degrees respectively.

Mast rotation [°]	**Track bearing pressure, right track [kPa]**	**Track bearing pressure, left track [kPa]**
0	186	186
25	110	215
90	23	110

4.4 Design of working platforms according to BR470

In 2004 the technical committee of the Federation of Piling Specialists, FPS, produced a guidance paper with the primary purpose of promoting safety in the design, construction and operation of ground supported working platforms (BRE, 2004). The FPS represents foundation contractors in the United Kingdom and the guide was published by the research consultancy Building Research Establishment, BRE, under the name “BR470 - Working platforms for tracked plant: good practice guide to the design, installation, maintenance and repair of ground-supported working platforms”. In the guide, both reinforced and un-reinforced granular platforms are covered and it is emphasised that the design must be conducted in accordance with the local site conditions. A thorough assessment of the site conditions is critical in obtaining a safe design, and weaker zones and variability of the ground conditions must be carefully appraised. It is stated that overturning of a rig is more likely to arise due to localized problems in the platform rather than general faults in the whole platform.

The guide proposes a design calculation treating a two-layer system, including a granular platform and a subgrade material (BRE, 2004). The subgrade may either be a cohesive, where the strength is characterised by an undrained shear strength, c_u, or a non-cohesive, where strength is characterised by the effective internal angle of shearing resistance, ϕ'. The basis of the analytical approach incorporates Meyerhof’s concept regarding punching shear resistance within the platform, a concept only valid when the platform material is stronger than the subgrade. For a cohesive subgrade the method is only valid for undrained shear strength between 20 and 80 kPa. When the thickness of the platform is larger than 1.5 times the width of the applied load, it is unlikely that the punching shear failure is critical, and hence another design approach is appropriate. The guide considers site conditions that are level, thus inclinations greater than 1:10 is not covered.

The design method proposed in BR470 does not fully follow the partial factor method described in EC7 (BRE, 2004). No partial factors as such are applied on soil strength parameters or loads. However, the loads are adjusted by certain loading factors corresponding to whether the rig operator is likely, or unlikely, to aid recovery of the rig from an impending platform failure. The latter case, referred to as “case 1” is associated with greater factors than the former case, “case 2”. The input value of track bearing pressures from a pile driving rig follows the standard BS EN 16228-1:2014. The FPS state that bearing pressures calculated in another way should not be used in BR470, as it potentially could lead to unsafe working platform design.

FPS conducted a study to investigate if the guide could be altered to include soils with undrained shear strength outside the initial range of 20-80 kPa (Miller, 2013). The study proposed that a correction factor should be applied to the bearing capacity equation. The correction factor in the modified method is applied to the coefficient of punching shear resistance, which is related to the ratio between the bearing capacity of the clay and the granular layer. In Appendix B the equations for the correction factor is presented.

5 Analysis

In this particular study five different analyses were conducted; comparison between numerical and analytical design methods, estimation of effect from timber mat, design of working platform in LSG covering both the partial factor and the unity approach, as well as analysis of an unsymmetrical loading condition. The specific setup in LSG used for the four latter analyses are described in section 5.2. The set material properties used in the analyses were concluded by the authors, see Table 5.1, and are not associated with a particular project. Material properties regarding shear strength of clay, $c_{u,clay}$, and thickness of dry crust, h_{DC}, were assumed to represent general soil conditions in Gothenburg. However, the shear strength of clay was set to constant with depth, thus the characteristic linear increase in shear strength of clay was not covered in the analyses. The set empirical values of internal friction angle, ϕ, and unit weight, γ_{WP}, of working platform material, as well as the shear strength of dry crust clay, $c_{u,DC}$, were specified in accordance with studied literature.

Table 5.1 Material properties used in the analyses.

$c_{u,clay}$ [kPa]	$c_{u,DC}$ [kPa]	h_{DC} [m]	ϕ [°]	γ_{WP} [kN/m^3]	$\gamma_{clay,DC}$ [kN/m^3]
12	30	1	45	22	15

The pile driving rig included in the analyses was a PM 23LC; a tracked machine fabricated by Junttan. The characteristics of the machine were previously specified in section 4.3.1. With the exception from the analysis covering unsymmetrical loading conditions, the track bearing pressures was fixed to 186 kPa. This track bearing pressures represent the pressure at the front edge of the tracks, thus no consideration was taken to the load distribution over each track. A visualisation of the applied model can be seen in Figure 5.1.

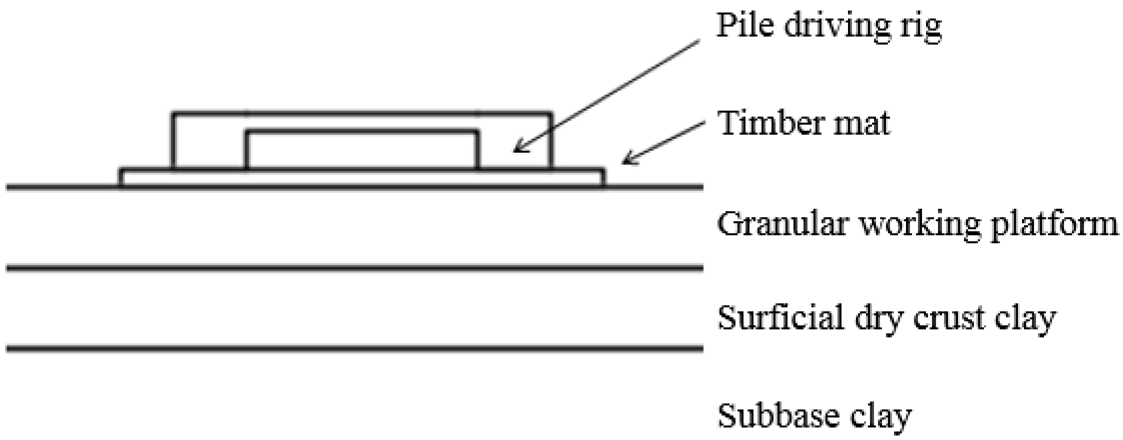


Figure 5.1 Model setup in LimitState:GEO.

5.1 Comparison between numerical and analytical methods

The required thickness of a working platform was analysed with the numerical software LSG, the established analytical approach proposed by Meyerhof, and the design calculation proposed in the report BR470. An extensive compilation of the equations used for working platform design in accordance with Meyerhof and BR470 can be found in Appendix A, B and C. As these analytical design methods are only valid for a two layered soil system, the surficial dry crust layer was excluded in the analyses. Two layers of soil were included in the analyses; working platform, and subbase clay.

The analysis in LSG was performed as a factor on load analysis and no partial factors were included. The geometrical objects defined in the software can be seen in Table 5.2. The nodal density very fine was used, implying that 2000 nodes were set as a target number in the problem domain. To obtain the required thickness of the working platform an iteration process was conducted until the adequacy factor on load was equal to 1.0. Neither of the two analytical methods fully covers the partial coefficient method, however, the BR-method incorporates a safety factor on the load.

Table 5.2 The geometrical objects and associated settings included in the software model.

Geometrical object	Setting
Width model	32 m
Height model	12.3 m
Width pile driving rig track	0.9 m
Width pile driving rig	4.7 m
Length pile driving rig track	3.525 m
Vertical boundaries	Fixed
Horizontal lower boundary	Fixed
Horizontal upper boundary	Free

Coefficients and factors that were used in the BR470 method are presented in Table 5.3. The material parameters in the analytical calculations are the same as presented in Table 5.1, however the shear strength of the clay was varied between 12, 25 and 30 kPa. The applied loading condition in BR470 was chosen as case 1, implying that the load was multiplied with a factor of 1.6. A load of 186 kPa was assumed to be distributed over the whole length of the pile driving rig's track.

Table 5.3 Coefficients and factors used in the analytical design method BR470.

Parameter	Input values
Punching shearing resistance coefficient	10
Punching shearing resistance coefficient factor	0.67
Load factor with a working platform, case 1	1.6

5.2 Model setup in LimitState:GEO

Prior to the analyses it was verified that the failure mechanism of any studied scenario did not tangent the set boundaries of the problem domain. The scenario resulting in the most widespread failure mechanism was set as a benchmark for the boundary setup, and to preclude uncertainties regarding the expanse of the failure mechanism, an oversized problem domain was set up. In the initial analyses the nodal density was set to “fine”, providing rapid analyses, however, to improve the accuracy of the model the nodal resolution “very fine” was used the final analyses. For the setup problem domain, a very fine resolution implies 2000 nodes and a scale factor of 0.81. To verify that the number of nodes was adequately set, the scale factor was changed to 1.5, generating 5791 nodes in the domain. This change altered the output adequacy factor with 1 percent, implying that 2000 nodes in the domain were adequately set and hence used in all analyses.

The setup geometry used in the analysis can be seen in Table 5.4. The pile driving rig was positioned symmetrically over a solid representing a timber mat of 6 meters, generating a distance from the edges of the timber mat to the outer edge of the track of 0.65 meters. Altering the thickness of the dry crust was done by manually replacing a specific distance of the subbase clay with dry crust. The external boundaries of the problem domain were specified to allow extrusion of soil at the boundary representing the ground surface, but to restrict extrusion in any other direction.

Table 5.4 Geometrical objects defined in the software and associated settings.

Width model	92 m
Height model	22.7 m
Width timber mat	6 m
Width pile driving rig track	0.9 m
Width pile driving rig	4.7 m
Vertical boundaries	Fixed
Horizontal lower boundary	Fixed
Horizontal upper boundary	Free

The constitutive material models assigned to solids and boundaries, as well as corresponding drainage behaviour can be seen in Table 5.5. The default setting where no materials are assigned to boundaries was retained. All soil material was assigned the Mohr Coulomb material model, and the tracks of the piling rig were modelled as rigid.

Table 5.5 The applied constitutive models, and drainage behaviour for each material used in the model.

Material	**Type**	**Drainage behaviour**
Clay	Mohr-Coulomb	Always undrained
Dry crust clay	Mohr-Coulomb	Always undrained
Granular material	Mohr-Coulomb	Always drained
Piling rig tracks	Rigid	-
Timber mat	Mohr-Coulomb/Cutoff	-

The timber mat was modelled with a combined yield surface, including the Mohr Coulomb and Cutoff material model. The software provides no explicit material model for timber material, by applying a combined material model a rough estimation of the real stress-strain relationship of wood material was provided. The timber mat was considered to be constructed of homogenous wood material, thus any other effect from the steel bolts but linking the timber logs together was disregarded. Prior to the final analysis a rigid material model was assigned to the solid representing the timber mat, providing a change in the adequacy factor on load with less than 1 percent. The load reduction induced by the timber mat was analysed by comparing the obtained adequacy factor on load for a model including no timber mat, with the adequacy factor on load obtained from a model including a timber mat. In these analyses four different platform thicknesses was covered; 0, 0.5, 1 and 1.5 meters.

Table 5.6 The range of values for timber mat strength and weight.

Unit weight [kN/m^3]	2.8	2.9	3.0	3.1	3.2	3.3	3.4	3.5	3.6
Limiting compression stress [kPa]	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4
Limiting tensile stress [kPa]	0.36	0.37	0.38	0.39	0.4	0.41	0.42	0.43	0.44
Shear strength [kPa]	1.7	1.85	2.00	2.15	2.3	2.45	2.60	2.75	2.90

5.3 Working platform design - partial factor approach

The review of the design procedure concerning overall stability and movements in ground, described in section 4.2.1, was used as a reference in the design of a working platform with the partial factor approach. The partial factor approach was in this particular report applied to one model, where material and loading parameters were fixed. Four different platform thicknesses was studied; 0.0, 0.5, 1.0 and 1.5 meters. Considering the output result from the partial factor analysis, an adequacy factor on load higher or equal to one, implies that design value of the effect of actions are less or equal to the design resistance, thus indicating a safe design. The result provided from the analyses was compared with that obtained from the unity approach analysis. And in order to obtain a result that directly could be compared to that obtained from the DA3 approach, the adequacy factor on load obtained from the unity approach was divided with the overall factor of safety 1.5.

The working platform was designed with respect to the ultimate limit state "GEO". This ULS was considered to be relevant for analysis of working platform as the state applies to geotechnical problems where the strength of soil has certain impact on providing resistance to the ground. Hence, partial factors were derived from tables in EC7 concerning this scenario. Design approach 3, DA3, was chosen for this particular design. DA3 applies to all geotechnical structures but piles in Sweden, consequently covers the design of a working platform. In DA3 partial factor are applied on both material parameters and actions. The working platform was categorised into SC2. This category was used as a working platform was considered to be associated with some risk of personal injury. The corresponding partial factor on load, γ_d, for SC2 is 0.91 (Swedish Road Administration, 2013).

In total stability analysis of ground, applied loads should be treated as geotechnical actions. Hence, determination of dimensional actions was carried out according to equation 5.1 (IEG, 2008b). In the equation permanent loads, G_{kj}, as well as variable loads, Q_{kj}, were multiplied with a partial factor considering the safety class, γ_d (Boverket, 2015). Furthermore, the partial factors, γ_G and γ_Q were applied to each load, taking favourable and unfavourable conditions into account. The load from the pile driving rig was considered to be a variable and unfavourable action, and the load from the working platform to be a permanent and unfavourable action. The corresponding partial factors, γ_G and γ_Q equal 1.10 and 1.4 respectively.

$$Geotechnical\ actions = \gamma_d * \gamma_G * G_{kj} + \gamma_d * \gamma_Q * Q_k \quad (5.1)$$

The determination of characteristic material parameters, X_k, was done in accordance with equation 5.2.

$$X_k = \eta * X_{selected} \quad (5.2)$$

The conversion factor, η, was calculated according to equation 5.3, and is applied to selected parameters to account for uncertainties related to ground properties and the certain construction. As conversion factors are not to be applied to empirically estimated values, a factor was applied solely on the shear strength of the subbase clay.

$$\eta = \eta_{1,2} * \eta_3 * \eta_{4,5,6,7} \quad (5.3)$$

Several estimations were done in order to obtain conversion factors to be applied on the shear strength of the subbase clay. It should be emphasised that the estimations were strictly hypothetical and no specific construction project was considered.

- The clay layer was assumed to be of the type “Standard Swedish Clay”
- The number of independent measurement points were estimated to three
- The number of different tests was estimated to be two to three, and the relative spread in the result was considered to be small.
- The failure surface was considered to be small and it was assumed that the shear strength along the slip surface was determined by a mean value. Furthermore, the distance from the survey point to the construction was estimated to be significant.

Table 5.7 Determined conversion factor, η_1- η_7, and resulting conversion factor η.

Soil parameter	**$\eta_{1,2}$**	**η_3**	**$\eta_{4,5,6,7}$**	**η-factor**
$C_{u,clay}$	0.95	1.0	0.95	0.9025

The determination of dimensional material parameters X_d, was done in accordance with equation 5.4. The partial factors for the soil parameters, γ_M, were derived from the national annexes and can be found in Table 5.8 (Boverket, 2015).

$$X_d = \frac{1}{\gamma_M} * X_k \quad (5.4)$$

Table 5.8 Partial factors assigned to soil parameters.

Soil Parameter	Symbol	Value
Angle of shearing resistance, tanϕ'	$\gamma_{\phi'}$	1.3
Effective cohesion, c'	$\gamma_{c'}$	1.3
Undrained shear strength, c_u	γ_{cu}	1.5
Weight density, γ	γ_{γ}	1.0

The input data and corresponding partial factors used in the analysis according to DA3 can be seen in Table 5.9. Partial factors for material and actions was defined in the scenario manager and automatically applied to relevant parameters, however, the conversion factor and the partial factor taking safety factor into account was applied to relevant parameter manually. Observe that only the shear strength of clay is associated with a conversion factor, η. Multiplying the partial factors γ_d and γ_G in equation 5.1, results in factor equal to one. Thus no resulting partial factor was applied to the permanent action resulting from self-weight of soil.

Table 5.9 Factors included in the partial factor approach DA3.

Factors	$C_{u,clay}$	$C_{u,DC}$	Φ	γ_{clay}	Q_k
γ_M	1.5	1.5	1.3	1.0	
η- factor	0.9025				
γ_d					0.91
γ_Q					1.4

5.4 Sensitivity analysis

The performed sensitivity analysis was carried out with the unity design approach as a basis, implying that no conversion or partial factors were assigned to any parameter. In the sensitivity analysis one input parameter at time was changed, keeping other parameters fixed. The considered range of values for each input parameter was defined in order to cover a wide range of input values, see Table 5.10. The shear strength of the dry crust clay was assumed to be higher than in the subbase clay, and as this parameter was analyses in steps of 5 kPa, no value lower than 15 kPa was assigned to this parameter. The unit weight of the clay and dry crust clay was set as 15 kN/m^3. However, altering this value did not affect the adequacy factor on load and was therefore fixed throughout the analysis.

Table 5.10 Range of values used in the sensitivity analysis.

$C_{u,clay}$ [kPa]	8	9	10	11	12	13	14	15	16
$C_{u,DC}$ [kPa]	*	15	20	25	30	35	40	45	50
h_{DC} [m]	0	0.25	0.5	0.75	1	1.25	1.5	1.75	2
ϕ [˚]	41	42	43	44	45	46	47	48	49
γ_{WP} [kN/m^3]	20	20.5	21	21.5	22	22.5	23	23.5	24

For each variation of input value, four different platform thicknesses were studied; 0.0, 0.5, 1.0 and 1.5 meters. For changes in internal friction angle and working platform unit weight, the scenario with no platform was excluded from the analysis. In accordance with the Swedish report TK GEO 13 the factor of safety was set to 1.5. Thus, considering the output result from the design with the unity approach, an adequacy factor on load higher or equal to 1.5 indicates a safe design.

5.5 Unsymmetrical loading conditions

In track bearing pressure calculations according to the standard BS EN 16228-1:2014, the most unfavourable combinations of operating and loading conditions of a plie driving rig are taken into consideration. However, these calculations are carried out for ideal ground conditions, allowing no deformations or failure. In real case scenarios ground conditions are not ideal and small deformations or surface irregularities in the working platform may result in a shift of centre of gravity, generating altered track bearing pressures. As deformations are omitted in LSG a hypothetical scenario was analysed, encountering the progressively shift of track bearing pressures from equal distribution over both tracks to all loading entirely on one track.

As well as in previous analyses track bearing pressures and pile driving rig dimensions applies to Junttan PM 23LC. The initial loading condition was set to 186 kPa, and the shift of the centre of gravity was manually altered by increasing and decreasing the track bearing pressures with 10 kPa respectively, thus generating unsymmetrical loading conditions. This was preceded until the track bearing pressure reached zero for one track, and 372 kPa for the other track. The adequacy on load was set to true for the load that was increased, and false for the load which was reduced. Thus, the output adequacy factor on load reflects with what factor one of the load has to be multiplied with in order for the system to reach collapse. The soil conditions that where applied are equal to the ones presented in Table 5.1 and a unity design approach was used. The setup geometry for the model is equal to that presented in section 5.2.

A rotational failure mechanism was anticipated for the unsymmetrical loading condition. This was incorporated in the model by delineation of rotating block below the foundation. Each loading condition is associated with a unique failure mechanism, implying that the delineation required manual adjustment after each change in load. This approach was deemed inefficient, and instead the lines were set wide enough to incorporate rotating block for all considered scenarios, see Figure 5.2.

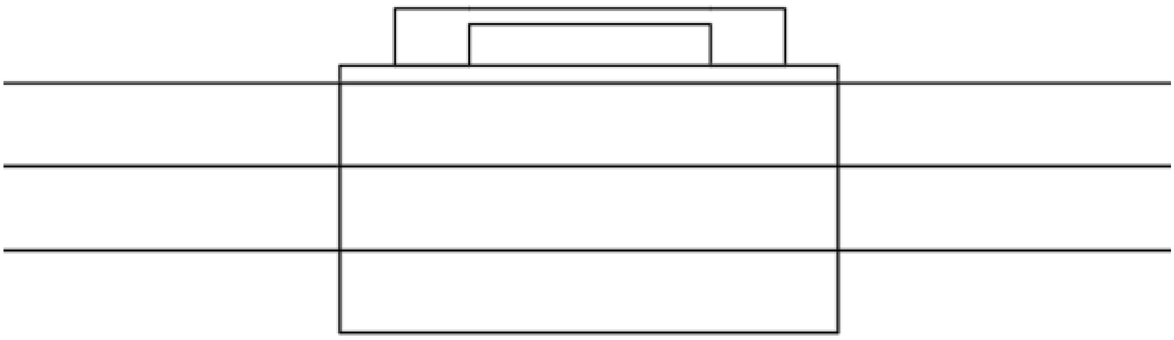


Figure 5.2 Model setup in LSG permitting a rotational failure mechanism by delineation.

6 Results

The results from the performed analyses are presented in text, tables and diagrams in section 6.1 to 6.5.

6.1 Comparison between numerical and analytical methods

The required thickness for a working platform according to LSG, Meyerhof and BR470, normal and modified scenario, are presented in Table 6.1. As BR470, normal scenario is only valid for shear strengths between 20 and 80 kPa, shear strength below 20 kPa was excluded from that analysis. BR470, modified scenario, resulted in a platform with greatest thickness for all analysed shear strengths. This is due to the applied punching shear coefficient factor, which reduces the bearing capacity. The Meyerhof method resulted in a less thick platform, compared to the other analytical methods. The numerical method, LSG, consistently resulted in the least thick platform. It should be emphasised that in the BR-method a safety factor is applied to the load, whereas no factors are applied in the analyses performed with Meyerhof and in LSG.

Table 6.1 Required working platform thicknesses according to the different methods.

Bearing capacity methods	**Working platform thickness [m] c_u=12 kPa**	**Working platform thickness [m] c_u=25 kPa**	**Working platform thickness [m] c_u=30 kPa**
BR470, normal scenario	-	0.73	0.66
BR470, modified scenario	1.23	0.92	0.82
Meyerhof, two layer	1.15	0.74	0.51
LSG, unity approach	0.89	0.37	0.1

6.2 Effect from timber mat

In all analyses it was observed that no discontinuity lines were encountered in the solid representing the timber mat, and thus no collapse of the material were considered. The adequacy factor on load with respect to analyses with and without timber mat can be seen in Table 6.2. From the results it is clear that the timber mat provides an effective load spreading measure, and the adequacy factor on load is at all times higher in those analyses containing timber mat.

Table 6.2 Adequacy factor on load associated with analyses with or without timber mat.

Thickness working platform [m]	**0.0**	**0.5**	**1.0**	**1.5**
Adequacy factor, no timber mat	0.811	1.159	1.586	1.894
Adequacy factor, timber mat	1.405	1.602	1.795	2.141

6.3 Working platform design - partial factor approach

The adequacy factor on load for the partial factor approach, DA3, as well as for the unity approach can be seen in Figure 6.1. The adequacy factor on load obtained from the unity approach exceeds the reference adequacy factor for platform thicknesses greater than 0.15 meters, whereas the adequacy factor on load obtained from DA 3 exceeds the reference adequacy factor for thicknesses greater than 1.45 meters. Observe that the adequacy factor on load obtained from the unity approach was divided by the overall factor of safety 1.5. Hence, this comparison is strictly dependent on the applied overall factor of safety. Applying an overall factor of safety of 2.1 would allocate the line representing the unity approach to about same magnitude as obtained from DA3. However, the line representing unity approach has a slightly steeper gradient than the line representing DA3. This implies that slightly higher values of adequacy factor on load may be obtained from the unity approach, as long as the assigned overall factor of safety is ≤ 2.1.

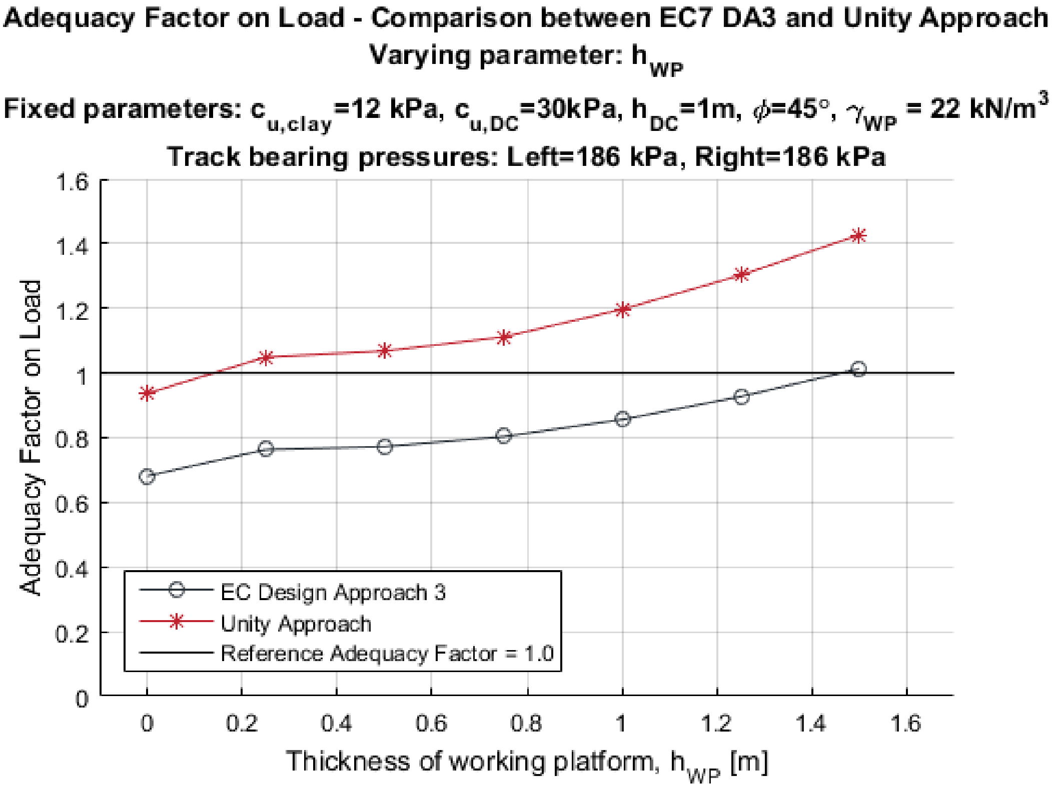


Figure 6.1 Comparison between the adequacy factors for the approaches EC7 DA3 and unity.

6.4 Sensitivity analysis

The relationship between the shear strength in the subbase clay and the adequacy factor on load for four different platform thicknesses are presented in Figure 6.2. Regardless of platform thickness, an increase in shear strength of subbase clay enhance the resisting forces of the soil, here shown as a linearly increase of adequacy factor on load. The diagram reflects that the shear resistance mobilised in the platform material increases with platform thickness. This behaviour is covered in all results obtained from the sensitivity analysis, and is clearly represented as the highest adequacy factor on load is at all times obtained for the thickest platform.

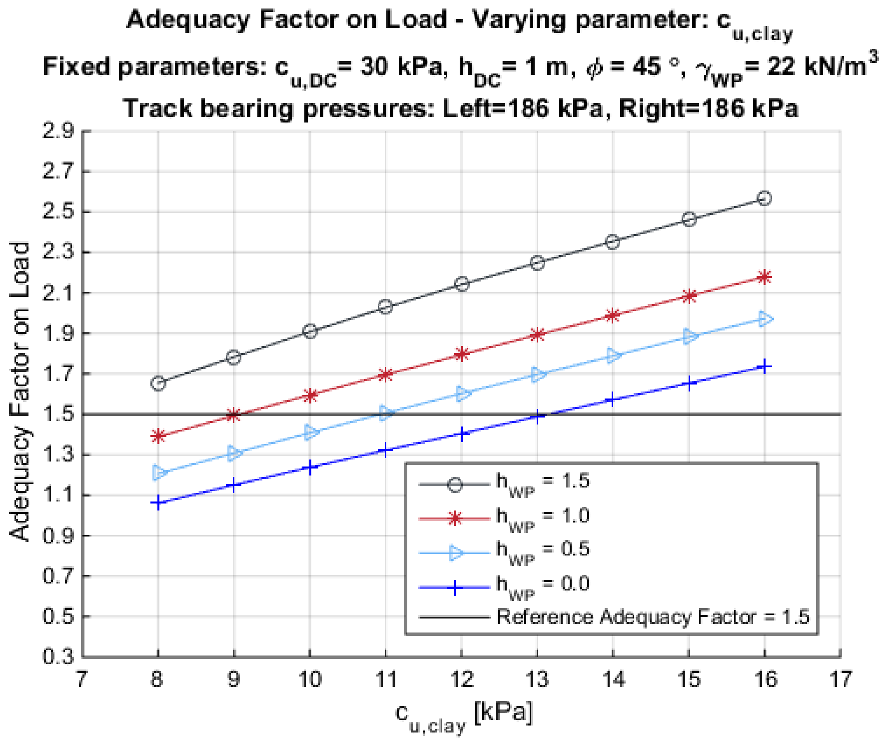


Figure 6.2 Adequacy factor on load with varying shear strength of clay

The relationship between the shear strength of dry crust clay and adequacy factor on load for four different platform thicknesses are presented in Figure 6.3. For all studied platform thicknesses a nearly linear increase of adequacy factor on load was obtained. For platform thicknesses up to 1.0 meters the increase of the curves diminishes slightly.

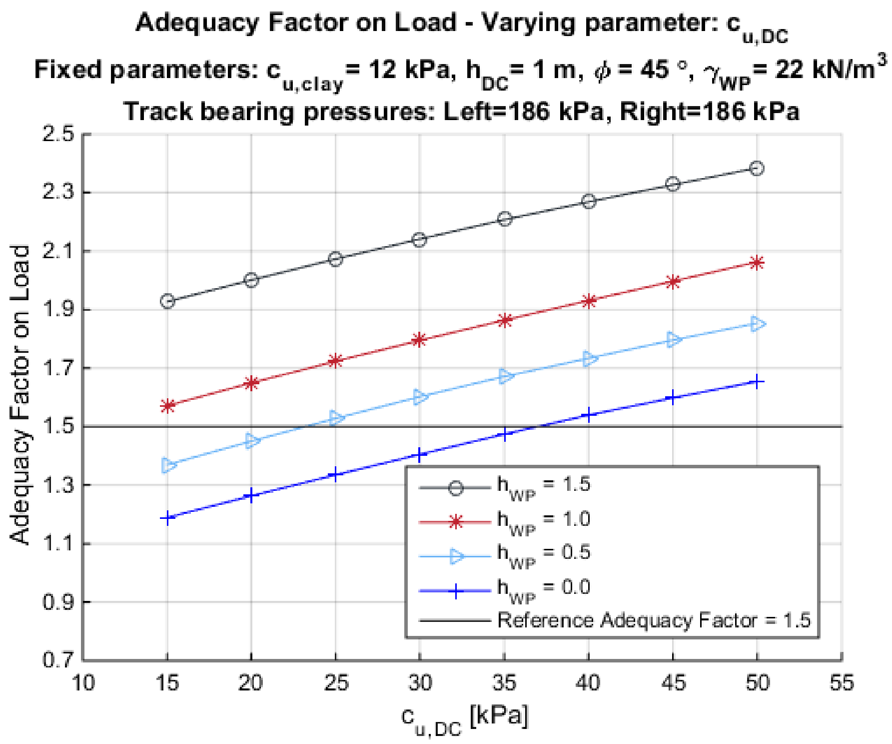


Figure 6.3 Adequacy factor on load with varying shear strength of dry crust clay.

The relationship between thickness of the surficial dry crust layer and the adequacy factor on load for four different platform thicknesses are presented in Figure 6.4. A prominent increase of adequacy factor on load is obtained when introducing a layer of dry crust in the model. Following increase in thickness of the dry crust layer generates a linear increase in adequacy factor on load.

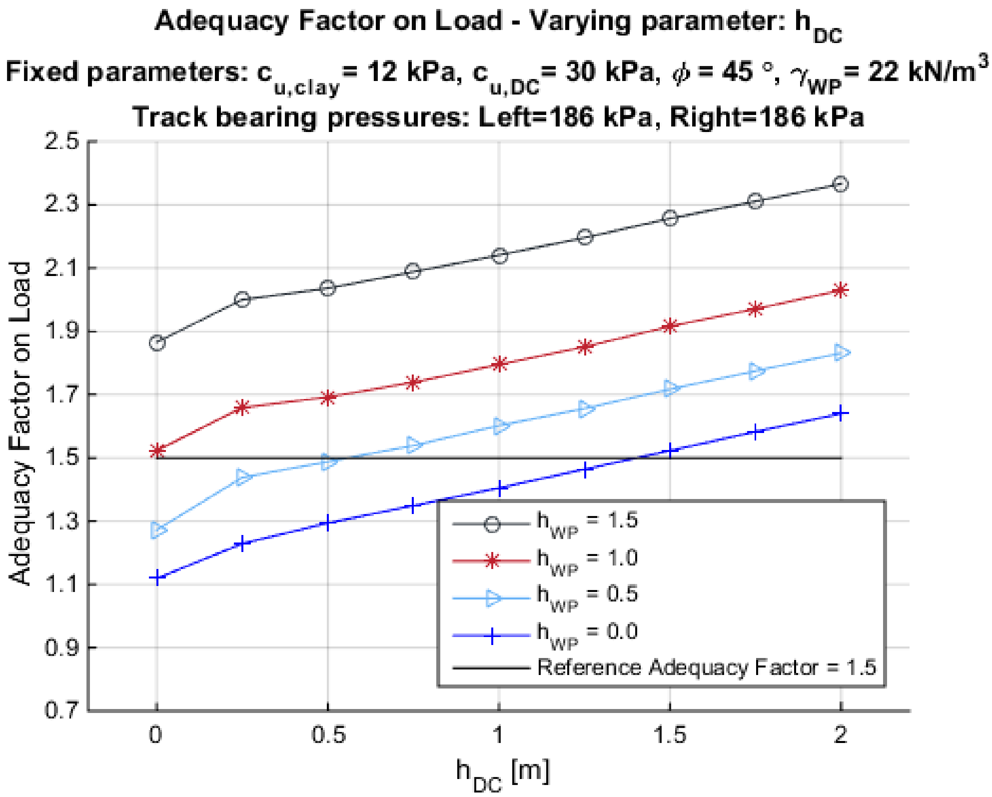


Figure 6.4 Adequacy factor on load with varying thickness of dry crust clay.

The relationship between the unit weight of the working platform and the adequacy factor on load for three different platform thicknesses are presented in Figure 6.5. For the platform thicknesses 0.5 meters, the unit weigh has an insignificant effect of the adequacy factor. For a platform thickness of 1.0 and 1.5 meters, an increase in unit weight from 20 to 24 kN/m^3 corresponds to an increase of the adequacy factor on load by 3.7 and 2.1 percent respectively. In the analyses, the unit weigh was increased for all material in the platform. However, from the result it can be seen that the resisting forces in the passive wedges are influenced to a greater extent than the active wedge, generating in an increase in adequacy factor on load.

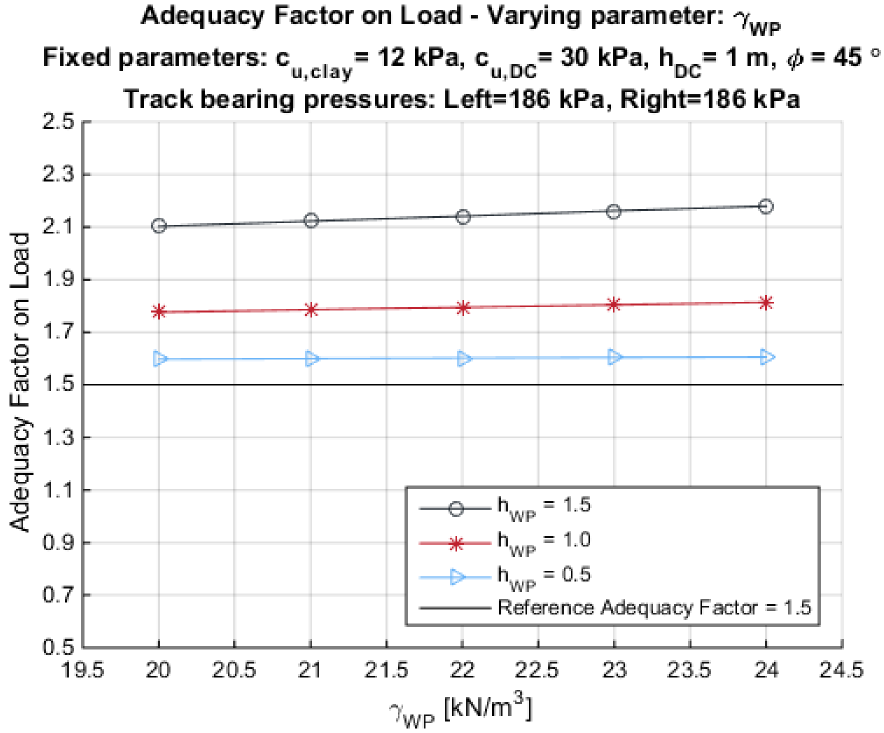


Figure 6.5 Adequacy factor on load with varying unit weight of platform material.

The relationship between the internal friction angel of platform material and the adequacy factor on load for three different platform thicknesses is presented in Figure 6.6. The varying inclination of the lines is a result from the additional shear strength that is mobilised in a material with a high internal frictional angle. For the platform of 1.5 meters the increase is notable, whereas the increase for a platform of 0.5 meters particularly small.

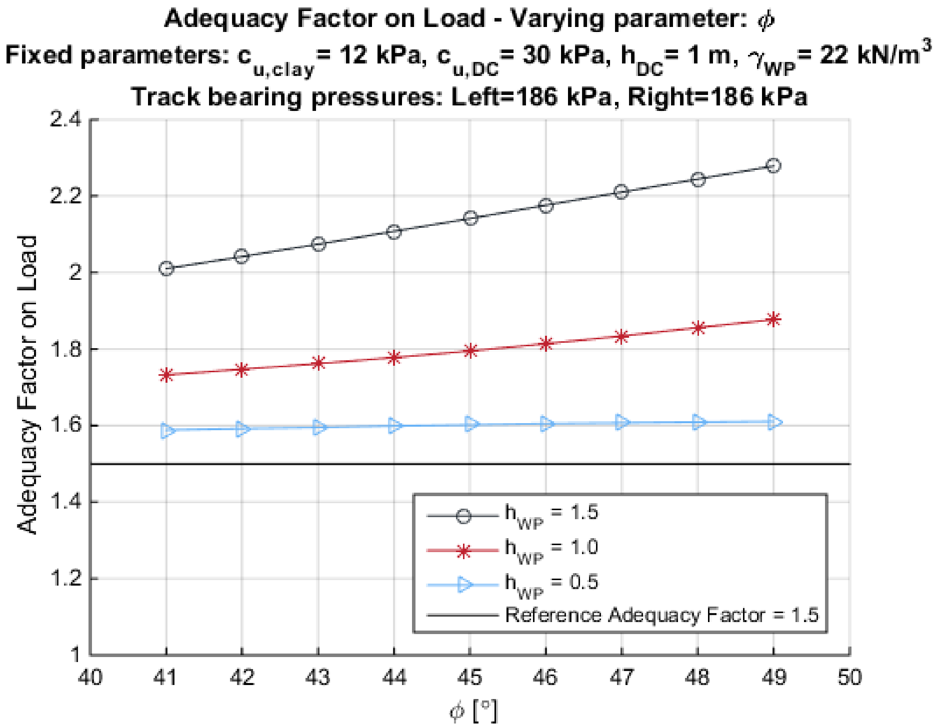


Figure 6.6 Adequacy factor on load with varying internal friction angle in platform material.

6.5 Unsymmetrical loading conditions

The result from the hypothetical scenario considering a progressive shift of track bearing pressures from equal distribution over both tracks, to load entirely on one track, are presented below. The unsymmetrically loading condition was studied both by permitting a rotational failure mechanism by delineation, see Figure 6.7, and by considering a translational failure mechanism solely, see Figure 6.8 . As can be seen there are significant differences in the result, and the analysis including delineation is associated with lower adequacy factor on the load for all platform thicknesses. It should be emphasized that the adequacy factor on load is restricted to only one load. Thus, compared to previous analyses, this setup provides an excessive adequacy factor on load at the initial loading of 186 kPa on each track.

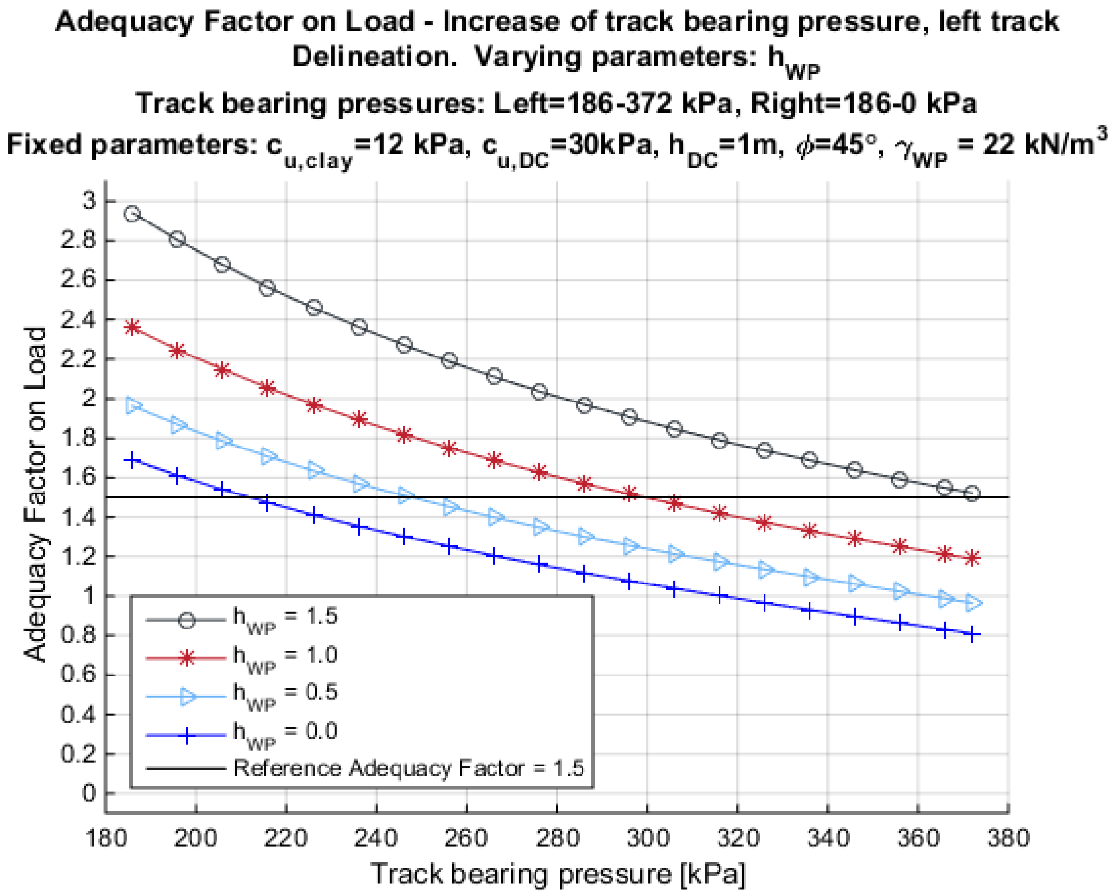


Figure 6.7 Adequacy factor on load for unsymmetrically loading condition, delineation.

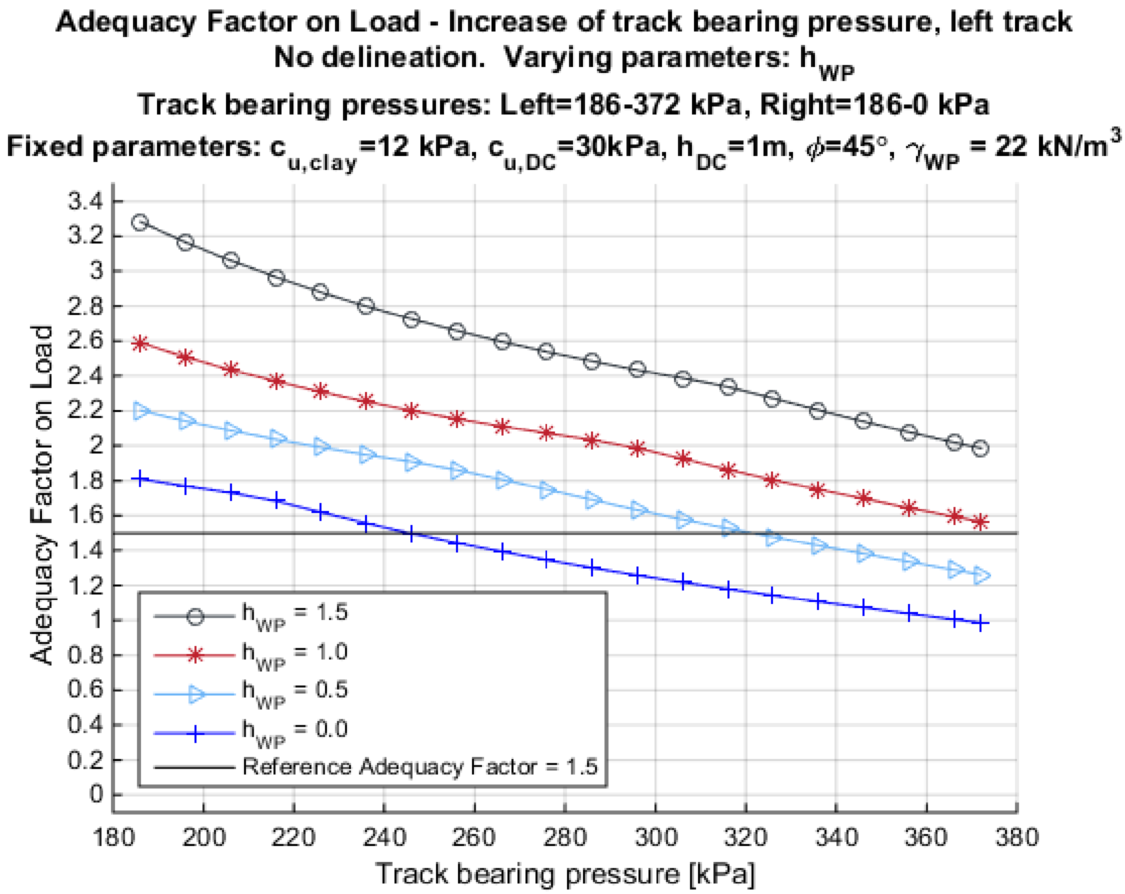


Figure 6.8 Adequacy factor on load for unsymmetrically loading condition, no delineation.

7 Discussion

The results obtained from the performed analyses with the analytical methods, and the numerical software LimitState:GEO are discussed in section 7.1 and 7.2 respectively. Furthermore, the influence of certain simplifications and assumptions are reviewed, as well as constraints of the applied model. In section 7.3 further studies related to the topic are suggested.

7.1 Analytical design

The results obtained from the two methods performed in accordance with BR470, clearly shows that the punching shear coefficient factor has a significant impact on the result. Also when the subgrade's shear strength is within the recommended values for the normal scenario, the required thickness for a platform for the two methods; normal and modified, differs considerably. Since the two methods from BR470 show a difference for the same shear strength of the subgrade an evaluation of the reliability and accuracy of the two methods would be desirable. Compared to the analytical methods, the numerical analysis in LSG resulted in a relative small required platform. This is likely due to the fact that no partial factors were considered on the load or material in this analysis, whereas in BR470 a factor is assigned to the load.

7.2 Analyses in LimitState:GEO

In all performed analyses an extensive problem domain was used. However, providing an adequately set nodal density, the size of the problem domain does not affect the accuracy of output result, though it may reduce the computational efficiency. From the performed sensitivity analysis it is clear that the material parameters with significant effect on the result is the shear strength of the subbase clay and dry crust clay, as well as the thickness of the dry crust layer. Thus, in real case scenarios these parameters should be assessed with great care. Furthermore, a more correct application of the model considering Gothenburg soil conditions would imply taking the increasing shear strength of clay into account. Linearly increasing shear strength would increase the resisting forces of the ground. However, as the obtained failure mechanisms are quite shallow, the additional mobilised shear strength will not affect the result considerably.

Assigning the timber mat with a combined, or rigid material model, had a minor effect on the adequacy factor on load. In consistency with real case scenarios, the result obtained from modelling the timber mat with a combined model, showed that no collapse was encountered in the timber mat. However, a more complex yield surface could be used to represent the stress-strain behaviour of the timber material more accurately, potentially improve the accuracy of the model to some extent. From the result covering adequacy factor on load with or without timber mat it is clear that the timber mat acts as a ground pressure reduction measure. Though, as a result of that no discontinuity lines were encountered inside the solid representing the timber mat, the post-solve function of bar charts displaying normal and shear stresses along discontinuity lines could not be used explicitly for analysis of the load distribution by the timber mat. As a consequence, the load spreading effects from a timber mat may not be consistent with the effects obtained in real case scenarios.

Only a portion of numerous of loading and operational combinations for a pile driving rig was covered in the analyses, and it should be emphasised that the results are strictly constrained to the adopted track bearing pressures. As a consequence of modelling in plane strain the triangular load distribution of track bearing pressures could not be accounted for. Instead, the pressures allocated at the edge of the tracks were used in the analysis. Although this simplification may not represent the track bearing pressures obtained in a real case scenario, it represents the conditions under which the ground is being subjected to the highest load generated from the pile driving rig in a specific operation condition.

Considering the unsymmetrical loading condition, there is no guarantee that the actual behaviour and track bearing pressures from a pile driving rig encountering irregularities in the ground have been reflected completely. The delineation was performed in a manner allowing rotations of solids in a wide range of soil. However, rotational failure is anticipated for eccentrically loaded footings, and the results obtained from the delineation analysis are more credible than the results obtained without delineation. Improved accuracy of the delineation analysis may be obtained through a more careful delineation setup, altering the additional lines prior to each increment of load.

In the analysis covering DA3, both the permanent and variable loads were considered to be unfavourable. From the initial analysis run by the software, the self-weight of the soil was considered to be accurately specified. However, the results from the sensitivity analysis displays that an increase of unit weight of the platform material generates a higher adequacy factor on load, thus the load from the self-weight might be seen as favourable. This scenario implicates that a lower partial factor should be assigned to the permanent load, and consequently returning a reduced load, theoretically generating an increase in adequacy factor on load. Though, the adoption of unfavourable or favourable load on the permanent load is considered to have a small impact on the obtained result.

7.3 Further studies

As this master thesis treats a limited amount of analytical and numerical methods, additional analyses with other bearing capacity methods are suggested. For instance, the limit equilibrium software Slope could be used in order to appoint the accuracy of the performed analyses. Furthermore, FE-modelling permitting deformations would potentially increase the accuracy of the modelled load spread from the timber mat. Furthermore, this study was not validated with any real case scenarios, accordingly it would be desirable to gather and study data associated with real overturning accident in more detail.

As there are no standards or regulations in Sweden regarding the design of working platforms, it would be of great interest to assess the implications of such guidance papers in terms of; health, cost and environmental aspects.

8 Conclusions

Based on the performed analyses it is clear that the resisting forces in the ground and the mobilised shear strength in the platform material determine the platform thickness. Furthermore, it is clear that the timber mat has a major effect on the bearing capacity of the ground, though the load spread function should be studied more in detail. The design method purposed in Eurocode 7 may be easily incorporated in the numerical modeling. The result from the unsymmetrical loading condition was shown to be strictly dependent on the software feature delineation, permitting rotational failure in the soil. In order to apply this particular numerical model in the design of a granular working platform, the obtained results must be verified to, and compared with actual measurements and more advanced models.

9 References

ASTM International, 2011. *ASTM D2487-11, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System),* West Conshohocken: ASTM International.

Aysen, A., 2002. *Soil Mechanics Basic Concepts and Engineering Applications.* Lisse: Swets & Zeitlinger.

Boverket, 2015. *Boverkets föreskrifter om ändring i verkets föreskrifter och allmänna råd (2011:10) om tillämpning av europeiska konstruktionsstandarder (eurokoder);* , Stockholm: Boverket.

BRE, 2004. *BRE470 - Working platforms for tracked plant: good practice guide to the design, installation, maintenance and repair of ground-supported working platforms,* Garston, Watford: BRE Bookshop.

BSI, 2014. *BS EN 16228-1:2014 - Drilling and foundation equipment. Safety. Common requirements,* London: Brittish Standard Institution Standards Limited.

Caldenius, C., 1925. *Contribution to the knowledge of the relationship between the make-up of ground and its bearing capacity as indicated by strength determinations and embankment loadings,* s.l.: Ing.-vetenskapsakad..

CEN, 2006. *Eurocode 7 - Geotechnical design - Part 2: Ground investigation And Testing,* Brussels: CEN.

CEN, 2010. *Svensk Standard SS-EN 1997-1:2005,* Stockholm: Swedish Standards Institue.

Chen, W.-F., 1975. *Limit analysis and soil plasticity - Developments in geotechnical engineering.* 7th ed. Oxford: Elsevier scientific publishing company.

Chen, W. F. & Davidson, H. L., 1972. *Bearing Capacity Determination By Limit Analysis,* Bethlehem: s.n.

Dahlgren, F. & Nyman, V., 2015. *Working platform on soft clay,* Gothenburg: Chalmers University of Technology.

Das, B., 2011. *Principles of Foundation Engineering.* 7th ed. Stamford: Cengage Learning.

Das, B. M., 2010. *Principles of Geotechnical Engineering.* Stamford: Cengage Learning.

De Jong, G., 1959. *Statics and kinematics in the failable zone of a granular material,* Delft: Technical University of Delft.

European Commission, 2016. *European Commission; Eurocodes; Building the future.*
[Online]
Available at: http://eurocodes.jrc.ec.europa.eu/home.php
[Accessed 14 April 2016].

Flodin, N. & Broms, B., 1981. *Historical Development of Civil Engineering in Soft Clay,* Stockholm: Royal Institute of Technology.

Helenelund, K., 1953. *Stability and failure of subsoil, with special reference to railway embankments in Finland,* s.l.: State Inst. Tech. Res., Publ. (No. 24).

Hill, R., 1951. On the state of stress in a plastic-rigid body at the yield point. *Philosophical Magazine,* Volume 42, pp. 868-875.

IEG, 2008b. *Tillämpningsdokument Kapitel 11 och 12, Slänter och bankar,* Stockholm: Svenska Geotekniska Föreningen.

Junttan, 2016. *Products; Pile driving rigs.* [Online]
Available at: http://www.junttan.com/products/pile-driving-rigs/
[Accessed 22 April 2016].

Kenney, T. C., 1975. *Weathering and changes in strength as related to landslides,* Ontario: Guelph Symposium on Geomorphology.

Knappett, J. & Craig, R., 2012. *Craig's Soil Mechanics.* 8th ed. Abingdon: Spon Press.

Kötter, F., 1903. *Die Bestimmung des Drucks an gekrümmten Gleitflächen, eine Aufgabe aus der Lehre vom Erddruck,* Berlin: Sitzungsberichte der Akademie der Wissenschaften.

LimitState:GEO, 2016. *Limit State GEO; Validation.* [Online]
Available at: http://www.limitstate.com/geo/validation
[Accessed 14 April 2016].

LSG, 2015. *LimitState:GEO Manual, Version 3.2.d,* Sheffield: LimitState Ltd.

Lutenegger, A. J., 1995. *Getechnical Behaviour of Overconsolidated Surficial Clay Crusts,* Washington , D.C.: Transportation Research Board.

Lyamin, A., Salgado, R., Sloan, S. & Prezzi, M., 2007. Two- and three dimensional bearing capacity of footings in sand. *Geotechnique,* 57(8), pp. 647-662.

Makrodimopoulos, M. & Martin, C. M., 2007. Upper bound limit analysis using simplex strain elements and second-order cone programming. *International journal for numerical and analytical methods in geomechanics,* 31(6), pp. 835-865.

Meyerhof, G., 1974. Ultimate bearing capacity of footings on sand overlying clay. *Canadian Geotechnical Journal,* Volume 11, pp. 223-229.

Meyerhof, G. & Hanna, A., 1978. Ultimate bearing capacity of foundations on layered soils under inclined load. *Canadian Geotechnical Journal,* Issue 15, pp. 565-572.

Meyerhof, G. & Hanna, A., 1979. Ultimate bearing capacity if foundations on a three layer soil. *Canadian Geotechnical Journal,* Issue 16, pp. 412-414.

Meyerhof, G. & Hanna, A., 1980. Design charts for ultimate bearing capacity of foundations on sand overlaying soft clay. *Canadian Geotechnical Journal,* Volume 17, pp. 300-303.

Miller, K. S., 2013. *Technical Note on Use of BR470 in Soft Clay.* [Online]
Available at:
http://www.fps.org.uk/fps/guidance/technical/otherguidance/Technical_Note_on_Use_of_BR470_in_Soft_Clay.pdf
[Accessed 06 04 2016].

Plant Safety Group, 2014. *Ground Conditions for Construction Plant* , London: Construction Plant-hire Association.

Prandtl, L., 1920. Über die Härte plastischer Körper. *Nachrichten von der Gesellschaft der Wissenschaften zu Göttingen, Mathematisch-Physikalische Klasse,* pp. 74-85.

Prandtl, L., 1921. Über die Eindringungsfestigkeit plastischer Baustoffe und die Festigkeit im Schneiden. *Math, Mech,* 1(1).

Raymond, G., 1972. Prediction of Undrained Deformations and Pore Pressures in Weak Clay Under Two Embankments. *Geotechnique,* 22(3), pp. 381-401.

Ringsten, B., 1984. *Dry crust - the Formation and Geotechnical Properties,* Gothenburg: Chalmers University of Technology.

SGI, 2000. *Skredriskanalys i Göta älvdalen - metodbeskrivning,* Linköping: Swedish Geotechnical Institute.

SGI, 2007. *Skjuvhållfasthet -utvärdering i kohesionsjord,* Linköping: Swedish Geotechnical Institute.

SGI, 2008. *Jords egenskaper,* Linköping: Statens geotekniska institut (SGI).

SGI, 2016. *SGI; jords hållfasthet; lera och kvicklera.* [Online]
Available at: http://www.swedgeo.se/sv/kunskapscentrum/om-geoteknik-och-miljogeoteknik/geoteknik-och-markmiljo/jords-hallfasthet/lera-och-kvicklera/
[Accessed 26 April 2016].

SIS, 2016. *Swedish Standard Institute; Eurokoder.* [Online]
Available at: http://www.sis.se/eurokoder
[Accessed 14 April 2016].

Skanska, 2013. *Temporära konstruktioner.* [Online]
Available at: OneSkanska.se
[Accessed 29 April 2016].

Skanska, 2015. *Granskningshandling pålkransbädd Kviberg 22:14,* Göteborg: Skanska Teknik.

Skanska, 2016. *Skanska; Om Skanska; Vårt samhällsansvar; Starka värderingar.* [Online]
Available at: http://www.skanska.se/sv/om-skanska/prioriterade-omraden/hallbarhet/vart-samhallsansvar/vara-varderingar/
[Accessed 15 April 2016].

Smith, C. & Gilbert, M., 2007. Application of discontinuity layout optimization to plane plasticity problems. *The Royal Society,* 463(2086), pp. 2461-2484.

Sokolovski, V., 1965. *Statics of Granular Materials,* Oxford: Pergamon Press.

Swedish Road Administration, 2013. *Trafikverkets tekniska krav för geokonstruktioner TK Geo 13,* Stockholm: Trafikverket.

Terzaghi, K., 1943. *Theoretical Soil Mechanics.* London: John Wiley and Sons, inc..

Terzaghi, K. & Peck, R. B., 1948. *Soil Mechanics in Engineering Practice.* 3rd ed. New York: John Wiley & Sons, Inc..

TräGuiden, 2003. *Konstruktionsvirke.* [Online]
Available at: http://www.traguiden.se/om-tra/materialet-tra/trabaserade-produkter/virkestyper-och-kvalitet1/konstruktionsvirke/
[Accessed 30 05 2016].

TWf, 2015a. *(Draft) Working Platforms - Design of granular working platforms for construction plant, a guide to good practice,* London: Temporary Works Forum.

TWf, 2015b. *TWf; News; Granular Working Platforms.* [Online]
Available at: https://www.twforum.org.uk/news/granular-working-platforms/
[Accessed 29 May 2016].

VTT, 2005. *Potential of strength grading of timber with combined measurements techniques,* Esbo: VTT.

A Meyerhof ultimate bearing capacity calculation

The equation for the ultimate bearing capacity, q_u, of a stronger soil, denominated 1, overlaying a weaker soil, denominated 2, is presented in equation A1.

$$q_u = \left(1 + 0{,}2\frac{B}{L}\right)5.14c_2 + \gamma_1 H^2\left(1 + \frac{B}{L}\right)\left(1 + \frac{2D_f}{H}\right)\frac{K_s tan\phi_1'}{B} + \gamma_1 D_f \leq \gamma_1 D_f N_{q(1)} F_{qs(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)} F_{\gamma s(1)}\ [kPa] \quad \text{(A.1)}$$

$F_{cs}, F_{qs}\ and\ F_{\gamma s}$ are shape factors relating to the equivalent layer. The equations for the shape factors are presented in equations A2 to A4.

$$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right) \quad \text{(A.2)}$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right) tan\emptyset' \quad \text{(A.3)}$$

$$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right) \quad \text{(A.4)}$$

K_s is the punching shear factor which is a function of q_2/q_1 and ϕ_1'. The ultimate bearing capacities $q_1$and q_2 are calculated with equation A.5 and A.6.

$$q_1 = \frac{1}{2}\gamma_1 B N_{\gamma(1)}\ [kPa] \quad \text{(A.5)}$$

$$q_2 = c_2' N_{c(2)}\ [kPa] \quad \text{(A.6)}$$

The punching shear coefficient is extracted from Figure A.1

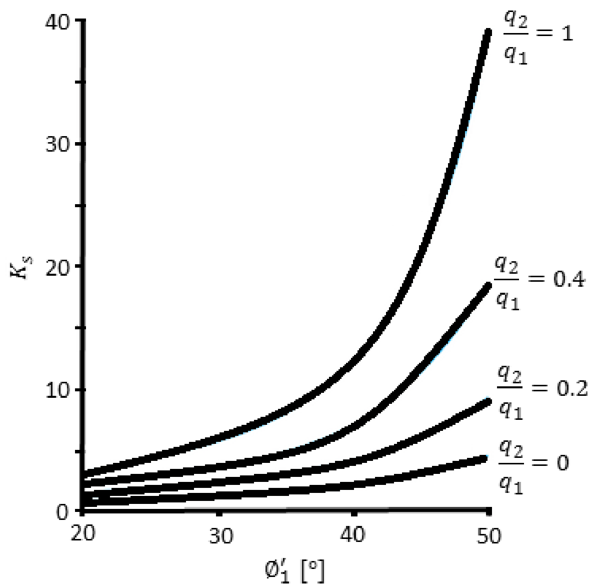


Figure A.1 Diagram for punching shear coefficient (Das, 2010).

N_γ, N_q and N_c are bearing capacity factors depending on the friction angle in the corresponding soil layer. Values of some of the factors for the corresponding friction angles are presented in the Table A.1.

Table A.1 Bearing capacity factors for different friction angles (Das, 2010).

ϕ’	$\mathbf{N_c}$	$\mathbf{N_q}$	$\mathbf{N_\gamma}$
25	20.72	10.66	10.88
30	30.14	18.4	22.4
35	46.12	33.3	48.03
40	75.31	64.2	109.41
45	133.88	134.88	271.76
50	266.89	319.07	762.89

B BR470 required platform calculation

The method for calculating the required platform thickness, proposed by FPS is presented in this Appendix. A full description of the method can be seen in the guide proposed by FPS (BRE, 2004). In the modified scenario, an additional punching shear coefficient factor is presented. This factor should be used in order to allow shear strength of the cohesive sub-grade outside the initial range of 20-80 kPa.

First the subgrade without a working platform is analysed. This is done in order to know if its bearing capacity is sufficient to support the load. The bearing resistance ($R_{d,clay}$) of the subgrade without a working platform at place are calculated with equation B.1.

$$R_{d,clay} = c_u N_c s_c \; [kPa] \quad \text{(B.1)}$$

Depending on the type of loading condition that applies the design load are calculated according to equation B2 and B3.

$$q_{1d} = 2.0 q_{1k} \; [kPa] \qquad \text{(B.2)}$$

$$q_{2d} = 1.5 q_{2k} [kPa] \qquad \text{(B.3)}$$

If the calculated design load is less than the bearing resistance, there is no need for a working platform. Otherwise the material in the working platform needs to have sufficient bearing resistance to support the load. The working platform's bearing resistance should be greater than the subgrade, and it is calculated with equation B4 and B5.

$$R_{d,WP} = 0.5 \gamma_p W_d N_{\gamma p} s_\gamma [kPa] \quad \text{(B.4)}$$

$$0.5 \gamma_p W_d N_{\gamma p} s_\gamma > c_{ud} N_c s_c \; [kPa] \qquad \text{(B.5)}$$

If a working platform is needed the design load for the different cases should be altered accordingly to equation B6 and B7.

$$q_{1d} = 1.6 q_{1k} \; [kPa] \qquad \text{(B.6)}$$

$$q_{2d} = 1.2 q_{2k} [kPa] \qquad \text{(B.7)}$$

The bearing resistance of the working platform material should be greater than the design load with a working platform q_{1d} and q_{2d}, according to equation B8 and B9.

$$q_{1d} < R_d \; [kPa] \quad \text{(B.8)}$$

$$q_{2d} < R_d \; [kPa] \quad \text{(B.9)}$$

If the working platform fails to achieve these requirements another platform material needs to be selected in order to have a working platform that could withstand the applied load. The punching shear coefficient ($K_p tan\delta$) are extracted the same way as in Appendix A. The thickness of the platform is calculated with the equation B10 and B11.

$$D_1 = \left\{ \frac{W_d (q_{1d} - c_{ud} N_c s_{c1})}{\gamma_p K_s tan\delta s_{p1}} \right\}^{0.5} [m] \qquad \text{(B.10)}$$

$$D_2 = \left\{ \frac{W_d (q_{2d} - c_{ud} N_c s_{c2})}{\gamma_p K_s tan\delta s_{p2}} \right\}^{0.5} [m] \qquad \text{(B.11)}$$

The punching shear coefficient factor (PSCF) is calculated with equation B12. In order to get a bearing capacity for the modified scenario same procedure as in the normal scenario occurs except factor should be multiplied to the punching shear coefficient ($K_s tan\delta$).

$$PSCF = 0.1704 * \ln\left(\frac{q_{ult,clay}}{q_{ult,WP}}\right) + 1.2021 \qquad \text{(B.12)}$$

$$q_{ult,clay} = c_u N_c s_c \ [kPa] \qquad \text{(B.13)}$$

$$q_{ult,WP} = 0.5\gamma_p W_d N_{\gamma p} s_\gamma \ [kPa] \qquad \text{(B.14)}$$

C Meyerhof required platform calculation

The required thickness for the working platform according to the method proposed by Meyerhof is derived from the equations in this appendix.

When the working platform (top layer) is stronger than the saturated clay layer (bottom layer) below, see Figure 2.1, the equation for the ultimate bearing capacity is calculated according to equation C.1. From the equation the required working platform thickness (H) can be assessed for a load if the other parameters are known.

$$q_u = \left(1 + 0.2\frac{B}{L}\right)5.14c_u + \gamma H^2\left(1 + \frac{B}{L}\right)\left(1 + \frac{2D_f}{H}\right)\frac{K_s tan\phi_1'}{B} + \gamma_1 D_f \qquad (C.1)$$

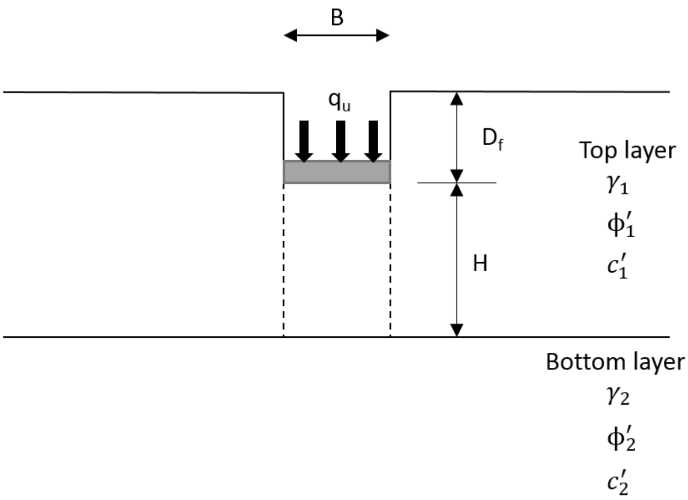


Figure C.1 Bearing capacity parameters for layered soil according to Meyerhof (Das, 2011).