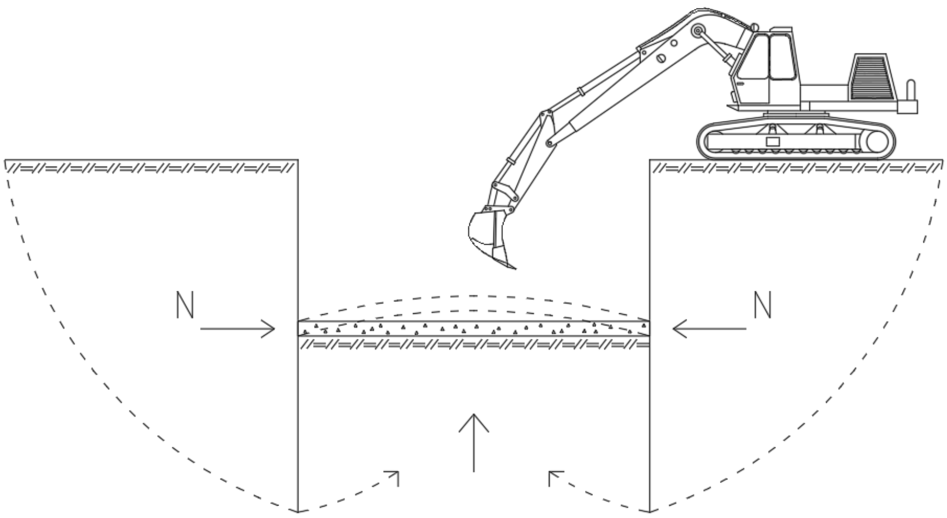

Design procedure of temporary plain concrete slab inside deep excavation

A study to evaluate the unfavorable actions when constructing temporary cast in-situ concrete slabs inside deep excavations

Master's thesis in Structural Engineering and Building Technology

Gustav Nilsson
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Gothenburg, Sweden 2021
www.chalmers.se

Master's thesis ACEX30 2021

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Cover: Illustration of performed excavation.

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Master's thesis in the Master's Programme Structural engineering and building technology

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Abstract

Regard urbanization and exploitation of already densified urban areas, development of areas with disadvantageous construction conditions are made. Regard this, deep excavations are commonly used to manage complex foundations, especially in Gothenburg, where deep excavations are made in soft clay. When performing excavation in clay, plain concrete slabs are commonly used as struts, but also as working platform to ensure a good working environment. For these concrete slabs there is no specified approach listed in Eurocode, or in no further governing guidelines within the industry.

The aim with the thesis is mainly about to evaluate and establish an appropriate design procedure for slabs made of plain concrete used during excavation in soft clay. Literature studies have been made to identify factors that influence the design of plain concrete slabs. The objective was to establish a suitable design procedure that could be used as a general approach, with the intention that the analogy of the design procedure should be inspired by people within the industry, but also on fundamental requirements stated in Eurocode 2.

To gather information from the industry, interviews with experts was performed. It was discovered during the interviews that there was no common procedure, or approach when designing these kind of plain concrete slabs, which strengthened our thesis. However, in summary the interviews contributed with important facts and thoughts that have inspired the analogy of the final design procedure. Finally the analytical and FE-analyses was performed, where the analytical design procedure was validated with the FE-software Abaqus CAE. Further a sensitivity analyse was conducted, where the slab was evaluated regarding different forced eccentricities. The result of the analyse confirmed that the design loads are strictly affected by eccentricities regarding initial conditions and heave.

Keywords: Excavation, Clay, Concrete slab, FE-analysis, Plain Concrete, Buckling.

Acknowledgement

The thesis was performed and completed at the department of architecture and civil engineering, at the division of structural engineering, within structural concrete at Chalmers University of Technology.

The study is mainly about to evaluate and establish an appropriate design procedure for slabs made of plain concrete used during excavation in soft clay. The work includes a literature study, interviews, finite element analyses and finally development of design procedure. The thesis has been carried out together with Geotechnical Engineers of Sweden AB (GEOS) and VBK konsulterande ingenjörer AB during the spring 2021.

The thesis has been conducted with Joosef Leppänen as examiner and supervisor at Chalmers University of Technology and we will hereby thank Joosef for his commitment and his willingness to help and support during the work with the thesis. Further, has the thesis been carried out with Joel Wessman (GEOS) as supervisor and Per Eriksson (VBK) as deputy supervisor, where we would like to thanks for valuable information, but also supporting and time demanding supervisions. Since the interviews have influenced the whole thesis, we would like to thank all interviewees for their participation. Thanks to Anders Kullingsjö och Johannes Tornborg from SKANSKA, Johnny Wallgren och Michael Nyström at PEAB, Rasmus Trygg at Tyréns and finally Anders Ryner.

We sincerely appreciate all the efforts and contributions that made this thesis feasible.

Gustav Nilsson & Rasmus Öhman, Gothenburg, May 2021

Notations

Abbreviation

FEA	Finite element analysis
FEM	Finite element method
w/c ratio	Water-cement ratio

Roman upper case letters

$B_{\%}$	Buckling percentage
E	Modulus of elasticity
E_{cd}	Design value of modulus of elasticity
E_{cm}	Mean modulus of elasticity
F_{br}	Bracing force (Winter)
I	Moment of inertia
K_0	Coefficient of earth pressure at rest
L	Half of l_0 (Winter)
M	Bending moment (y-direction)
M_{Ed}	Design moment
M_{Rd}	Moment capacity
M_0	1^{st} order moment
M_2	2^{nd} order moment
N	Normal force (x-direction)
N_B	Euler's buckling load
N_{Ed}	Design value of the axial force
T	Shear force (z-direction)
P_e	Buckling load (Winter)

Roman lower case letters

c	The cohesion of soil (GEO)
e_h	Eccentricity due to bottom heave
e_i	Unintended eccentricity
e_{min}	Minimum eccentricity of normal force
e_0	Intended eccentricity
e_2	Additional second order eccentricity
f_{cd}	Design compressive strength
$f_{cd,pl}$	Design compressive strength for plain concrete
f_{cm}	Mean compressive strength
$f_{cm}(t)$	Mean compressive strength at a specific time
f_{ck}	Characteristic compressive strength
$f_{ck,cube}$	Characteristic compressive strength from "cube test"
$f_{ck,cylinder}$	Characteristic compressive strength from "cylinder test"
f_{ctd}	Design tensile strength
$f_{ctd,pl}$	Design tensile strength for plain concrete
$f_{ctk0.05}$	Tensile strength, characteristic lower bond value
$f_{ctk0.95}$	Tensile strength, characteristic upper bond value
f_{ctm}	Mean tensile strength
$f_{ctm,fl}$	Mean Tensile strength in bending
$f_{ct,sp}$	Tensile strength from splitting test
h_w	Height of cross section
i	Radius of gyration
l_c	Effective buckling length
l_0	Buckling length
m	Number of members contributing to the total effect
s	Coefficient depended on cement type
u	Water pressure (GEO)

Greek letters

α_{cc}	Coefficient that consider the long term effects on compressive strength
$\alpha_{cc.pl}$	Compressive coefficient for plain concrete
α_{ct}	Coefficient that consider the long term effects on tensile strength
$\alpha_{ct.pl}$	Tensile coefficient for plain concrete
α_h	Reduction factor for element geometry
α_m	Reduction factor for number of members
β	Brace stiffness (Winter)
$\beta_{cc}(t)$	Age dependent coefficient when determine concrete strength
β_i	Ideal brace stiffness (Winter)
Δ	Second order eccentricity (Winter)
Δ_T	Total eccentricity (Winter)
Δ_0	Initial eccentricities (Winter)
γ	Soil weight (GEO)
γ_C	Partial coefficient for concrete
γ_{cE}	Partial coefficient of loading effect of concrete
γ_w	Water weight (GEO)
λ	Slenderness ratio
λ_{lim}	Slenderness that decides whether the element is slender or not
σ_c	Compressive stress
σ'_{h0}	Effective horizontal stress (GEO)
σ_0	Vertical stress (GEO)
σ'_0	Effective vertical stress (GEO)
τ_{cp}	Maximum shear force that occur in the cross sections center of gravity
θ_i	Unintended inclination
ν	Poisson's ratio
ϕ	The friction angle (GEO)
Φ	Factor considering eccentricities, second order effects and effects of creep
ψ	The dilatancy angle (GEO)

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1
Introduction

Since the global trend is urbanization and further exploitation of already densified built-up urban areas, development of areas with disadvantageous construction conditions are made. An example of unfavourable conditions could be areas with bad geotechnical properties, like construct on soft soil. This fact, together with the knowledge about increased urbanization require a new way of design housing, where high-rising buildings is preferably used due to the lack of space within the cities. Regard this, deep excavations is commonly used to manage to construct the complex foundations in a proper manner, especially in Gothenburg, where deep excavation commonly is conducted in soft clay.

1.1 Background

When performing excavation with sheet pile walls in clay, cast in-situ concrete slabs are commonly used as load carrying struts, but also as a working platform to ensure a good working environment. It is also preferably used when phased excavation is practised, then the clay is excavated in phases, where strips of the clay is excavated in phases and further replaced with concrete at the bottom level. When one strip of the concrete is cured, the excavation proceeds alongside the cured concrete strut.

For these previous mentioned temporary concrete slabs there is no specified approach listed in Eurocode, or in no further governing guidelines within the industry. This could be explained by that the question of issue is adjacent to both geotechnical engineering and structural engineering, which make it even more complex. According to lack of information and the fact of that the problem is located between two professions, companies forces to produce their own analytical models for these temporary used constructions. According to this, there will likely be variations between the approaches which will generate different designs for the same cases. However, it would be beneficial to have a common procedure for these slabs to facilitate for companies and ensure an efficient design, both economical, due to utilization ratio and to reduce environmental impact.

1.2 Aim

The aim is to evaluate the interaction, sensitivity and behavior between geotechnical and constructional properties. This to make it possible to identify factors that influence the design of provisional plain concrete slabs and further establish a suitable design procedure that could be used as a general approach in future. The intention is that the analogy of the design procedure should be inspired by people within the industry, but also on fundamental requirements stated in Eurocode 2.

1.3 Objective

The objective is to evaluate, compare and finally enhance a suitable design approach for temporary plain concrete slabs used together with sheet pile walls during excavation. The procedure will be based on knowledge from actives within the profession, theories from relevant literature and further be verified with a FE-software.

1.4 Method

To ensure and to establish a deeper understanding regard the main problem of the thesis a literature study was performed. The literature study was performed to gathered knowledge and to facilitate the work with the final design procedure, but also to gather information about a fair workload and to make it possible to identify limitations and further future work. Below in Figure 1.1, a flowchart that describes the workflow is found.

Alongside the literature study, interviews with persons within the industry was performed. These persons were selected based on their experience and knowledge within the industry, but also on their experience of working in the area of Gothenburg. The purpose was to gather knowledge, but also obtain important insight in how professions are working with these kind of problems. The interviews were conducted as discussions, where the interviewees described their work but also answered on prestated questions.

Based on the interviews and the literature studies an analytical design procedure was established. The design should be conservative and ensure a safe construction site. Further, the analytical model was evaluated with the FE-software Abaqus where the mode of action is determined and evaluated. A sensitivity analysis was also conducted, this to evaluate how sensitive the system is regarding deformations and initial imperfections. The FE-analysis together with the first draft of the analytical design procedure have ended up in a final design procedure preferably used for excavation in soft clay.

Finally, the conducted analyses are discussed together with knowledge gathered from the interviews and the literature studies. From here, further work, source of errors and improvements are further suggested.

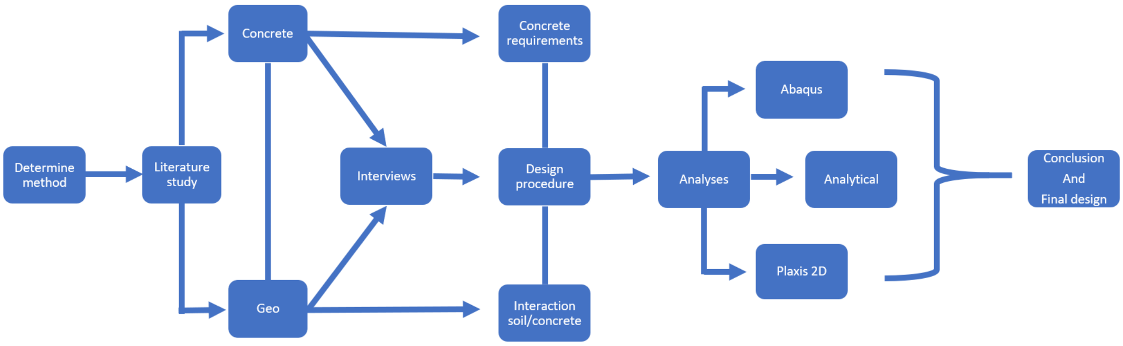

Figure 1.1: *A flowchart that illustrate the methodology of the working progress.*

1.5 Limitations

To ensure a reasonable workload and obtain essential information for the thesis, limitations are made, where one of the most fundamental limitations is the exclusion of other soils than clay. This limitation is made regard time, but also due to the fact of that the complex clay is over-represented in the area of Gothenburg.

When concrete structures are analysed, concrete shrinkage is an important factor that over time could decrease the life of reinforced concrete by induce cracking and further corrosion of the reinforcement, (Palm, 2016, p.52). For temporary plain concrete structures the drying shrinkage (early effects) affect most, this due to temporary use. However, early age shrinkage could be handled and reduced if it is handled in a proper way at the first hours when it is casted at site, so this phenomenon will not be further taken into consideration in this thesis.

Creep is another long term effect, that will in an analytical way reduce the modulus of elasticity dependent on time, (Al-Emrani et al., 2014, B.239). Further, is the creep phenomenon a stress-dependent deformation that will influence and decrease the stiffness of the structural member over long time. Since the slabs is used temporary, this long-term effect is further delimited.

Since the studied concrete slab only consists of plain concrete, the elastic part of the material was used. With this limitation, the failure is assumed to be governing of the first induced crack or concrete crushing. As mentioned in Section 3.3.2, the concrete begin to act non-linear after the induce of the first crack, but this is however neglected further in the thesis.

The adhesive forces at the interface between the clay and concrete, together with the negative pore pressure is neglected, since these phenomenon is favourable and will not affect the design approach in an unconservative way. However, these parameters should preferably be further evaluated in future work, or by experimental trials, since they could delimit the buckling length even more and further generate a less conservative design.

According to Yura (1996), Winter's approach could be extended by infinite bracing springs. In this thesis only one spring is used to reduce the buckling length to it's half, since winter's original work was made with one spring only. However, this can also be evaluated further in future work and research.

2
Literature study - Geotechnics

This first part of the literature study will consist of the geotechnical aspects and phenomenon that occurs during excavation in clay. Since this is not the main focus in the rapport, only a brief overview is going to be presented to give an understanding how and why the geotechnical effects affect the design of temporary concrete slab. Since clay is the most common soil when working with geotechnics in Gothenburg, the main focus will be to identify properties and phenomenon that is characterized for the Swedish clay.

2.1 Excavation

When working with geotechnical construction the strength parameters is based on the characteristic of the surrounding soil. So, first properties of the soil need to be determined, it is then stated what necessary arrangement that is needed to ensure a safe excavation. When performing excavations in clay there is often need for support structures to ensure a safe excavation. For temporary support, sheet pile walls are often used, these are commonly made out of steel (Axelsson & Mattsson, 2016, p.32).

The sheet piles are driven down where they further hooks into each other to create a continuous wall, i.e. a retaining wall. The sheet pile walls needs to be designed with enough capacity to ensure a safe excavation regarding fundamental requirements as failure, but also deformations induced as an effect from excavation that could affect surrounding building and infrastructures (Kullingsjö & Tornborg, 2020, p.4). To minimize the risk of deformations and failure stabilizing measures are used, these are commonly tie back anchors, axially loaded struts and load distributing walers. The strut, sheet pile walls and the waler are displayed in Figure 2.1 below.

Figure 2.1: *Sheet pile structure with associated stabilizing member.*

According to Kullingsjö and Tornborg (2020, p.4), the use of axially loaded struts within the excavation is space consuming and therefore usually problematic during the construction of the building inside the excavation. Tie back anchors are preferably used when the working space is limited, but with the disadvantage of high costs and complex installation in deep soils and couldn't always be installed due to obstacles outside the excavation. So, plain concrete is commonly used at the bottom of the excavation.

A typical working order starts with soil excavation, then whalers and struts are installed to ensure a safe working progress. As mentioned, the axial loaded struts is space consuming and will therefore preferably be disassembled. This could be done by cast plain concrete at the bottom and use this cured concrete as an axially loaded strut. Therefore is it possible to replace the the struts as support for the sheet pile walls.

2.2 Phased excavation

To minimize the use of expensive and space consuming load carrying systems, phased excavations with counter-casted concrete could be used to ensure global stability, where the 3D-effect is taken into account. Phased excavation could be implemented for all depths and is performed in phases until the excavation is completed.

When the sheet pile walls are installed, the excavation work starts. The first phase of excavation is commonly performed to the level of the waler, where the walers usually are welded to the sheet piles (Kullingsjö & Tornborg, 2020). The excavation continues in different "strips", where the soil is excavated in phases and successively replaced with counter-casted concrete. The progress with the excavation is allowed to continue when the casted concrete have cured long enough to ensure a required strength (further reading about the strength, dependent on curing is found in Section 3.1.2.5). Loading of the concrete slab occurs when the soil alongside the cured concrete is excavated, then the normal force generated from the sheet pile wall is transferred to the concrete slab which consequently becomes axially loaded. This procedure continues until the bottom of the excavation is replaced completely by concrete, this concrete slab is further acting as support for the sheet pile walls, but also working-platform to enhance the working environment. If the excavation is deep, there could still be need of stabilizing measures, i.e. steel struts at a higher level to ensure global stability.

An illustration of a common working progress with phased excavation is shown in Figure 2.2 and 2.3 below.

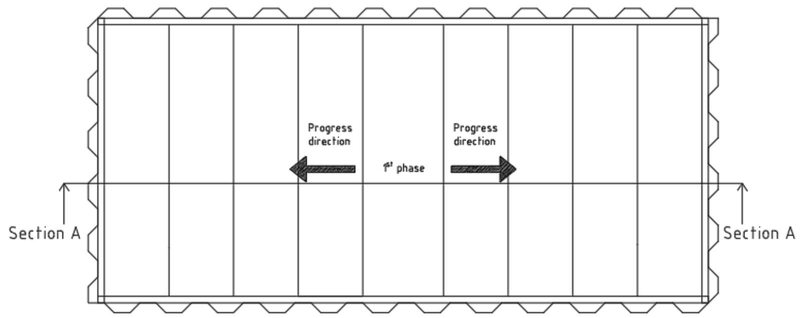

Figure 2.2: *Authors own illustration of phased excavation, plan view.*

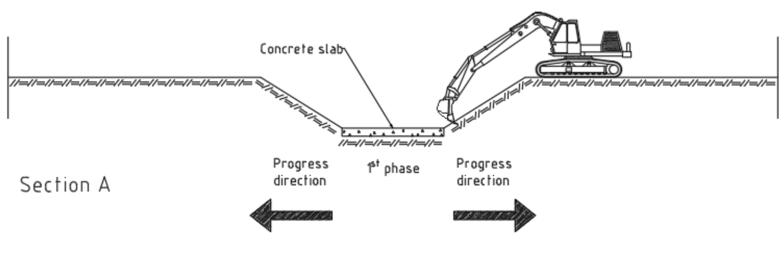

Figure 2.3: *Authors own illustration of phased excavation, section A view.*

2.3 Material Properties

Clay is an inherently occurring material, where the general definition is that clay needs to consist of 40 weight percent fined-grained soil, with additional content of 40% of clay minerals (SGI, 2020a). The behaviour of clay is hard to define generally due to it's complex and unique crystal structure, where the soil is kept together with both chemical and physical bonds regarding the in going minerals and particles in the specific clay.

The micro structure of clay is dependent on several historical factors, the surrounding environment at sedimentation, the loading history, and further the clay content within the material. These factors influence the clay properties and further the micro structure of the clay, which is characterized to be compressible regarding the relatively open and fine-grained structure. How open the structure is depends on the content of salt in the settled environment. The structure tends to be open with an increased amount of saltwater in the settled environment, (Sällfors, 2013).

2.3.1 Permeability

Permeability defines a materials ability to let free water flow through it's soil body (SGI, 2019). Clay has low permeability, which means the water flow through the soil is low.

2.3.2 Cohesion and adhesion

Clay is a cohesive soil, which means that the shear strength is depended on both friction and cohesion, (SGI, 2020b). The physical force cohesion is molecular forces of attraction between the particles in the soil, where a cohesive soil is characterized by its strong binding between its particles.

Further, corresponding behaviour between different materials is entitled, adhesion (Anderberg & Andersson, 2016, p.11). Since the concrete slab will be casted directly on the clay material, there will be adhesion forces between the concrete and clay due to the cohesion within the clay. This force will be favourable since it will act downwards and further counteract the buckling upwards of the concrete. This kind of "suction" force will further be neglected since it is favourable, but should be further evaluated with physically experiments to determine a suction factor for this kind of vertical displacements in the future.

2.4 Unfavourable design actions

To ensure a safe working environment and further a stable excavation, the fundamental global stability needs to be determined. The global stability of the excavation could be disturbed of several possible instability phenomenons, like collapse of the sheet pile walls (the walls fall into the excavation), failure within the sheet pile walls (undersized) and heave of the bottom (Axelsson & Mattsson, 2016, p.369). The design procedure of the sheet pile structure will not be addressed further, since the main focus will be the heave of the clay and other effects on the excavation bottom that could affect the design procedure of the concrete slab.

2.4.1 Instability in the bottom of excavation

Since the concrete slab is casted directly on top of the clay, the movement and deformations of the clay are decisive further for the design of the concrete, especially regarding up-lifting deformations that affect 2^{nd} order effects, but also skew bottom. Deformation of the bottom could occur dependent on several independent and interconnected actions and phenomenons, like mechanical failure in the clay, hydraulic heave, reverse event consolidation and volume increases (regard i.e piling inside or outside the excavation).

2.4.1.1 Mechanical Bottom heave

When performing excavation in clay a reversed stability phenomenon could occur, when the crown of the active side of the excavation is loaded, partly of the overburden, but also external loads, like working equipment or machines. This failure mode occurs when the mobilized shear stresses generated from the overburden and the external load q_{mark} exceeds the capacity of the soil. The failure mode is illustrated in Figure 2.4 (Axelsson & Mattsson, 2016, p.370).

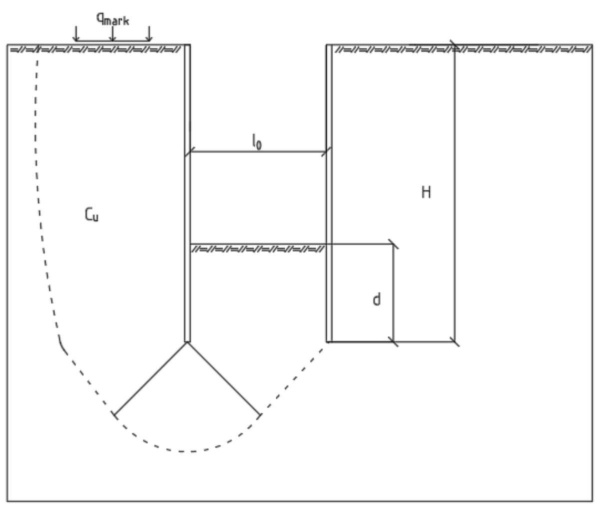

Figure 2.4: *Authors own illustration, based on Axelsson and Mattsson (2016, p.370), that illustrate the failure mode.*

2.4.1.2 Hydraulic bottom heave

Hydraulic bottom heave could occur if the aquifer layer (coarse-grained soil) beneath the bottom of the excavation is exposed to a higher pore pressure than the weight of the sealed overburden (clay) (Axelsson & Mattsson, 2016, p.373). This pore pressure (P(u)) is trapped beneath the clay, but when the clay is unloaded (excavated) the overburden is decreased and further an increased risk of heave of the underlying clay.

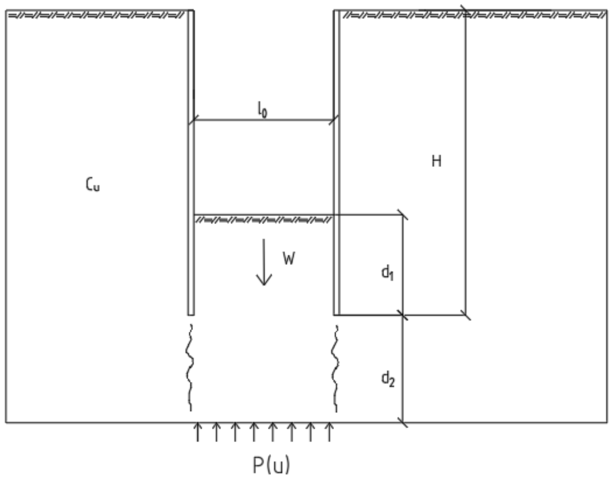

Figure 2.5: *Authors own illustration, based on Axelsson and Mattsson (2016, p.370), that illustrate the failure mode of hydraulic bottom heave.*

2.4.1.3 Reverse event consolidation

Reversed event consolidation could occur as a secondary effect of unloading the soil inside the excavation (Kullingsjö & Tornborg, 2020). When unloading the soil the current equilibrium is disturbed, and the soil tries to return to equilibrium. This mode of action could in a rough manner be compared to a reversed consolidation procedure.

2.4.2 Horizontal load from sheet pile walls

The sheet pile walls function is keep the surrounding soil in place and distribute the horizontal load to the struts and concrete slab, (Sällfors, 2013). The vertical effective stress σ_0', is determined as:

$$\sigma_0' = \sigma_0 - u \tag{2.1}$$

with:

$$\sigma_0 = \sum_{i=1}^{N} \gamma_i \cdot z_i \tag{2.2}$$

$$u = \gamma_w \cdot z \tag{2.3}$$

where:

γ	The weight of the soil.
z	Thickness of soil layer.
u	Water pressure.
γ_w	The weight of water.

Unlike the vertical stress, the effective horizontal stress $\sigma_{h0}^{'}$ can not be determined in an analytical way. Instead is the coefficient of earth pressure at rest K_0 used, which is an empirical value. K_0 describes the relation between the vertical stress and the horizontal stress and varies due to the consolidation rate of the clay. Further, $\sigma_{h0}^{'}$ is determined as:

$$\sigma_{h0}^{'} = K_0 \cdot \sigma_0^{'} \tag{2.4}$$

Where K_0 for normally consolidated clay is assumed to be between 0.6-0.8, and for over consolidated clay K_0 can be greater than 1.

3

Literature study - Concrete

This chapter describes the behaviour of concrete and its material properties. Since the design procedure is established for a plain concrete slab, no contribution from reinforcement has been considered in the thesis.

3.1 Material Properties

Concrete is a well used material, mostly due to its high compressive strength, permanence and formability, (Burström, 2007, p.204-205). Concrete is a composite material containing cement, water and aggregate, where the proportions of the mixture can be varied to provide suitable concrete properties for its purpose. Plain concrete is therefore a suitable material to use for temporary slabs in excavations. Its favorable abilities in compression are useful since the slab will transfer the horizontal loads between the sheet pile walls like a strut.

3.1.1 Concrete behaviour in compression

To illustrate concrete behaviour in compression, the stress-strain relation is studied, (Al-Emrani et al., 2013, p.B31). Figure 3.1 shows a typical stress-strain relation for concretes with varying compressive strengths. The relationship can be assumed linear until 50-60% of the compressive strength is reached and Hooke´s law can be applied. The figure shows that concrete with higher compressive strength has a more brittle failure than concrete with lower compressive strength.

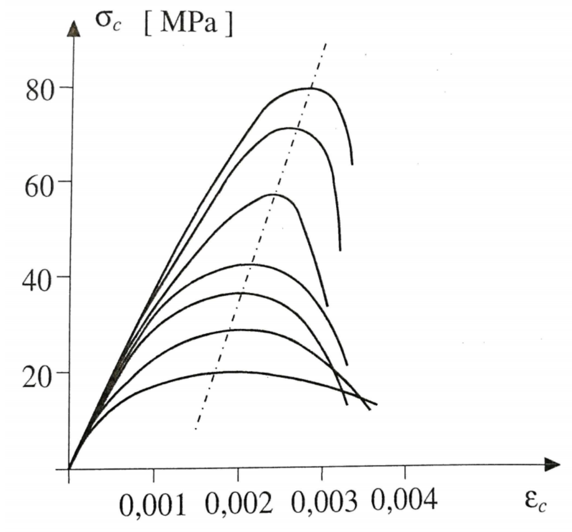

Figure 3.1: *Typical stress-strain relation for concrete under compression. Impact of varying strength (Al-Emrani et al., 2013, p.B31).*

Figure 3.2 shows the concrete behaviour during different loading rates. For short term loads the compressive strength and Young's modulus increases while the ultimate compressive strain decreases, (Al-Emrani et al., 2013, p.B32).

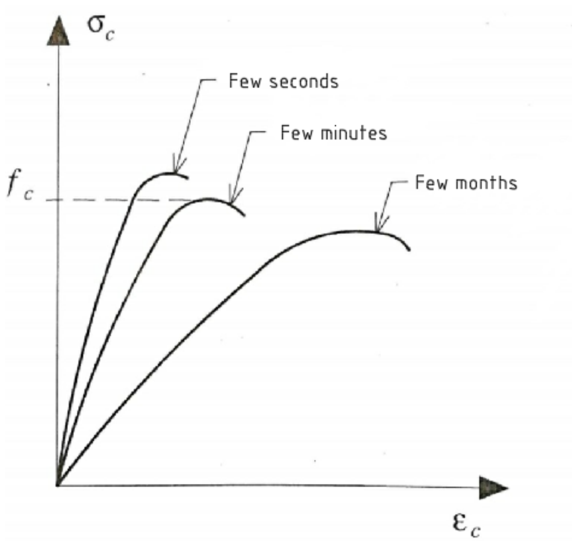

Figure 3.2: *Compressive strength depending on loading rate, figure is taken and translated to English from Al-Emrani et al. (2013, p.B32).*

If the element is subjected to bi-axial or tri-axial loading the compressive strength affects due to changed stress state. During axial and transversely compression the compressive strength increase due to reduction of the concretes transverse expansion. In the same way, transversely tension together with axial compression will reduce it, so called confinement effect. According to Al-Emrani et al. (2013, p.B33) the uniaxial compressive strength can be reduced with 50% if the transversely tension stress is close to the uniaxial tension strength.

3.1.2 Concrete strength

In this section the strength parameters for concrete is defined. Worth mentioned is that Section 3.1.2.1 and 3.1.2.2 defines concrete after 28 days, while the concrete slab analysed in this thesis is younger when it is subjected to loading. This requires a reduction of the strength parameters which is defined in Section 3.1.2.5.

3.1.2.1 Compressive strength

The compressive strength of concrete is determined by standardized uniaxial laboratory tests by loading a cylinder until failure, (Al-Emrani et al., 2013, p.B35-B36). In some countries, like Sweden, it is common to use a cube instead of a cylinder which gives a higher compressive strength. The concrete classes are defined as $f_{ck}/f_{ck,cube}$ in Eurocode, where f_{ck} is the characteristic compressive strength for tested cylinders and correspond to 5%-fractile of the compressive strength achieved during the laboratory test. According to CEN (2004, p.29) the mean compressive strength, f_{cm} can be determined as:

$$f_{cm} = f_{ck} + 8MPa \tag{3.1}$$

3.1.2.2 Tensile strength

The tensile strength of concrete in pure tension can be determined by a splitting test, (Al-Emrani et al., 2013, p.B39). In the splitting test, the crack will occur in a controlled location between the loading points, while for pure tension the crack location will occur in the weakest section. According to CEN (2004, p.28), the mean tensile strength in pure tension could approximately be determined as:

$$f_{ctm} = 0.9 \cdot f_{ct,sp} \tag{3.2}$$

where $f_{ct,sp}$ is the tensile strength from splitting test.

According to Al-Emrani et al. (2013, B40-B41) a relationship between compressive strength and tensile strength can be used instead, since it´s difficult to determine the tensile strength in pure tension by tests. The relationship between the two strength parameters is that the tensile strength increases with increasing compressive strength, but with a lower magnitude. The Equations (3.3) and (3.4) is found in CEN (2004, p.29) and are based on this relationship.

$$f_{ctm} = 0.3 \cdot (f_{ck})^{\frac{2}{3}} \leqslant C50/60 \tag{3.3}$$

$$f_{ctm} = 2.2 \cdot ln(1 + \frac{f_{ck} + 8}{10}) > C50/60 \quad (3.4)$$

Due to the uncertainty when establish a precise tensile strength, characteristics upper and lower bond values are necessary. These are used with respect to the type of design performed, depending on whether a low value is favorable or unfavorable. According to CEN (2004, p.29) Equations (3.5) and (3.6) can be used to determine these characteristic values.

$$f_{ctk0.05} = 0.7 \cdot f_{ctm} \quad (3.5)$$

$$f_{ctk0.95} = 1.3 \cdot f_{ctm} \quad (3.6)$$

where $f_{ctk0.05}$ is the lower bond value $f_{ctk0.95}$ is the upper bond value.

3.1.2.3 Modulus of elasticity

The modulus of elasticity E, represents of the relationship between the deviation in stress and the deviation in strain, where the relationship keeps linear for non-extreme stresses which gives a constant E, (Burström, 2007, p.224). Since concrete is approximately consisting of 70% aggregate with a specific modulus of elasticity and the remaining part is a mixture between cement and water, the compressive strength and E is depended on the *w/c ratio* and the choice of aggregate. This means that a relationship between the compressive strength and the modulus of elasticity can be used.

According to Al-Emrani et al. (2013, p.B43) the mean modulus of elasticity E_{cm}, approximately can be determined by using the linear elastic part from the stress-strain curve shown in Figure 3.3 where E_{cm} is the secant modulus between $\sigma_c = 0$ and $\sigma_c = 0.4f_{cm}$ and can be determined as:

$$E_{cm} = 22 \cdot (\frac{f_{cm}}{10})^{0.5} \quad (3.7)$$

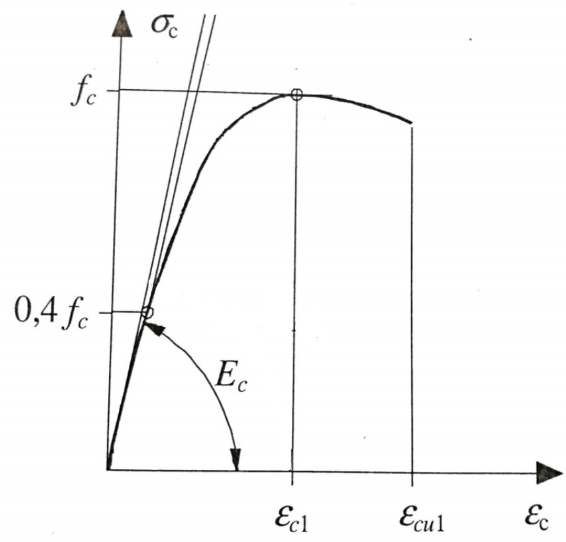

Figure 3.3: *Idealized stress-strain relation for concrete under compression according to Al-Emrani et al. (2013, p.B43), the figure is modified by authors.*

3.1.2.4 Design strength parameters

According to CEN (2004), a further reduction of the strength parameters need to be performed to ensure safety in the ultimate limit state. The design strengths parameters for a reinforced concrete section can be determined as:

$$f_{cd} = \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_C} \tag{3.8}$$

$$f_{ctd} = \alpha_{ct} \cdot \frac{f_{ctk0.05}}{\gamma_C} \tag{3.9}$$

$$E_{cd} = \frac{E_{cm}}{\gamma_{cE}} \tag{3.10}$$

where α_{cc} and α_{ct} is a coefficient that consider the long term effects. These are national parameters that is stated in CEN (2004), for use in a Country may be found in it's National Annex.

α_{cc} Recommended value: 1.0
α_{ct} Recommended value: 1.0

γ_C is the partial coefficient for concrete and γ_{cE} is the partial coefficient of loading effect of concrete.

γ_C Recommended value: Persistent and Transient 1.5
γ_{cE} Recommended value: 1.2

However, for plain concrete, due to less ductile properties compared to reinforced concrete, lower design strengths are to be considered. Instead these lower design strengths are determined as:

$$f_{cd,pl} = \alpha_{cc,pl} \cdot \frac{f_{ck}}{\gamma_C} \tag{3.11}$$

$$f_{ctd,pl} = \alpha_{ct,pl} \cdot \frac{f_{ctk0.05}}{\gamma_C} \tag{3.12}$$

where $\alpha_{cc,pl}$ and $\alpha_{ct,pl}$ is a coefficient that consider the long term effects for plain concrete.

$\alpha_{cc,pl}$ Recommended value: 0.8
$\alpha_{ct,pl}$ Recommended value: 0.8

3.1.2.5 Time effects

As earlier mentioned, the concrete in the temporary slab will be loaded before the concrete has reached its full strength. The difference in strength between young concrete and fully cured concrete can be significant and is therefore an important aspect to consider. According to CEN (2004, p.27), Equation (3.13) can be used to determine the mean compressive strength at a specific time when the concrete is younger than 28 days. Important to mentioned is that this equation is based on specific conditions, where the mean curing temperature is 20°C.

$$f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm} \tag{3.13}$$

with

$$\beta_{cc}(t) = exp(s \cdot (1 - (\frac{28}{t})^{1/2})) \tag{3.14}$$

where $\beta_{cc}(t)$ is an age dependent coefficient where t is defined as days and s a coefficient dependent on cement type as:

s = 0.2 CEM 42,5 R, CEM 52,5 N and CEM 52,5 R (Class R)
s = 0.25 CEM 32,5 R, CEM 42,5 N (Class N)
s = 0.38 CEM 32,5 N (Class S)

The remaining strength parameters can then be determined by the relationships described in the sections above.

3.1.2.6 Temperature effects

The temperature conditions at the excavation bottom will in general result in a lower curing temperature than 20°C, which gives a slower increase of the concrete strength than assumed in Section 3.1.2.5. According to Cementa (2007), the early stage after casting is crucial for the concrete strength since the cooler the concrete becomes, the harder it gets for the hardening process to start. Figure 3.4 illustrates the concrete strength increment during different curing temperatures, which shows the significant impact of the temperature.

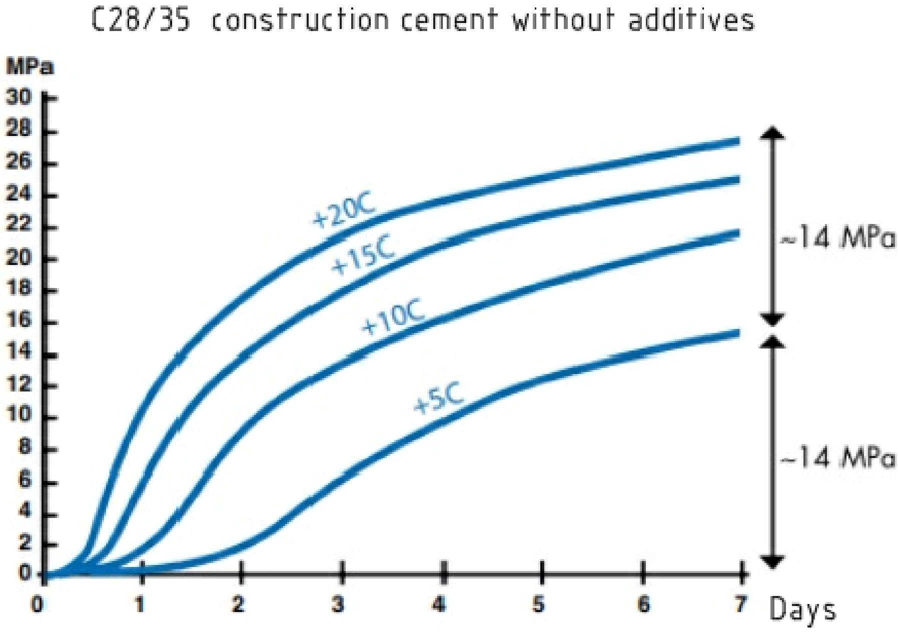

Figure 3.4: *Concrete strength during hardening with different curing temperatures (Cementa, 2007, p.6), where the figure is translated by authors.*

In order to treat this problem, a normal solution is to use rapidly curing cement. This type of cement shortens the time the concrete is fresh and reduces the risk of cooling during this period.

3.2 Accounted failure modes

In this section accounted and limited failure modes will be described in a general manner. The fundamental theories and behavior will also be further presented.

3.2.1 Buckling according to elasticity theory

The global analysis of buckling could be divided into two unfavourable sub-moments, 1^{st} and 2^{nd} order moments, which are illustrated in Figure 3.5. The 1^{st} order moment is influenced by the intentional moment (i.e transversely load), but also contribution from the axially load regarding intentional and unintentional eccentricities. The 1^{st} order moment is according to Al-Emrani et al. (2014, p.B356) described as:

$$M_0 = M(x) + N_{Ed} \cdot [e_0(x) + e_i(x)] \quad (3.15)$$

where:

$e_0(x)$ = Total intended eccentricity.
$e_i(x)$ = Total eccentricity regarding imperfections.
$M(x)$ = Moment regard transversely load.

However, as seen in Equation (3.16), an extra eccentricity regarding unfavourable deformations from the underlying clay is added and taken into account. This eccentricity represent the bottom heave based on the theory stated in Section 3.1.2.5. Further, the effect of the unfavourable magnitude of the eccentricity will be evaluated by a sensitivity analysis in Section 5.4.3.

$$M_0 = M(x) + N_{Ed} \cdot [e_0(x) + e_i(x) + e_h(x)] \quad (3.16)$$

where:

$e_h(x)$ = Total eccentricity regarding unfavourable bottom heave.

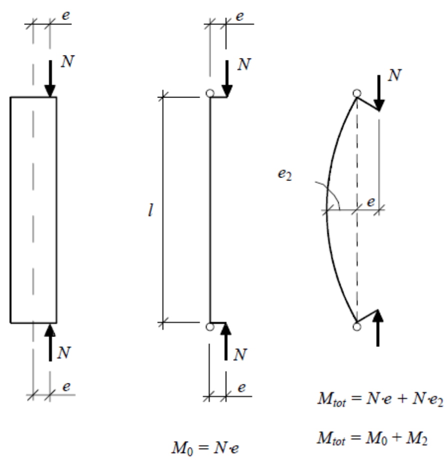

Figure 3.5: *Calculation model when determine the total acting moment, (Al-Emrani et al., 2014, p. K8).*

The 2^{nd} order moment is influenced by the deformed shape of the slab when it is subjected to loading, where the deformed shape will induce an extra eccentricity due to the axial force generated from the sheet pile wall and further generate an extra moment corresponding to the increased eccentricity. The total moment regarding 1^{st} and 2^{nd} order moments is derived as:

$$M_{Ed} = M_0 + N_{Ed} \cdot e_2 = M_0 + M_2 \tag{3.17}$$

3.2.1.1 First order eccentricities

The first order moment does not consider the deformation of the element, but includes intentional moments due to external forces and intentional eccentricity, (Al-Emrani et al., 2014, p.B355). It also includes unintentional moment due to geometrical imperfections.

When considering the unintentional moment, it is assumed that the element is not completely straight, which will induce an unintended eccentricity e_i, dependent on an unintended inclination θ_i. According to CEN (2004, p.55), e_i and θ_i can be determined by the Equations (3.18) and (3.19) and they are further illustrated in Figure 3.6. However, the conditions to perform a slab with minimal imperfections in a excavation is most likely worse than what Eurocode states, which will induce a larger eccentricity. It must be ensured that the clay layer surface is in correct level at all points, otherwise a higher eccentricity need to be considered.

$$e_i = \theta_i \cdot \frac{l_o}{2} \tag{3.18}$$

$$\theta_i = \theta_0 \cdot \alpha_h \cdot \alpha_m \tag{3.19}$$

with

$$\begin{aligned} \alpha_h &= \frac{2}{\sqrt{l}} \\ \frac{2}{3} \leqslant &\alpha_h \leqslant 1 \end{aligned} \tag{3.20}$$

$$\alpha_m = \sqrt{0.5 \cdot (1 + \frac{1}{m})} \tag{3.21}$$

where:

θ_0	Recommended value 0.005
l_0	Buckling length
l	Element length
m	Number of members contributing to the total effect

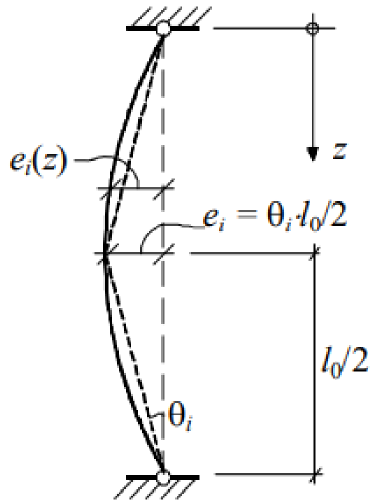

Figure 3.6: *Illustration of the initial imperfection, (Al-Emrani et al., 2014, p.B352).*

The intentional moment include the bending moment from external forces, such as transverse loads and moment due to intentional eccentricity of normal force (Al-Emrani et al., 2014, p.B355-B356). These moments can be replaced by an equivalent moment, $N \cdot e_0$, where e_0 is the intended eccentricity and can be calculated with Equation (3.22). An illustration of how intentional moments can be replaced by an equivalent moment can be seen in Figure 3.7

$$e_0 = \frac{M + N \cdot e}{N} \tag{3.22}$$

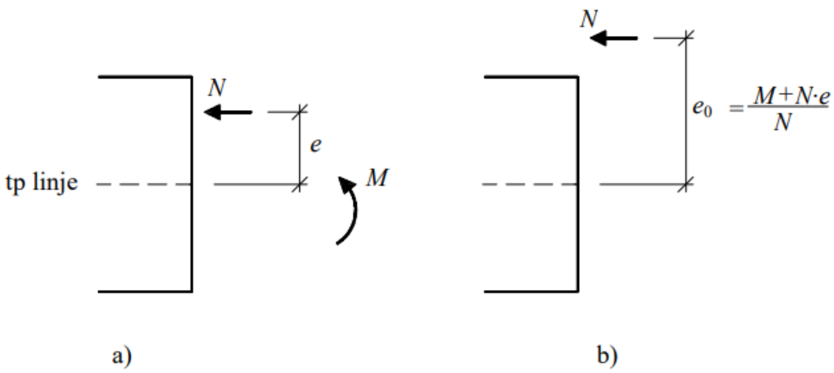

Figure 3.7: *Load cases with multiple intentional moment can be replaced by an equivalent load case with only one eccentric normal force. a) original load case, b) equivalent load case, (Al-Emrani et al., 2014, p.B356).*

3.2.1.2 Second order eccentricities

As mentioned in Section 3.2.1, the 2^{nd} order moment is influenced by the deformed shape of the slab when it is subjected to loading, where this deformation results in an extra eccentricity, e_2. This additionally eccentricity is according to Al-Emrani et al. (2014, p.K15) derived from the double integral of the curvature of the slab and is further derived bellow:

$$e_2 = \int_0^{l_0/2} \int \frac{1}{r}(x)\, dx \tag{3.23}$$

$$\frac{1}{r}(x) = \frac{M(x)}{EI} = \frac{N \cdot ((e_1 + e_2) \cdot sin(\pi \cdot \frac{x}{l_0}))}{EI} \tag{3.24}$$

Insert the above given Equations in Equation (3.17), the additional eccentricity could be written as:

$$e_2 = \frac{e_i}{\left(\frac{\pi^2 \cdot EI}{N \cdot l_0^2}\right) - 1} \tag{3.25}$$

When analysing Equation (3.25) together with the expression for the buckling load Equation (3.43), it could be observed that the additional eccentricity goes toward infinity when the axial force is equal to the Euler's buckling load N_B, stated as Equation (3.43). So, the additional eccentricities denoted as e_2, could be rewritten according to Al-Emrani et al. (2014, p.K20), as:

$$e_2 = \frac{e_i}{\left(\frac{N_B}{N}\right) - 1} \tag{3.26}$$

The total design moment regarding 1^{st} and 2^{nd} order effects can thereafter be expressed as:

$$M_{Ed} = \frac{M_0}{\frac{N_B}{N} - 1} + M_0 = \left(\frac{1}{\frac{N_B}{N} - 1} + 1\right) \cdot M_0 \tag{3.27}$$

Equation (3.27) could according to Al-Emrani et al. (2014, K.22) be rewritten as a general case dependent on the curvature of the acting loads as:

$$M_{Ed} = \left(\frac{\beta}{\frac{N_B}{N} - 1} + 1\right) \cdot M_0 \tag{3.28}$$

where Figure 3.8 below states the β-factor as a function of load case.

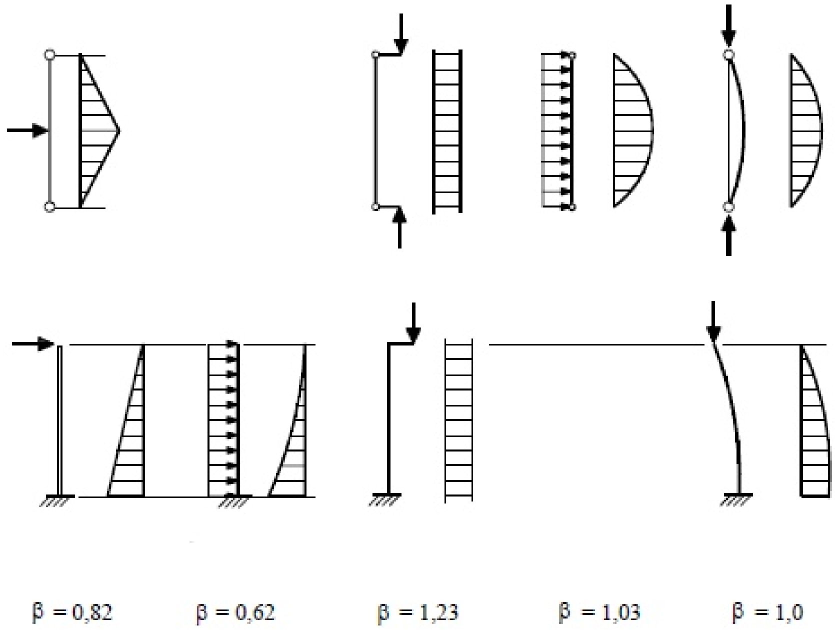

Figure 3.8: *The β-factor for different load cases, (Al-Emrani et al., 2014, p.B362).*

As a final remark, according to CEN (2004, p.65), the second order effects could be neglected if they are less than 10% of the corresponding first order moment. However, if the second order moment is less than 10%, the contribution is small and will therefore in the analytical part still be assumed to contribute to the design moment, this to ensure a design on the safe-side.

3.2.1.3 Slenderness

The design procedure for element exposed to compression are depended on its slenderness ratio λ, (Al-Emrani et al., 2014, p.B357). The second order effects needs to be considered for a slender element, but not for a non-slender element. The slenderness ratio can be determined by following expressions:

$$\lambda = \frac{l_o}{i} \tag{3.29}$$

with

$$i = \sqrt{\frac{I_c}{A_c}} \tag{3.30}$$

where:

i Radius of gyration
I_c Moment of inertia
A_c Sectional concrete area

According to CEN (2004, Page 62), an element can be considered as non-slender if the slenderness ratio is below a certain value λ_{lim}. How to determine this value is shown in Appendix B. However, a slab for this research's purpose will most likely be quite thin with long span width, which will induce a slender element in most cases.

3.2.1.4 Critical buckling length

When reading Section 3.3.1 about Euler's buckling theory, a critical buckling length (l_0) is stated dependent on acting boundaries. However, when evaluate critical buckling length for these kind of concrete slabs, resting on soil, the Euler's buckling cases tend to be quite conservative due to favourable actions. Regarding the gravity load (self-weight) and the interface between clay and concrete (adhesion, further neglected in this rapport), the buckling length will be shorter than the given by Euler in Figure 3.13.

The favourable effects on the buckling length would be generated from the gravity load and will therefore only be affected by the density of plain-concrete. The density of plain concrete varies between 2000 - 2600 kg/m^3 (Palm, 2016, p.39). Generally the weight of plain concrete is set to 25 kN/m^3, but since the gravity load in this case is favourable it would in a conservative manner be set to 23 kN/m^3 (Palm, 2016, p.39).

Due to imperfections and eccentricities of the axial force P, the concrete will subject an uplifting force. 1958 George Winter published his work *Lateral bracing of columns and beams*, where he derived a fictitious bracing force F_{br}, at mid height. Winter illustrates that for a certain brace stiffness β, the buckling length l_0 could be reduced by it's half to L. According to Figure 3.9, F_{br} is defined by the displacement and the brace stiffness at the node n (Yura, 1996, p.822). Among engineers, this bracing force is commonly known as 2% of the axial force P which is based on strength only. By performing experiment, Winter stated that both a defined strength and a defined stiffness are needed to determine the effective bracing, (Winter, 1958). By using the test results, Winter established an inter-relationship between the brace stiffness and the brace force and developed mathematical models for needed strength and stiffness.

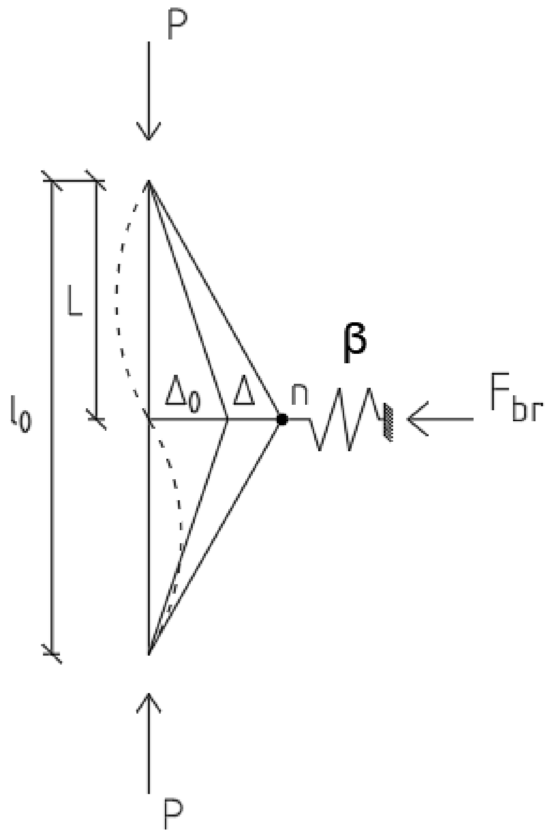

Figure 3.9: *Authors own figure based on Winter's theory, (Winter, 1958).*

According to Yura (1996), full bracing occur and the element buckles between it´s braces, (along L), when the brace stiffness is equal or larger than the ideal brace stiffness, β_i. The ideal brace stiffness is the brace stiffness for the element without initial imperfections, i.e. perfectly straight as illustrated in Figure 3.10.

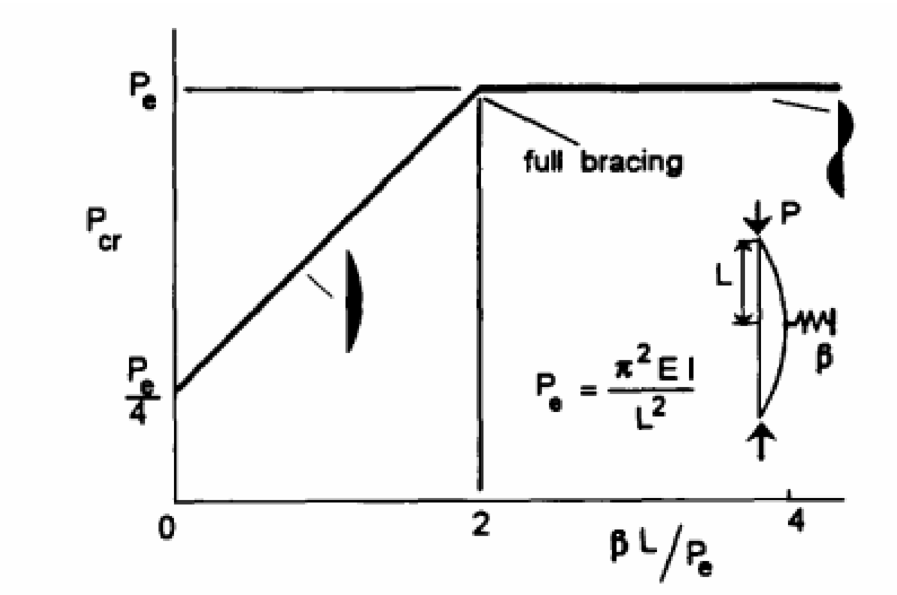

Figure 3.10: *Ideal brace stiffness at full bracing, (Yura, 1996, p.821).*

To determine the ideal brace stiffness, Winter developed a model, illustrated in Figure 3.11, where an axial force displaced the element by Δ. By take moment equilibrium around the connection between the spring and the element and cancel Δ, following expression occur:

$$P = \frac{\beta \cdot L}{2} \tag{3.31}$$

At full bracing, $P = P_e$, which gives:

$$P_e = \frac{\beta \cdot L}{2} \longrightarrow \beta = \frac{2 \cdot P_e}{L} \tag{3.32}$$

When comparing Equation (3.32) with Figure 3.10, it can be seen that they correspond to each other when $\beta = \beta_i$ as

$$\beta_i = \frac{2 \cdot P_e}{L} \tag{3.33}$$

Where P_e is the critical buckling load and is determined by Equation (**??** with the length l_0 as L.

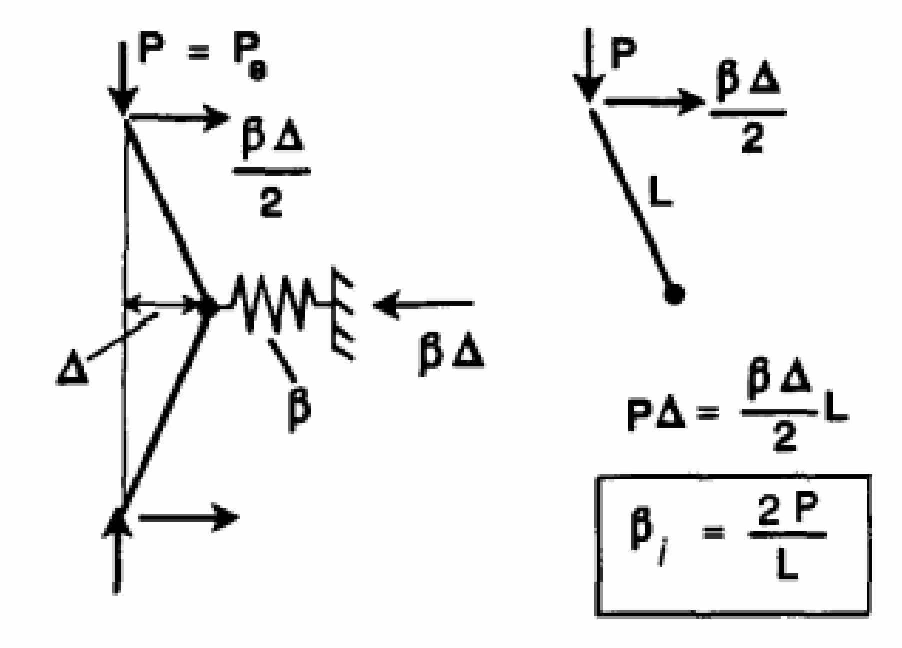

Figure 3.11: *Winter´s model, (Yura, 1996, p.822).*

When comparing the theoretical work with the experimental, Winter realised that using $\beta = \beta_i$ was not applicable for a real column. This since the element used in the experiment contained initial imperfections. Winter also noticed that when using exact elastic theory for an ideal element, no displacement Δ occur. This means that no brace force occur either, while the experimental results showed that the element developed a brace force. With this knowledge, the model in Figure 3.11 was extended to the model in Figure 3.9 by adding an eccentricity due to imperfection, Δ_0 with its corresponding second order eccentricity, Δ.

Taking moment equilibrium around same point as earlier gives following expression:

$$\frac{\beta \cdot L \cdot \Delta}{2} = P \cdot (\Delta_0 + \Delta) \tag{3.34}$$

which gives:

$$\beta = \frac{2 \cdot P \cdot (\Delta_0 + \Delta)}{L \cdot \Delta} \longrightarrow \beta = \frac{2 \cdot P}{L} \cdot (1 + \frac{\Delta_0}{\Delta}) \tag{3.35}$$

The total eccentricity can be expressed as $\Delta_T = \Delta_0 + \Delta$. By using Equation (3.34) Δ_T can be determined as:

$$\frac{\beta \cdot L(\Delta_T - \Delta_0)}{2} = P \cdot \Delta_T \longrightarrow \Delta_T = \frac{\beta \cdot L \cdot \Delta_0}{\beta \cdot L - 2 \cdot P} = \frac{\Delta_0}{1 - \frac{2 \cdot P}{L \cdot \beta}} \tag{3.36}$$

With defined eccentricities and brace stiffness, the brace force can be determined. The brace force represent the minimum force needed to achieve full bracing, i.e. the buckling length can be reduced by half.

$$F_{br} = \beta \cdot \Delta = \frac{2 \cdot P}{L} \cdot (1 + \frac{\Delta_0}{\Delta}) \cdot \Delta = \frac{2 \cdot P}{L} \cdot (\Delta + \Delta_0) = \frac{2 \cdot P}{L} \cdot \Delta_T \tag{3.37}$$

These equations shows a relationship between the brace stiffness and brace force. Yura (1996) tried to illustrate this in Figure 3.12 by plotting the axial load against the brace force for different brace stiffness.

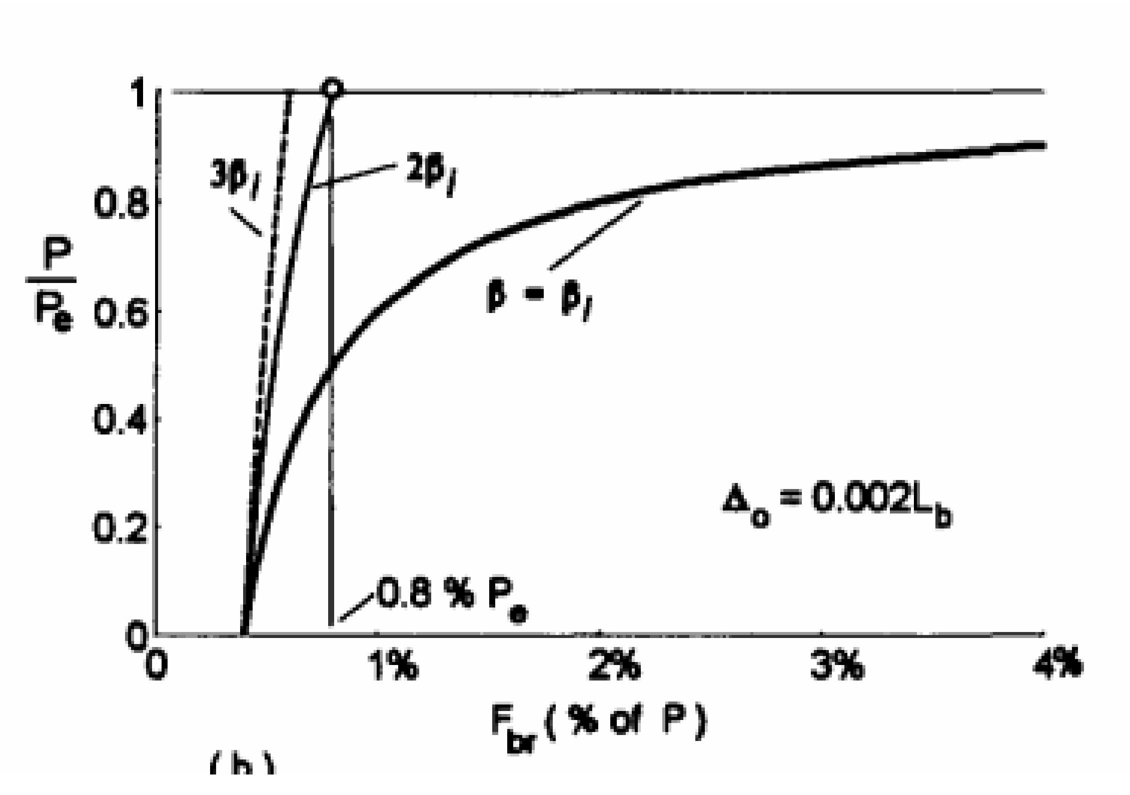

Figure 3.12: *Axial load against the brace force for different brace stiffness with* $\Delta_0 = \frac{L}{500}$*, (Yura, 1996, p.823).*

Winter suggested that the second order eccentricity should be set equal to the eccentricity due to imperfection, (Winter, 1958). By doing this, Equation (3.35) gives that $\beta = 2 \cdot \beta_i$, which in Figure 3.12 shows a lower percentage than 2%. However, it is shown in the Figure 3.12 that the brace force is valid for an initial imperfection of $\Delta_0 = \frac{L}{500}$, and it is highly depended on Δ_0, where the relationship between Δ_0 and F_{br} is:

$$Larger\ \Delta_0 \longrightarrow Larger\ F_{br} \tag{3.38}$$

By using $\beta = 2 \cdot \beta_i$, the buckling percentage, $B_{\%}$, can be derived to be independent of elements dimensions and properties, where the total displacement can be simplified as:

$$\Delta_T = \frac{\Delta_0}{1 - \frac{2 \cdot P}{2 \cdot \beta_i \cdot L}} = \frac{\Delta_0}{1 \frac{P}{(\frac{2 \cdot P_e}{L}) \cdot L}} = \frac{\Delta_0}{1 - \frac{P}{2 \cdot P_e}} \tag{3.39}$$

Defining Δ_0 as a function of L, gives $\Delta_0 = \frac{L}{X}$, where the bracing force is derived as:

$$F_{br} = \frac{2 \cdot P}{L} \cdot \frac{L}{X \cdot (1 - \frac{P}{2 \cdot P_e})} = \frac{2 \cdot P}{X \cdot (1 - \frac{P}{2 \cdot P_e})} \tag{3.40}$$

The buckling percentage is the percentage of the axial force that will represent the needed bracing force and given as:

$$B_{\%} = \frac{F_{br}}{P} \cdot 100 = \frac{2 \cdot P}{X \cdot (1 - \frac{P}{2 \cdot P_e})} \cdot \frac{1}{P} \cdot 100 = \frac{2}{X \cdot (1 - \frac{P}{2 \cdot P_e})} \cdot 100 \tag{3.41}$$

At full bracing $P = P_e$, gives:

$$B_{\%} = \frac{2}{0.5 \cdot X} \cdot 100 \tag{3.42}$$

As earlier mentioned, the slab considered in this work is subjected to an additional eccentricity due to bottom heave. This eccentricity is added to the initial eccentricity, which means that $\Delta_0 = e_i + e_h$. Table 3.1 shows buckling percentage for different Δ_0 that can be used when applying Winter´s theory, the calculations can be found in Appendix A. As seen in Table 3.1, the buckling percentage is higher than 2% when the initial imperfections is bigger then $l_0/400$.

Δ_0 function of l_0	Buckling percentage $B_{\%}$ [%]
$l_0/100$	8
$l_0/200$	4
$l_0/300$	2.7
$l_0/400$	2
$l_0/500$	1.6
$l_0/600$	1.3
$l_0/700$	1.1
$l_0/800$	1
$l_0/900$	0.9
$l_0/1000$	0.8

Table 3.1: *Buckling percentage for different initial eccentricities.*

Since the slab will be subjected to upward buckling, the bracing force will be represented by the self-weight of the slab. The moment from the self weight and the moment from the bracing force are weighted against each other. When the maximum moment from the self weight is equal or larger than the maximum moment from the bracing force, full bracing has been achieved and the buckling length l_0 can be reduced by half.

3.2.2 Local crushing of concrete

In the interaction between the sheet pile wall and the concrete slab, stress concentrations will occur when the clay strip alongside is excavated. These stresses needs to be evaluated and compared with the compressive strength to avoid local crushing of the concrete.

3.2.3 Shear failure

Due to transversal actions, shear forces will occur in the element. The shear force will result in a shear stress that could induce shear cracking. These stresses needs to be evaluated and compared with the shear strength to avoid shear failure.

3.2.4 Loading of uncured concrete

As mentioned in Section 3.1.2.5, the effect of curing will have an impact on the concrete strength. Since the concrete will be loaded earlier than 28 day, a possible failure mode could be, failure in the uncured concrete. Therefore, Equation (3.14) and (3.13) should be used in a proper manner to evaluate the needed curing time for a corresponding concrete stress.

3.3 Material Models and theories

To establish a proper model and further a general design procedure, the work will be based on recognized and validated theories and further specific structural material models. In this section theories and structural material models of importance are explained.

3.3.1 Euler's buckling theory

Buckling is an instability phenomenon that could occur when a slender element is exposed to compression and is loaded above the critical buckling load, N_B (Nationalencyklopedin, n.d.). For slender compression elements, Euler's buckling theory could be implemented, where buckling occur in the elastic part of the material. Non-slender compression elements treats with a plastic theory that was developed for instance of Friedrich Engesser, Euler's buckling cases does not apply for these non-slender elements. Since, commonly the slab addressed in this work is slender the buckling in this thesis would address the elastic buckling for slender compression elements.

Euler buckling occur for ideal-elastic materials when the the axial load exceeds the critical buckling load addressed as Equation (3.43). As could be observed, this load is dependent on the buckling length of the compressed element. Commonly the critical buckling length is determined using Euler's buckling cases (Al-Emrani &

Åkesson, 2020, p.11). Euler's buckling philosophy depends on the boundary conditions of the element, and corresponding buckling length could be determined using Figure 3.13.

$$N_B = \frac{\pi^2 EI}{l_0^2} \tag{3.43}$$

where:

E	Modulus of elasticity
I	Moment of inertia
l_0	Euler's critical buckling length.

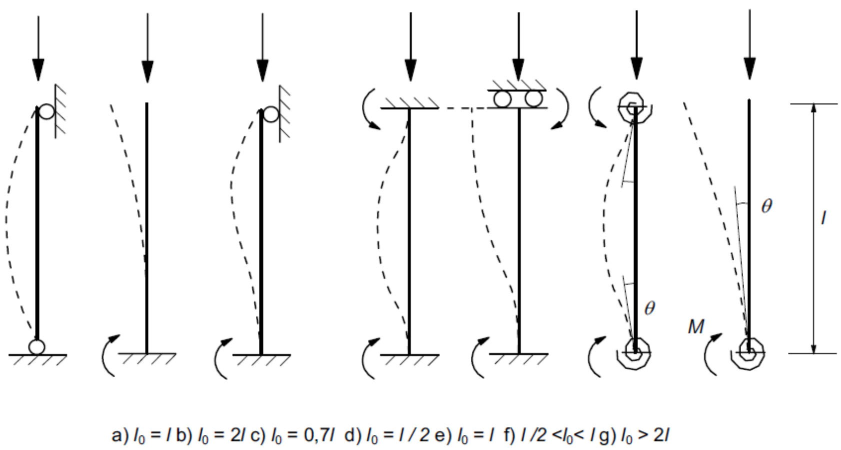

Figure 3.13: *Euler's buckling cases with corresponding buckling lengths. (CEN, 2004, p.64).*

3.3.2 Structural response of concrete

Since concrete is a composite material, different material models are used regarding the response of the material in different phases. Linear elastic response is normally used in the uncracked state, where the concrete is assumed to be homogeneous and perfectly isotropic (Plos, 1996, p.8). At this phase the displacement at corresponding stress state will return to zero when it's unloaded, i.e no plastic deformations occur during loading.

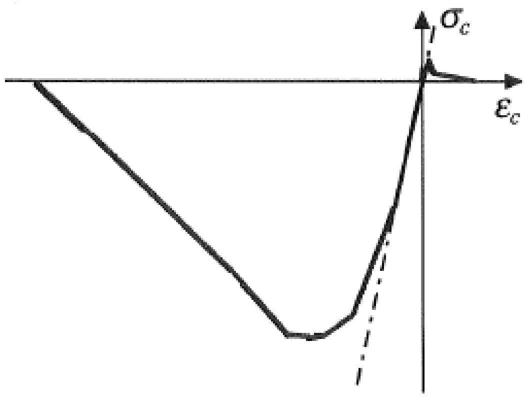

Figure 3.14: *Linear elastic response compared to uniaxial stress-strain relation for concrete, (Plos, 1996, p.8).*

Concrete is acting linear until cracking occur, after the induce of the first crack the response of the concrete tends to be non-linear according to Figure 3.14. According to Plos (1996, p.8), models based on plasticity and fracture mechanisms gives a more proper estimation of the behavior when cracking has occurred and when the concrete gets close to failure in compression.

Generally for homogeneous materials, the material could be defined with a stress-displacement curve, but since concrete is a composite material, the behaviour after cracking is dependent on the geometry and where the proportion of material will influence the cracking and further the failure. However, the non-linear effects will be neglected further, this since focus will be on plain concrete without reinforcement.

3.3.3 Technical beam theory

To solve structural problems analytical, models is developed as an attempt to imitate the reality. These models are based on fundamental physical laws and properties, but also influenced of assumptions and acknowledged theories.

The used beam theory that is applied in the thesis is based on the definition of a statically problem, which make it dependent on fundamental equilibrium and compatibility conditions. For the sectional forces acting on the structure, the equilibrium equations yields (Erochko, 2020):

$$\frac{dN}{dx} = 0 \tag{3.44}$$

$$\frac{dT}{-q(x)} = 0 \tag{3.45}$$

$$\frac{dM}{T(x)} = 0 \tag{3.46}$$

The excavation is commonly performed in different stages, where the clay is excavated in strips and further replaced by plain concrete. The concrete is therefore casted in smaller strips. Due to this, the slab could preferably be modelled with beam-like conditions. According to Erochko (2020) the mode of action for a concrete beam (slab) could in a proper way be explained by applying classic Bernoulli-Euler beam theory. The beam theory explain the behaviour of beams when it's exposed to axial loading and bending. Bernoulli-Euler beam theory consists of some fundamental conditions, these yields:

- Plane cross-sections remains plan during loading and deformation, i.e shear deformations are neglected (Palm, 2016, p.28).

- The cross-section remains perpendicular to the neutral axis during deformation, further the deformed angles of the beam (θ) in every cut are small.

3.3.3.1 Design load

The total acting design load is derived from classic beam theory and is expressed as:

$$\sigma = \frac{N_{Ed}}{A_c} + \frac{M_{Ed}}{I_c} \cdot z \tag{3.47}$$

where the normal force is the axial force applied from the sheet pile walls and the moment is the total moment, booth from 1^{st} and 2^{nd} order moments. This equation will be used further to evaluate the tensile stress and compressing stress. The expression of the total stresses is based on parameters that easily could be derived from the geometry and shape of the studied cross-section, where:

I_c The moment of inertia of concrete cross section
A_c Area of concrete cross section
z Distance from compression or tensile edged to the centroidal axis.

3.4 Procedures to design plain concrete slabs

If the slenderness λ in Section 3.2.1.3 is greater then λ_{lim} the following procedures could be used in some manner. To determine which of them, λ (defined as Equation 3.29) needs to be determined and thereafter choose which analytical design procedure dependent on the slenderness criterion below:

Criterion	Method
$\lambda \leq 86$	Simplified method
$\lambda > 86$	General method

Table 3.2: *Methods dependent on the slenderness of the analysed slab.*

Further, the simplified method is found in Section 3.4.1 and the general method is found in Section 3.4.2, where they are further explained.

3.4.1 Simplified method

If the slenderness (λ) is below 86, this simplified method could be used as design procedure. According to CEN (2004, p.197), in absence of a more rigorous approach for more "stocky" slabs, this simplified method could be used. However, the simplified method is therefore preferably used for small excavations, where the length (l_0) is quite short, this to ensure reliable and safe results. The slab has to be designed so it fulfills the requirement from both Equation (3.60) and (3.53).

3.4.1.1 Combination of bending and normal force

According to Section 12.6.5.2 in CEN (2004, p.197), the capacity for a plain concrete slab exposed to bending with an axial compressive force can be determined as:

$$N_{Rd} = b \cdot h_w \cdot f_{cd,pl} \cdot \Phi \tag{3.48}$$

$$N_{Rd} \geq N_{Ed} \tag{3.49}$$

where:

b Width of cross section
h_w Height off cross section

The design compressive strength $f_{cd,pl}$, is determined by Equation (3.11) with consideration to the time effects discussed in Section 3.1.2.5. Φ is a factor that consider the eccentricities, second order effects and effects of creep. Φ is determined as:

$$\Phi = 1.14 \cdot (1 - \frac{2 \cdot e_{tot}}{h_w}) - 0.02 \cdot \frac{l_0}{h_w} \leq 1 - \frac{2 \cdot e_{tot}}{h_w} \tag{3.50}$$

where l_0 is determined in the first step in the analytical design procedure.

$$e_{tot} = e_0 + e_i \tag{3.51}$$

The eccentricities e_0 and e_i are determined according to Section 3.2.1.1. For the slab analysed in this thesis, an additional eccentricity, e_h, will occur due to bottom heave from the underlying clay.

$$e_{tot} = e_0 + e_i + e_h \tag{3.52}$$

3.4.1.2 Combination of shear force and normal force

According to CEN (2004, p.191), the capacity for a plain concrete slab exposed to shear force and normal force is depended on both the tensile strength and compressive strength. If the element should be able to resist the combination of shear force and normal force, the relationship need to be fulfilled:

$$\tau_{cp} \leq f_{cvd} \tag{3.53}$$

where τ_{cp} is the maximum shear stress that occur in the cross sections center of gravity, (Al-Emrani et al., 2013, p.B97). For a homogeneous cross section following expression can be used:

$$\tau_{cp} = \frac{S(0) \cdot V}{I \cdot b} \tag{3.54}$$

where:

S	First moment of area
V	Shear force
I	Moment of inertia
b	Width of cross-section

For a rectangular cross-section, Equation (3.54) can be simplified as:

$$\tau_{cp} = \frac{3 \cdot V}{2 \cdot b \cdot h_w} = k \cdot \frac{V_{Ed}}{A_{cc}} \tag{3.55}$$

where:

k	Recommended value $\frac{3}{2} = 1.5$
V_{Ed}	Design shear force
A_{cc}	Compressive area

The design shear force in this thesis is defined as fully braced member i.e. the bracing force, F_{br}.

f_{cvd} is depended on the relationship between σ_{cp} and $\sigma_{c,lim}$. σ_{cp} is the acting compressive stress and $\sigma_{c,lim}$ is a stress depended on the concrete compressive and tensile strengths, which are determined according to Section 3.4.1.1. The capacity of the slab is determined as:

if $\sigma_{cp} \leq \sigma_{c,lim}$:

$$f_{cvd} = \sqrt{f_{ctd,pl}^2 + \sigma_{cp} \cdot f_{ctd,pl}} \tag{3.56}$$

if $\sigma_{cp} > \sigma_{c,lim}$:

$$f_{cvd} = \sqrt{f_{ctd,pl}^2 + \sigma_{cp} \cdot f_{ctd,pl} - (\frac{\sigma_{cp} - \sigma_{c,lim}}{2})^2} \tag{3.57}$$

where:

$$\sigma_{cp} = \frac{N_{Ed}}{A_{cc}} \tag{3.58}$$

$$\sigma_{c,lim} = f_{cd,pl} - 2 \cdot \sqrt{f_{ctd,pl} \cdot (f_{ctd,pl} + f_{cd,pl})} \tag{3.59}$$

When looking at the contribution from σ_{cp} in Equation (3.56) and (3.57), it can be seen that the compressive axial force has a favourable effect on the capacity.

3.4.2 General design procedure

If the slenderness (λ) is greater than 86, a general design procedure should be used to ensure a proper design. Since the slab only consist of plain concrete, nonlinear effects has been neglected and assumptions have been made that failure occur if the section cracks. The stresses are to be determined in critical sections and controlled against the design strength parameters.

3.4.2.1 Buckling load

The first step in the general procedure is to evaluate the acting loads regard fundamental requirements as:

$$N_B \geq N_{Ed} \tag{3.60}$$

where N_b is the buckling load, determined by using Euler's buckling theory stated in Section 3.3.1.

3.4.2.2 Stresses from bending and normal force

To ensure the capacity of the plain concrete slab, the maximum stresses at compressive and tensile sides are determined and further compared to the capacities of the concrete. Stadium I is assumed, where the cross-section remains uncracked until failure, i.e. cracking or concrete crushing occurs. The design stresses are determined by using the theory from Section 3.3.3.1, where the compressive stress is denoted as σ_{cc} and is given by:

$$\sigma_{cc} = \sigma_N + \sigma_M = \frac{-N_{Ed}}{A_c} + \frac{M_{Ed}}{I_c} \cdot (-\frac{t}{2}) \tag{3.61}$$

The design compressive stress is thereafter compared with the compressive strength of concrete. The design compressive strength of plain concrete $f_{cd,pl}$, is determined by Equation (3.11) and the requirement that needs to be fulfilled is stated as:

$$f_{cd,pl} \geq \sigma_{cc} \tag{3.62}$$

The tensile stress is denoted as σ_{ct} and is further determined as:

$$\sigma_{ct} = \sigma_N + \sigma_M = \frac{-N_{Ed}}{A_c} + \frac{M_{Ed}}{I_c} \cdot (\frac{t}{2}) \tag{3.63}$$

The design tensile stress is thereafter compared with the design tensile strength of concrete. The design tensile strength of plain concrete $f_{ctd,pl}$, is determined by Equation (3.12) and the requirement that needs to be fulfilled is stated as:

$$f_{ctd,pl} \geq \sigma_{ct} \tag{3.64}$$

3.4.2.3 Local crushing edge

To ensure safety against local crushing of the concrete, the capacity against axial compressive force is determined. The same methodology as in Equation (3.61) is used, but only the contribution from the σ_N. Instead of determine the design normal stress, the design strength against normal force is determined.

$$\sigma_N = \frac{N_{Ed}}{A_c} \rightarrow f_{cd,pl} = \frac{N_{Rd}}{A_c} \rightarrow N_{Rd} = f_{cd,pl} \cdot A_c \tag{3.65}$$

$$N_{Rd} \geq N_{Ed} \tag{3.66}$$

3.4.2.4 Stresses from shear force

The design strength of the shear capacity is also chosen with the analogy of uncracked cross-section, where the strength is set to $f_{ctd,pl}$, which is determined by Equation (3.12). When determine the design shear stresses in the slab, focus is on the appearance of web shear cracks, this due to the fact of plain concrete.

The design shear stress is determined according to Engström (2011, p.164), and is given as:

$$\tau_{max} = \frac{V_{Ed}}{b_c \cdot z_{in}} \tag{3.67}$$

where:

b_c	The width of the concrete cross-section.
V_{Ed}	Force needed for full bracing, F_{br}.
z_{in}	Internal level-arm between maximum compression and tension side.

If the element should be able to resist the design shear stress τ_{max}, the relationship below should be fulfilled:

$$f_{ctd,pl} \geq \tau_{max} \tag{3.68}$$

4 Interviews

This chapter includes interviews with persons within the profession, with the purpose to give indispensable information about their thoughts regarding design of concrete slabs used during excavation. Their experience and knowledge will also inspire further work, but of course inspire the final design procedure.

4.1 Methodology

The conversations between authors and professionals was performed during spring, where each interview started with a brief review conducted by each consulted company. The conversation was accomplished by questions that the authors had specified, with the purpose to evaluate differences and similarities between professionals within the industry. The interviewees was prepared with stated questions before the interview and the pre-stated questions are found below:

- How do you treat the contact surface (interaction) between the plain concrete and the sheet pile wall, as free to rotate, partially restrained or fully fixed?

- How do you determine a reasonable buckling length, with respect to favourable actions generated from the gravity load?

- Is the groundwater pressure taken into consideration beneath the concrete slab, or is the free water assumed to be pumped away?

- Do you take any unfavourable deformations regarding the clay into consideration, deformations like i.e unloading of the clay or heave of the bottom beneath the slab?

- Do you take the hardening time into consideration, i.e days less than 28 days before loading when performing phased excavation? Further, is the temperature at curing taken into consideration?

- Is the concrete slab designed with or without reinforcement?

- Further, is the concrete modelled in some kind of FE-software or is it only analysed in an analytical way?

- How do you treat eccentricities due to imperfections? Except theoretically imperfections, do you take others into consideration, i.e rough bottom?

These stated questions was based on newfound knowledge and question marks found during the earlier performed literature study. The stated questions was free to answer for the interviewed persons and therefore could some answers be missing. The questioned persons was chosen from different companies to ensure a proper dispersion and further a representable data of how the industry are working. The interviews were conducted in Swedish and has later been translated by the authors and subsequently been approved by the interviewees.

Name	**Company**	**Date for interview**
Anders Kullingsjö	SKANSKA AB	2021-02-23
Johannes Tornborg	SKANSKA AB	2021-02-23
Johnny Wallgren	PEAB AB	2021-02-23
Michael Nyström	PEAB AB	2021-02-23
Rasmus Trygg	Tyréns AB	2021-03-09
Anders Ryner	Retired	2021-03-01

Table 4.1: *Interviewed professionals within the industry.*

4.2 The conversations

In this section the conversation will be displayed and retold in a manner where the most important features are depicted. Further citations and reflections from the interviewees of importance for the thesis and to testify the need of the work will be emphasized. The results of the interviews will therefore be presented in a couple of subsections that emerged during the time of processing and validation of the interviews.

4.2.1 General remarks and feedback

An overall response during the interviews was that the asked persons welcomes such work that is implemented in this thesis. A recurrent comment is that there is missing guidelines when performing such designs of concrete slabs used during excavation. Below is a citation stated from Tornborg during the interview:

> "That's a relevant problem to study since we use plain concrete for working platforms in almost all projects conducted in Gothenburg. Therefore it's a good idea to use the concrete as a construction element and not only with the purpose to improve the working environment."

When talking to the professionals at Peab AB, Nyström also expresses that there is lack of guidelines when performing this kind of designs:

> "I think that the area of the thesis is interesting, this since there is no guidelines how to handle these slabs."

Another striking remark is that the view of the mode of action and further how to implement the design of these slabs differ a lot between the asked companies. These differences was determined during the interviews and confirms our thesis and further the need of a general future design procedure.

4.2.2 Design philosophy

As an attempt to design these concrete slabs a design philosophy is to be chosen, due to the fact that concrete is designed in different ways in Eurocode dependent on the chosen mode of action. Therefore, a design philosophy is to be chosen to make it possible to use some guidelines and requirements stated in Eurocode to ensure a sufficient design.

There are several design decisions that is to be made, especially possible failure modes, and how the concrete will behave during axial loading. During the literature study three possible failure modes occurred, theses are found in Section 3.2. Accounted failure modes are further also listed below:

- Buckling of slender structural element.
- Local crushing of compressed concrete.
- Crushing of uncured concrete.

So, how does the industry count for different failure modes and is the opinion of the failure mode common for all interviewees? This question was answered already during the first two interviews at the first question about a reasonable buckling length where the opinion was strictly divided. According to Wallgren:

> "We don't take the buckling phenomenon into consideration, our opinion is that there is suction between the clay and the concrete. This suction is limited to time, due to permeability, so if we could provide a limited time period of 1-2 months and with the requirement that the concrete is casted directly on top of the clay we could assume suction. This assumption entails that the slab is stiffened and buckling could be further neglected. So, the counted failure mode we count for is failure due to compression in the contact zone between the sheet pile wall and concrete. The buckling reasoning is abandoned and replaced of a time limit, where the suction is limited regard equalized pore pressure."

Nyström that is employed at the same company and is colleague to Wallgren adds important information:

> “This reasoning is utmost dependent on the fact that concrete is casted directly on top of the clay. Therefore is it prohibited to sprinkle any-type of gravel at the interface between the concrete and clay.”

When comparing this arguments with another stated during the conversation with Kullingsjö and Tornborg differences occurred. Read Kullingsjö:

> “In a former project (Göta tunneln), it was decided to base the buckling length on the analogy of axially loaded piles with corresponding springs that illustrates the enveloping soil. The adopted analogy state that when the pile is loaded axially the pile starts to deform like a sinus curve, where the length of one mode constitutes the buckling length. So, will the concrete buckle in one bow, like a classic Euler case, or will it buckle in sinus modes? The thing is, when the slab starts to lift from the ground, the gravity load will be favourable and counteract this phenomenon up to a certain limit, which could generate a sinus curve. Further we conducted an analysis to determine the maximum length of the sinus modes.”

To get a different point of view and further validate the risk of buckling an interview with an external expert outside the area of Gothenburg Anders Ryner was conducted. According to Ryner:

> “I don’t think you should try to count for buckling, since you have the gravity load acting in a favourable direction and it not likely to buckle down due to the underlying clay. So, I don’t think buckling is such a problem as far as the bottom of the excavation is reasonable smooth. So, it is of importance to study the condition of the bottom of the excavation to ensure if buckling could occur.”

As could be observed from the citations above, is that the approaches differ a lot, which support the thesis of a fragmented design approach within the industry. Another thing that appeared during the interview was, that most of the consulted persons commonly try to take the curing time into consideration. Read Kullingsjö:

> “We take the hardening time into consideration, this is important when performing phased excavation, especially closest to the sheet pile wall. The curing time is important, since there will be load concentrations at the end of the slab when the concrete is loaded as a result of the excavated clay alongside the slab. So, the curing time needs to be evaluated and determine when the phased excavation could continue to prevent crushing of concrete.”

Tornborg that is employed at the same company and colleague to Kullingsjö adds:

> "Based on the normal force acting on the specific concrete area, a required strength is determined. The required strength is then prescribed in the construction drawings and it's usually not a problem for the progress of construction, since accelerators could be employed as an additive in the concrete."

This reasoning was confirmed by Trygg, Ryner, Nyström and Wallgren, read e.g Nyström:

> "I use to prescribe a required needed strength in my delivered documents and commonly the concrete quality is increased to ensure the prescribed strength."

And Ryner:

> "One should not think that you can fully load the slab the day after, if the concrete was casted at afternoon before, this since the concrete need to cure to provide a rigid support. This curing will probably take a couple of day, but of-course the curing is dependent on the chosen concrete class. So, most often is it good to increase the concrete strength to provide sufficient strength in an early age of curing."

A final question regarding the design philosophy that was discussed during the sessions was the use of reinforcement while designing the concrete slabs. Is reinforcement commonly used or is plain concrete preferably used? An overall conclusion after compiling the answers was that in almost 9 out of 10 times the slabs is designed without reinforcement. Reinforcement is only used in those cases where the slab will be loaded with heavy vertical loads (i.e machines) inside the excavation or if there is need of notches in the slab to provide the possibilities of piling at the bottom of the excavation. Read Kullingsjö:

> "Commonly our goal is to use plain concrete. We just use reinforcement when it's especially needed, for example when we know that the slab will face heavy vertical loads and further tends to buckle downwards in the underlying soft clay. For those cases we use and prescribe reinforcement."

This reasoning is strengthened and further explained by Trygg:

> "Sometimes is it basically necessary to prescribe reinforcement during the design. The thing is, when it's starts to be advanced design of reinforced concrete we tend to leave the goal with your thesis. This since the biggest uncertainties are connected to the use of plain con-

> crete."

4.2.3 Unfavourable design actions

As mentioned in Section 3.2.1, buckling according to elasticity theory, imperfections and deformations will have a huge impact on the acting design moment. Since the concrete is casted directly on top of the clay that is recently unloaded there could be imperfections regarding skewness and heave of the bottom of the excavation that could generate a corresponding eccentricity to the axial force. However, when reading Section 3.2.1.4, about how to determine the buckling length based on Winter's approach the initial imperfections is also fundamental. So, is this critical phenomenon taken into consideration or is it neglected? Read Kullingsjö:

> "Our aim is to take it into consideration, but if this unfavourable action is larger than the gravity load needs to be determined for each case. Another important parameter for the bottom heave is if you are performing phased excavation, since it's hard to determine how large the heave will become when continuing the phased excavation. So, usually a safety margin is used, where the thickness of the concrete is increased based on experience from other projects.

Follow up question from Authors: Regarding unloading of the clay while excavating, do you know how long it takes for the bottom to heave?

Tornborg answering:

> "It's depends on many factors, but a part of the heave will, depending on e.g. soil characteristics and excavation geometry occur instantaneously, but it's a small part, say in the order of 20% in Gothenburg clay, of the total stress change will transform to heave directly. The remaining heave will take longer time, but the passage of time of the total heave is quicker than we thought for a couple of years ago, e.g. if piling has been executed before. The soil permeability and modulus but also e.g. the geometry of the excavation will influence the process of heave, where a large excavation likely will generate a longer time period and vice versa.

A suggestion from Ryner that could improve the future design procedure was stated:

> "Further you should actually study and evaluate the level and the skewness of the bottom after unloading. In other words determine the order of magnitude of the initial imperfections in the system. Because if you do, rules could be established for different situations based on the initial imperfections."

4.2.4 Final remarks and ideas

During the interviews important knowledge and thoughts regarding further work and the final design procedure appeared, where some of them are listed below and will further be used in the thesis.

Trygg stated important thoughts regarding the sensitivity analysis. He suggested to use predetermined deformations instead of trying to evaluate exact values for the observed case, since then the system could be analysed to determine worst case scenarios, and further the behaviour with respect to forced deformations combined with axial loading. Read Trygg:

> “Maybe it’s a good idea to base this forced deformations regarding heave on experience and simplified calculations, despite having advanced FE-software’s. However, the FE-software is a good tool when determine the general behaviour, but the FE-modelling require a lots of assumptions which also generates corresponding uncertainties and further tend to catch the mode of action for just one specific case. So, to catch the general trends a sensitivity analysis with predetermined forced deformations a good way to evaluate and determine the worst case scenarios.”

As mentioned previously during the conversation with Ryner, he suggested a deeper evaluation of the bottom of the excavation. This reasoning was however further specified and explained as:

> “It would be beneficial to establish a table that itemize load cases dependent on initial imperfections of the bottom of the excavation. For example, if you have variations of +/- 2 cm you have some requirement, +/- 5 cm some others and so on. A system like that would be really good!”

Due to the uncertainties and assumptions that is made during the design procedure, it would be of interest to perform a real case study as further work, this to determine the real behaviour and mode of action. The proposal of the case study was stated by Wallgren:

> “A case study with two sheet pile walls and a 10 m long concrete slab that is loaded up to failure by the use of hydraulic jack would be of interest to study. This to make it possible to derive the analytical part from and further prove the chosen mode of action. This would be a great future work to conduct as a future work.”

5

Analyses

In this chapter several analyses are made and described, this to evaluate and validate the stated theory, but also to determine similarities and differences between models and theory. However, three different analyses will be performed and these are:

- Plaxis 2D analysis - With the purpose to determine reasonable axial force acting on the concrete slab.
- Analytical - With the purpose to establish a proper design procedure based on stated requirements.
- Abaqus CAE analysis - With the purpose to validate the analytical approach and further evaluate how sensitive the system is regard eccentricities.

5.1 Reference case

For simplicity, a reference case is used during the three analyses. The reference case is located in Gothenburg, where the enclosing soil is made out of characteristic Gothenburg soft clay.

The geometry is illustrated in Figure 5.1, where the depth of the excavation is 5 m, the length of the sheet pile walls is 13 m and the width of the excavation is 10 m. The concrete slab will obtain the same length i.e. 10 m.

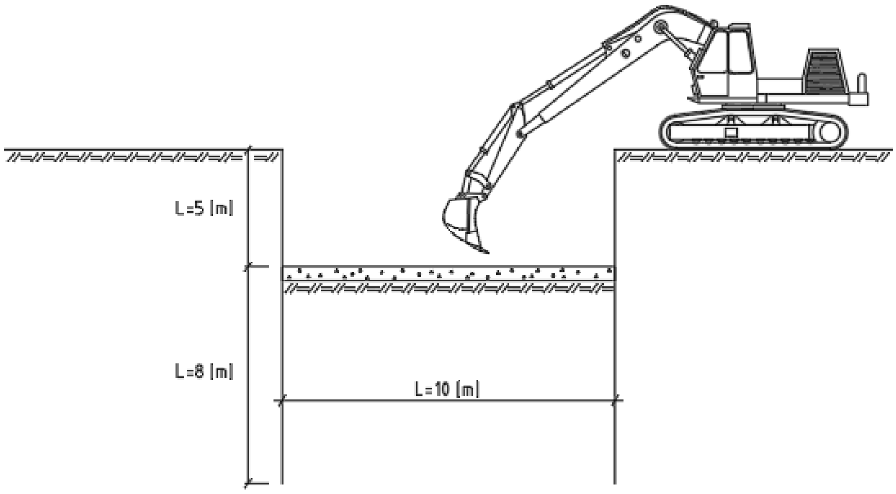

Figure 5.1: *Own illustration of reference excavation.*

5.2 A brief FE-Modelling in PLAXIS 2D

To determine a reasonable axial force acting on the concrete slab, a FE-model of an arbitrary excavation using sheet pile walls was modelled in Plaxis 2D. The model was simplified, with the purpose to estimate reasonable unfavourable actions. The updated mesh setting was used to take 2^{nd} order effects into consideration.

The input material parameters used for the Plaxis 2D analysis could further be found in Appendix C.

5.2.1 Material model

The used built-in Plaxis 2D features are further described in this section, this to enable to understand the behaviour and limitations of the chosen material model. Further, the plain strain with 15-nodes elements was used.

5.2.1.1 Node-to-node anchor

Node-to-node anchor is an element in Plaxis 2D that is commonly used to determine the axial force acting in the normal direction of the anchor, due to this the node-to-node anchor only sustain displacement in the normal direction (van der Sloot, 2020, p.35). Node-to-node anchor is in a physically way defined as a spring, with a defined stiffness (EA), that spans between two nodes. The concrete slab was modeled as a node-to-node anchor at the bottom of the excavation, this to determine acting normal forces.

5.2.1.2 Interfaces

To obtain reasonable behavior between the clay and the adjacent structural elements interface elements was used. The interface allows for displacements between i.e. plate and soil dependent on a given rawness that is pre-defined. Regard this, slipping and gapping between clay and structural objects could occur (van der Sloot, 2020, p.45).

The interface between the concrete slab and clay was set to 0.68 according to Anderberg (2018, p.18).

5.2.1.3 Soil model

Since the model only was used as an estimation of the acting forces, an easy perspicuous soil model was used, Mohr-Coulomb (van der Sloot, 2020, p.8). The Mohr-Coulomb material model is based on linear elastic perfectly-plastic material, where the model is based on five material parameters:

E Modulus of elasticity
c The cohesion of the soil
ψ The dilatancy angle
φ The friction angle
ν Poisson's ratio

The clay was modelled as undrained and according to van der Sloot (2020, p.11), the friction angle should be set equal to zero and the cohesion set equal to c_u, this to enable the undrained behaviour of the material. Therefore, the Mohr-Coulomb failure criterion is reduced to the Tresca failure criterion according to Figure 5.2.

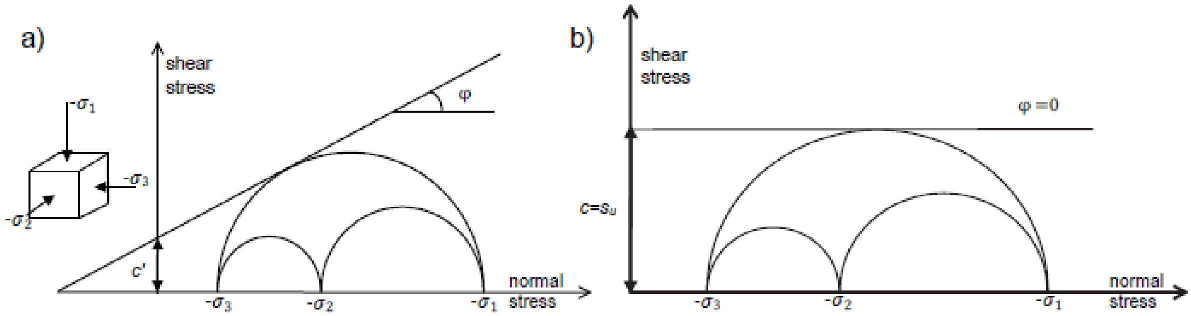

Figure 5.2: *In a) is the Mohr-Coulomb failure criterion and in b) reduced to a Tresca failure criterion (van der Sloot, 2020, p.39).*

5.3 Numerical analytical analysis

As mentioned in the Section 1.2, the objective of the thesis is to end up with a proper and efficient design procedure. This design procedure is an analytical and numerical analysis, that is based on fundamental requirements and parameters that are collected from Eurocode 2. The full design procedure and it's input parameters are found in Appendix E.

However, the design procedure is divided into three different sub-parts:

- Determination of buckling length.
- Definition of design parameters.
- Determination of slenderness and choice of suitable design procedure.
  - The general design procedure, for λ greater than 86.
  - The simplified method, for λ less than 86.

The studied case is designed according to the general procedure, an example for a case designed according to the simplified method is shown in Appendix F. However, these two procedures will be further compared during the analysis.

In the analytical analysis the reference case will be evaluated and further compared with the FE-analysis. Applied axial force is taken from the results obtained from the Plaxis 2D analysis in Section 6.1.

5.3.1 Determine buckling length

Euler's buckling length (l_0), see Section 3.3.1, is evaluated regarding the possibility to reduce the initial length to it's half. The analysis is based on Winter's theory stated in Section 3.2.1.4. The analysis is therefore based on the analogy of bracing, where full bracing is required to obtain a stiffened slab and further the ability to reduce the initial length to its half. The reduction of length is not always possible, therefore is the evaluation of buckling length the first step in the procedure.

5.3.2 Definition of design parameters

The determination of proper buckling length is followed by defining input parameters for the evaluated concrete. The analytical approach enable two choices, pre-set the strength, f_{ck} to a fixed value or specify the curing time. With the defined characteristic compressive strength, remaining necessary design parameters are determined according to Section 3.1.2.

5.3.3 Design Procedure

With a defined buckling length, determination of the slenderness is possible. The slenderness magnitude decides which design procedure should be used. In this last step the slab is designed based on the criterion in Section 3.4.1 respective Section 3.4.2. An additional analysis will be performed to study the differences between the two procedures.

5.3.4 General

When designing the slab it might be necessary to perform two calculations, failure during the excavation phase and failure due to large deformation after a certain time period. Retells Kullingsjö, Chapter 4:

> "The curing time is important, since there will be load concentrations at the end of the slab when the concrete is loaded as a result of the excavated clay alongside the slab. So, the curing time needs to be evaluated and determine when the phased excavation could continue to prevent crushing of concrete."

and Tornborg:

> "It's depends on many factors, but a part of the heave will, depending on e.g. soil characteristics and excavation geometry occur instantaneously, but it's a small part, say in the order of 20% in Gothenburg clay, of the total stress change will transform to heave directly. The remaining heave will take longer time, but the passage of time of the

> total heave is quicker than we thought for a couple of years ago, e.g. if piling has been executed before. The soil permeability and modulus but also e.g. the geometry of the excavation will influence the process of heave, where a large excavation likely will generate a longer time period and vice versa.

When consider failure in the uncured concrete, respect need to be taken to the extra pressure that occur when the next clay strip is excavated. The normal force will then be 50% greater, (if the strips are of equal width), until the next concrete strip has sufficient capacity. This will have a major impact on the design. However, as Tornborg mentioned, only a small part of the bottom heave will occur instantaneously which reduce the risk of buckling.

If it is known that large deformations will occur in a close future, it is recommended to perform a second control. All strips are then cast and the calculation can be preformed per length meter. The concrete will by then have cured for a longer time, which has increased the design strength.

5.4 FE-modelling

For the verification and further evaluation of the behavior of these slabs, some FE-analyses were performed. The software Abaqus CAE is used for the analyses, since it allows for good estimations regard buckling analyses. These analyses were conducted:

- Mesh Convergence study
- Verification of the analytical approach, regard the reference case.
- Sensitivity analysis regarding heave of the bottom.

5.4.1 Material model

The first step is to define the geometry and the material properties for the slab to be analysed. Regard the information that was obtain during the interviews in Chapter 4, the curing of the concrete commonly never reaches 28 day for these slabs, so non-linear properties was hard to define without a physical experiment where the properties are measured. However, to get rid of the problem, an elastic model for the concrete properties was used, where only the modulus of elasticity is defined.

Due to the fact of elastic material model, failure criterion is hard to define for a material with different characteristics in different loading directions. The focus was therefore on stresses in the concrete, where compression and tension stresses were determined, with the aim to determine similarities and differences between the analytical approach and FE-analysis. The determined stresses was thereafter compared with the tensile and compressive strength to ensure that the slab did not cracked or crushed.

5.4.1.1 Solid

The solid element in Abaqus could be used for both linear static and non linear analyses (Simuleon, 2017b). The solid element allows for analyse of the stresses within the element, but also in an easy way provide information regard the maximum tensile and compressing stresses which arises at the upper and lower surface. Regard this, the solid element was used during the analyses, this to make it possible to compare the tensile and compressing stresses with the analytical design procedure.

5.4.2 FE-analyses

When running the model, the whole analyse procedure was divided into two different steps, general eigenvalue buckling and linear static analysis. The eigenvalue buckling is the first analysis to run, where information regard the deformed shape of the structure is captured and saved as an input file for the linear static analysis. Further below, the two analysis procedures are described.

5.4.2.1 General eigenvalue buckling

The general eigenvalue buckling is commonly used to determine the critical buckling load for an axially loaded member. This eigenvalue procedure increments the load stiffness matrix until the stiffness of the model matrix becomes singular (Simuleon, 2017a). For those cases where the the singularities occurs, an eigenvalue is stored i.e. a buckling mode.

As mentioned in Section 5.4.2, information about the deformed shape generated from the buckling analysis is of importance for further analysis. The deformed shape obtained from the eigenvalue one i.e. buckling mode one, is further stored as an input file for the linear static analysis. This deformation was further used with the purpose to apply eccentricities to the system, where the deformed shape was scaled to its prescribed eccentricity. The eigenvalue buckling procedure was therefore used to investigate the influence of imperfections in the system.

5.4.2.2 Linear static analysis

In Section 5.4.2.1 it is stated that the deformed shape from the buckling analysis is saved as an input file and further scaled to implement eccentricities to the system in the linear static analysis. This deformed shape is applied to the system before the element is subjected to load.

In this analysis, the linear general static approach was conducted, where proper boundary conditions and load steps was defined. The load was applied at one side of the slab, where the boundaries allows the structure to move in the loading direction. On the other hand, the displacements at the boundaries were set to zero (fixed).

Since the analysis include history-dependent response of the structure i.e. deformations, Newton's method was used (Simuleon, 2017c). By using this method, the solution is obtained by increment's, where the load was iterated in a series of increments, where the solution is given for equilibrium in each step.

The aim by using this static analysis is to obtain concrete stresses that corresponds to applied static axial loading. This was done, by plotting the increased stress state dependent on the applied (incremented) load. Further, the stresses will be compared with the stresses obtained in the numerical analysis.

5.4.2.3 Mesh convergence study

To ensure a proper mesh size and sufficient equilibrium conditions, a mesh convergence study has been performed. The mesh study was performed to ensure that the results have converged and further obtain reliable results. However, the convergence study was done by analysing the maximum compressive and tensile stress in the mid-section of the beam. This results have further been plotted and compared with different mesh-sizes until the size of error was acceptable.

5.4.3 Sensitivity analysis

The same methodology have been used for the sensitivity study as for the validation, but with the change of increased scaling of the eccentricity when performing the linear static analysis. To capture the impact of slenderness, the same cross section used in the validation have been used, but with different lengths.

An analysis of each eccentricity was executed, where the result of concrete stresses (tensile and compressive) were plotted with corresponding axial force. The observed eccentricities varies between 5 mm - 30 mm in steps of a 5 mm. The slab exposed to 5 mm eccentricity is designed to resist an axial force of 500 kN, where larger eccentricities resulted in a failure load with lower magnitude. Thereafter the behavior of the system will be discussed and evaluated with the purpose to determine the sensitivity regard forced deformations of the bottom of the excavation (eccentricities).

The analogy of the analysis is influenced by Ryner's thought during the interview session, where he mentioned that it would be beneficial to establish a table that itemize load cases dependent on imperfections of the bottom of the excavation (see citation in Section 4.2.4). It was not only Ryner's thoughts that inspired to the analogy, Trygg also stated during the interview, that a sensitivity analysis with predefined forced deformations would be a good approach to catch general behaviours.

6

Results

In this chapter all results will be briefly presented and be further discussed in Chapter 7. The results listed below are obtained from the analytical procedure, FE-analyses in Abaqus and from the analysis made in Plaxis 2D.

6.1 Results from Plaxis 2D

As mentioned previously, the main purpose with the Plaxis 2D analysis is to evaluate and determine a reasonable axial force. This obtained axial force was further used in the analytical and the FE-analysis.

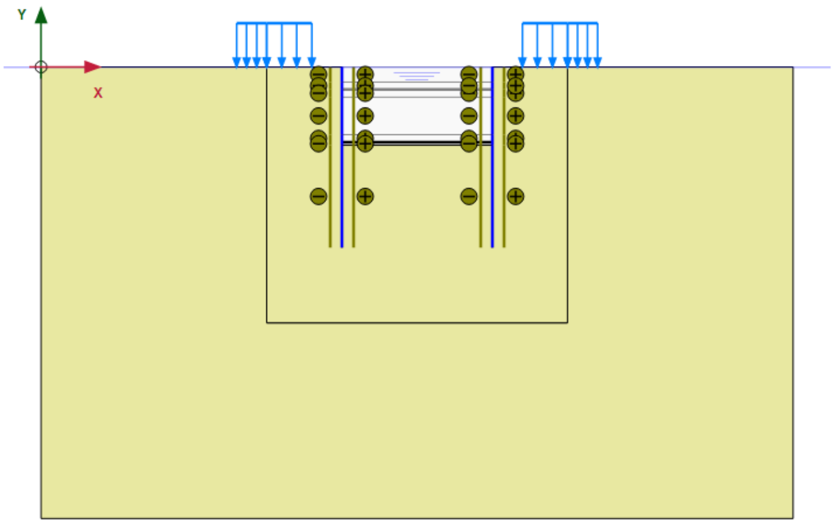

Figure 6.1: *Illustration of the geometry used in Plaxis 2D.*

To ensure reliable results and further small deviations a mesh sensitivity study was performed, where the normal force in the concrete slab was studied for different meshes listed in the Table 6.1 below. However, as seen the results could be assumed to be converged at the level of fine mesh, this since the quota of error between the fine and very fine mesh only reach 0.22%, where the same goes for the difference between fine and very fine mesh. So, the value of the axial force that should be used is, 269.4 kN, but was further rounded up to 270 kN in further analysis in Plaxis 2D.

Mesh quality	Axial force	Elements size
Very coarse	262,3 kN	1635
Coarse	265,9 kN	3206
Medium	267,6 kN	5381
Fine	269,4 kN	10476
Very fine	270,0 kN	17834

Table 6.1: *Acting axial force dependent on mesh.*

6.2 Results from design procedure

As could be seen in Appendix E, where the calculations for the reference case are stored, the buckling length was initial set to 10 m. However, the performed analysis for determination of the buckling length permitted a reduction of the buckling length to it's half, 5m.

Since the evaluated slab was slender, with λ greater than 86, the general procedure was required. The whole procedure is found in Appendix E, and the results of interest are listed in Table 6.2 below.

Denomination	Design stress [MPa]	Design strength [MPa]	Utilization [%]
Compressive stress	4.81	$f_{cd.pl} = 5.33$	90
Tensile stress	0.31	$f_{ctd.pl} = 0.52$	68
Shear stress	0.058	$f_{ctd.pl} = 0.52$	8

Table 6.2: *Concrete stresses determined during the analytical design procedure.*

The results listed in Table 6.2 shows the results obtained from the analytical design procedure. These will further be evaluated and compared to the ones obtained from the FEA results listed in Section 6.3 below.

6.2.1 Difference between simplified method and the general procedure

In Appendix F a slab designed using the simplified method is shown. The geometry of the slab yields, 6 m long, a thickness of 150 mm. The slab is further exposed to forced transverse eccentricity and an axial acting force. The same slab was designed according to the general approach and the difference in needed characteristic strength is shown in Table 6.3 below.

	Simplified method	General procedure
f_{ck} [MPa]	6	5

Table 6.3: *Different characteristic strength for different method used in design procedure.*

6.3 Results from Abaqus

In this section, the results obtained from the performed FEA (Abaqus) are listed, these will further be discussed and evaluated in Chapter 7. As mentioned in Section 5.4.2.1 a buckling analysis was first performed to evaluate the deformed shape of the first buckling mode. Below in Figure 6.2, the result and the deformation of the first buckling mode is shown:

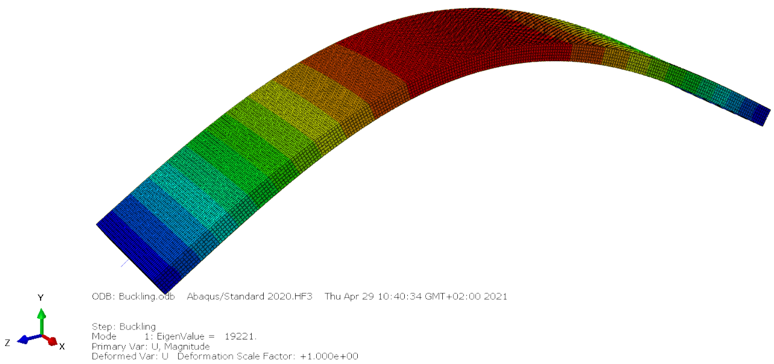

Figure 6.2: *Figure of the deformed shape obtain during the general eigenvalue buckling analysis.*

A linear static analysis was performed, where the load was incremented as stated in Section 5.4.2.2. The resulted behaviour obtained during the analysis is shown in Figure 6.3 below, where the slab is visualized in the final incremental loading step. All the results regard validation and sensitivity that were obtained during the analyses are further found below in the chapter.

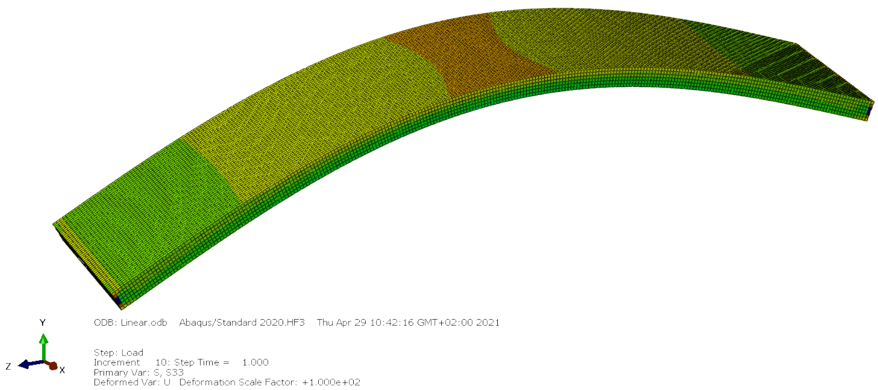

Figure 6.3: *Figure of the analysed slab regard stresses in the final increment.*

6.3.1 Mesh convergence study

To obtain reliable results, the mesh size was evaluated and a couple of analyses for the same load case was performed. The results from these three analyses are listed in Table 6.4 below, where concrete stresses are listed with corresponding mesh.

Tensile stress [MPa]	Compressive stress [MPa]	Number of elements [-]
4.451	13.038	56760
4.453	13.040	75600
4.455	13.041	88128

Table 6.4: *Concrete stresses dependent on mesh.*

As seen in Table 6.4, the result has converged already at a number of elements of 75 600, with an error of only 0.05% for the worst case (tension). With this result, further analyses regard validation and sensitivity was made with a number of elements of 75 600.

6.3.2 Validation of design procedure

To ensure a conservative and safe analytical design procedure, the results obtained regarding the reference case from the analytical analysis is compared to the ones generated from the FE-analysis.

Method	Tensile stress [MPa]	Compressive stress [MPa]	Axial force [kN]
Abaqus	0.18	4.70	270
Analytical	0.31	4.81	270
Error	69 %	2.4 %	-

Table 6.5: *Comparison of result between Abaqus analysis and analytical approach.*

The results obtained from the analytical design procedure based on Eurocode 2 and the literature study gave higher design values regard stresses compared to the FE-analysis. As seen in Table 6.5 above, the compressive stresses obtained from the analyses agree in a reasonable margin of error to each other. The tensile stresses varies a lot as seen in Table 6.5. This could be due to the small magnitude of stresses occur, which implies a more sensitive error. However, it is shown that the analytical design procedure is more conservative and on the safe side. A graph of the results is shown in Figure 6.4. Further, the original size of the figure is found in Appendix G.

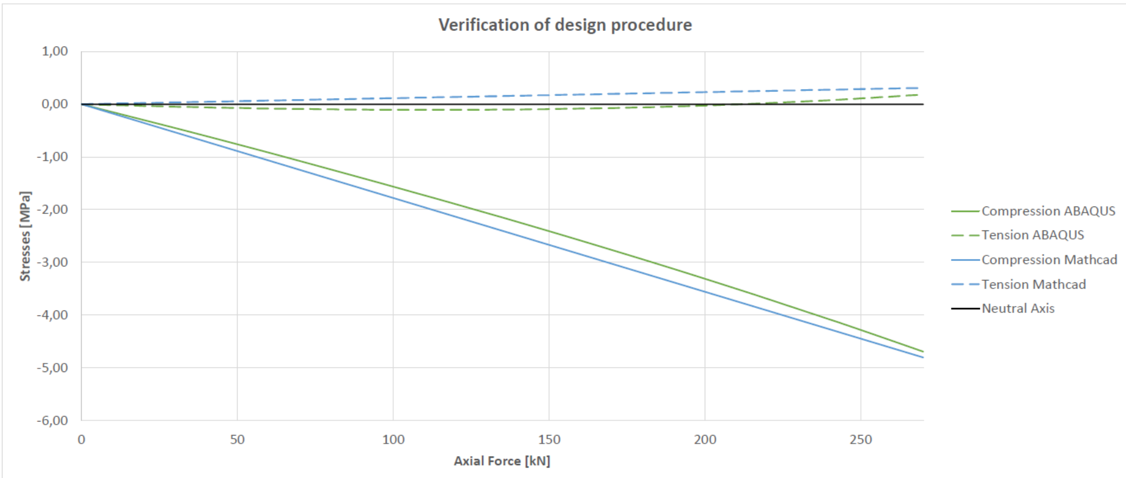

Figure 6.4: *Comparison between the results obtained from the analytical and FEA that validate that the design procedure is conservative and on the safe side.*

6.3.3 Sensitivity analysis

The needed characteristic compressive strength for a 5 m long slab was determined to f_{ck} = 11 MPa to ensure enough capacity when the slab is exposed to 5 mm eccentricity and loaded by 500 kN. The result is found in Table 6.6 and Figure 6.5. Further, the original size of the figure is found in Appendix H.

Eccentricity [mm]	Failure load [kN]	Failure mode
5	500	Crushing
10	425	Crushing
15	370	Crushing
20	325	Crushing
25	225	Cracking
30	150	Cracking

Table 6.6: *Concrete failure modes with corresponding failure load dependent eccentricities for a 5 m long slab.*

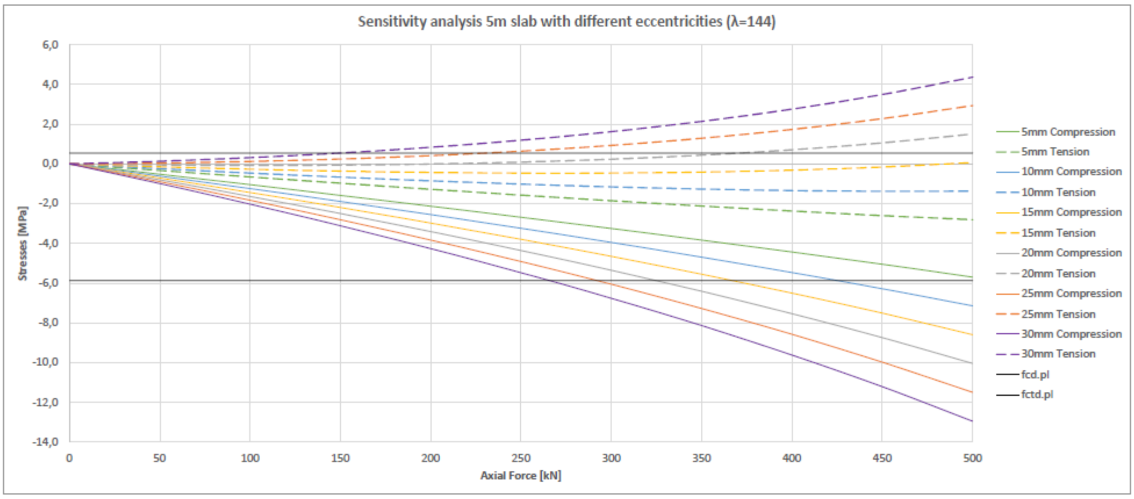

Figure 6.5: *Sensitivity results for a 5 m slab regarding eccentricities, where the concrete stresses are plotted with corresponding normal force.*

The needed characteristic compressive strength for a 6 m long slab was determined to f_{ck} = 12 MPa to ensure enough capacity when the slab is exposed to 5 mm eccentricity and loaded by 500 kN. The result is found in Table 6.7 and Figure 6.6. Further, the original size of the figure is found in Appendix H.

Eccentricity [mm]	Failure load [kN]	Failure mode
5	500	Crushing
10	425	Crushing
15	370	Crushing
20	290	Cracking
25	200	Cracking
30	125	Cracking

Table 6.7: *Concrete failure modes with corresponding failure load dependent eccentricities for a 6 m long slab.*

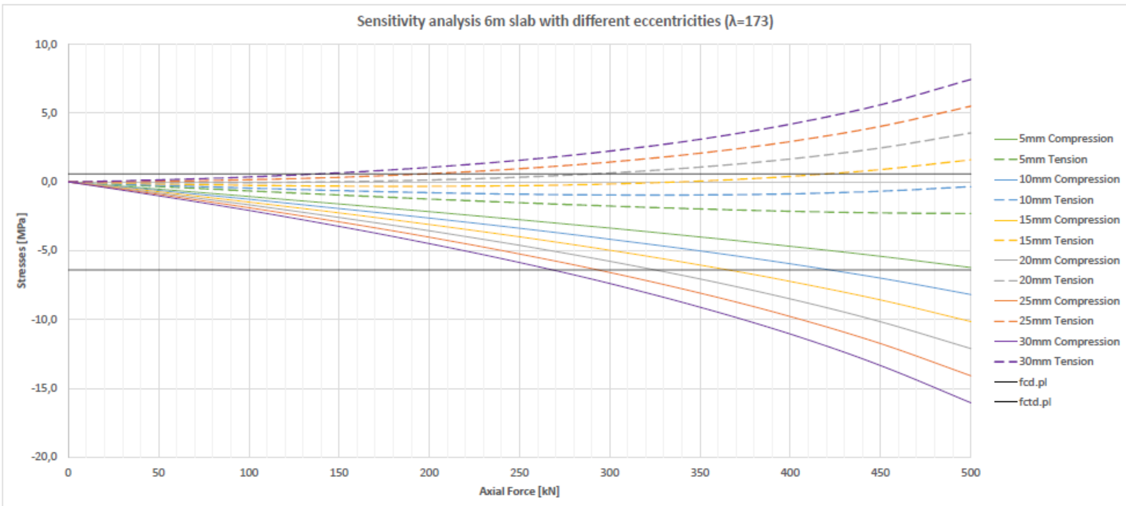

Figure 6.6: *Sensitivity results for a 6 m slab regarding eccentricities, where the concrete stresses are plotted with corresponding normal force.*

The needed characteristic compressive strength for a 7 m long slab was determined to $f_{ck} = 15$ MPa to ensure enough capacity when the slab is exposed to 5 mm eccentricity and loaded by 500 kN. The result is found in Table 6.8 and Figure 6.7. Further, the original size of the figure is found in Appendix H.

Eccentricity [mm]	Failure load [kN]	Failure mode
5	500	Crushing
10	425	Crushing
15	340	Cracking
20	250	Cracking
25	190	Cracking
30	140	Cracking

Table 6.8: *Concrete failure modes with corresponding failure load dependent eccentricities for a 7 m long slab.*

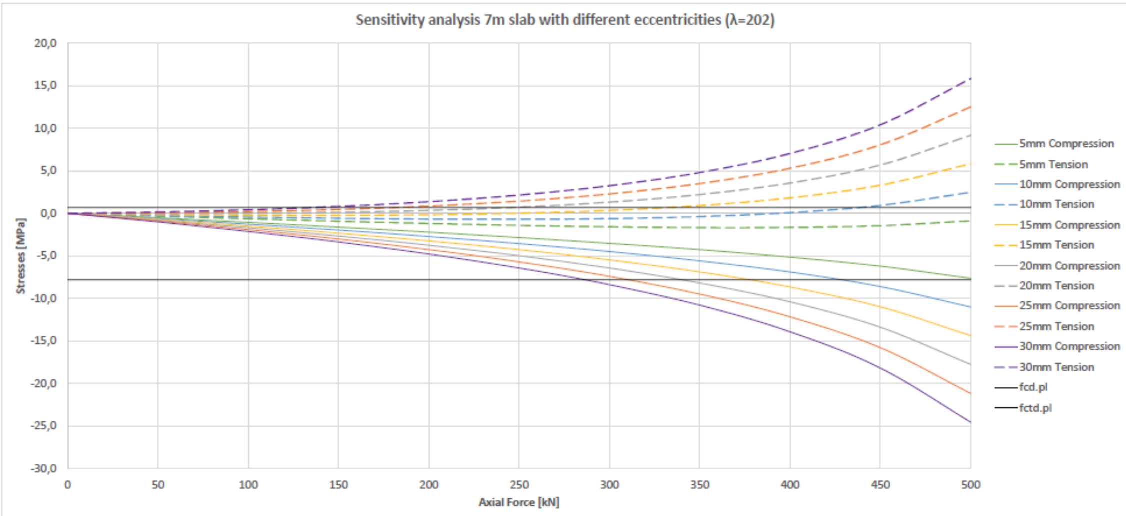

Figure 6.7: *Sensitivity results for a 7 m slab regarding eccentricities, where the concrete stresses are plotted with corresponding normal force.*

7 Discussion

In this chapter the discussions will be linked to the objectives, but also to the results obtained during the analyses.

The main objective has been achieved in this thesis, by established a general design procedure that is based on published theories, new-found knowledge, but also on conversations with professions within the industry. The design procedure has further been validated and ensured to be conservative by using the FE-software, Abaqus.

As mentioned, the aim has been to evaluate the sensitivity of the system regard the interaction between clay and concrete. A way to progress this was discovered during the interviews in Chapter 4, where forced deformations could be applied to the system. Due to this, some analyses was performed with the purpose to determine how the maximum design load varies dependent on eccentricities that would represent unfavourable deformations of the clay.

7.1 Discussion about the final design procedure

While using the design procedure in Appendix E, it is shown that the length of the slab could be reduce to it's half, by using Winter's theory. The idea of using Winter's theory was at the beginning proposed by the supervisor Per Eriksson, that presented it as a common known 2% buckling rule. This buckling percentage has been retold among engineers as a stiffening force that corresponds to 2% of the applied axial load, but there was no founded reference of the expression. However, afterwards working with the theory chapters, Winter's theory about lateral bracing of column and beams was found (derivations in Section 3.2.1.4).

While progressing with Winter's approach as an attempt to enhance an efficient establish of the buckling length, it was discovered that the used method (one bracing force) is suited for middle wide excavations. This was found out while working with the analyses, where the reduction of the buckling length to it's half only is efficient for excavations (length of slabs) less than i.e. 20 m, otherwise the axial load will overcome the critical buckling load in an early stage. However, the use of only one bracing force was implemented and a deeper research of how to reduce the buckling length even more was added to the future research. Below in Section 7.3.1 an example of how to work with the future problem stated.

The results obtained from the analytical design procedure based on Eurocode 2 and the literature study gave higher design values regard stresses compared to the FE-analysis. In Table 6.5 it is shown that the analytical design procedure is more conservative and on the safe side, which confirms that the main objective has been achieved in the thesis. However, there is still potential for improvements and further research, some ideas of this could be found below in Section 7.3.

As has been explained in the thesis and can be seen in Appendix E, the procedure includes two different ways to design the slab, the general design procedure and the simplified method. During the numerical analysis it was observed that in most cases the general approach was recommended to be used due to the magnitude of slenderness ratio, and thoughts of exclude the simplified method came up. It can be seen in Section 6.2.1 that the simplified method gives a more conservative design than the general approach. However, the general design procedure has been confirmed by Abaqus CAE in Section 6.3.2 to be enough conservative. The decision to only use the general procedure has been made since it allows for more optimized and efficient slabs. The final design procedure will therefore be in accordance with Appendix E.

A final remark about the established design procedure is that it only take plain concrete into consideration i.e. the procedure doesn't take any reinforcement into account. This was concluded during the interview session, where newfound knowledge was gained. It can be read in Chapter 4, that almost 9 out of 10 design cases are designed without reinforcement and that reinforcement only is used in those cases where the slab will be loaded with heavy vertical loads (i.e machines) inside the excavation. However, this mode of action is delimited since it is favourable effects that will push the slab downwards the clay, retells again Kullingsjö, from Chapter 4:

> "Commonly our goal is to use plain concrete. We just use reinforcement when it's especially needed, for example when we know that the slab will face heavy vertical loads and further tends to buckle downwards in the underlying soft clay. For those cases we use and prescribe reinforcement"

7.2 Discussion about the sensitivity analysis

As an attempt to evaluate the influence of the clay, while designing the concrete slab a sensitivity analysis was performed in Abaqus, where the results can be found in Section 6.3.3. The chosen approach was primarily proposed by Trygg during the interview session, retells again Trygg, from Chapter 4:

> "So, to catch the general trends a sensitivity analysis with predetermined forced deformations is a good way to evaluate and determine the worst case scenarios"

Moreover, the results is reasonable, where the maximum design load is decreased due to increased forced deformation. This results could be observed in Figures 6.5 - 6.7 and Tables 6.6 - 6.8, where three different lengths of slabs are analysed regard increased imperfections. A remark is that the more slender slabs is to a greater extent affected in a negative way, where the capacity is decreased quicker.

A final remark in this discussion about sensitivity, the choice of imperfection is crucial since only 5mm larger imperfection reduce the capacity in compression from 500 kN to 425 kN, which is a reduction of 15%.

7.3 Improvements and future research

During the study, delimitations, new way of thinking and newfound insights have occurred. Some of these have inspired the final result, but some of them have consciously been left out of the work with the purpose to study them deeper in future researcher work. Some of the affected are discussed and listed below in this section.

7.3.1 Further development of Winter's approach

According to Yura (1996, p.822), Winter's model could be expanded with several more bracing-nodes, see Figure 7.1. By using the information stated by Yura (1996, p.822), an even more efficient buckling length could be established in the future, where the number of possible bracing nodes is evaluated. Moreover, the expanded approach could maybe then enable even thinner slabs then the final established one, where money, material and further the environment are saved.

In Appendix I a further reduction of the buckling length can be found using Winter´s theory several times. The buckling length is reduced by half until the required moment exceeds the bracing moment from the self weight. By this the design procedure can be efficient for larger excavations as well. However, the eccentricities is here assumed to vary linear, which is not completely reliable since depended on heave, the forced deformation from the clay will vary differently. With a linear eccentricity the same buckling percentage could be considered for all reduction phases, while for nonlinear eccentricities the buckling percentage needs to be evaluated for each reduction phase. Important to mention is that this method is not verified and therefore not included in the final design procedure. This reduction method needs to be further evaluated and compared with Winter´s extended version, this to ensure it's validity.

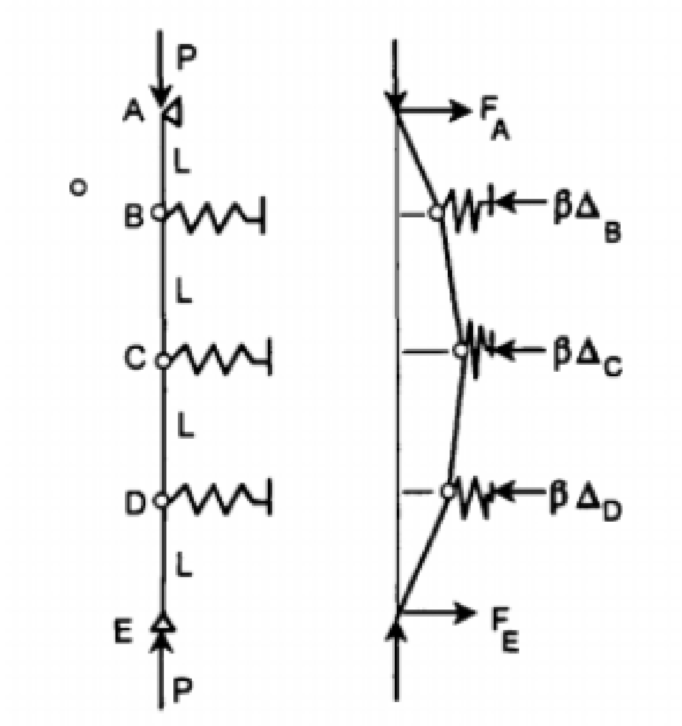

Figure 7.1: *Expanded version of Winter's approach according to, Yura (1996, p.823).*

7.3.2 Physically experiments

As mentioned in the Section 4.2.4, Wallgren stated a proposal of a physically experiment, where the mode of action should be proper evaluated, this to provide the industry with fundamental information about how the slab is acting during loading. Retells again Wallgren, from Chapter 4:

> "A case study with two sheet pile walls and a 10 m long concrete slab that is loaded up to failure by the use of hydraulic jack would be of interest to study. This to make it possible to derive the analytical part from and further prove the chosen mode of action"

By evaluate the mode of action during loading, the "real" behaviour could be determined, this to determine how the slab would buckle, i.e. sinusoidal nodes. Moreover, the interaction between the clay and concrete should be evaluated, this to determine i.g. how favourable the adhesion and the negative pore pressure are and if this phenomenons could be derived as bracing and stiffening of the slab.

In Figure 7.2, the future set-up of a possible experiment is shown. As can be seen, the slab is located between two hydraulic jacks, with the purpose to compress the slab until failure. During the progress of loading, measurement of the deformations should be done (i.g. inclinometer measurement) with the purpose to determined the mode of deformation while/if buckling occurs.

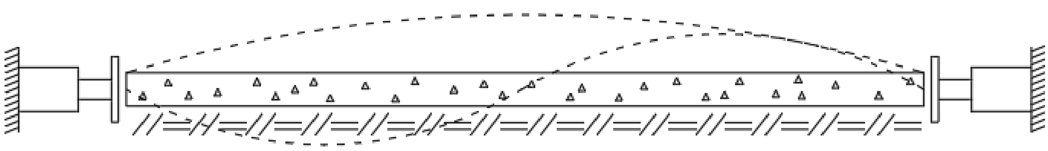

Figure 7.2: *Illustration of the geometry of a possible future physically experiment.*

Moreover, the adhesion and the negative pore pressure could be of interest, this since they are favourable actions (in this case) and could further reduce the buckling length even more. However, it would be favourable to determine these effects by lifting the slab in the vertical direction to evaluate applied force required to separate the concrete from the clay. This applied force could hence be used to derive a "suction" factor that work as a stiffening (bracing) and prevent the slab to buckle up-wards.

7.3.3 Non-linear analysis

During the FEA working with Abaqus, a simplified evaluation of the results was made, where the stresses obtain from the analysis was compared to the strength of the observed concrete. This course of action was performed since the use of an linear elastic material didn't provide the analysis with enough information regard the failure of concrete. As mentioned in Section 3.3.2, it is stated that models based on plasticity and fracture mechanisms gives a more proper estimation about the mode of action of the material when it's tend to fail. So, it would be of interest to evaluate material parameters needed to perform a non-linear analysis in Abaqus and compare the results from a better adapted material model with the used linear elastic and further validate the analytical procedure.

This was neglected during the study since the analysed concrete was not fully cured and therefore the input parameters regard non-linear models was missing. However, would it be beneficial to do more research in the future regard this, especially determine the mode of action right after the first induced crack occur.

7.3.4 Influence of geotechnics

The influence of geotechnical properties was at the beginning intended to influence the design procedure in a more advanced manner, than it has done. During the literature study, but also during the interview sessions it became clear that the scope of the influence regard geotechnics was really complex and hard to determine in only one master thesis. To handle this, which can be read, only forced deformations from the clay have been used.

As an attempt to do the design procedure even more reliable, a procedure to determine a reasonable magnitude of the heave of the clay should be established, this since its now taken based on experience. The use of experience is however of-course really important, but to establish a general procedure it would beneficial to establish a method that could be used by i.g. inexperienced engineers as well. So, a final future research that is recommended, is to study the heave of unloaded clay and try to end up with a procedure that could determine the magnitude of the heave based on i.g. the shear strength.

Moreover, as mentioned in Section 7.3.2, the influence of adhesion and negative pore pressure needs to be further investigated.

8
Conclusion

The aim of this thesis was to evaluate the interaction, sensitivity and behavior between geotechnical and constructional properties. This to make it possible to identify factors that influence the design of provisional concrete slabs and further establish a suitable design procedure that could be used as a general design approach. This final chapter gives the conclusions of the performed study.

One of the most important and astounding knowledge gained during the work with the thesis, is that the concerns stated in Section 1.1 about a disunite industry is confirmed, and therefore the need of a general design procedure that could facilitate for the companies. Regard this, this new-produced design procedure could be a movement in right direction to facilitate for the companies within the industry.

Due to the fact stated in Section 7.2, the eccentricities affect the capacity in a crucial way. Regard this, a conservative approach is required when determine the forced deformations regard heave and skew bottom of the excavation. To handle this in a proper way, and further ensure right chosen imperfections while performing the design, is to inspect the bottom of the excavation at site before the concrete is casted. This could prevent that insufficient parameters is used and further ensure a safe working environment.

What characterizes the study is that uncertainties and limitations occurs in almost every step, where fundamental properties needs to be carefully chosen. As mentioned in Chapter 7, further research and work could be further improved, i.g. the determination of the buckling length. Regardless this, the established design procedure is a movement in right directions, since it can be used as a general and safe procedure while progress the design of plain concrete slabs used during excavations.

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A

Appendix 1 - Calculation buckling percentage

INPUT DATA

```
clc
clear all

L_0 = 10;
L = L_0 / 2;
b = 1;
t = 0.15;

E = 8e9;
I = (b * t^3) / 12;
```

LOAD

```
Pe = (pi^2 * E * I) / L^2;           % BUCKLING LOAD

P = linspace(0.0001 , Pe , 1000);
```

IDEAL BRACE STIFFNESS

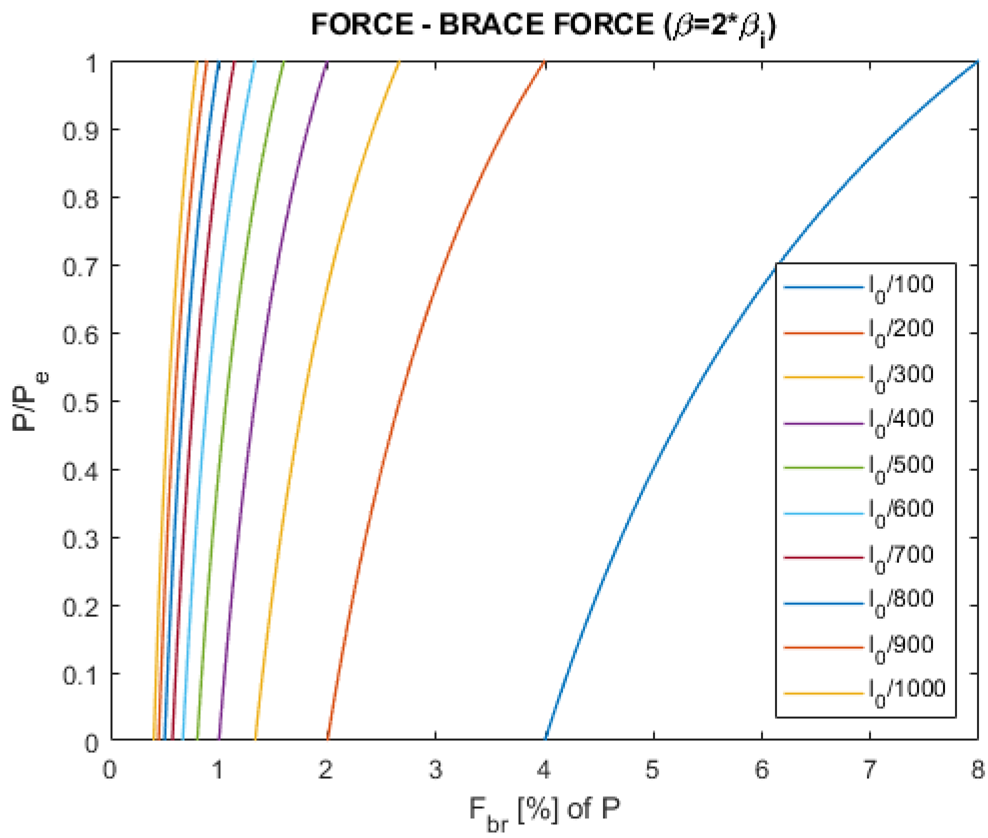
```
beta_i = (2 * Pe) / L;               % IDEAL BRACE STIFFNESS
```

PLOTS

```
% INITIAL ECCENTRICITY
delta_0 = [L/50 L/100 L/150 L/200 L/250 L/300 L/350 L/400 L/450 L/500];

% -------------------------------------------------------------------%
% FORCE - DISPLACEMENT
%
% DETERMINE THE TOTAL DISCPLACEMENT BY USING EQUATION (3) IN Yura (1996).
% -------------------------------------------------------------------%

delta_t = zeros ( length(P) , length(delta_0));

force = zeros (length(P) , 1);

for i = 1 : length(P)

    force(i) = P(i) / Pe;

    for j = 1 : length(delta_0)

    delta_t(i,j) = delta_0(j) / (1 - (2 * P(i)) / (2*beta_i * L));

    end
```

```
end

% -----------------------------------------------------------------%
% FORCE - BRACE FORCE
%
% DETERMINE THE BRACE FORCE BY USING EQUATION (4) IN Yura (1996).
% -----------------------------------------------------------------%

fbr = zeros ( length(P) , length(delta_0));

PERCENTAGE = zeros ( length(P) , length(delta_0));

for i = 1 : length(P)

    for j = 1 : length(delta_0)

    fbr(i,j) = (2 * P(i) / L) * delta_t(i,j);

    PERCENTAGE(i,j) = (fbr(i,j) / P(i)) * 100;

    end

end

for i = 1 : length(delta_0)

    figure(1)
    plot(PERCENTAGE(:,i) , force , 'linewidth' , 1)
    hold on

end
    axis([0 8 0 1])
    xlabel('F_{br} [%] of P')
    ylabel('P/P_e')
    legend('l_0/100', 'l_0/200', 'l_0/300', 'l_0/400', 'l_0/500',...
        'l_0/600', 'l_0/700', 'l_0/800', 'l_0/900', 'l_0/1000',...
        'Location', 'SouthEast')
    title('FORCE - BRACE FORCE (\beta=2*\beta_i)')



ECCENTRICITY = {'L/50' ; 'L/100' ; 'L/150' ; 'L/200' ; 'L/250'...
             ; 'L/300' ; 'L/350' ; 'L/400' ; 'L/450' ; 'L/500'};

ECCENTRICITY_0 = {'L_0/100' ; 'L_0/200' ; 'L_0/300' ; 'L_0/400'...
               ; 'L_0/500' ; 'L_0/600' ; 'L_0/700' ; 'L_0/800}'...
               ; 'L_0/900' ; 'L_0/1000'};

PERCENTAGE = [PERCENTAGE(end,1) ; PERCENTAGE(end,2) ; PERCENTAGE(end,3)...
            ; PERCENTAGE(end,4) ; PERCENTAGE(end,5) ; PERCENTAGE(end,6)...
            ; PERCENTAGE(end,7) ; PERCENTAGE(end,8) ; PERCENTAGE(end,9)...
            ; PERCENTAGE(end,10)];


BucklingPercentage = table(ECCENTRICITY , PERCENTAGE)

BucklingPercentage = table(ECCENTRICITY_0 , PERCENTAGE)
```

```
BucklingPercentage =

  10×2 table

    ECCENTRICITY    PERCENTAGE
    ____________    __________

     {'L/50' }            8   
     {'L/100'}            4   
     {'L/150'}       2.6667   
     {'L/200'}            2   
     {'L/250'}          1.6   
     {'L/300'}       1.3333   
     {'L/350'}       1.1429   
     {'L/400'}            1   
     {'L/450'}      0.88889   
     {'L/500'}          0.8   


BucklingPercentage =

  10×2 table

    ECCENTRICITY_0    PERCENTAGE
    ______________    __________

     {'L_0/100' }            8   
     {'L_0/200' }            4   
     {'L_0/300' }       2.6667   
     {'L_0/400' }            2   
     {'L_0/500' }          1.6   
     {'L_0/600' }       1.3333   
     {'L_0/700' }       1.1429   
     {'L_0/800}'}            1   
     {'L_0/900' }      0.88889   
     {'L_0/1000'}          0.8   
```



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B

Appendix 2 - Extract from Al-Emrani et. al (2013) - λ_{lim}

B358

Ett grovt och konservativt värde på gränsvärdet λ_{lim} kan uppskattas som

$$\lambda_{\text{lim}} = \frac{20 \cdot 0{,}7 \cdot 1{,}1 \cdot 0{,}7}{\sqrt{n}} = \frac{10{,}8}{\sqrt{n}} \qquad \text{(B11-11)}$$

Enligt EC 2 är λ_{lim} en nationell parameter, men rekommenderat värde kan bestämmas som

$$\lambda_{\text{lim}} = \frac{20 \cdot A \cdot B \cdot C}{\sqrt{n}} \qquad \text{(B11-12)}$$

där $n = \frac{N_{Ed}}{f_{cd} \cdot A_c}$ (relativ normalkraft)

$A = \frac{1}{1 + 0{,}2\varphi_{ef}}$ (om φ_{ef} inte är känd, kan antas $A = 0{,}7$)

φ_{ef} = effektivt kryptal (*effective creep factor*)

$B = \sqrt{1 + 2\omega}$ (om ω inte är känd, kan antas $B = 1{,}1$)

$\omega = \frac{f_{yd} A_s}{f_{cd} A_c}$ (mekaniskt armeringsinnehåll)

(*mechanical reinforcement ratio*)

A_s = total tvärsnittsarea hos längsgående armering

$C = 1{,}7 - r_m$ (om r_m inte är känd kan antas $C = 0{,}7$)

$r_m = \frac{M_{01}}{M_{02}}$ (förhållande mellan första ordningens moment vid ändarna, $|M_{02}| \geq |M_{01}|$)

$r_m = 1$ för icke avstyvade konstruktionsdelar i allmänhet och för avstyvade konstruktionsdelar där första ordningen moment huvudsakligen beror på imperfektioner eller transversell last

Om ändmomenten M_{01} och M_{02} ger dragning på samma sida ska r_m sättas positiv annars negativ, se fig. B11.12.

C

Appendix 3 - Input parameters to Plaxis 2D

In this chapter the input parameter for the Plaxis 2D analysis is listed.

C.1 Soil parameters

Table C.1: *Clay parameters.*

Parameter	Value	Comment
E	$320 \cdot c_u$	Conservative chosen
ν	*0.4 [-]*	-
c_u	*(15 + $(1 \cdot Z_0)$) [kPa]*	Increasing strength with depth
ψ	*0*	Due to little dilatancy in clay
φ	*0*	Due to undrained modelling
k_x	$1 \cdot 10^{-4}$ *[m/day]*	Horizontal permeability
k_y	$1 \cdot 10^{-4}$ *[m/day]*	Vertical permeability

C.2 Structural elements

Table C.2: *Sheet pile walls parameters plate like.*

Parameter	Value	Comment
EA	$3.08 \cdot 10^6 [kN]$	
EI	$6.9 \cdot 10^4 [kN \cdot m^2]$	
w	1.15 [kN/m]	

Table C.3: *Concrete parameters node-to-node.*

Parameter	Value	Comment
EA	$3,6 \cdot 10^6 [kN]$	Using 30 [GPa] as concrete stiffness

D

Appendix 4 - Input parameters to the reference case

The geometry is illustrated in Figure D.1, where the depth of the excavation is 5 m, the length of the sheet pile walls are 13 m and the width of the shaft is 10 m. According to the width of the shaft, the concrete slab will obtain the same length ie 10 m.

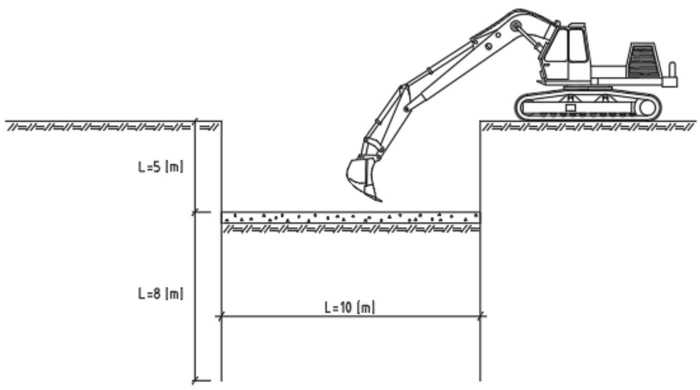


Figure D.1: *Own illustration of reference excavation.*

D.1 Used parameters

In this section the input parameter for the Abaqus and the analytical analyse are listed.

Table D.1: *Concrete properties*

Parameter	Value	Comment
E_c	26 GPa	-
ν	*0.2 [-]*	-

D.1.1 Load and forced displacement

Table D.2: *Load and forced displacement*

Parameter	Value
Axial force	$270[kN]$
Forced displacement	$20[mm]$

E

Appendix 5 - Design procedure for reference case

Deisgn procedure for reference case

Input data

$b := 1m$ Width of slab

$t := 120mm$ Slab thickness

$l_0 := 10m$ Buckling length according to Eulers

$N_{Ed} := 270kN$ Design axial force from sheet pile walls

$\rho_c := 23\frac{kN}{m^3}$ Density concrete, favourble effects

$e_h := 20mm$ Eccenricity in midsection due to bottom heave

$I_c := \frac{b \cdot t^3}{12} = 1.44 \times 10^{-4} \cdot m^4$ Moment of inertia for the concrete cross-section

$A_c := b \cdot t = 0.12\,m^2$ Concrete area

Initial eccentricities

Unintended eccentricity due to imperfections

$\alpha_m := 1$ Only the slab member contributing to the total effect

$$\alpha_h := \begin{cases} \frac{2}{3} & \text{if } \frac{2}{3} \geq \frac{2m^{0.5}}{\sqrt{l_0}} \\ 1 & \text{if } 1 \leq \frac{2m^{0.5}}{\sqrt{l_0}} \\ \frac{2m^{0.5}}{\sqrt{l_0}} & \text{otherwise} \end{cases} = 0.667$$

$\theta_i := 0.005 \cdot \alpha_h \cdot \alpha_m = 3.333 \times 10^{-3}$

$e_i := \theta_i \cdot \frac{l_0}{2} = 16.667 \cdot mm$

$e_i := e_i = 16.667 \cdot mm$ Change if larger initial imperfection than Eurocode prescibe want be considered, otherwise set e_i=e_i.

Intended eccentricity due to external forces

$M_{Ed} := 0$

The transversal load from bottom heave is here considered as a eccentricity.

$e := 0mm$

The axial force is assumed to act in the gravity of center

$$e_0 := \frac{M_{Ed} + N_{Ed} \cdot e}{N_{Ed}} = 0 \cdot mm$$

Total eccentricity

$$e_{tot} := e_i + e_0 + e_h = 36.667 \cdot mm$$

$$A := \frac{l_0}{e_{tot}} = 272.727$$

The total initial eccentricity is $\Delta_0 = l_0 / A$

Determine buckling length based on bracing force

Δ_0 function of l_0	Buckling percentage $B_{\%}$ [%]
$l_0/100$	8
$l_0/200$	4
$l_0/300$	2.7
$l_0/400$	2
$l_0/500$	1.6
$l_0/600$	1.3
$l_0/700$	1.1
$l_0/800$	1
$l_0/900$	0.9
$l_0/1000$	0.8

Choose the buckling percentage from the table above based on the initial eccentricity

$B_{\%} := 2.7\%$

$F_{bracing} := B_{\%} \cdot N_{Ed} = 7.29 \cdot kN$

Bracing force dependent on buckling percentage

$M_{required} := \frac{F_{bracing} \cdot l_0}{4} = 18.225 \cdot kN \cdot m$

Moment required to achieve full bracing.

$g := \rho_c \cdot t \cdot b = 2.76 \cdot \frac{kN}{m}$

$M_g := \frac{g \cdot {l_0}^2}{8} = 34.5 \cdot kN \cdot m$

Moment from self-weight

$L := \begin{cases} \frac{l_0}{2} & \text{if } M_g \geq M_{required} \\ l_0 & \text{otherwise} \end{cases} = 5 \cdot m$

If the moment from the self weight is greater than the required moment, reduce Eulers buckling length by half

$e_{tot} := \begin{cases} e_{tot} & \text{if } L = l_0 \\ \frac{e_{tot}}{2} & \text{otherwise} \end{cases} = 18.333 \cdot mm$

Assume eccentricities varies linear. If the buckling length is reduced by half, reduce the eccentricity by half

Design strength parameters

Input data

$Initial_{strength} := 2$

If the curing time is to be considered, choose 1
If a fix strength is prescribed, choose 2

$f_{ck} := 12MPa$

If 1 is chosen above: set characteristic compressive strength after 28 days.
If 2 is chosen above: set the prescribed characteristic compressive strength.

With consideration of curing time

$\alpha_{cc.pl} := 0.8$ $\alpha_{ct.pl} := 0.8$ $\gamma_C := 1.5$ $\gamma_{cE} := 1.2$

$f_{cm} := f_{ck} + 8MPa = 20 \cdot MPa$

$$s = 0.2 \text{ CEM 42,5 R, CEM 52,5 N and CEM 52,5 R (Class R)}$$
$$s = 0.25 \text{ CEM 32,5 R, CEM 42,5 N (Class N)}$$
$$s = 0.38 \text{ CEM 32,5 N (Class S)}$$

$s := 0.25$

$t_1 := 3$ Curing time in days

$$\beta_{cc} := \exp\left[s \cdot \left[1 - \left(\frac{28}{t_1}\right)^{\frac{1}{2}}\right]\right] = 0.598$$

$f_{cm.t} := \beta_{cc} \cdot f_{cm} = 11.965 \cdot MPa$ Mean compressive strength at t days

$f_{ck.t} := f_{cm.t} - 8MPa = 3.965 \cdot MPa$ Characteristic compressive strength at t days

$$f_{ck.t} := \begin{cases} f_{ck.t} & \text{if } Initial_{strength} = 1 \\ f_{ck} & \text{otherwise} \end{cases} = 12 \cdot MPa$$

Design compressive strength used when determine other needed properties.

$f_{cm.t} := f_{ck.t} + 8MPa = 20 \cdot MPa$ Mean compressive strength

$E_{cm.t} := 22GPa \cdot \left(\frac{f_{cm.t}}{10MPa} \right)^{0.5} = 31.113 \cdot GPa$ Mean modulus of elascticity

$f_{ctm.t} := 0.3 \cdot \left(\frac{f_{ck.t}}{MPa} \right)^{\frac{2}{3}} \cdot MPa = 1.572 \cdot MPa$ Mean tensile strength

$f_{ctk.0.05} := 0.7 \cdot f_{ctm.t} = 1.101 \cdot MPa$ Characteristic tensile strength, lower bound

$f_{cd.pl} := \alpha_{cc.pl} \cdot \frac{f_{ck.t}}{\gamma_C} = 6.4 \cdot MPa$ Design compressive strength

$f_{ctd.pl} := \alpha_{ct.pl} \cdot \frac{f_{ctk.0.05}}{\gamma_C} = 0.587 \cdot MPa$ Design tensile strength

$E_{cd} := \frac{E_{cm.t}}{\gamma_{cE}} = 25.927 \cdot GPa$ Design modulus of elasticity

Slenderness

$i := \sqrt{\frac{I_c}{A_c}} = 0.035\,m$

$\lambda := \frac{L}{i} = 144.338$

"USE SIMPLIFIED PROCEDURE" if $\lambda \leq 86$ = "USE GENERAL PROCEDURE"
"USE GENERAL PROCEDURE" otherwise

General procedure - Plain concrete

Design moment

First order effects

$M := 0$

Moment due to transversial load. If the buckling length not is reduced by half, add moment from self weight as a negative value.

$M_0 := M + N_{Ed} \cdot (e_{tot}) = 4.95 \cdot kN \cdot m$

First order moment

Second order effects

$$N_B := \frac{\pi^2 \cdot E_{cd} \cdot I_c}{L^2} = 1.474 \times 10^3 \cdot kN$$

$\beta := 1$

Beta is equal 1 for this kind of problem. Expression is derived on page K21, in bärande konstruktioner, del2.

Total moment

$$M_{Ed} := M_0 \cdot \left[1 + \frac{\beta}{\left(\frac{N_B}{N_{Ed}} - 1 \right)} \right] = 6.06 \cdot kN \cdot m$$

$$\begin{vmatrix} \text{"OK"} & \text{if } N_{Ed} \le N_B \\ \text{"NOT OK"} & \text{otherwise} \end{vmatrix} = \text{"OK"}$$

Check of the buckling capacity reffering to Euler's linear buckling cas.

Slab capacity

Linear elastic analysis assuming uncracked section.

Controlling compressive stresses

$\sigma_{ccN} := \frac{-N_{Ed}}{A_c} = -2.25 \cdot MPa$ — Stress due to axial force

$\sigma_{ccM} := \frac{M_{Ed}}{I_c} \cdot \left(\frac{-t}{2}\right) = -2.525 \cdot MPa$ — Stress due to moments

$\sigma_{cc} := \sigma_{ccN} + \sigma_{ccM} = -4.775 \cdot MPa$ — Total stress

$\left| \begin{array}{ll} \text{"OK"} & \text{if } |\sigma_{cc}| \le f_{cd.pl} \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right. = \text{"OK"}$

Controlling tensile stresses

$\sigma_{ctN} := \frac{-N_{Ed}}{A_c} = -2.25 \cdot MPa$ — Stress due to axial force

$\sigma_{ctM} := \frac{M_{Ed}}{I_c} \cdot \left(\frac{t}{2}\right) = 2.525 \cdot MPa$ — Stress due to moments

$\sigma_{ct} := \sigma_{ctN} + \sigma_{ctM} = 0.275 \cdot MPa$ — Total stress

$\left| \begin{array}{ll} \text{"OK"} & \text{if } \sigma_{ct} \le f_{ctd.pl} \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right. = \text{"OK"}$

Check of shear capacity

$z_{in} := t = 0.12\,m$ — Internal level arm

$V_{Ed} := F_{bracing} = 7.29 \cdot kN$ — Design shear force regard fully braced member

$\tau_{max} := \frac{V_{Ed}}{1m \cdot z_{in}} = 0.061 \cdot MPa$ — Design shear stress, regard web shear crack (plain concrete)

$\tau_{Rd} := f_{ctd.pl} = 0.587 \cdot MPa$ — Shear capacity, lower bound

$\left| \begin{array}{ll} \text{"OK"} & \text{if } \tau_{max} \le \tau_{Rd} \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right. = \text{"OK"}$

Check local crushing

$N_{Rd} := f_{cd.pl} \cdot A_c = 768 \cdot kN$

$\left| \begin{array}{ll} \text{"OK"} & \text{if } N_{Ed} \le N_{Rd} \\ \text{"NOT OK"} & \text{otherwise} \end{array} \right. = \text{"OK"}$

F

Appendix 6 - Design procedure - Simplified method

Design procedure - simplified method

Input data

$b := 1m$ Width of slab

$t := 150mm$ Slab thickness

$l_0 := 6m$ Buckling length according to Eulers

$N_{Ed} := 270kN$ Design axial force from sheet pile walls

$\rho_c := 23 \frac{kN}{m^3}$ Density concrete, favourble effects

$e_h := 8mm$ Eccenricity in midsection due to bottom heave

$I_c := \frac{b \cdot t^3}{12} = 2.812 \times 10^{-4} \cdot m^4$ Moment of inertia for the concrete cross-section

$A_c := b \cdot t = 0.15\, m^2$ Concrete area

Initial eccentricities

Unintended eccentricity due to imperfections

$\alpha_m := 1$ Only the slab member contributing to the total effect

$$\alpha_h := \begin{cases} \frac{2}{3} & \text{if } \frac{2}{3} \geq \frac{2m^{0.5}}{\sqrt{l_0}} \\ 1 & \text{if } 1 \leq \frac{2m^{0.5}}{\sqrt{l_0}} \\ \frac{2m^{0.5}}{\sqrt{l_0}} & \text{otherwise} \end{cases} = 0.816$$

$$\theta_i := 0.005 \cdot \alpha_h \cdot \alpha_m = 4.082 \times 10^{-3}$$

$$e_i := \theta_i \cdot \frac{l_0}{2} = 12.247 \cdot mm$$

$e_i := e_i = 12.247 \cdot mm$ Change if larger initial imperfection than Eurocode prescibe want be considered, otherwise set e_i=e_i.

Intended eccentricity due to external forces

$M_{Ed} := 0$

The transversal load from bottom heave is here considered as a eccentricity.

$e := 0mm$

The axial force is assumed to act in the gravity of center

$$e_0 := \frac{M_{Ed} + N_{Ed} \cdot e}{N_{Ed}} = 0 \cdot mm$$

Total eccentricity

$$e_{tot} := e_i + e_0 + e_h = 20.247 \cdot mm$$

$$A := \frac{l_0}{e_{tot}} = 296.334$$

The total initial eccentricity is $\Delta_0 = l_0 / A$

Determine buckling length based on bracing force

Δ_0 function of l_0	Buckling percentage $B_{\%}$ [%]
$l_0/100$	8
$l_0/200$	4
$l_0/300$	2.7
$l_0/400$	2
$l_0/500$	1.6
$l_0/600$	1.3
$l_0/700$	1.1
$l_0/800$	1
$l_0/900$	0.9
$l_0/1000$	0.8

Choose the buckling percentage from the table above based on the initial eccentricity

$B_{\%} := 2.7\%$

$F_{bracing} := B_{\%} \cdot N_{Ed} = 7.29 \cdot kN$

Bracing force dependent on buckling percentage

$M_{required} := \frac{F_{bracing} \cdot l_0}{4} = 10.935 \cdot kN \cdot m$

Moment required to achieve full bracing.

$g := \rho_c \cdot t \cdot b = 3.45 \cdot \frac{kN}{m}$

$M_g := \frac{g \cdot l_0^{\ 2}}{8} = 15.525 \cdot kN \cdot m$

Moment from self-weight

$$L := \begin{vmatrix} \frac{l_0}{2} \ \text{ if } M_g \geq M_{required} \\ l_0 \ \text{ otherwise} \end{vmatrix} = 3 \cdot m$$

If the moment from the self weight is greater than the required moment, reduce Eulers buckling length by half

$$e_{tot} := \begin{vmatrix} e_{tot} \ \text{ if } L = l_0 \\ \frac{e_{tot}}{2} \ \text{ otherwise} \end{vmatrix} = 10.124 \cdot mm$$

Assume eccentricities varies linear. If the buckling length is reduced by half, reduce the eccentricity by half

Design strength parameters

Input data

$Initial_{strength} := 2$

If the curing time is to be considered, choose 1
If a fix strength is prescribed, choose 2

$f_{ck} := 6MPa$

If 1 is chosen above: set characteristic compressive strength after 28 days.
If 2 is chosen above: set the prescribed characteristic compressive strength.

With consideration of curing time

$\alpha_{cc.pl} := 0.8$ $\alpha_{ct.pl} := 0.8$ $\gamma_C := 1.5$ $\gamma_{cE} := 1.2$

$f_{cm} := f_{ck} + 8MPa = 14 \cdot MPa$

$$s = 0.2 \text{ CEM 42,5 R, CEM 52,5 N and CEM 52,5 R (Class R)}$$
$$s = 0.25 \text{ CEM 32,5 R, CEM 42,5 N (Class N)}$$
$$s = 0.38 \text{ CEM 32,5 N (Class S)}$$

$s := 0.25$

$t_1 := 3$ Curing time in days

$$\beta_{cc} := \exp\left[s \cdot \left[1 - \left(\frac{28}{t_1}\right)^{\frac{1}{2}}\right]\right] = 0.598$$

$f_{cm.t} := \beta_{cc} \cdot f_{cm} = 8.375 \cdot MPa$ Mean compressive strength at t days

$f_{ck.t} := f_{cm.t} - 8MPa = 0.375 \cdot MPa$ Characteristic compressive strength at t days

$$f_{ck.t} := \begin{cases} f_{ck.t} & \text{if } Initial_{strength} = 1 \\ f_{ck} & \text{otherwise} \end{cases} = 6 \cdot MPa$$

Design compressive strength used when determine other needed properties.

$f_{cm.t} := f_{ck.t} + 8MPa = 14 \cdot MPa$ Mean compressive strength

$E_{cm.t} := 22GPa \cdot \left(\frac{f_{cm.t}}{10MPa}\right)^{0.5} = 26.031 \cdot GPa$ Mean modulus of elascticity

$f_{ctm.t} := 0.3 \cdot \left(\frac{f_{ck.t}}{MPa}\right)^{\frac{2}{3}} \cdot MPa = 0.991 \cdot MPa$ Mean tensile strength

$f_{ctk.0.05} := 0.7 \cdot f_{ctm.t} = 0.693 \cdot MPa$ Characteristic tensile strength, lower bound

$f_{cd.pl} := \alpha_{cc.pl} \cdot \frac{f_{ck.t}}{\gamma_C} = 3.2 \cdot MPa$ Design compressive strength

$f_{ctd.pl} := \alpha_{ct.pl} \cdot \frac{f_{ctk.0.05}}{\gamma_C} = 0.37 \cdot MPa$ Design tensile strength

$E_{cd} := \frac{E_{cm.t}}{\gamma_{cE}} = 21.692 \cdot GPa$ Design modulus of elasticity

Slenderness

$i := \sqrt{\frac{I_c}{A_c}} = 0.043\,m$

$\lambda := \frac{L}{i} = 69.282$

$$\begin{cases} \text{"USE SIMPLIFIED PROCEDURE"} & \text{if } \lambda \le 86 \\ \text{"USE GENERAL PROCEDURE"} & \text{otherwise} \end{cases} = \text{"USE SIMPLIFIED PROCEDURE"}$$

Simplified procedure

Combination of bending and normal force

$$\phi := 1.14 \cdot \left[1 - \frac{2 \cdot (e_{tot})}{t}\right] - 0.02 \cdot \frac{L}{t} = 0.586$$

$$\phi := \begin{cases} \phi & \text{if } \phi \le 1 - \frac{2 \cdot (e_{tot})}{t} \\ 1 - \frac{2 \cdot (e_{tot})}{t} & \text{otherwise} \end{cases} = 0.586$$

$$N_{Rd} := A_c \cdot f_{cd.pl} \cdot \phi = 281.337 \cdot kN$$

$$\begin{cases} \text{"OK"} & \text{if } N_{Rd} \ge N_{Ed} \\ \text{"NOT OK"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Combination of shear force and normal force

$$k := 1.5$$

$$V_{Ed} := F_{bracing} = 7.29 \cdot kN$$

$$\tau_{cp} := k \cdot \frac{V_{Ed}}{A_c} = 0.073 \cdot MPa$$

$$\sigma_{cp} := \frac{N_{Ed}}{A_c} = 1.8 \cdot MPa$$

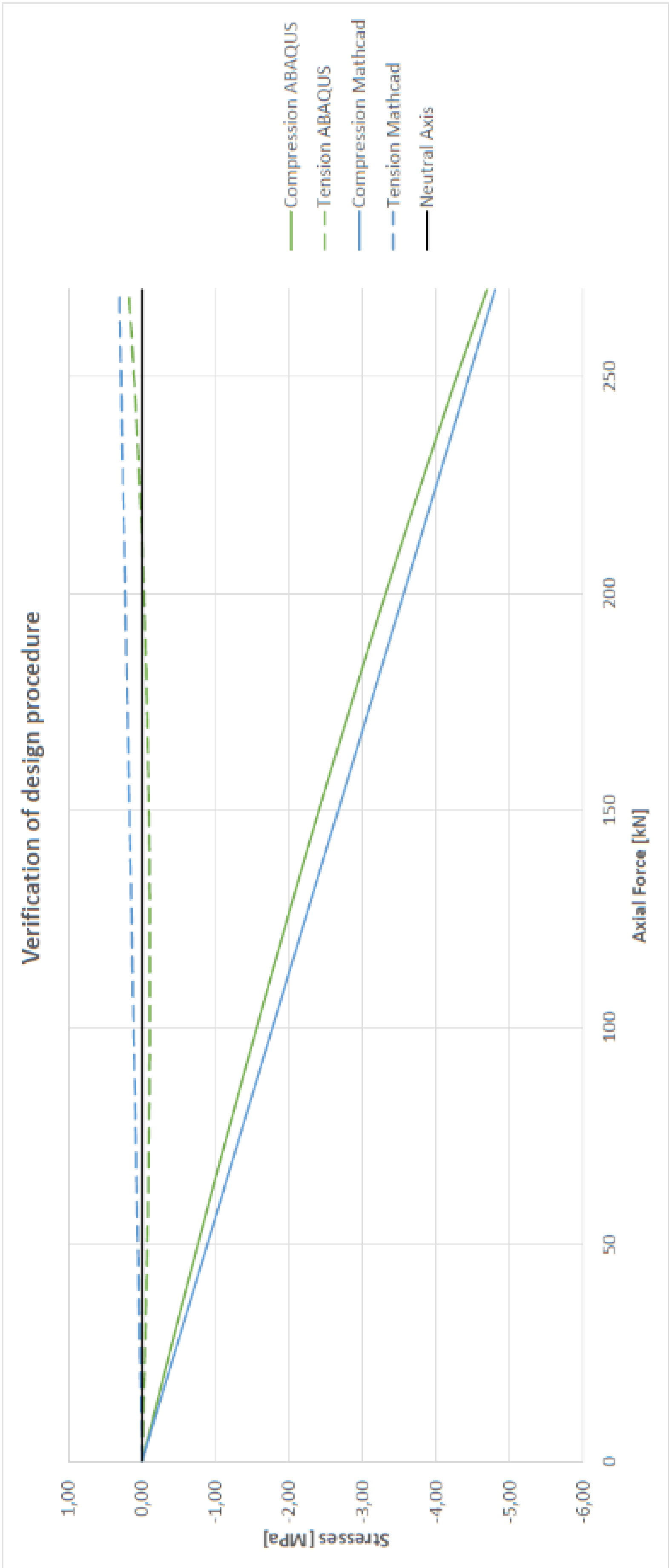
$$\sigma_{c.lim} := f_{cd.pl} - 2 \cdot \sqrt{f_{ctd.pl} \cdot (f_{ctd.pl} + f_{cd.pl})} = 0.902 \cdot MPa$$

$$f_{cvd} := \begin{cases} \sqrt{(f_{ctd.pl})^2 + \sigma_{cp} \cdot f_{ctd.pl}} & \text{if } \sigma_{cp} \le \sigma_{c.lim} \\ \sqrt{(f_{ctd.pl})^2 + \sigma_{cp} \cdot f_{ctd.pl} - \left[\frac{(\sigma_{cp} - \sigma_{c.lim})}{2}\right]^2} & \text{otherwise} \end{cases} = 0.775 \cdot MPa$$

$$\begin{cases} \text{"OK"} & \text{if } \tau_{cp} \le f_{cvd} \\ \text{"NOT OK"} & \text{otherwise} \end{cases} = \text{"OK"}$$

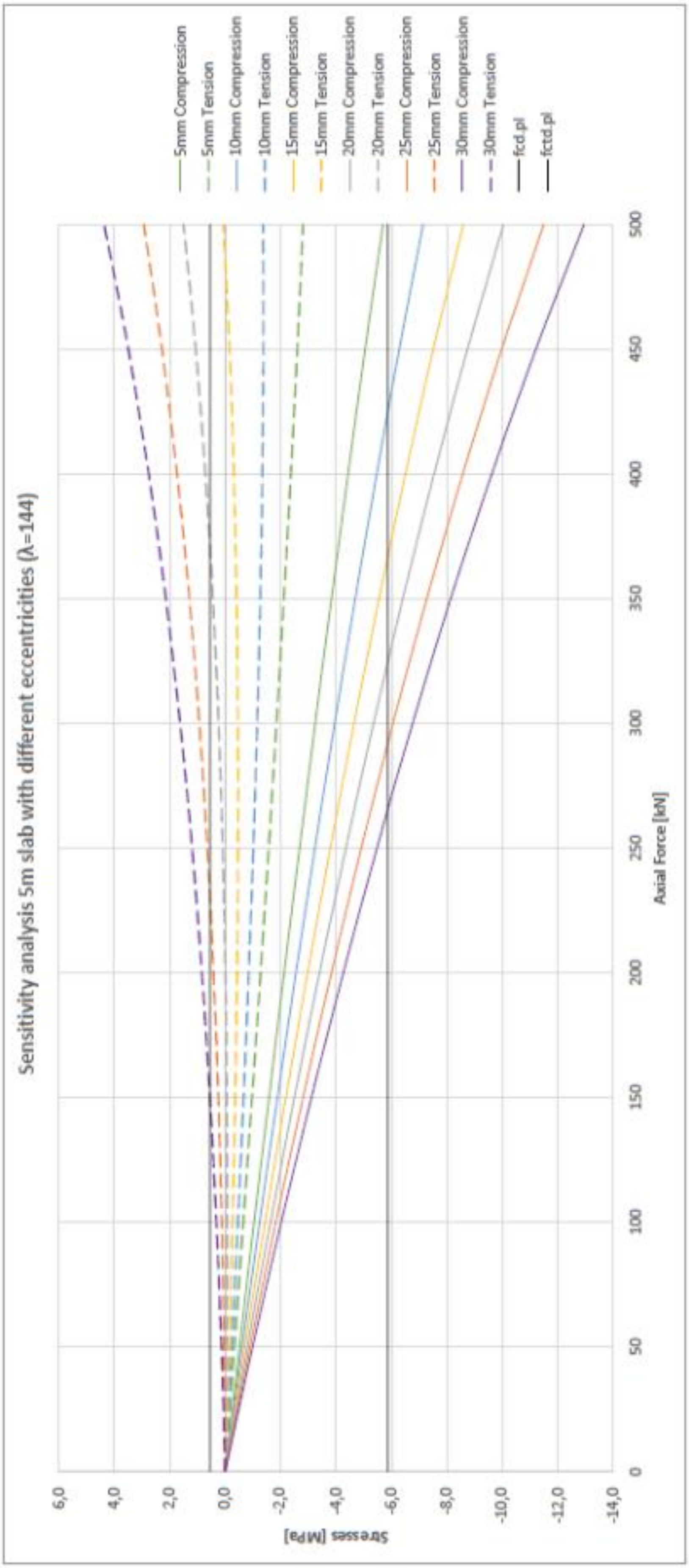
G

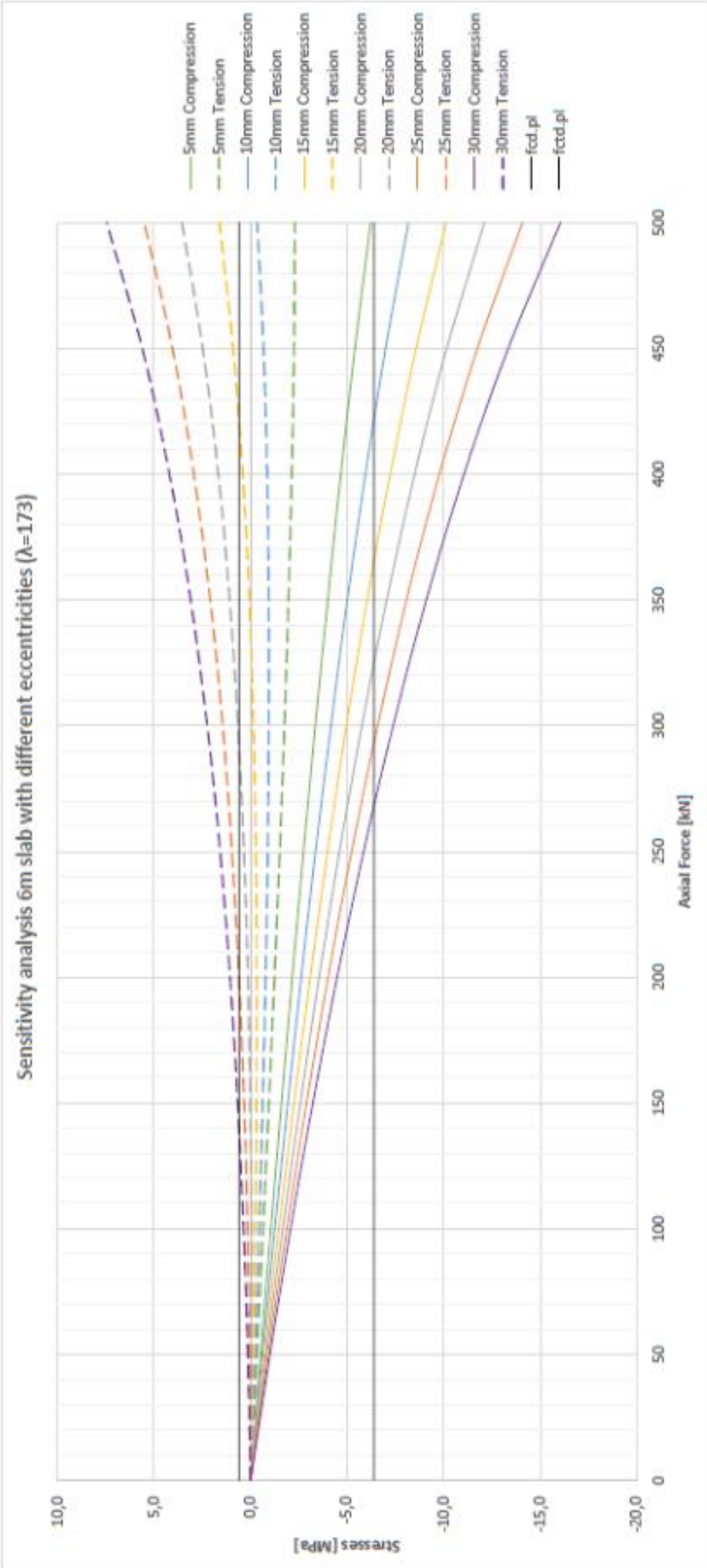
Appendix 7 - Plot from verification of design procedure

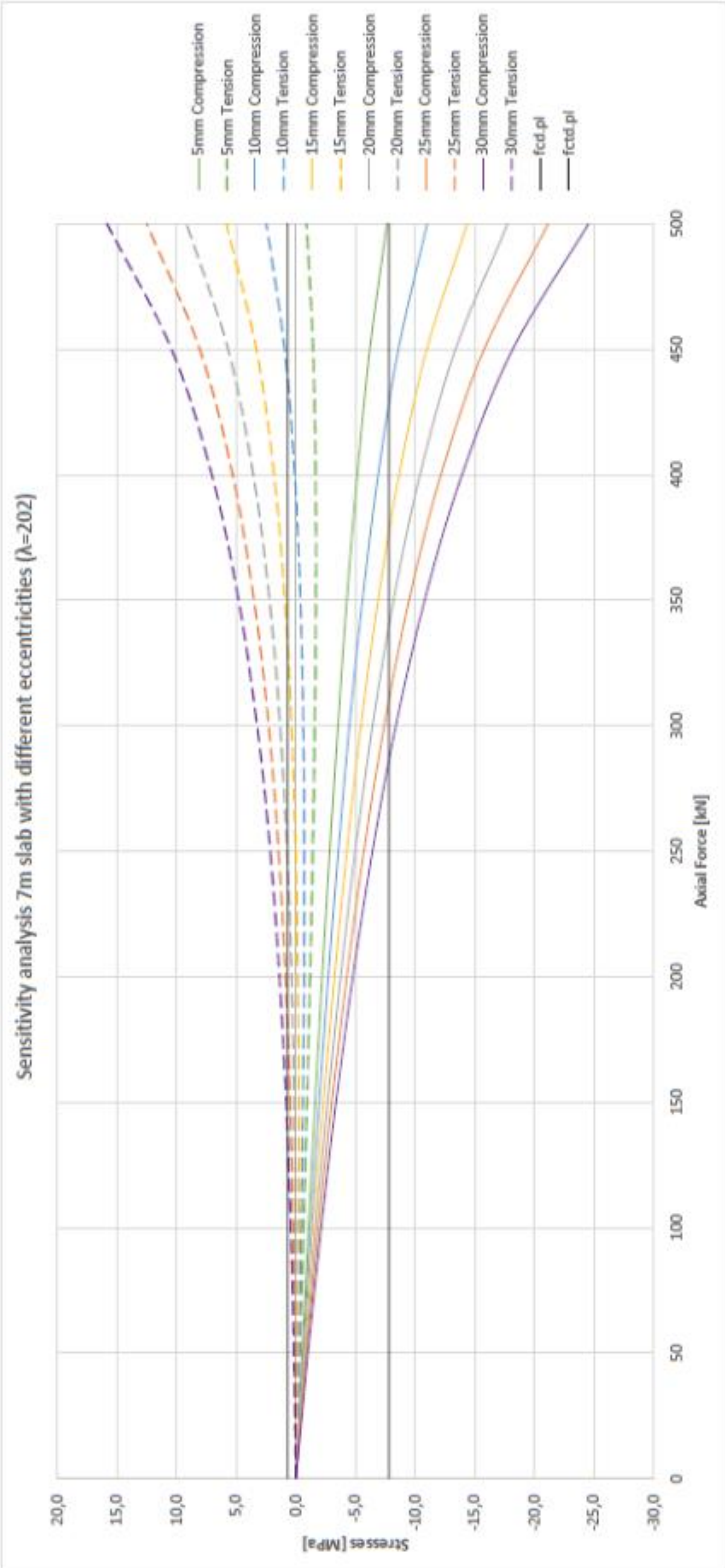

Verification of design procedure
Compression ABAQUS
Tension ABAQUS
Compression Mathcad
Tension Mathcad
Neutral Axis
Stresses [MPa]
1,00
0,00
-1,00
-2,00
-3,00
-4,00
-5,00
-6,00
Axial Force [kN]
0
50
100
150
200
250

H

Appendix 8 - Plots from sensitivity analysis


Sensitivity analysis 5m slab with different eccentricities (λ=144)
5mm Compression
5mm Tension
10mm Compression
10mm Tension
15mm Compression
15mm Tension
20mm Compression
20mm Tension
25mm Compression
25mm Tension
30mm Compression
30mm Tension
fcd.pl
fctd.pl
Stresses [MPa]
6,0
4,0
2,0
0,0
-2,0
-4,0
-6,0
-8,0
-10,0
-12,0
-14,0
Axial Force [kN]
0
50
100
150
200
250
300
350
400
450
500


Sensitivity analysis 6m slab with different eccentricities (λ=173)
5mm Compression
5mm Tension
10mm Compression
10mm Tension
15mm Compression
15mm Tension
20mm Compression
20mm Tension
25mm Compression
25mm Tension
30mm Compression
30mm Tension
fcd.pl
fctd.pl
Stresses [MPa]
Axial Force [kN]
10,0
5,0
0,0
-5,0
-10,0
-15,0
-20,0
0
50
100
150
200
250
300
350
400
450
500


Sensitivity analysis 7m slab with different eccentricities (λ=202)
5mm Compression
5mm Tension
10mm Compression
10mm Tension
15mm Compression
15mm Tension
20mm Compression
20mm Tension
25mm Compression
25mm Tension
30mm Compression
30mm Tension
fcd.pl
fctd.pl
Stresses [MPa]
20,0
15,0
10,0
5,0
0,0
-5,0
-10,0
-15,0
-20,0
-25,0
-30,0
Axial Force [kN]
0
50
100
150
200
250
300
350
400
450
500

I

Appendix 9 - Further reduction of buckling length

Further reduction of buckling length NOT VERIFIED!

Input data

$b := 1m$ — Width of slab

$t := 120mm$ — Slab thickness

$l_0 := 40m$ — Buckling length according to Eulers

$N_{Ed} := 270kN$ — Design axial force from sheet pile walls

$\rho_c := 23 \frac{kN}{m^3}$ — Density concrete, favourble effects

$e_h := 60mm$ — Eccenricity in midsection due to bottom heave

Initial eccentricities

Unintended eccentricity due to imperfections

$\alpha_m := 1$ — Only the slab member contributing to the total effect

$$\alpha_h := \begin{cases} \frac{2}{3} & \text{if } \frac{2}{3} \geq \frac{2m^{0.5}}{\sqrt{l_0}} \\ 1 & \text{if } 1 \leq \frac{2m^{0.5}}{\sqrt{l_0}} \\ \frac{2m^{0.5}}{\sqrt{l_0}} & \text{otherwise} \end{cases} = 0.667$$

$$\theta_i := 0.005 \cdot \alpha_h \cdot \alpha_m = 3.333 \times 10^{-3}$$

$$e_i := \theta_i \cdot \frac{l_0}{2} = 66.667 \cdot mm$$

$e_i := e_i = 66.667 \cdot mm$ — Change if larger initial imperfection than Eurocode prescibe want be considered, otherwise set e_i=e_i.

Intended eccentricity due to external forces

$M_{Ed} := 0$

The transversal load from bottom heave is here considered as a eccentricity.

$e := 0mm$

The axial force is assumed to act in the gravity of center

$$e_0 := \frac{M_{Ed} + N_{Ed} \cdot e}{N_{Ed}} = 0 \cdot mm$$

Determine buckling length based on bracing force

Δ_0 function of l_0	Buckling percentage $B_{\%}$ [%]
$l_0/100$	8
$l_0/200$	4
$l_0/300$	2.7
$l_0/400$	2
$l_0/500$	1.6
$l_0/600$	1.3
$l_0/700$	1.1
$l_0/800$	1
$l_0/900$	0.9
$l_0/1000$	0.8

Choose the buckling percentage from the table above based on the initial eccentricity

$$g := \rho_c \cdot t \cdot b = 2.76 \cdot \frac{kN}{m}$$

Determine buckling percentage

$$e_{tot} := e_i + e_0 + e_h = 126.667 \cdot mm$$

$$A := \frac{l_0}{e_{tot}} = 315.789$$

The total initial eccentricity is $\Delta_0 = l_0 / A$

$B_{\%} := 2.7\%$

Choose buckling percentage from table. This buckling percentage will be the same for all reduction phases since if the buckling length is reduced by half, so is the eccentricity.

$$F_{bracing} := B_{\%} \cdot N_{Ed} = 7.29 \cdot kN$$

Bracing force dependent on buckling percentage

First reduction

$M_{required.1} := \frac{F_{bracing} \cdot l_0}{4} = 72.9 \cdot kN \cdot m$

Moment required to achieve full bracing.

$M_{g.1} := \frac{g \cdot {l_0}^2}{8} = 552 \cdot kN \cdot m$

Moment from self-weight

$L_1 := \begin{cases} \frac{l_0}{2} & \text{if } M_{g.1} \geq M_{required.1} \\ l_0 & \text{otherwise} \end{cases} = 20 \cdot m$

If the moment from the self weight is greater than the required moment, reduce Eulers buckling length by half

$Reduction_1 := \begin{cases} \text{"REDUCTION"} & \text{if } L_1 < l_0 \\ \text{"NO REDUCTION"} & \text{otherwise} \end{cases} = \text{"REDUCTION"}$

Second reduction

$M_{required.2} := \frac{F_{bracing} \cdot L_1}{4} = 36.45 \cdot kN \cdot m$

$M_{g.2} := \frac{g \cdot {L_1}^2}{8} = 138 \cdot kN \cdot m$

$L_2 := \begin{cases} \frac{L_1}{2} & \text{if } M_{g.2} \geq M_{required.2} \\ L_1 & \text{otherwise} \end{cases} = 10 \cdot m$

$Reduction_2 := \begin{cases} \text{"REDUCTION"} & \text{if } L_2 < L_1 \\ \text{"NO REDUCTION"} & \text{otherwise} \end{cases} = \text{"REDUCTION"}$

Third reduction

$$M_{required.3} := \frac{F_{bracing} \cdot L_2}{4} = 18.225 \cdot kN \cdot m$$

$$M_{g.3} := \frac{g \cdot L_2^{\,2}}{8} = 34.5 \cdot kN \cdot m$$

$$L_3 := \begin{cases} \frac{L_2}{2} & \text{if } M_{g.3} \geq M_{required.3} \\ L_2 & \text{otherwise} \end{cases} = 5 \cdot m$$

$$Reduction_3 := \begin{cases} \text{"REDUCTION"} & \text{if } L_3 < L_2 \\ \text{"NO REDUCTION"} & \text{otherwise} \end{cases} = \text{"REDUCTION"}$$

Fourth reduction

$$M_{required.4} := \frac{F_{bracing} \cdot L_3}{4} = 9.113 \cdot kN \cdot m$$

$$M_{g.4} := \frac{g \cdot L_3^{\,2}}{8} = 8.625 \cdot kN \cdot m$$

$$L_4 := \begin{cases} \frac{L_3}{2} & \text{if } M_{g.4} \geq M_{required.4} \\ L_3 & \text{otherwise} \end{cases} = 5 \cdot m$$

$$Reduction_4 := \begin{cases} \text{"REDUCTION"} & \text{if } L_4 < L_3 \\ \text{"NO REDUCTION"} & \text{otherwise} \end{cases} = \text{"NO REDUCTION"}$$

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