Evaluation of seismic induced liquefaction and related effects on dynamic behaviour of anchored quay walls

Using UBC3D-PLM constitutive model

MSc. Thesis

W.A. van Elsäcker

July 2016
Evaluation of seismic induced liquefaction and related effects on dynamic behaviour of anchored quay walls

Using UBC3D-PLM constitutive model

by

W.A. van Elsäcker

Final report

to obtain the degree of Master of Science
at the Delft University of Technology,

Date: July 26th 2016

Thesis committee: Prof. dr. ir. S.N. Jonkman TU Delft, chairman
Dr. ir. J.G. de Gijt TU Delft
Dr. ir. R.B.J. Brinkgreve TU Delft
Ir. F. Besseling Witteveen+Bos
Preface

This report describes the research I conducted as a part of my thesis to finish the master Hydraulic Engineering and obtain the degree of Master of Science at the Delft University of Technology. During my master I discovered that I enjoyed designing structures with a strong soil structure interaction. I searched for a graduation topic where typically this interaction was important. By reading other master theses I saw that already substantial research was done to dynamic analysis of anchored quay walls and to evaluation of earthquake induced liquefaction, but not together. This triggered me to combine these topics and perform this challenging research.

Witteveen+Bos offered me the opportunity to perform my research, for which I am very grateful. I enjoyed working at the office, both in Rotterdam and in Deventer. Many thanks to all my colleagues for their interest and support during the research. I would like to thank Floris Besseling in particular for taking place in the graduation committee and being my daily supervisor, your energy and enthusiasm is contagious. I learned a lot from our discussions and all your feedback during the research.

Furthermore, the research would not have been succeeded without the guidance and feedback of the whole graduation committee. I want to thank Bas Jonkman for being the chairman of the committee, your enlightening view on the conclusions and report helped a lot. Many thanks to Ronald Brinkgreve for his support with PLAXIS and to Jarit de Gijt for his valuable knowledge about quay walls.

Last but not least I want to thank my family and friends for their loving support and drinking beers during my study period. Especially my parents supported me in every way they could during my study, I can't thank you enough.

Willem van Elsäcker
Delft, July 2016
Abstract

Liquefaction may have large influence on performance of anchored quay walls and is important to include in seismic design. Design codes provide evaluation methods that provide hardly any insight in the actual development of excess pore pressures and don't consider soil structure interaction. There is a need for tools that include these aspects, since performance based design principles are more often adopted in earthquake engineering. In this research effects of excess pore pressures on dynamic earth pressures and dynamic behaviour of an anchored are investigated. Different evaluation methods are analysed on their performance. Initially prescribed pseudo-static methods in design codes are applied, followed by a dynamic analysis using finite element model PLAXIS.

After a broad literature overview of available pseudo-static methods and sensitivity analysis. A case study in Akita Port is adopted where two similar anchored quay walls were both hit by the Nihonkai Chubu Earthquake in 1983. One quay wall survived the seismic event, the other suffered severe damage. Damage was related to occurrence of liquefaction in the backfill. Pseudo-static- and dynamic analyses are applied to assess both structures. Performance of both analyses is evaluated and effects of liquefaction are investigated.

Pseudo-static analysis of the quay wall without liquefaction are, after reduction of the seismic coefficient, reasonably in correspondence with observations. Active earth pressures are well predicted, while passive earth pressures are overestimated. The relatively simple method is however suitable to give a first indication of failure/non-failure of the structure. On the other hand the pseudo-static method is not capable to realistically include effects of excess pore pressures. Large stiffness differences between layers lead to exaggerated bending moments and displacements, again passive resistance is overestimated. Since Mononobe-Okabe is a limit equilibrium method, it is not well able to capture progressive failure.

By performing a dynamic analysis, using the finite element model PLAXIS, insight is obtained in the actual soil structure interaction. The Hardening Soil small strain (HSsmall) constitutive material model is adopted to model the static and dynamic behaviour of soil not vulnerable to liquefaction. A calibrated dynamic calculation lead to reasonable accurate for the case without liquefaction. Bending moment distribution and displacements are in line with observations. Different earthquake motions lead to relatively large spread in displacement of the structure, less differences are found for bending moment distribution.

To include effects of liquefaction the user-defined UBC3D-PLM constitutive material model is assigned to potential liquefiable layers. Liquefaction resistance depends on the type of soil and on the stress state of the soil. By calibrating the UBC3D-PLM model a reasonably accurately prediction of the onset of liquefaction can be obtained for the considered stress state. Performance of the model deteriorates for variation of the stress state, especially for initial static shear stresses. Therefore a calibration method is developed to get around this limitation. Initially zones with a specific stress state are identified around the structure. Subsequently the model is calibrated to accurately predict the onset of liquefaction for each considered stress state, leading to a calibrated model parameter set for each zone. After adopting all parametersets for each zone a calibrated dynamic calculation is performed of the system, leading to reasonably accurate results of the development of excess pore pressures around the structure. Calculated displacements and bending moments are in line with observations in the field and the failure mode is well predicted. Insight is gained in the development of excess pore pressures in time at different locations. Leading to the conclusion that the UBC3D-PLM model is capable to reasonable accurately evaluate liquefaction and corresponding effects on an anchored quay wall.

Finally link is made between the followed procedure and applicability of this procedure for performing a dynamic analysis including liquefaction for one of the anchored quay walls in the Eemshaven in Groningen. Important aspects that are typical for Groningen are the location of the bedrock and corresponding translation of the earthquake motion and characteristics of the motion.
4.2 Pseudo-static analysis ........................................... 55
  4.2.1 Liquefaction potential .................................. 55
  4.2.2 Seismic loading ........................................ 56
  4.2.3 Structural properties ................................... 57
  4.2.4 Results .................................................. 62
4.3 Conclusion ................................................... 68
5 Validation performance UBC3D-PLM model 69
  5.1 Defining loading paths .................................... 70
  5.2 Model parameters UBC3D-PLM model ..................... 71
  5.3 Liquefaction resistance sands Akita Port ............... 73
  5.4 Performance UBC3D-PLM according to literature ....... 74
  5.5 Initial model parameters ................................. 76
  5.6 Undrained cyclic direct simple shear test .............. 77
    5.6.1 Results initial model parameters .................. 78
    5.6.2 Calibration of model ................................ 79
    5.6.3 Effects of state parameters ....................... 84
  5.7 Undrained cyclic triaxial test ........................... 93
    5.7.1 Results initial model parameters .................. 93
    5.7.2 Calibration of model ................................ 95
    5.7.3 Effects of state parameters ....................... 100
  5.8 Conclusion ................................................ 103
6 Dynamic analysis Akita Port ......................... 105
  6.1 Time records earthquakes ................................ 106
    6.1.1 Nihonkai Chubu Earthquake ......................... 106
    6.1.2 Reference earthquake records ...................... 106
  6.2 Site response analysis ................................... 107
    6.2.1 Equivalent linear analysis ......................... 109
    6.2.2 Non-linear site response analysis ................. 110
    6.2.3 Results site response analysis .................... 116
  6.3 Dynamic analysis Ohama No.1 Wharf .................... 119
    6.3.1 Model configuration ................................ 119
    6.3.2 Results dynamic analysis ........................... 119
    6.3.3 Comparison with pseudo-static analysis .......... 121
    6.3.4 Conclusion ........................................... 123
  6.4 Dynamic analysis Ohama No.2 Wharf .................... 125
    6.4.1 Static analysis ...................................... 125
    6.4.2 Parameter selection ................................ 127
    6.4.3 Model configuration ................................ 128
    6.4.4 Results soil deformation ............................ 129
    6.4.5 Response structure .................................. 134
    6.4.6 Conclusion ........................................... 136
7 Procedure dynamic analysis Eemshaven ........... 139
  7.1 Introduction ............................................. 139
  7.2 Wilhelminahaven ......................................... 140
  7.3 Description of procedure ................................ 141
8 Conclusions and recommendations ................... 145
  8.1 Conclusions ............................................... 145
  8.2 Recommendations ......................................... 147
Bibliography ....................................................... 149
List of Figures ................................................... 153
List of Tables ................................................... 161
## Contents

A  Case studies analysis pseudo-static methods ........................................ 163
   A.1  Introduction into model ........................................................... 163
   A.2  Results models ......................................................................... 163
      A.2.1  Reference case ................................................................. 164
      A.2.2  Partially saturated backfill ................................................ 165
      A.2.3  Backfill with liquefiable layer ........................................... 167
B  Interpretation soil investigation ............................................................... 171
   B.1  Ohama No.1 Wharf ................................................................. 172
   B.2  Ohama No.2 Wharf ................................................................. 174
C  Constitutive material models ................................................................. 177
   C.1  HSSmall model ....................................................................... 177
   C.2  UBC3D-PLM model ................................................................ 180
      C.2.1  Development excess pore pressures .................................... 182
D  Performance UBC3D-PLM model ............................................................. 185
   D.1  Undrained cyclic direct simple shear test .................................... 185
      D.1.1  Effects of state parameters ................................................. 186
   D.2  Undrained cyclic triaxial test .................................................... 189
      D.2.1  Effects of state parameters ................................................. 190
E  Earthquake Motions .............................................................................. 193
   E.1  Nihonkai Chubu Earthquake, Japan, M = 7.7, May 1983 .......... 194
   E.2  Imperial Valley Earthquake, USA, M = 6.6, October 1979 ........ 195
   E.3  Landers Earthquake, United States of America, M = 7.3, June 1992 196
   E.4  Kocaeli Earthquake, Turkey, M = 7.6, August 1999 .................. 197
F  Results site response analysis ................................................................. 199
   F.1  Ohama No.1 Wharf ................................................................. 200
   F.2  Ohama No.2 Wharf ................................................................. 204
G  Model parameters dynamic analysis Ohama No.2 Wharf ....................... 209
Introduction

1.1. Problem description

In the past few years the increased seismic activity in Groningen due to natural gas extraction has led to public discussion. The Eemshaven is situated in the area vulnerable to induced earthquakes. Quay walls are important structures in the port area for transferring goods between vessels and the land based infrastructure. Operation cannot continue if these structures suffer severe damage, which could lead to large economical losses. It is therefore important to guarantee that these structures do not suffer significant damage during a seismic event. Typically anchored quay walls in the Eemshaven were not designed for seismic loading. Assessment of the structural performance is required to provide insight in the safety of the structure, especially because of the presence of potential liquefiable soils in the backfill.

Deltares published several reports in which effects of induced earthquakes on important infrastructure in Groningen are investigated. Typically conservative approximations were adopted since there is still much uncertainty, especially the behaviour of structures in potentially liquefiable sand layers requires extra attention (Deltares, 2014b). In addition to these findings, Deltares performed more research to the risk of the occurrence of liquefaction in sand layers in Groningen. Pseudo-static analysis was considered to be too conservative compared to the dynamic finite element calculations for assessment of these structures (Deltares, 2014a).

Gazetas also investigated the performance of an anchored quay wall subjected to a strong seismic event by applying different design methods. Conclusions are in line with results found by Deltares; pseudo-static limit equilibrium methods generally lead to unnecessarily conservative designs. Gazetas also stated that the risk of liquefaction must be excluded in applying this method, because the method isn’t capable to deal with the change of behaviour of the soil (Gazetas, G. et al., 2015).

Design codes provide evaluation methods that provide hardly any insight in the actual behaviour, don’t consider soil structure interaction. Applicability is questionable, especially in case of potentially liquefiable layers. Failure of quay walls however is often related to the occurrence of liquefaction. There is a need for tools that include these aspects and overcome the limits.

Dynamic calculations using effective stress finite element models can be applied to obtain more insight in the actual soil structure interaction. However often liquefaction is excluded in such calculations. Research is done on evaluation of liquefaction and effects on gravity based structures by applying dynamic finite element calculation (Galavi, V. et al., 2013). In the Netherlands mostly more slender and flexible quay walls are constructed, like the anchored quay walls in the Eemshaven. Much less insight in the development of excess pore pressures and the effects on these type of structures is yet obtained.
1.2. Objectives
The main objective of this research is to provide more insight in the effects of liquefaction on dynamic earth pressures by application of effective stress finite element models. By gaining knowledge about the effects and soil structure interaction, simplified methods prescribed in design codes are assessed on their performance. The other way around, available constitutive models used in the effective stress finite element models are evaluated on their capability of modelling this problem. The following research question is defined to reach the main objective:

*How does the occurrence of liquefaction due to earthquakes affect the magnitude of dynamic earth pressures and the behaviour of anchored quay walls, and how can this be analysed?*

The following associated sub questions are answered throughout the research, supporting the main research question:

- How do pseudo-static design methods account for effects of excess pore pressures on dynamic (earth) pressures?
- To what extent are pseudo-static methods capable of realistically predicting dynamic earth pressures on anchored quay walls, with and without effects of development of excess pore pressures?
- How can the HSsmall and UBC3D-PLM constitutive material models be calibrated to realistically model the soil behaviour around an anchored quay wall under seismic loading?
- To what extent is the UBC3D-PLM model capable predicting the development of excess pore pressures together with the loss of strength of the soil and the corresponding dynamic behaviour of the anchored quay wall?
- Which procedure has to be followed in the structural assessment of the anchored quay wall with relieving platform in the Wilhelminahaven in Groningen subjected to an induced earthquake?

1.3. Outline of research
Focus of this research is on local site effects and especially the response of the structure in relation to the occurrence of liquefaction. In Figure 1.1 on the left side the path of the seismic waves from source to bedrock is indicated, which is not considered. On the right side the focus area of the research is shown, which involves analysis of the liquefaction potential of the soils and the dynamic response of the structure.

![Figure 1.1: On the left the path of seismic waves from source to structure leading to local site effects; on the right the focus area of this research.](image-url)
In Figure 1.2 a flowchart of the research is presented. The structure of the report broadly follows the order of this flowchart.

In chapter 2 a theoretical background on the research topic is given. An introduction into different assessment theories is given and basic theory of liquefaction is elaborated. Finally a broad overview of the pseudo-static Mononobe-Okabe theory is given. Sensitivity of the Mononobe-Okabe theory is analysed at the beginning of chapter 3. It continues with an analysis of the performance of this theory for different configurations.

From chapter 4 on a case study in Akita Port is analysed. Two similar anchored quay walls located close to each other were hit by the Nihonkai Chubu Earthquake. One quay wall survived the seismic event without severe damage, the other quay wall suffered major damage. Failure was linked to the occurrence of liquefaction in the backfill. In chapter 4 the pseudo-static Mononobe-Okabe theory is applied to analyse dynamic earth pressures and the response of both structures to the seismic event. Liquefaction potential is determined with cyclic stress approach by Idriss, I.M. and Boulanger, R.W. (2008). Performance of the pseudo-static method for both cases is evaluated.

The user-defined constitutive material model UBC3D-PLM is adopted to model the development of excess pore pressures. In chapter 5 the UBC3D-PLM model is calibrated and validated. The model is calibrated for two soil types by fitting results of experimental element tests. Subsequently different model parameters sets are developed to calibrate the model for varying initial stress states.

For soil layers not vulnerable to liquefaction the Hardening Soil small strain (HSsmall) constitutive model is adopted. Chapter 6 contains the dynamic analysis of the Akita port case using PLAXIS. Both structures are modelled in order to analyse differences in loading and behaviour of the structures. Besides it is analysed to what extent the UBC3D-PLM model realistically predict the development of excess pore pressures. Performance of both models is evaluated and compared to observations in the field and results of pseudo-static analysis.

In chapter 7 a case study in the Wilhelminahaven in Groningen is introduced. Lessons learned from the Akita Port case are used point out important aspects in dynamic analysis. Result is a detailed description how to set up a calibrated finite element model to dynamically assess the anchored quay wall with relieving platform in the Wilhelminahaven for an induced earthquake including the liquefaction potential.

Finally in chapter 8 conclusions of the research are presented together with recommendations for future research.
Figure 1.2: Flowchart of research
Theoretical background

In this chapter a general theoretical background is given as introduction to the research. First information about earthquakes in general is provided, followed by the seismic hazards for port structures. After that a short introduction into performance based design for earthquake engineering is given. Several possible failure modes of anchored quay walls are elaborated, which all have to be checked. This can be done according to several methods, varying from simplified methods to full dynamic analysis. One of the major hazards for anchored quay walls is the occurrence of liquefaction. An introduction into this phenomenon and available evaluation methods are presented. At last the generally used pseudo-static Mononobe-Okabe methods is considered in detail. Several additions to the original method are available accounting for the presence of water and excess pore pressures.

2.1. Earthquakes

2.1.1. Sources

The main source of earthquakes is tectonic activity, triggering the tectonic earthquakes. Areas where no tectonic activity is present also earthquakes may appear, so-called induced earthquakes related to human activities like gas extraction or mining. The last type of earthquakes appear in Groningen.

Figure 2.1: (left) Global distribution of earthquake occurrence associated with plate boundaries (Bosboom, J., 2015). (right) Earthquake density in province of Groningen in 2014-2015 together with the faults in subsoil indicated (TNO, 2015)
Tectonic earthquakes

Tectonic earthquakes occur all over the world, mainly around tectonic faults where at least two adjacent plates move with respect to each other, see Figure 2.1. The earth's surface consists of different plates that all move. Around faults the plates can deform slowly and continuously, but also sudden deformation can occur leading to seismic waves. Elastic strain energy is stored in the ground near the boundary of the plates as shear stresses increase on the fault plane due to relative movement of two plates. If the critical shear stress of the material is reached it fails and the accumulated strain energy is released. Due to this last phenomenon waves are released which are felt as earthquakes (Kramer, S.L., 1996).

Induced earthquakes

The occurrence of induced earthquakes is often related to human activities. In this research only induced earthquakes due to the gas extraction in Groningen are considered. Gas is extracted in Groningen from a porous sandstone layer at a depth of about three kilometers. The initial pressure in this layer is very high, but due to the extraction of gas the pressure in the pores lowers. This leads to an increase of stress in the sandstone layers and to compaction of the layer. Due to interaction of the reservoir with the surrounding rock the reservoir is not able to deform freely, which results in the development of differential stresses. Vertical effective stress increases more compared to the horizontal effective stress (Mulders, F.M.M., 2003).

Differential compaction behaviour and anisotropic material behaviour leads to reactivation of pre-existing geological fault planes. Shear stress develops between the fault planes until it suddenly fails resulting in the accumulated strain energy to be released. The sudden differential movement along natural faults causes seismic waves. Because earthquakes are triggered mainly by compaction of reservoirs, the location of the hypocenter of the earthquake is relatively shallow compared to tectonic earthquakes (Mulders, F.M.M. (2003), Namplatform (2015)).

In Figure 2.1 the earthquake density in the province of Groningen together with the faults in the subsoil are presented. Location of the epicenter of the induced earthquakes is related to the locations of the faults in the soil.

2.1.2. Seismic waves

Seismic waves are generated by the movement of faults. Different type of seismic waves can be distinguished. The so called body waves that move from the source to the site through the earth. Once they reach the surface surface waves can be identified, which can be subdivided into Rayleigh waves and Love waves. The surface waves are in this research of minor interest and are therefore not described in detail.

The body waves on the other hand are responsible for the loading of the subsoil. Two types of body waves can be distinguished: compression- (primary) and shear (secondary) waves. The compression waves cause strains strain in the propagation direction, where shear waves cause ground strains perpendicular to the propagation direction. These shear waves can again be subdivided into shear vertical (or SV wave) and a shear horizontal (or SH wave). The SV wave causes the up-down movement and the SH wave causes a left-right movement.

The propagation velocity is dependent only on the material properties of the soil, density and stiffness. Since the compression stiffness modulus of the soil larger is than the shear modulus, the compression waves travel faster through the soil. The wave celerity of both compression- ($v_p$) and shear ($v_s$) waves can be determined as follows:

$$v_p = \sqrt{\frac{E_{oed}}{\rho}}$$

(2.1)

$$v_s = \sqrt{\frac{G_0}{\rho}}$$

(2.2)
2.2. Seismic hazards port structures

2.1.3. Size
The size of an earthquake is an important parameter in earthquake engineering, both qualitative as quantitative ways of describing the size have been developed.

Earthquake intensity
The intensity of an earthquake is a qualitative description of the local effects, based on observed damage and reactions from humans. Historical data is present in which qualitative descriptions of effects of earthquakes are included, which was used to develop a scale. Originally the Rossi-Forel (RF) scale of intensity was used for many years. Another scale has been introduced in most English-speaking countries, which is the Mercalli intensity scale (MMI) (Kramer, S.L. (1996) PIANC (2001)).

Earthquake magnitude
Once modern instrumentation became available, more objective quantitative measurement of the size of an earthquake was possible. This quantitative description of an earthquake is called earthquake magnitude. The most known earthquake magnitude scale is the Richter scale. Richter defined the local magnitude as the logarithm of the maximum trace amplitude recorded on a Wood-Anderson seismometer located 100 km from the epicenter of the earthquake (Kramer, S.L., 1996). The Richter local magnitude (ML) however not always the most appropriate description of an earthquake, because it doesn't distinguish different types of seismic waves.

2.2. Seismic hazards port structures
Seismic waves travel from the fault where they are generated to the bedrock at the site. Local soil deposits influence characteristics of these seismic waves. The ground motions travel through the local deposits to the ground surface where it affects the considered structure. Different types hazards induced by the seismic action threat the structure. In this section different main three seismic hazards are described.

2.2.1. Strong shaking
Bedrock motion
Bedrock motions used for seismic analysis for a specific site are characterized through a seismic hazard analysis. The bedrock motion is based on the earthquake source parameters and the wave propagation effects in the soil deposits on the source-to-site path (PIANC, 2001).

In earthquake engineering the intensity of the bedrock motions is defined in terms of peak ground acceleration (PGA), which is the largest peak in the accelerogram of the recorded earthquake. The level of bedrock motion is in a probabilistic seismic hazard analysis defined as a function of a return period. In codes and standards for specific regions the bedrock motion for a certain probability of exceedance is often specified which has to be used in seismic design methods.
Local site effects
The bedrock ground motion can significantly be influenced by the dynamic response characteristics of local soil deposits. The amplitude, frequency content and duration may be altered due to these local site effects. The effects depend on the local material properties but also on the characteristics of the intensity and frequency of the bedrock motion, there is interaction between the type of motion and the type of material.

Generally soft layers with low stiffness towards the surface tend to amplify the ground motions. Stiffer soil deposits transfer energy to higher frequency range while softer soil properties lead to transfer of energy in the lower frequency range. The stiffness of a soil deposit is non-linearly stress and strain dependent, which makes it complex to exactly predict the behaviour of the ground. For very small deformations the stiffness is relatively high corresponding to small damping ratio. Larger deformations of the soil lead to a lower stiffness and thus a higher damping ratio.

In engineering practice two types of methods are commonly used for the determination of the local site response. The local site effects can be evaluated by using site amplification factors or by a site-specific soil response analysis (Kramer, S.L. (1996), PIANC (2001), Habets, C.J.W. (2015)):

- **Site amplification factors and response spectrum**
  In design codes and standards site amplification factors are often specified for typical regions or soil types. The bedrock PGA and response spectrum can be scaled to define the corresponding values at ground surface.

- **Site-specific soil response analysis**
  Site-specific response analysis is generally performed using a 1D model (2D model with limited width) to obtain the time histories of the motion at ground surface. The non-linear behaviour of the soil is often modelled in first instance by an equivalent linear model. For lower loading levels, low strain levels are expected. In this range the soil behaves more or less elastically and the soil response can be evaluated using equivalent linear methods. The non-linear behaviour of the soil becomes more more relevant for larger strains. Finite element models with suitable constitutive material modes are capable describing this non-linear soil behaviour.

2.2.2. Liquefaction
In loose saturated cohesionless soils under cyclic loading the water pressure in the pores may accumulate, because the soil tends to densify due to the cyclic loading. When the pore pressure increases, the effective stress in the soil decreases. If the excess pore water pressure has risen to a value that the effective stress reaches zero, the strength and shear stiffness of the soil is lost completely. The particles are suspended and the soil behaves like a heavy fluid. Large movements can occur in the ground and after drainage of the water densification and settlement of soil layers is likely to occur. The hazard of earthquake-induced liquefaction is related to the resistance of the soil against the building up of excess pore water pressures and the magnitude and duration of the strong shaking.
Failure as a result of effective stresses that become zero is called cyclic mobility. The failure behaviour of the soil changes significantly when initial static shear stresses are present. Effective stresses do not reach zero, since failure already occurs because of flow failure. In this case shear strength of the soil is already exceeded by the initial static shear level, leading to much larger strains compared to cyclic mobility.

Liquefaction forms a threat for anchored quay walls because of the potential loss of lateral stability and the load increase due to the occurrence of liquefaction. Different evaluation methods to determine the liquefaction potential are elaborated, but also effects on the performance of anchored quay walls are considered (Kramer, S.L. (1996), (PIANC, 2001)).

Possible consequences of liquefaction on anchored quay walls strongly depend on local conditions. Aspects to be considered are local site conditions, the characteristics of the earthquake loading and the nature of structures on site. Depending on the location of the potential liquefiable layer the consequence for the retaining structure will be different. A liquefiable layer in the backfill has other consequences than a liquefiable layer in the passive soil wedge of the structure. Changes of the soil properties are on the basis of the consequences for the anchored quay walls (Zerki, A. et al., 2014).

Three main consequences on anchored quay walls due the occurrence of liquefaction are:

- Loss of lateral stability due to loss of shear strength of passive soil wedge.
- Increase of lateral pressure due to lateral deformations of the soil because of loss of shear strength.
- Increase of lateral pressure and settlement of the structure due to densification of the soil.

These general consequences are on the basis of hazards for structures due to liquefaction. Engineering analysis methods are still not able to reasonably account for all the factors that influence the liquefaction potential, neither predicting the accurately the deformation patterns as a result of the occurrence of liquefaction. Approximations and assumptions are included in liquefaction analysis methods to overcome these problems, which makes them less accurate. After results are found an attempt has to be made to quantify the uncertainty in the predictions (Boulanger, R.W. and Idriss, I.M., 2014).

2.2.3. Tsunami

Besides direct increased loading or decreased resistance, earthquakes may also trigger a tsunami that eventually hits the structure. Forces induced by tsunamis on port structures can be very high compared to other loads. Tsunami hazards won’t be considered in this research, it is however for some regions an important loading event to take into account in the design procedure.

2.3. Performance-based design

Instead of applying conventional force-based design, a trend within seismic design of port structures is the use of performance-based methodology. Structures are designed on displacement criterion in stead of a limiting force. Especially for port structures allowing deformation for smaller events wouldn’t mean that structure doesn’t fulfil its function anymore or even fails, as long as it stays within acceptable limits.

The key parameters are in that case not the internal forces, but the deformation of ground and structure and the corresponding stress states. Conventional limit-equilibrium-based methods are not well suitable for the evaluation of these parameters (PIANC, 2001). This new design approach leads to the need of using more sophisticated design methods. Habets, C.J.W. (2015) did research to the applicability of different types of design methods for performance based design of anchored quay walls. The performance of pseudo-static-, simplified dynamic- and full dynamic analysis for designing a typical anchored quay wall was investigated. It was concluded that a reduction factor of 45% to 50% could be applied on the the pseudo-static method for the considered case to avoid over dimensioning. The full dynamic analysis using PLAXIS 2D lead to the most accurate results. In this research risk of the occurrence of liquefaction was however excluded.
Performance criteria
Before being able to assess the performance of a structure, the reference loading level has to be defined on which the performance is assessed. In earthquake engineering generally two types of design motions are used. The first is an earthquake motion that is likely to occur during the lifetime of the structure, and secondly a more rare seismic event with larger accelerations. The magnitude of these motions depends on the location and with a certain probability of occurrence.

Damage criteria
If the earthquake motions are known the acceptable level of damage given these motions has to be specified. How much damage is considered to be acceptable depends on the needs of the users and on the function of the structure. Different types of damage can be distinguished. Firstly, the structural damage which is related to the amount of work to restore the function of the structure. Next to this type of damages also the economical losses related to time and costs to restore the serviceability of the structure are part of the total damage.

The amount of acceptable damage is related to the required performance of the structure. In design codes often several (importance/performance) classes are distinguished, where for the higher classes less damage is accepted. These are important or valuable structures where loss of life or high economical losses are a consequence of failure of the structure.

2.4. Seismic failure anchored quay walls
Typical failure modes during a seismic event depend both on structural and geotechnical conditions, see Figure 2.4. The failure modes indicated with (a) are related to unacceptable deformation or failure at anchor, (b) are related to failure at sheet pile wall and tie-rod and (c) shows failure at the passive wedge.

![Figure 2.4: Failure modes of anchored sheet pile quay wall (PIANC, 2001)](PIANC, 2001)
Important design aspect is to determine what the preferred sequence and degree of ultimate limit states of different structural elements is. The structure should always warn before it fails, in other words it should deform to indicate that the capacity is reached (PIANC, 2001).

Damage criteria can be subdivided in two categories and are specified according to the following parameters: displacements (sheet pile or anchor) or stresses (sheet pile, tie-rod or anchor). In Figure 2.5 several criteria are presented. Depending on the function and importance of the structure the damage criteria based are determined on beforehand, as discussed in performance-based design method. In the important variables to consider in the damage criteria are shown.

Figure 2.5: Parameters related to displacements (a) and stresses (b) used for specifying the damage criteria for an anchored quay wall (PIANC, 2001)

Considering all failure modes, the damage criteria should be defined by the most restrictive condition but sudden failure of the structure must be avoided at all time (PIANC, 2001).

2.5. Seismic analysis

In performance based design the seismic response of the anchored quay wall is evaluated with respect to allowable limits, as displacements, stresses and strains. Several analysis methods are available for the evaluation of local site effects and structural assessment with different levels of sophistication can be distinguished:

- Simplified analysis
  Appropriate for evaluation of approximate threshold limit for displacements and elastic response limit, also able to give an estimate of the order-of-magnitude of the permanent displacements.

- Simplified dynamic analysis
  Based on assumed failure modes the extent of displacements, stresses and strains can be evaluated.

- Dynamic analysis
  The failure modes can be evaluated together with the extent of the displacement, stresses and strains.

The less advanced models are generally used for the preliminary design phase. For higher performance classes and more detailed designing more sophisticated analysis is performed. It is however desirable for each type of method to validate the applicability for the analysis of the structure by suitable case histories or model test.

In this research only the simplified analysis (pseudo-static method) and the dynamic analysis are considered. This is done because the pseudo-static methods are prescribed in design methods and dynamic analysis is emerging, since computational possibilities has increased significantly.
2.5.1. Simplified analysis
Pseudo-static seismic design methods are originally based on the assumption that the structure doesn’t experience any displacement. This results in relatively high internal forces. In most of the seismic design codes reduction factors are included that account for some deformation, based on the importance level and allowable damage. Allowing some deformation results in lower internal forces (Kramer, S.L. (1996), PIANC (2001), Nederlands Normalisatie-instituut (2005)).

The simplified analysis is based on the limit equilibrium method, for some type of structures supported by statistical analysis of case histories. The seismic design codes and standards all adopted this type of analysis, each with own background and thus (slight) differences. One of the most used method to determine the total earth pressures during a seismic event is the method developed by Mononobe-Okabe. This method is extensively described in section 2.7, including extensions that account for other configurations than the method was originally based on.

In the simplified analysis one peak ground acceleration (PGA) is used that accounts for the considered earthquake motions. When a pseudo-static method is used, this PGA is translated into an equivalent seismic coefficient representing the earthquake motion. These parameters are obtained from the analysis of local site effects. The required capacity of the structure to resist the seismic loading is determined based on structural and geotechnical conditions. This capacity is often given in terms of a critical acceleration or seismic coefficient beyond which the schematized structure starts to move. Occurrence of liquefaction can not be taken into account in this type of method but effects can artificially be included through extra factors.

2.5.2. Dynamic analysis
For the dynamic analysis generally finite element models (FEM) or finite difference models (FDM) are used that include soil-structure interaction. For finite element modelling a constitutive material model has to be adopted, which is able to describe the behaviour of the soil. It describes the relation between the stresses and the strains in the soil. The models are formulated with soil properties and state parameters. The state parameters are related to a certain point in time, and thus these parameters may change in time. A set of time histories of the earthquake motions is applied at the base of the system. Depending on the magnitude of the earthquake motion relative to the structure’s elastic limit, the structure is modelled either linear or non-linear. The soil representation depends on the expected strain level in the soil during the design earthquake. It is mostly modelled by effective stress model.

Results obtained from this type of analysis include the failure modes of the system and the extent of the displacements, stresses and strain states. The analysis is however sensitive to the modelling choices made and the input of parameters (Kramer, S.L. (1996), PIANC, 2001).

2.6. Liquefaction
When loose cohesionless soil is loaded under cyclic loading it tends to contracts. In case the saturated soil is loaded under undrained conditions, the load acts in a short time, this causes an increase in the excess pore water pressure, see Figure 2.6. Consequently the effective stresses in the soil decreased, in some cases even to zero. Reaching an effective stress state of zero also means that the shear resistance is zero (since \( \tau = \sigma'_{v0} \cdot tan(\varphi) \)) and the soil acting like a fluid.

Damage from liquefaction itself (sand boils) is of minor interest. It is the loss of strength and stiffness in the liquefied stage and the associated deformations of the ground that causes damage to structures.

Characteristics of the soil that have influence on the potential hazard of the occurrence of liquefaction are the particle size, particle shape, gradation plasticity characteristics. The triggering factors depend on the magnitude of the earthquake, the duration and the peak ground acceleration.

Significant strains that result in ground deformations during earthquake loading can also develop in cohesive soils, like clays and plastic silts. In case of cohesive soils it is preferable to use the term “cyclic softening” instead of liquefaction. Distinction is made because of the difference in shear strength characteristics between cohesionless and cohesive soils. This has influence on the choice and outcome of procedures for evaluating the seismic response of soils (Boulanger, R.W. and Idriss, I.M., 2014).
2.6.1. Evaluation liquefaction hazard

The liquefaction potential at a specific site for a design earthquake can be determined according to semi-empirical relations (most commonly stress-based) or using dynamic finite element effective stress models. Within the semi-empirical procedures, the safety factor is evaluated which is the ratio between the amount of cyclic shear stress (CRR) required to cause liquefaction and the equivalent cyclic shear stress (CSR) that is induced by the design earthquake. In dynamic methods, a one-dimensional wave propagation analysis in terms of effective stresses is performed. This gives the possibility to calculate the pore pressure ratio at any depth (Laera, A. and R.B.J. Brinkgreve, 2015). In this section, for both types of approaches, examples of analysis methods are presented.

Semi-empirical methods

There have been several approaches to assess the potential of liquefaction to occur. The stress-based approach is most widely used. The semi-empirical procedure introduced by Seed, H.B. and Idriss, I.M. (1971) and updated by Idriss, I.M. and Boulanger, R.W. (2008) is generally used.

The basis of the development of analytical procedures for the assessment of liquefaction potential is empirical data, which provides the link between the liquefaction resistance and in-situ conditions. Laboratory test can help improving the liquefaction potential assessment methods.

The assessment leads to a safety factor (FoS) that represents the safety against liquefaction, and must be at least larger than 1. By empirical fitting this safety factor is related to a value for the excess pore pressure ratio \( r_u \) in a specific soil layer. This excess pore pressure ratio is defined as the excess pore pressure over the initial vertical effective stress (Boulanger, R.W. and Idriss, I.M., 2014).

- **Liquefaction potential by Idriss and Boulanger (2008)**

  The safety factor is the ratio between the resistance against liquefaction represented by cyclic resistance ratio (CRR) over the cyclic loading by means of cyclic stress ratio (CSR). If the safety factor is lower than 1.0, the soil is completely liquefied. For values slightly higher than 1.0 the soil is not liquefied, but the shear strength is already reduced significantly. Equation 2.3 is presents the safety factor developed by Idriss and Boulanger (2008). Besides the ratio of the resistance over the loading, also some correction factors are included. These factors are used to fit the site-specific properties to the set of case histories.

  \[
  Fos_{liq} = \frac{CRR_{75}}{CSR} \cdot MSF \cdot K_\sigma \cdot K_a
  \]  

  (2.3)
• Cyclic stress ratio (CSR)
Vertical propagating horizontal shear waves are primarily responsible for the shear stresses induced in the soil column. With the following relation the induced cyclic stress ratio can be determined:

\[ CSR = 0.65 \cdot \frac{a_{max}}{g} \frac{\sigma_v}{\sigma_{v0}} \cdot r_d \] (2.4)

In this relation \(a_{max}\) is the horizontal peak ground acceleration in \(m/s^2\), \(g\) is the gravitational acceleration in \(m/s^2\), \(\sigma_v\) is the total vertical stress in \(kN/m^2\), \(\sigma_{v0}\) is the total effective vertical stress in \(kN/m^2\) and \(r_d\) is the shear stress reduction coefficient. The value of 0.65 is used to represent the earthquake-induced cyclic stresses by a representative value, which is 65% of the peak cyclic stress. This value is chosen somewhat arbitrary, but was selected since the beginning of the development of liquefaction evaluation procedures. All adjustment factors are and empirically derived liquefaction correlations from were derived for this reference stress.

• Cyclic resistance ratio (CRR; 7.5)
The cyclic resistance ratio can either be expressed in terms of normalized cone resistance \((q_{c1N})\) based on results of a CPT-test, or in terms of the corrected SPT blow counts \((\langle N \rangle_{60})\) based on a SPT-test. For both type of tests the results and corresponding cyclic resistance are compared with the results of case histories. An empirical method is developed for both type of tests where normalized values of the test results are used to determine the cyclic resistance.

\[ \text{CRR based on normalized corrected SPT blow count } \langle N \rangle_{60,cs} \]:
The measured SPT blow count \(N_{SPT}\) is affected by the procedure and type of equipment and therefore has to be standardized. The standardized N-value can be calculated as:

\[ \langle N \rangle_{60} = C_E \cdot C_R \cdot C_B \cdot C_S \cdot N_{SPT} \] (2.5)

In this equation several correction factors are included for the type of equipment and the followed procedure. The variable \(C_E\) (also \(E_M\)) is the energy ratio correction factor for the type of equipment, \(C_B\) is a correction factor for the borehole diameter, the \(C_R\)-factor corrects for the rod length and \(C_S\) is correction factor for the sampling method. In Figure 2.7 the magnitude of the correction factors are presented.

![Figure 2.7: On the left values for \(C_E, C_S\) and \(C_B\) developed by Skempton, A.W. (1986) and on the right energy ratio correction factors by Clayton, C.R.I. (1990)](image)

In order to account for the stress state of the soil the standardized N-value is multiplied by an overburden factor \((C_N)\):

\[ \langle N \rangle_{60} = C_N \cdot \langle N \rangle_{60} \] (2.6)
2.6. liquefaction

The value of $C_N$ has to be determined in an iterative calculation:

$$C_N = \left( \frac{p_a}{\sigma'_{v0}} \right)^\beta \leq 1.7$$

$$\beta = 1.338 - 0.0768 \sqrt{(N_1)_{60}}$$

In which $p_a$ is the atmospheric pressure, $\sigma'_{v0}$ is the effective vertical stress and $(N_1)_{60}$ is the normalized standardized blow count.

The liquefaction case histories suggest that the higher the amount of fines in the soil, the larger the resistance against liquefaction is. The value for $(N_1)_{60,cs}$ is the value for equivalent clean sand including the influence of the fines content (FC in %), represented by $\Delta(N_1)_{60}$.

$$(N_1)_{60,cs} = (N_1)_{60} + \Delta(N_1)_{60} \quad (2.7)$$

With:

$$\Delta(N_1)_{60} = exp\left( 1.63 + \frac{9.7}{FC+0.01} - \left( \frac{15.7}{FC+0.01} \right)^2 \right) \quad (2.8)$$

Finally the $(N_1)_{60,cs}$ value is used in the empirical relation to determine the cyclic resistance ratio of the soil:

$$CRR_{7.5} = exp\left( \frac{(N_1)_{60,cs}}{14.1} + \left( \frac{(N_1)_{60,cs}}{126} \right)^2 + \left( \frac{(N_1)_{60,cs}}{23.6} \right)^3 + \left( \frac{(N_1)_{60,cs}}{25.4} \right)^4 - 2.8 \right) \quad (2.9)$$

This value is true for an overburden pressure of 100 kPa and an earthquake magnitude of 7.5. For other earthquake magnitudes or overburden pressures this value $CRR_{7.5}$ has to be corrected according to the magnitude scaling factor (MSF) or the overburden correction factor ($K_o$), those are described below.

CRR based on normalized cone resistance $q_{c1N}$:

The cyclic resistance ratio based on the cone tip resistance $q_c$ is not dependent on the type of equipment. However to be able to compare the resistance of the soil with the case histories the cone tip resistance has to be normalized.

The cone tip resistance is multiplied by the overburden correction factor $C_N$ to account for the stress state of the soil. This $C_N$ value can again be found with an iterative calculation.

$$q_{c1N} = C_N q_c \quad (2.10)$$

With:

$$C_N = \left( \frac{p_a}{\sigma'_{v0}} \right)^\beta \leq 1.7$$

$$\beta = 1.338 - 0.249(q_{c1N})^{0.264}$$

To account for the fines content (FC) the normalized cone tip resistance has to be adjusted according to equation 2.11.

$$q_{c1N,cs} = q_{c1N} + \Delta q_{c1N} \quad (2.11)$$

Where:

$$\Delta q_{c1N} = \left( 5.4 + \frac{q_{c1N}}{16} \right) \cdot exp\left( 1.63 + \frac{9.7}{FC+0.01} - \left( \frac{15.7}{FC+0.01} \right)^2 \right) \quad (2.12)$$

Finally this normalized cone tip resistance $q_{c1N}$ can be used in the empirical relationship to determine the cyclic resistance ratio of the soil, see equation 2.13. Again this value is valid for an earthquake with magnitude of 7.5 and an overburden pressure of 100 kPa.

$$CRR_{7.5} = exp\left( \frac{q_{c1N,cs}}{540} + \left( \frac{q_{c1N,cs}}{67} \right)^2 - \left( \frac{q_{c1N,cs}}{80} \right)^3 + \left( \frac{q_{c1N,cs}}{114} \right)^4 - 3 \right) \quad (2.13)$$
2. Theoretical background

- **Magnitude scaling factor (MSF)**
  In literature all correlations are based on earthquakes with a magnitude of 7.5. The induced CSR has to be adjusted in case of an earthquake with magnitude other than 7.5, which is done with the magnitude scaling factor (MSF). For an earthquake with magnitude 7.5 this value is equal to 1.0. The MSF increases for decreasing magnitudes and has a maximum of 1.8, based on the fact that every earthquake has at least one loading cycle.

  \[
  MSF = \min (6.9 \cdot \exp\left(-\frac{M}{4}\right) - 0.058; 1.8)
  \] (2.14)

- **Overburden correction factor \((K_\sigma)\)**
  The atmospheric pressure was used as the reference stress in the calculation of CSR and CRR. When the effective stress level in the soil higher, the over burden correction factor \((K_\sigma)\) has to be applied. The factor depends on the vertical effective stress \((\sigma'_0)\) and on a correction factor based on soil test results (either SPT- or CPT test).

  \[
  K_\sigma = 1 - C_\sigma \cdot \ln\left(\frac{\sigma'_0}{P_a}\right)
  \] (2.15)

  With:

  \[
  C_\sigma = \frac{1}{18.9 - 2.55 \sqrt{\left(N\right)_{60,cs}}} \leq 0.3
  \] (2.16)

  \[
  C_\sigma = \frac{1}{37.3 - 8.27 \left(q_{c1,Neq}\right)^{0.264}} \leq 0.3
  \] (2.17)

- **Static shear stress correction factor \((K_\tau)\)**
  Within for example slopes or embankment dams static shear stresses are present, which affect the CRR. Several relationships have been developed for the value of \(K_\tau\), taking into account the effect of initial static shear on the liquefaction potential. The validity of these relationships for in-situ sands is difficult to assess, because there is an insufficient amount of data available. The following expressions are developed by Idriss and Boulanger (Idriss, I.M. and Boulanger, R.W., 2008):

  \[
  K_\tau = a + b \cdot \exp\left(-\frac{\xi_R}{c}\right)
  \] (2.18)

  \[
  a = 1267 + 636\alpha^2 - 634 \cdot \exp(\alpha) - 632 \cdot \exp(-\alpha)
  \] (2.19)

  \[
  b = \exp(-1.11 + 12.3\alpha^2 + 1.31 \cdot \ln(\alpha + 0.0001))
  \] (2.20)

  \[
  c = 0.138 + 0.126\alpha + 2.52\alpha^3
  \] (2.21)

  \[
  \alpha = \frac{\tau_s}{\sigma'_{vc}}
  \] (2.22)
2.6. Liquefaction

\[
\xi_R = \frac{1}{Q - \ln \left( \frac{100(1+2K_0)\sigma'_{cc}}{3p_a} \right)} - \sqrt{\frac{((N_1)_{60}}{46)}
\]

\[
\xi_R = \frac{1}{Q - \ln \left( \frac{100(1+2K_0)\sigma'_{cc}}{3p_a} \right)} - \left(0.478(q_{c1N})^{0.264} - 1.063\right)
\]

With the following constrains: \(0 \leq 0.35\) and \(-0.6 \leq \xi_R \leq 0.1\)

- **Excess pore pressure ratio** \((r_u)\)
  
  If the factor of safety against liquefaction is still larger than 1.0 the soil is not fully liquefied, but excess pore pressures can still be present. In order to predict to what extent excess pore pressures develop, a relation is developed between the factor of safety against liquefaction and the excess pore pressure ratio \((r_u)\) (Marcuson, W. et al., 1991)

  \[
  r_u = \frac{2}{\pi} \cdot \arcsin \left( \frac{FoS^{1/2\theta}}{\beta} \right)
  \]

  In this equation the values \(b\) and \(\theta\) are empirical constants. Tokimatsu and Yoshimi (1983) prescribes empirical constants are \(b = 0.20\) and \(\theta = 0.70\). Application of these empirical constants leads to the relation indicated by the red line in Figure 2.9, which is originally developed for sand. In the NPR 9998 this is the representative value of the excess pore pressure ratio.

![Figure 2.9: Excess pore pressure ratio against the factor of safety against liquefaction for FoS > 1 (Marcuson, W. et al., 1991).](image)

**Finite element methods**

Where the semi-empirical models only give a value for the excess pore pressures based on the safety factor against liquefaction, finite element models can provide more insight in the actual development of excess pore water pressure in the soil. In addition the models also can give an impression of the expected deformations. To obtain proper results an extensive calibration procedure is required. For this procedure many properties of the soil have to be known, which include mostly a lot of uncertainty (Winde, H.P., 2015).

To predict the onset of liquefaction the constitutive model must be able to cope with cyclic loading and able to accumulate excess pore pressures. The building up of pore water pressure that results in the gradual loss of strength of the soil can be captured with an effective stress model. An example of a material model that is able to accumulate excess pore pressures is the UBC3D-PLM constitutive model, which is a user-defined model within PLAXIS (Makra, A. (2013), Petalas, A. and Galavi, V. (2013)). In Appendix C the background of this model is elaborated.
2.6.2. Failure modes related to liquefaction

Depending on the location of the liquefiable layer the effects on the performance of the anchored quay wall is different. In Figure 2.10 different situations are presented.

(a). Because of the increase of pore pressures, the effective stresses in the soil decrease. This leads to a loss of resistance of the anchor, which could ultimately result in pulling out of the anchor is embedded in a liquefiable layer.

(b). Presence of a liquefiable layer in the backfill leads to an increase of loading on the retaining structure. When excess pore pressures develop, the resistance between particles decreases leading to an increase of horizontal earth pressure. Together with the excess pore pressures this leads to an increase of load against the wall.

(c). When a the sheet pile is embedded in a liquefiable layer the stability of whole structure could be affected if excess pore pressures develop in the passive soil wedge. Effective stresses decrease, leading to lower resistance of the soil. This could ultimately cause stability problems of the wall.

Loss of stability of the anchor and/or the sheet pile has to be prevented at any time, because this leads to large deformations and mostly complete failure of the structure.

![Figure 2.10: Possible failure mechanisms related to the occurrence of a liquefiable layer. (a). Failure of the anchor because of decreased soil strength. (b). Structural failure of the sheet pile wall due to load increase in the (partly)liquefied layer. (c). Loss of stability of sheet pile wall because of decrease of passive soil strength.](image-url)
2.6.3. Mitigation measures

The performance of port structures subjected to seismic loading can be improved significantly by remediation of liquefiable soils. Remediation solutions can generally be divided into two categories:

- **Soil improvement**
  which is focused on reducing the probability of the occurrence of liquefaction. This can be done either by improving the performance of the soil or by increasing the dissipation rate of excess pore pressure or by a combination.

- **Structural design**
  to minimize the damage in case of a liquefaction occurs. One of the strategies is reinforcing the structure in terms of strength and stiffness. Another option is reducing the consequences of liquefaction by the use of flexible joints or other structural measures. A combination of measures is common practice.

See Figure 2.11 for a flow chart with possible choices in the liquefaction remediation strategy. After selecting a strategy different remediation measures can be taken. Suitable measures are selected and weighted against one another, and often a combination of different measures is very effective. No general procedure is available for selecting the most efficient remediation, so for each situation engineering judgement is required to select the most optimal solution (PIANC (2001), Boulanger, R.W. and Idriss, I.M. (2014)).

The type of soil improvement to be applied is based on an assessment of the specific liquefaction related

![Figure 2.11: Suggested basic strategy for liquefaction remediation by PIANC (PIANC, 2001)](image-url)

failure mechanism. Because effects of liquefaction are often uncertain and hard to predict, soil improvement is generally preferred above structural design measures. ((PIANC, 2001), Zerki, A. et al. (2014)).
2.7. Pseudo-static methods

The Mononobe-Okabe method is the most used pseudo-static theory for the determination of the dynamic earth pressures during a seismic event. The total loading during a seismic event is a combination of both static and seismic loads. In this section methods to calculate the total loads on an anchored quay wall during a seismic event are presented.

Several seismic design codes from all over the world are available for designing earthquake resistant anchored quay walls. Pseudo-static methods are described within these seismic design codes. No overview of what theory is described in what design code is made. The design codes are used to analyse which methods are prescribed to determine loads on retaining structures by seismic loading.

First static loading is considered, where static earth and water pressures are considered. Then the way the seismic action is defined is discussed. Once the seismic action is defined the original Mononobe-Okabe method is described to determine the total seismic earth pressures. The effects of the presence of water is also discussed. Distinction is made between free standing water and water within the hydraulic backfill. The method to determine the dynamic pressure of free standing water is treated. Also the effects of the earth pressure by the presence of water is elaborated. Finally the effects of excess pore pressures within the backfill are discussed, where two methods to take these effects into account are presented.

2.7.1. Static lateral earth pressure

Static lateral earth pressures are the result from the presence of an earth body next to the retaining structure. The magnitude of the lateral earth pressure depends on several properties of the soil itself.

The magnitude of the lateral earth pressure however also depend strongly on the movement of the wall with respect to the initial situation. Without any movement neutral earth pressures \((K_0)\) act on the retaining structure. Active earth pressures \((K_A)\) develop when the retaining structure moves away from the soil. Very little movement is enough to mobilize the minimum active earth pressure on the wall, which is smaller than the neutral earth pressure. On the other hand, when the retaining structure moves towards the soil, larger passive earth pressures \((K_P)\) develop (Verruijt, A., 2010). Two most commonly used theories to determine static lateral earth pressures are developed by Coulomb and Rankine.

Coulomb (1776)

The method applied in this research is the theory of Coulomb (Coulomb, C.A., 1776), which is originally developed for cohesionless soils. The method is valid under the assumption that the retaining structure can move sufficiently to fully develop the active and passive earth pressures. The weight of a soil wedge above a planar failure surface results in a force acting on the retaining structure. The size of the soil wedges for both the minimum active and maximum passive conditions are determined using force equilibrium.

The active thrust on the retaining structure in cohesionless soil is for the critical failure surface is:

\[ P_A = \frac{1}{2} K_A \gamma_s H_{wall}^2 \]  (2.26)

With:

\[ K_A = \frac{\cos^2(\varphi - \theta)}{\cos^2 \theta \cos(\delta + \theta) \left[ 1 + \sqrt{\frac{\sin(\delta + \varphi) \sin(\varphi - \theta)}{\cos(\delta + \theta) \cos(\beta - \theta)}} \right]^2} \]  (2.27)

In the same way the maximum passive resulting force can be determined:

\[ P_P = \frac{1}{2} K_P \gamma_s H_{wall}^2 \]  (2.28)

With:

\[ K_P = \frac{\cos^2(\varphi + \theta)}{\cos^2 \theta \cos(\delta - \theta) \left[ 1 - \sqrt{\frac{\sin(\delta + \varphi) \sin(\varphi + \theta)}{\cos(\delta - \theta) \cos(\beta - \theta)}} \right]^2} \]  (2.29)

For homogeneous backfills with flat surfaces by the absence of surface loads the lateral earth pressure distribution is triangular, with a point of action at one third of the layer above the base of the wall layer.
2.7. Pseudo-static methods

The Coulomb theory overestimates the passive soil pressure for an increasing wall-friction angle. This can be explained by the fact that the shape of the failure plane becomes more curved with increasing wall-friction angles, there where Coulomb assumes planar failure plane. Resulting in an increasing error. For this reason Coulomb should not be applied when $\delta > \phi/2$.

2.7.2. Dynamic earth pressure

An important issue during earthquakes involves predicting the loads imposed on the retaining wall. Estimating the actual loading during an earthquake is complicated, therefore seismic earth pressures are often estimated using simplified methods. In this research the pseudo-static Mononobe-Okabe method is described to determine dynamic pressures acting on the retaining wall. Several extensions on the methods are available depending on the situation. However the method also has some major limitations, which are also described and shown.

Mononobe-Okabe (1929)

The Mononobe-Okabe method is an extension of the static earth pressure theory of Coulomb and is developed by (Okabe, S., 1926) and Mononobe, N. and Matsuo, H. (1929). The method forms the basis of a pseudo-static analysis of seismic earth pressure. Accelerations due to an earthquake act on the soil wedges that are defined by Coulomb. Next to the static earth pressures, additional horizontal and vertical pseudo-static forces act on the soil wedge. The total active thrust is increased compared to the static case, and the total passive thrust is lower compared to the static situation. The magnitude is related to the mass of the wedge and the accelerations (Mononobe, N. and Matsuo, H. (1929), Okabe, S. (1926)). The method is originally developed for a dry cohesionless homogeneous backfill.

In case of a dry backfill the inclination angle of the seismic coefficient with respect to the vertical is determined as follows:

$$\theta = \tan^{-1}\left(\frac{k_h}{1 - k_v}\right)$$

Active earth pressure

The total dynamic active earth pressure resultant force according to Mononobe-Okabe is determined as follows:

$$P_{AE} = \frac{1}{2} \cdot K_{AE} \cdot \gamma H_{wall}^2 \cdot (1 - k_v)$$

The dynamic active earth pressure coefficient $K_{AE}$ is given by:

$$K_{AE} = \frac{\cos^2 (\rho - \Psi - \theta)}{\cos \theta \cos^2 \Psi \cos (\delta + \Psi + \theta) \left[1 + \sqrt{\frac{\sin (\delta + \Psi) \sin (\delta - \beta + \Psi) \sin (\delta - \beta - \Psi) \cos (\beta - \Psi)}{\cos (\delta + \Psi + \theta) \cos (\delta - \beta - \Psi)}}\right]^2}$$

This equation is valid under the condition that: $\rho - \beta \leq \theta$

The Mononobe-Okabe method gives the total active earth pressure. In order to determine the point of action of the total thrust, the thrust is first into a static component $P_A$, found by Coulomb, and a dynamic component $\Delta P_{AE}$:

$$P_{AE} = P_A + \Delta P_{AE}$$
2. Theoretical background

The height point of action of the static component is known to be at one third of the height of the wall. The dynamic component acts at approximately $0.6 \cdot H$ above the base of the wall. The total active thrust thus acts at a height:

$$h = \frac{P_A(H/3) + \Delta P_{AE}(0.6 \cdot H)}{P_{AE}}$$  \hspace{1cm} (2.34)

**Passive earth pressure**

Regarding the total passive earth pressure, the resultant force can be determined in the same way as for the active earth pressure:

$$P_{PE} = \frac{1}{2} \cdot K_{PE} \cdot \gamma \cdot H_{wall}^2 \cdot (1 - k_v)$$  \hspace{1cm} (2.35)

The dynamic passive earth pressure coefficient $K_{PE}$ can be found by:

$$K_{PE} = \frac{\cos^2(\varphi + \Psi - \theta)}{\cos \theta \cos^2 \Psi \cos(\delta - \Psi + \theta) \left[ 1 - \frac{\sin(\delta + \varphi) \sin(\varphi + \beta - \theta)}{\cos(\delta - \Psi + \theta) \cos(\beta - \Psi)} \right]^2}$$  \hspace{1cm} (2.36)

This equation is valid under the condition that: $\varphi - \beta \leq \theta$

The Mononobe-Okabe method also gives the total passive thrust. This force can again be divided into a static and dynamic component:

$$P_{PE} = P_P + \Delta P_{PE}$$  \hspace{1cm} (2.37)

The static component can again be determined according to Coulomb, together with the total passive thrust the dynamic component can be found.

### 2.7.3. Layered soil

Often the backfill behind a retaining structure is not homogeneous. Originally the considered methods to calculate the lateral (dynamic) earth pressure are developed for a homogeneous backfill. In order to take layering of the backfill into account another approach is suggested.

For each soil layer the properties have to be identified. Then the earth pressure coefficient for both active and passive side can be determined for each layer. The resultant loads on the wall for all the individual soil layers can be determined. This can be done both for static and for dynamic conditions (Carl Borg, R. (2007), Ports and Harbours Bureau, Ministry of Land, Infrastructure, Transport and Tourism (MLIT) et al. (2009)).
2.7.4. Effects of water

Retaining structures are often built in waterfront areas, resulting in presence of water in the backfill. Often free standing water is present in front of a retaining structure. Therefore it is of importance to consider the effects of water both in front of and in the backfill of a retaining structure. This is an extension of the Mononobe-Okabe method, which was originally developed for a dry backfill (Kramer, S.L. (1996), PIANC (2001)).

Dynamic water pressure

The dynamic response of free standing water due to the seismic action results in a hydrodynamic water pressure on the structure. This hydrodynamic water pressure can be determined according to method by Westergaard, H. (1931). This solution is originally derived for the case of a vertical and rigid dam that retains a semi-infinite reservoir of water. This reservoir was excited by a harmonic horizontal motion of its rigid base. The solution is however generally used, also for different configurations (Kramer, S.L., 1996).

The amplitude of the hydrodynamic pressure according to Westergaard is:

\[ p_w = \frac{7}{8} k_h \gamma_w \sqrt{z_w H} \]  
(2.38)

The resultant hydrodynamic thrust is given by:

\[ P_w = \frac{7}{12} k_h \gamma_w H^2 \]  
(2.39)

The total water pressure on the retaining structure is the sum the hydrostatic and the hydrodynamic water pressures.

Water in backfill

The presence of water in the backfill behind a retaining structure influences the seismic loads on the structures. Several cases can be distinguished (Transportation Research Board (2008), Carl Borg, R. (2007), Kramer, S.L. (1996)):

- **Inertial forces altered within the backfill**

  Inertial forces in saturated soils depend on the relative movement between the soil and porewater. If the permeability of the soil is small enough \((k \leq 10^{-3} \text{ cm/s})\) water in the pores moves with the soil during earthquake shaking. In that case it is assumed that the horizontal inertia force is proportional to the total saturated weight of the soil, \(\gamma_{sat}\), and the gravity force is proportional to the buoyant unit weight, \(\gamma_b = \gamma_{sat} - \gamma_w\).

  The horizontal seismic coefficient is modified, which is taken into account in the determination of the inclination angle of the seismic coefficient with respect to the vertical:

  \[ \theta = \tan^{-1} \left( \frac{\gamma_{sat} - \gamma_w}{k_h \frac{\gamma_{sat} - \gamma_w}{1 - k_v}} \right) \]  
(2.40)

- **Hydrodynamic pressures develop within the backfill**

  When the permeability of the backfill soil is very high, the soil and water may act independently. The porewater can be assumed to remain more or less stationary while the soil skeleton moves back and forth. Inertial forces are in such cases proportional to the dry unit weight of the soil.

  \[ \theta = \tan^{-1} \left( \frac{\gamma_{dry}}{\gamma_{sat} - \gamma_w} \frac{k_h}{1 - k_v} \right) \]  
(2.41)

  Hydrodynamic water pressures can also develop under these free pore-water conditions. These dynamic water pressures can be calculated according to Westergaard and must be added to the hydrostatic water pressures to obtain the total loading by the water on the wall.
There is however no agreement about the transition zone between the state where water and soil move independently and there they move as a single body. Permeability values $k$ ranging from $10^{-2}$ cm/s to 10 cm/s have been reported. Research is needed to provide better insight in the location of this transition zone.

- **Partially saturated backfill**

Earth pressures from partially submerged backfills may be computed using an average unit weight, in that case the saturated unit weight may be replaced by the average unit weight. This average weight is based on the relative volumes of soil within the active wedge that are above and below the phreatic surface:

$$\gamma_{eq} = \lambda^2 \gamma_{sat} + (1 - \lambda^2) \gamma_b$$

(2.42)

Another method for the determination of the soil weight properties is given in PIANC (2001). The equivalent buoyant unit weight of the backfill soil is given by:

$$\gamma_e = \frac{\gamma_{wet} \left( (H + D_{emb})^2 - (H_{sub} + D_{emb})^2 \right) + \gamma_b (H_{sub} + D_{emb})^2}{(H + D_{emb})^2}$$

(2.43)

The equivalent saturated unit weight can be calculated according to:

$$\gamma_{e-sat} = \frac{\gamma_{wet} \left( (H + D_{emb})^2 - (H_{sub} + D_{emb})^2 \right) + \gamma_{sat} (H_{sub} + D_{emb})^2}{(H + D_{emb})^2}$$

(2.44)

- **Apparent Seismic Coefficient**

Another approach is directly adjusting the horizontal seismic coefficient for the (partial) saturated backfill, which is called the apparent seismic coefficient (Ports and Harbours Bureau, Ministry of Land, Infrastructure, Transport and Tourism (MLIT) et al., 2009):

$$k_h' = \frac{2 \left( \sum \gamma_i h_i + \sum \gamma h_j + \omega \right) + \gamma h}{2 \left[ \sum \gamma_i h_i + \sum (\gamma - 10) h_j + \omega \right] + (\gamma - 10) h} \cdot k_h$$

(2.45)
2.7. Pseudo-static methods

Excess pore pressures within backfill

In loosely packed saturated cohesionless soils excess pore water pressures can develop that affect the magnitude of the dynamic earth pressures. The original Mononobe-Okabe method has to be modified to take into account the loss of strength of the soil (Matsuzawa, H. et al., 1985). Due to excess pore pressures active earth pressures increase and the passive earth pressures decrease with respect to the saturated situation. Next to the adjustment of the dynamic earth pressures also the water pressure acting on the retaining structure have to be adjusted as a result of excess pore pressure. A distinction is made between the increase of earth pressures and water pressures, although the two are both related to the pore pressure ratio \( r_u \).

The pore pressure ratio represents the amount of excess pore water pressure in the backfill with respect to the effective earth pressure:

\[
ru = \Delta u / \sigma'_v
\]

There is no rigorous approach adapting the Mononobe-Okabe solution to take into account effects of excess pore water pressure for determining the total earth pressures. There are however two approaches suggested:

- **Adjusting seismic inclination angle**
  
  The first approach is to adapt the seismic inclination angle in order to take the effect of excess pore pressure into account. This is done by adjusting the effective unit weight of the soil in the following way (Kramer, S.L. (1996), Ebeling, R.M. and Morrison, E.E. (1993)):

\[
\gamma_{liq} = \gamma_b (1 - r_u)
\]

This can be explained by the fact that the submerged unit weight of the soil decreases due to the increasing water pressure. The effective unit weight can be substituted into the equation to determine the seismic inclination angle, which gives:

\[
\theta = \tan^{-1} \left( \frac{\gamma_{sat} k_h}{\gamma_b (1 - r_u) (1 - k_v)} \right)
\]

This adjusted seismic inclination angle is substituted in the original Mononobe-Okabe equation to determine the total dynamic earth pressures.

- **Adjusting angle of internal friction**
  
  Another approach is using a reduced effective stress internal friction angle, in which the effects of the excess pore water pressures are approximated within the analysis using a simplified shear strength relationship.

\[
\Phi_{liq,d}' = \tan^{-1} (1 - r_u) \tan(\Phi_d')
\]

This adjusted internal friction angle is substituted in the original Mononobe-Okabe equation, together with the seismic inclination angle for saturated soil. The wall friction angle is kept equal.

Effect of excess pore pressure on water pressure

Due to the increasing water pressure in the liquefied layer an equivalent hydrostatic thrust based on increased unit weight of water must be added to the soil thrust. The unit weight of the fluid can be determined as follows:

\[
\gamma_{w,eq} = \gamma_w + r_u \gamma_b
\]

This equivalent unit weight can be substituted to determine the water pressure including the excess pore pressures.

Note that when \( r_u \) approaches 1.0 (fully liquefied backfill), the soil acts like a heavy fluid of equivalent unit weight, \( \gamma_{w,eq} = \gamma_{sat} = \gamma_w + \gamma_b \). On the other hand when \( r_u \) is equal to zero the equivalent unit weight is equal to the static unit weight of water.

In Figure 2.15 and Figure 2.16 the components that contribute to the total horizontal pressure for increasing excess pore pressures are presented. The earth pressures are determined according to both described methods. As can be seen there is an exchange in unit weight between the soil and the water for increasing \( r_u \) value. The unit weight of the soil decreases with the same amount as the unit weight of the water increases.
For the active side, the earth pressure decreases however not with the same amount as the water pressure increases, because of the trend of the earth pressure coefficient. This coefficient increases with increasing $r_u$ value leading to an increase of the total lateral pressure. Since the total lateral pressure is a combination of the lateral earth pressure and the water pressure. For the passive side it is the other way around, the total pressure decreases due to decreasing earth pressure coefficient.

Figure 2.15: Contributions of change unit weight soil and water and earth pressure coefficients to total horizontal pressures by increasing excess pore pressure ratio $r_u$ for adjusting $\theta$ approach.

Figure 2.16: Contributions of change unit weight soil and water and earth pressure coefficients to total horizontal pressures by increasing excess pore pressure ratio $r_u$ for adjusting $\phi'$ approach.
2.7.5. Seismic action

Within the Mononobe-Okabe theory a seismic coefficient is included to account for the magnitude of the earthquake. It is based on the peak ground acceleration and local soil conditions.

Each seismic design code prescribes a method to calculate the horizontal seismic coefficient. Expressions according to different design codes are presented in Table 2.1. The vertical seismic coefficient is assumed to be equal to zero \((k_v = 0)\) for slender structures according to each design code.

Table 2.1: Relationships for horizontal seismic coefficients according to different design codes

<table>
<thead>
<tr>
<th>Eurocode</th>
<th>(k_h = \alpha \cdot (S/r))</th>
<th>(\alpha = \text{ratio acceleration at bedrock to g} )</th>
<th>[-]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(S = \text{soil factor} )</td>
<td>(r = \text{factor depending ductility structure} )</td>
<td>[-]</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>PIANC</th>
<th>(k_h = a_{\text{max}}/g)</th>
<th>(a_{\text{max}} = \text{peak ground acceleration} )</th>
<th>([\text{m/s}^2])</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(k_h = (1/3)(a_{\text{max}}/g)^{1/3})</td>
<td>(a_{\text{max}} &lt; 0.20\text{g})</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(k_h = (1/3)(a_{\text{max}}/g)^{1/3})</td>
<td>(a_{\text{max}} \geq 0.20\text{g})</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>OCDI</th>
<th>(k_c = 0.6(a_{\text{max}}/g))</th>
<th>(a_{\text{max}} = \text{peak ground acceleration} )</th>
<th>([\text{m/s}^2])</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(\alpha_c = \text{corrected peak ground acceleration} )</td>
<td>(D_a = \text{allowable deformation structure} )</td>
<td>([\text{cm}])</td>
</tr>
<tr>
<td></td>
<td>(D_r = \text{standard deformation [10 cm]} )</td>
<td>(D_a/\alpha_{fc}/g + 0.03)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TEC</th>
<th>(E1 \text{Earthquake:} )</th>
<th>(A_{10} = \text{peak ground acceleration E1} )</th>
<th>([\text{m/s}^2])</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(k_h = (2/3)(A_{10}/g))</td>
<td>(A_{20} = \text{peak ground acceleration E2} )</td>
<td>([\text{m/s}^2])</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>(E2 \text{Earthquake:} )</th>
<th>(A_{20} = \text{peak ground acceleration E2} )</th>
<th>([\text{m/s}^2])</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(a_{\text{max}} = \text{peak ground acceleration} )</td>
<td>(D_a = \text{allowable deformation structure} )</td>
<td>([\text{cm}])</td>
</tr>
<tr>
<td></td>
<td>(D_r = \text{standard deformation [10 cm]} )</td>
<td>(A_{20} &lt; 0.20\text{g})</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(A_{20} \geq 0.20\text{g})</td>
<td>(A_{20} \geq 0.20\text{g})</td>
<td></td>
</tr>
</tbody>
</table>

**Eurocode**

The reference peak ground acceleration is chosen by National Authorities for each seismic zone. This corresponds to the seismic action for the no-collapse requirement. For important structures topographic amplification effects have to be taken into account calculating the peak ground acceleration. For non-gravity walls, the vertical acceleration may be neglected in the seismic design procedure.

For walls larger than 10 meters an average value of the peak horizontal soil accelerations over the height of the wall may be used as input for determining the horizontal seismic coefficient. A reduction factor may be applied on the determined seismic coefficient.

When saturated cohesionless soils susceptible to the development of high excess pore pressures the factor \(r\) may not be taken larger than 1.0 (Nederlands Normalisatie-instituut, 2005).

**PIANC**

The acceleration \(a_{\text{max}}\) is equal to the Peak Ground Acceleration (PGA), which is obtained by a site response analysis.

The prescribed relationship is an upper boundary for the seismic coefficient and is empirically obtained. Also an equation is given for the average value of the seismic coefficient (PIANC, 2001).
2. Theoretical background

**OCDI**

The allowable standard deformation $D_a$ of an anchored sheet pile quay wall for Level 1 earthquake ground motion is 15 cm. For Level 2 earthquake the allowable deformation at the top is equal to 50 cm. Level 2 earthquake motions are mainly set to determine whether seismic resistance is at rational level with respect to the safety of the public, it is the most damaging motion from all earthquake scenarios. An upper bound seismic coefficient has to be used, when the expression regarding a Level 2 earthquake exceeds this value (Ports and Harbours Bureau, Ministry of Land, Infrastructure, Transport and Tourism (MLIT) et al., 2009).

**TEC**

Distinction is made between two types of earthquakes. A level E1 earthquake represents ground motions that are likely to occur relatively often during lifetime of the structure, with relatively low intensities at that location. It corresponds to an earthquake with probability of exceedance of 50% in 50 years corresponding to a return period of 72 years. Level E2 earthquakes are not likely to occur during lifetime of a structure, but have a high intensity. The probability of exceedance is 10% in 50 years or a corresponding to return period of 475 years.

The effective ground accelerations $A_{10}$ and $A_{20}$ can be obtained by dividing spectral acceleration ($0.4S_{MS}$), corresponding to period $T = 0$ s in the design spectrum, by the gravitational acceleration. These can be found in the code (Turkish Ministry of Public Works and Settlement, 2008).

**Comparison seismic coefficients**

Differences are observed between expressions for the horizontal seismic coefficients in the considered design codes. In the Eurocode acceleration at bedrock level is used in the expression while in the OCDI, PIANC and TEC the ground acceleration at surface level is used as input. In the expression for the horizontal seismic coefficient in the Eurocode the soil type is included, accounting for the altering of the signal.

In Figure 2.17 seismic coefficients against horizontal acceleration according to different design codes are presented. For lower accelerations levels relationships are reasonably in correspondence to each other. For higher horizontal acceleration levels the Eurocode predicts much higher horizontal seismic coefficients compared to other design codes. The expression in OCDI is bounded by 0.25. PIANC and TEC prescribe a lower gradient of the development seismic coefficient from acceleration of 0.20 g. The Eurocode predicts for higher acceleration levels higher seismic coefficients compared to other design codes, since it is not bounded or corrected for high acceleration levels. High seismic coefficients in the Mononobe-Okabe lead to exaggerated values for the seismic earth pressure coefficients.
2.7.6. Limitations Mononobe-Okabe
The Mononobe-Okabe is a direct extension of the static Coulomb theory. For that reason the same limitations that related to the Coulomb theory also hold for the Mononobe-Okabe theory. For the active soil wedge, the critical failure surface is flatter for dynamic loading compared to the critical failure surface for static conditions, which means a load increase. For the passive side it is the other way around and a load/resistance reduction.

All the modifications described above to the original method are meant to deal with conditions other than the ones the theory was based on. There remain however limitations to the method. These are described in this section:

- **Passive earth pressure for higher wall friction angles**
The Mononobe-Okabe method will overestimate the actual total passive earth pressure for higher wall friction angles, like $\delta > \phi' / 2$. This overestimation is due to the fact that the most critical sliding surface is much different from a planar surface as assumed in the Mononobe-Okabe theory, for relatively rough walls. The method only approximates the real shape of the curved slip plane. For smooth walls the critical sliding surface is much more identical to the predicted one.

This overestimation is an unsafe approach for anchored sheet pile walls, because these type of structures owe their stability to the passive earth pressure. For that reason some design codes, like the Eurocode 8, suggest to assume a wall friction angle $\delta$ of zero for the passive side, which is on the other hand a very conservative approach. Knowledge is however limited in this field, which may justifies this approach in some cases. The Mononobe-Okabe also predicts higher active earth pressures for higher wall friction angles, however introduced error is much smaller and it is a conservative approach (Visone, C. and Santucci de Magistris, F., 2009).

- **High values for horizontal seismic coefficient**
For high values of the seismic coefficient or for steep backslopes the Mononobe Okabe method gives total earth pressures that tend to go to infinity. The sliding plane is not in equilibrium and limit equilibrium methods are not valid anymore. The earth pressures become very large because of the infinite size of the soil wedge, which has a horizontal failure surface. For sloping backfills this limit is reached earlier, namely when the failure plane angle approaches the angle of the backslope. This already results in an infinite mass of the active soil wedge.

- **Including of non-homogeneous and cohesive soil**
The Mononobe-Okabe theory is originally not suitable for non-homogeneous and cohesive fills (Transportation Research Board, 2008).

- **Liquefiable soils**
The determination of the appropriate seismic earth pressure coefficient with the Mononobe-Okabe method is not suitable for soils that experience significant loss of strength during an earthquake (Visone, C. and Santucci de Magistris, F., 2009). Although not recommended by Visone, C. and Santucci de Magistris, F. (2009), design codes prescribe approaches to include the development excess pore pressures in the Mononobe-Okabe method (see section 2.7.4). Performance of these approaches are investigated in this research.

- **Complete mobilisation of ground**
In order to mobilize the maximum passive earth pressure some deformation is required. In static situation this is about 2 to 5 percent of the embedded wall height (Transportation Research Board, 2008). This also holds for the active earth pressure, however in less deformation is required to reach the active state. Limited knowledge is available to what extent this holds for seismic loading. It is assumed that in for the seismic earth pressures the same deformation is required to mobilize the maximum and minimum earth pressures.

- **Soil structure interaction not taken into account**
Soil structure interaction is not taken into account. The dynamic behaviour of both structure and soil is too complex to include in the simplified method.
In this chapter performance of the Mononobe-Okabe method is analysed. First behaviour of the method for varying input parameters is analysed, in order to provide insight in the sensitivity and limits of the application of the method. Secondly the pseudo-static method is applied to an experimental case of an anchored quay wall, to investigate the applicability of the method. Input parameters are calibrated in order to fit bending moments in the wall. The calibrated input parameters are then used to model other case studies in order to analyse the performance of pseudo-static methods for different configurations. This chapter provides insight in how the method performs for different configurations, what sensitivity of parameters is and what limits of the methods are.

### 3.1. Sensitivity method

Behaviour of the Mononobe-Okabe method including it’s modifications is investigated for varying input parameters. The total active and passive earth pressure coefficients are plotted against the horizontal seismic coefficient for different internal friction angles $\phi'$ and wall friction angles $\delta$. Behaviour of the method is analysed based on these graphs. First results for the earth pressure coefficients according to the original Mononobe-Okabe method for a dry backfill are presented. Secondly the same procedure is done for the modified Mononobe-Okabe for a saturated backfill. At last two approaches to account for excess pore pressures are considered for increasing excess pore pressure ratio $r_u$, differences in earth pressure coefficients between these approaches are analysed.

#### 3.1.1. Dry backfill

In Figure 3.1 both total active and passive seismic earth pressure coefficients for a dry backfill are presented in graphs for different internal friction angles and wall friction angles.

For an increasing internal friction angle the active earth pressure coefficient decreases and on the other hand the passive earth pressure coefficient increases with increasing internal friction angle. This result is in line with what is expected, larger internal friction means more clamping together of the particles resulting in larger shear strength capacity.

Another aspect is the influence of wall friction angle, which is a parameter to account for friction between soil and wall. Concerning the active earth pressure coefficient, differences for varying wall friction angles are small. Influence on the passive earth pressure coefficients is however significant. For increasing wall friction angle, passive earth pressure coefficient increases especially for high internal friction angles. This is explained by the fact that the Mononobe-Okabe method overestimates passive earth pressures for increasing wall friction angles, as was mentioned earlier. Roughly above $\delta = \phi' / 2$ the method isn't accurate anymore, because of the altered shape of the failure soil wedge (Visone, C. and Santucci de Magistris, F., 2009).

Soil properties are often not exactly known. For prediction of the wall friction angle for the passive soil wedge several approaches are available, varying from a wall friction angle of zero (very conservative) to the same as for the active side. Looking at trends for the earth pressure coefficients, relatively small changes in soil properties may already lead to large variations in earth pressures, especially for passive earth pressures.
Variation of the horizontal seismic coefficient leads significant differences in earth pressures, especially for higher values. Sensitivity of the method is large for the chosen input.

The lines are cut-off at certain horizontal seismic coefficient. This implies the limit of the Mononobe-Okabe method, as described earlier. For the active side that point represents the moment the failure soil wedge becomes infinitely large and for the passive side the size of the failure soil wedge approximates zero.

Figure 3.1: Total seismic active earth pressure coefficient (left) and total seismic passive earth pressure coefficient (right) against the horizontal seismic coefficient for varying $\phi$ and $\delta$ for a dry backfill.
3.1.2. Saturated backfill

In Figure 3.2 both total active and passive seismic earth pressure coefficients for saturated soil are presented against the horizontal seismic coefficient for different internal friction angles and wall friction angles. The Mononobe-Okabe method is applied with adjusted seismic inclination angle to account for fully saturated soil.

Analysing the presented results in Figure 3.2 the same trends can be observed as for the earth pressure coefficients for the dry backfill. The gradient is however larger, resulting in higher active earth pressure coefficients for the saturated backfill for the same seismic coefficient. The passive earth pressure coefficients are on the other hand lower with respect to the dry case. This can be explained by the fact that the unit weight of the soil is increased and the friction between particles decreased due to the presence of water.

Limit of the method accounting for a saturated backfill is reached earlier. This can also be seen in the figures, lines are cut-off at lower horizontal seismic coefficients. For a small internal friction angle, the limit is already reached for low seismic loadings.

![Figure 3.2](image)

Figure 3.2: Total seismic active earth pressure coefficient (left) and total seismic passive earth pressure coefficient (right) against the horizontal seismic coefficient for varying $\phi$ and $\delta$ for a saturated backfill.
3.1.3. Excess pore pressures within backfill

In Figure 3.3 total seismic earth pressure coefficients are presented for saturated backfill with excess pore pressures for different wall friction angles and increasing $r_u$ values. Both approaches to account for the effects of excess pore pressures are applied. First the approach where the seismic inclination angle $\theta$ (solid lines) is adjusted and secondly for the method where the internal friction angle $\phi'$ (dotted lines) is adjusted.

![Graphs showing total seismic active and passive earth pressure coefficients for varying $\phi'$ and $\delta$ values with $r_u$ varying from 0.25 to 0.75.](image)

Figure 3.3: Total seismic active earth pressure coefficient (left) and total seismic passive earth pressure coefficient (right) against the horizontal seismic coefficient for varying $\phi$ and $\delta$ values with $r_u$ varying from 0.25 to 0.75.

In Figure 3.3 both total active and passive seismic earth pressure coefficients for a layer with excess pore pressures are shown. Increasing excess pore pressure ratios $r_u$ from 0.25 to 0.75 are adopted to determine the earth pressure coefficients. The internal friction angle is kept constant, behaviour for changing friction angles is already shown in the cases with dry and saturated backfill. The wall friction angles are varied for increasing excess pore pressure ratios.
A difference between the two methods is observed in the results. First of all the difference in earth pressure coefficients for a horizontal seismic coefficient of zero. This difference can be explained by the fact that the $q'$ is adjusted according to the $r_u$ value, independent from the $k_h$ value. The line is translated with respect to the saturated case to take into account the effect of the excess pore pressure ($r_u$).

For the adjusting-$\theta$-method the expression for the seismic inclination is changed. This expression depends on both chosen $r_u$ and $k_h$ and thus changes with increasing $k_h$ value. The trend of the line is changed, with as starting point the earth pressure coefficient for a saturated backfill.

In reality the magnitude of the $r_u$ value depends also on the magnitude of $k_h$. This relation is not taken into account in this analysis and $r_u$ is just varied independent from the $k_h$ value. In an analysis where the $r_u$ and $k_h$ value are coupled the methods may behave more identical to each other.

For lower $r_u$ values the two methods approach each other better than for higher $r_u$ values, especially for the active earth pressure coefficients.

The method where $\theta$ is adjusted gives higher passive earth pressure coefficients compared to the method where $q'$ is adjusted. The active earth pressure coefficients are lower for low $k_h$ values according to the method with adjusted $\theta$ compared to the method with adjusted $q'$. However the trend is steeper so for higher values of $k_h$ the active earth pressures are higher for the adjusted $\theta$ method. It is very unlikely that large values for $r_u$ occur for small seismic coefficients, because the magnitude of $r_u$ depends on the $k_h$ values. Therefore it is not to be expected that the lines show a realistic outcomes for high $r_u$ and low $k_h$ values. The $r_u$ values are chosen rather arbitrarily with respect to the $k_h$ values. The lines do however show the trend and the difference between the methods and the sensitivity to the input values.
### 3.2. Analysis case studies

In the previous section the sensitivity of the Mononobe-Okabe method and modifications was analysed. In this section the implications of these modifications are assessed, with emphasis on different methods to account for the presence of excess pore pressures.

Several case studies with typical configurations are developed in order to analyse the influence of the configuration on earth pressures and moment distribution of the structure. The Coulomb theory is applied to calculate the static earth pressure and Mononobe-Okabe theory together with the suggested modifications is used to calculate the total dynamic earth pressures. Finally the Westergaard equation is used to account for the dynamic water pressures.

Starting point of the analysis is an experimental case study where a scale model of an anchored sheet pile structure in a centrifuge was subjected to several earthquake motions on a shaking table (Higuchi, S. et al., 2012). The soil parameters parameters required as input for the Mononobe-Okabe method are calibrated to this reference case. Based on this calibrated model other configurations are considered and the differences in earth pressures and resulting moment distribution are analysed.

#### 3.2.1. Reference experimental case study

In order to verify outcomes of the Mononobe-Okabe method, there is searched for a representative experimental reference case. An experimental case study is only suitable if the considered setup is comparable to the structure of interest and if it is documented well.

In the paper "Evaluation of the seismic performance of dual anchored sheet pile wall" by Higuchi, S. et al. (2012) the behaviour between a single anchored and dual anchored sheet pile wall is investigated. This is done by performing shake table tests in a centrifuge. The structure is subjected to different subsequent seismic events. During each event the distribution of the bending moment over the height of the structure, the maximum tension force in the anchor and the displacements were measured. The case with a single anchor is only of interest for this research.

**Test setup**

The dimensions of both the scaled test-setup and the full scale structure are given in the paper. As can been seen in Figure 3.4 the height of the wall is 15 meters (scaled to 500 mm), water depth is 9.5 meters (317 mm) and the embedment depth is 2.5 meters (83 mm). The anchor is installed 1.5 meters (50 mm) below the top of the wall and the connection between the horizontal anchor and the batter piles is located 12 meters (400 mm) behind the wall (Higuchi, S. et al., 2012).

In the experiment coarse silica sand with $D_{50} = 1.2$ mm is used with a relative density of $RD = 80\%$ which corresponds to a $\rho_s = 2g/cm^3$ (Higuchi, S. et al., 2012). The magnitudes of the maximum amplitudes of the four events are 0.1 g, 0.2 g, 0.3 g and 0.6 g. In Figure 3.4 subsequent seismic events are presented (Higuchi, S. et al., 2012).

![Figure 3.4: Conceptual test setup of the reference experimental centrifuge test case (left) together with the subsequent seismic events (right) (Higuchi, S. et al., 2012).](image-url)
Results
A summary of the maximum bending moments, the maximum anchor tensile forces and the displacements of the wall are presented in Table 3.1 and the bending moment distribution is presented in Figure 3.5.

Results from the experiment show that the maximum bending moments decreased from Case-200 to Case-300, while the displacements increased. This indicates that the passive soil wedge in front of the wall has failed. Another indication that this mechanism occurred is that the point of contra-flexure has moved up, which indicates that the passive soil wedge has moved up. This aspect also has to be taken into account in the modelling in order to approximate the results of the experiment (Higuchi, S. et al. (2012), Habets, C.J.W. (2015)).

Table 3.1: Results for maximum bending moment, anchor tensile force and displacement for different seismic events (Higuchi, S. et al., 2012)

<table>
<thead>
<tr>
<th>Event</th>
<th>Bending moment [kNm]</th>
<th>Anchor force [kN]</th>
<th>Displacement at anchor [mm]</th>
<th>Displacement seabed [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static case</td>
<td>207</td>
<td>108</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Case-100</td>
<td>301</td>
<td>182</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Case-200</td>
<td>547</td>
<td>261</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>Case-300</td>
<td>449</td>
<td>262</td>
<td>95</td>
<td>95</td>
</tr>
<tr>
<td>Case-600</td>
<td>728</td>
<td>308</td>
<td>225</td>
<td>225</td>
</tr>
</tbody>
</table>

Figure 3.5: Maximum bending moments over height of the experimental reference case sheet pile wall for each event combined in one figure.
3.2.2. Characteristic cases
The described experimental case study is modelled in D-Sheet Piling. This model is calibrated to the results of the experiment. The properties of the soil and structure in the experimental model are determined and used as input in the model. These input parameters are then adjusted to match between the results of the D-Sheet model and the results of the experiment.

The validated parameters from the experiment are then used as input for other cases. Different cases are modelled in order to analyse the performance of the Mononobe-Okabe theory and corresponding modifications for different cases. The same structure is used for all cases, but the configuration of the backfill is changed. The found the moment distribution, tension force in anchor and the displacements are compared and differences are analysed.

The following cases are considered:

- Reference case
- Saturated backfill
- Presence of layer with excess pore pressures

The bending moment distribution, the displacements and the anchor forces are calculated with the help of the program D-Sheet Piling developed by Deltares. The different cases are modelled and results are presented. A short introduction into the used model, the considered construction stages and the input parameters is given.

Description D-Sheet model
In order to take into account change in lateral earth pressures during seismic events it is chosen to manually insert the earth pressure coefficients \(K_A, K_0, K_P\) according to the Coulomb and Mononobe-Okabe methods.

According to documentation in the paper of the reference case three construction stages can be distinguished:

- **Stage 1: Installation sheet pile and anchors**
- **Stage 2: Deepening / excavating in front of wall**
  After finishing the installation of the sheet pile and the anchors, the ground in front of the wall is excavated to the required depth of -9.5 meters.
- **Stage 3: Seismic event**
  Once the excavation was finished, subsequently the several seismic events were applied on the structure. This last calculation stage was adjusted for each seismic event. The bottom level in front of the sheet pile wall was in the previous stage at -9.5 meters. For Case-300 and Case-600 the bottom level is modelled at -9.0 meters to take into account the effect of the passive wedge failure and resulting height increase.

Due to the relatively small spacing between the anchors in the test set-up, arching effect is expected. The anchor system may start to work like a relieving floor resulting in lower active soil pressures on the retaining wall (Habets, C.J.W., 2015). This effect is taken into account in the model by reducing the height of the surface level until the level of the anchors, +1.5 meters. The soil above the anchors doesn't contribute to the horizontal loading on the wall.

For the determination of the bending moment distribution and the displacement of the wall the surface is lowered to the level of the anchors. Because the anchors behave more or less like a relieving platform, horizontal loading on the wall is lower. On the other hand for the calculation of the anchor forces the whole soil wedge has to be taken into account, since it still loads the system as a whole. Both situations are modelled, and the different input models for each case are presented in Annex A. For the stronger seismic events this arching effect is abolished, this will be explained treated later in more detail.
Properties soil and structure
Properties of the sheet pile and anchor were derived from earlier research by Higuchi, S. et al. (2012) and Habets, C.J.W. (2015). In these reports the used soil type and compaction were also documented. In Table 3.2 the properties of the sheet pile, anchor and soil that were initially used in the model are presented.

Note:
In several design codes (Nederlands Normalisatie-instituut (2005), Visone, C. and Santucci de Magistris, E (2009)) the wall friction angle is set to zero for the calculation of the passive earth pressures. This lower limit and conservative approach is motivated by the fact that the wall friction coefficient is an uncertain factor (especially at the passive side) and has large influence on the magnitude of the passive earth pressure. In the considered case study initially a wall friction angle of $\delta = (1/2)\phi'$ is adopted, which thus results not in a lower limit but an estimation based on literature (CUR, 2012). Within the calculation no safety factors are taken into account yet, because the parameters are calibrated to real results.

Table 3.2: Properties used in the model of the experimental case derived from research by Higuchi, S. et al. (2012) and Habets, C.J.W. (2015).

<table>
<thead>
<tr>
<th>Sheet pile properties</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending stiffness</td>
<td>[kN/m/m']</td>
<td>132300</td>
</tr>
<tr>
<td>Maximum elastic moment</td>
<td>[kN/m/m']</td>
<td>1118</td>
</tr>
<tr>
<td>Section area</td>
<td>[cm²/m']</td>
<td>268</td>
</tr>
<tr>
<td>Profile height</td>
<td>[mm]</td>
<td>200</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Anchor properties</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level below surface level</td>
<td>[m]</td>
<td>1.50</td>
</tr>
<tr>
<td>E-modulus</td>
<td>[kN/m²]</td>
<td>2.10⁰⁸</td>
</tr>
<tr>
<td>Section area</td>
<td>[m²/m']</td>
<td>9.24⁻⁰⁴</td>
</tr>
<tr>
<td>Anchor length</td>
<td>[m]</td>
<td>12.0</td>
</tr>
<tr>
<td>Angle</td>
<td>[°]</td>
<td>0.00</td>
</tr>
<tr>
<td>Yield force</td>
<td>[kN/m']</td>
<td>328</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Soil parameters</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight</td>
<td>$\gamma_{dry}$ [kN/m³]</td>
<td>20</td>
</tr>
<tr>
<td>Saturated unit weight</td>
<td>$\gamma_{sat}$ [kN/m³]</td>
<td>22</td>
</tr>
<tr>
<td>Cohesion</td>
<td>[kN/m³]</td>
<td>0</td>
</tr>
<tr>
<td>Internal friction angle</td>
<td>$\phi$ [°]</td>
<td>45</td>
</tr>
<tr>
<td>Wall friction angle</td>
<td>$\delta$ [°]</td>
<td>$\phi / 3$</td>
</tr>
<tr>
<td>Modulus of subgrade reaction</td>
<td>k₁ [kN/m³]</td>
<td>65000</td>
</tr>
<tr>
<td>Modulus of subgrade reaction</td>
<td>k₂ [kN/m³]</td>
<td>32500</td>
</tr>
<tr>
<td>Modulus of subgrade reaction</td>
<td>k₃ [kN/m³]</td>
<td>16250</td>
</tr>
<tr>
<td>Relative density RD</td>
<td>[%]</td>
<td>80</td>
</tr>
<tr>
<td>Permeability</td>
<td>[m/s]</td>
<td>0.0005</td>
</tr>
</tbody>
</table>

Seismic action
The used expression for the horizontal seismic coefficient ($k_h$) is:

In case $a_{max} < 0.2g$:

$$k_h = \frac{a_{max}}{g} \quad (3.1)$$

When $a_{max} \geq 0.2g$:

$$k_h = \frac{1}{3} \left( \frac{a_{max}}{g} \right)^{1/3} \quad (3.2)$$

Both equations 3.1 and 3.2 are plotted in Figure 3.6. As can be observed in the figure the equations represent the upper boundary for the horizontal seismic coefficient, based on the 129 case histories. Also a dotted line is shown, which represents the average value.

Because the considered retaining structure is a slender structure the $k_v$ value may assumed to be equal to zero (PIANC, 2001).
Schematization structure

The resultant forces on the structure are schematized in Figure 3.7. Active and passive earth pressures are determined using the Coulomb theory and the additional dynamic components are calculated with the Mononobe-Okabe theory. On both sides hydrostatic water pressures are present. In the dynamic phase additional dynamic water pressures according to Westergaard are applied on the outside of the structure.

<table>
<thead>
<tr>
<th>Resultant forces [kN/m]</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_A$</td>
<td>Active earth pressure</td>
</tr>
<tr>
<td>$P_P$</td>
<td>Passive earth pressure</td>
</tr>
<tr>
<td>$P_{H,W}$</td>
<td>Hydrostatic water pressure</td>
</tr>
<tr>
<td>$P_{H,W,d}$</td>
<td>Resultant hydrostatic and dynamic water pressure</td>
</tr>
<tr>
<td>$\Delta P_{AE}$</td>
<td>Dynamic active earth pressure</td>
</tr>
<tr>
<td>$\Delta P_{PE}$</td>
<td>Dynamic passive earth pressure</td>
</tr>
<tr>
<td>$F_{A,m}$</td>
<td>Anchor force</td>
</tr>
</tbody>
</table>
3.2.3. Reference case
Initially the retaining structure of the experiment is considered. The surface level is at +3.0 meters and in front of the wall the bottom level is -9.5 meters. Water is present until 3.0 meters below the top of the wall. The same soil type is present both in front of the wall as in the backfill. See Figure 3.8 for schematization of the reference case.

Figure 3.8: Schematization of the reference case modelled in D-Sheet Piling.

In Table 3.3 the initial seismic horizontal seismic coefficients for each event are presented. These seismic coefficients are determined using equations 3.1 and 3.2. The earth pressure coefficients ($K_{AE}$ and $K_{PE}$) according to Mononobe-Okabe are also given.

Table 3.3: Horizontal seismic coefficient and total seismic horizontal earthpressure coefficients according to original Mononobe-Okabe method.

<table>
<thead>
<tr>
<th>Event</th>
<th>$k_h$</th>
<th>$K_{AE}$</th>
<th>$K_{PE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static case</td>
<td>0.000</td>
<td>0.16</td>
<td>21.38</td>
</tr>
<tr>
<td>Case-100</td>
<td>0.100</td>
<td>0.21</td>
<td>19.92</td>
</tr>
<tr>
<td>Case-200</td>
<td>0.200</td>
<td>0.27</td>
<td>18.44</td>
</tr>
<tr>
<td>Case-300</td>
<td>0.223</td>
<td>0.28</td>
<td>18.10</td>
</tr>
<tr>
<td>Case-600</td>
<td>0.281</td>
<td>0.33</td>
<td>17.24</td>
</tr>
</tbody>
</table>

Calibrating to reference case
Input parameters used in the model are based on documentation of the experimental reference case, as presented in Table 3.2. The internal friction angle and wall friction angle do not follow directly from documentation. As a first estimation the wall friction angle and internal friction angle are adopted as according to the CUR166.

Model parameters are then fit to the static results, the initial internal friction angle and wall friction angle are calibrated for this case study. The calibrated parameters also used to model the seismic events, where subsequently only the horizontal seismic coefficient is adjusted to fit model results the experimental results.

As was already concluded in section 3.1 adjusting the internal friction angle has large influence on the moment distribution. Results of the static case fit the best for an internal friction angle $\phi' = 45^\circ$. This is a high value especially in seismic conditions, where the internal friction angle generally decreases as shaking continues. In the paper by Higuchi, S. et al. (2012) it is however stated that the soil is very densely packed to exclude the occurrence of liquefaction. Because of the absence of a correction factor all corrections made are included in the calibration of the value for the internal friction angle.
Secondly, the wall friction angle was calibrated to the static experimental results. A wall friction angle of \((1/3)\varphi\) provides the best match with experimental results. This low value can be explained by the fact that plexiglas was used for the wall in the experiment, which is smooth compared to for example a steel sheet pile wall.

For the seismic loading events these values are also adopted. The horizontal seismic coefficient is calibrated to match the results. In Table 3.4 the original \(k_h\) and the calibrated values are presented. The measured bending moment distribution during the experimental (solid lines) is compared with the results from the D-Sheet model (dotted line), see Figure 3.9. The D-Sheet model predicts higher passive earth pressures compared to the experiment results. Leading to higher bending moments at the embedded part of the structure. For the static case and Case-100 event the model with a lowered surface (+1.5 m) is used to account for the arching effect of the anchors.

![Figure 3.9: Comparison bending moment distribution experimental reference case and D-sheet model for static case and seismic events Case100 and Case200.](image)

From the seismic events Case-200 and higher a better fit was found between the experiment and model results if the surface behind the sheet pile wall was modelled until +3.0 meters. It can be concluded that the arching effect is eliminated from this event on because of the heavy shaking. Results of the fitted bending moment distributions for seismic loading case Case-200 to Case-600 are presented in Figure 3.10. After seismic event Case-300 something substantially changed in the experimental model that has large influence on the moment distribution of the wall. This is explained by the fact that the retaining wall moved forwards substantially compared to the former events. Some displacement is required to fully mobilize the passive earth pressures. Another explanation could be that the passive soil wedge moved upwards due to the forward displacement of the wall. What exactly happened in the model is unfortunately not documented. The increase in passive soil resistance could not be reproduced by the model, leading to a more altered bending moment line for the higher seismic events.
3.2. Analysis case studies

As can be seen in Table 3.4 the $k_h$ values can be reduced with a factor 2 for most seismic events. Only for the Case-600 the reduction factor is substantially lower. These reduced seismic coefficients are applied in the Mononobe-Okabe and Westergaard solution to obtain the new loading events.

In Figure 3.11 the calibrated values for the seismic coefficients are plotted in the original figure by PIANC. The found values are lower than the line for the upper bound. Values are more consistent with the average values. Based on this analysis one might suggest to lower the seismic coefficient in design. This can however not be stated yet, since the type of signal and structure are also of importance in considering the effects of an earthquake. These variations are not yet considered. However for this case application of a simplified method leads to an overestimation of the loading.

The same input parameters are for the modelling of the other cases. In this way a comparison between the results of the cases can be made and the modifications to the original Mononobe-Okabe method can be assessed.

Table 3.4: Calibrated horizontal seismic coefficients and total seismic horizontal earth pressure coefficients for the reference case.

<table>
<thead>
<tr>
<th>Event</th>
<th>Original $k_h$ [-]</th>
<th>Calibrated $k_h$ [-]</th>
<th>Reduction factor [-]</th>
<th>$K_{AE}$ [-]</th>
<th>$K_{PE}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static case</td>
<td>0.000</td>
<td>0.00</td>
<td>$I$</td>
<td>0.16</td>
<td>12.47</td>
</tr>
<tr>
<td>Case-100</td>
<td>0.100</td>
<td>0.05</td>
<td>2.00</td>
<td>0.18</td>
<td>12.10</td>
</tr>
<tr>
<td>Case-200</td>
<td>0.200</td>
<td>0.10</td>
<td>2.00</td>
<td>0.21</td>
<td>11.72</td>
</tr>
<tr>
<td>Case-300</td>
<td>0.223</td>
<td>0.11</td>
<td>2.03</td>
<td>0.21</td>
<td>11.65</td>
</tr>
<tr>
<td>Case-600</td>
<td>0.281</td>
<td>0.23</td>
<td>1.25</td>
<td>0.28</td>
<td>10.78</td>
</tr>
</tbody>
</table>

Figure 3.10: Comparison bending moment distribution experimental reference case and D-sheet model for static case and seismic events Case300 and Case600.
3.2.4. Saturated backfill

A second case study is considered with the same retaining structure and type of backfill. The presence of water is however now taken into account in calculating the dynamic earth pressures. The original Mononobe-Okabe method is adjusted to account for the presence of water. The permeability of the soil is assumed to be small enough so water and soil doesn’t move independently from each other.

The ground water level is not equal to the surface level so part of the soil is not fully saturated. An average unit weight over the whole soil profile is be determined, represented by an equivalent saturated unit weight. See also Annex A for the input in the model. In Table 3.5 the applied earth pressure coefficients are presented using an equivalent saturated unit weight, based on the calibrated horizontal seismic coefficients.

<table>
<thead>
<tr>
<th>Event</th>
<th>$k_h$</th>
<th>$K_{AE}$</th>
<th>$K_{PE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static case</td>
<td>0.00</td>
<td>0.16</td>
<td>12.47</td>
</tr>
<tr>
<td>Case-100</td>
<td>0.05</td>
<td>0.19</td>
<td>11.94</td>
</tr>
<tr>
<td>Case-200</td>
<td>0.10</td>
<td>0.23</td>
<td>11.40</td>
</tr>
<tr>
<td>Case-300</td>
<td>0.11</td>
<td>0.24</td>
<td>11.29</td>
</tr>
<tr>
<td>Case-600</td>
<td>0.23</td>
<td>0.35</td>
<td>10.04</td>
</tr>
</tbody>
</table>

Results model

Earth pressures over the height of the structure for the static situation and for seismic event Case-100 are presented in Annex A. The maximum bending moments, anchor forces and displacements are summarized in Table 3.6. The presence of water in the backfill has a large influence on the lateral earth pressure coefficients and thus on the bending moment distribution and displacement of the retaining structure. The maximum values are higher compared to the reference case.

In Figure 3.12 results of the bending moment distribution for the reference case (dotted lines) and the saturated case (solid lines) are presented. For higher loading events, differences in lateral earth pressure coefficients between the original method and saturated case become larger. Active earth pressures become increasingly higher and passive earth pressures increasingly lower.
### Table 3.6: Maximum bending moment, anchor force and displacement during each seismic event for a partly saturated backfill.

<table>
<thead>
<tr>
<th>Event</th>
<th>Maximum bending moment [kNm]</th>
<th>Maximum anchor force [kN/m']</th>
<th>Maximum displacement [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static case</td>
<td>209</td>
<td>113</td>
<td>27</td>
</tr>
<tr>
<td>Case-100</td>
<td>317</td>
<td>155</td>
<td>42</td>
</tr>
<tr>
<td>Case-200</td>
<td>574</td>
<td>206</td>
<td>81</td>
</tr>
<tr>
<td>Case-300</td>
<td>525</td>
<td>193</td>
<td>71</td>
</tr>
<tr>
<td>Case-600</td>
<td>930</td>
<td>309</td>
<td>131</td>
</tr>
</tbody>
</table>

![Image of bending moment distribution](image)

Figure 3.12: Comparison resulting bending moment distribution from D-Sheet models for respectively the case with dry backfill and with saturated backfill.

### 3.2.5. Excess pore pressures

Depending on the type and state of the soil excess pore pressures can develop during an earthquake, which have influence on lateral pressures on the retaining structure. In this section two cases are considered with potential liquefiable layers at different locations next to the structure.

The earth pressure coefficients of the liquefiable layers are determined according to two different methods, which are described in Chapter 3. In the first method the internal friction angle $\phi$ is decreased for increasing excess pore pressure ratio $r_u$. The second method the seismic inclination angle $\theta$ is increased according to a changing $r_u$. The main goal of this section is to show the differences in between these two methods. In both cases and for both methods an excess pore pressure ratio ($r_u$) of 0.5 is adopted.

The macro stability of the 15 meter long wall is for this configuration too low in the static situation. Not enough passive resistance could be developed to make equilibrium with the active pressure. For that reason the length of the wall is increased with 2.0 meters in order to provide enough passive resistance.

Soil properties of the liquefiable layer are different than the other sand layers, a loosely packed clean sand layer with corresponding soil properties from Eurocode 7 is adopted (Nederlands Normalisatie-instituut, 2012). Properties are presented in Table 3.7.
46

3. Analysis pseudo-static methods

Table 3.7: Soil properties liquefiable layer.

<table>
<thead>
<tr>
<th>Soil parameters</th>
<th>$\gamma_{dry}$ [kN/m³]</th>
<th>$\gamma_{sat}$ [kN/m³]</th>
<th>C [kN/m²]</th>
<th>$\varphi$ [°]</th>
<th>$\delta$ [°]</th>
<th>RD [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight</td>
<td>20</td>
<td>22</td>
<td>0</td>
<td>45</td>
<td>$\varphi / 3$</td>
<td>80</td>
</tr>
<tr>
<td>Saturated unit weight</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cohesion</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Internal friction angle</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wall friction angle</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relative density</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Case 1: liquefiable layer within backfill

The first case considered is a retaining structure with a liquefiable layer in the backfill. This leads to an increased earth pressure against the structure, compared to the case without liquefiable layer. In Figure 3.13 a schematization of the case is presented.

The earth pressure coefficients according to both methods are presented in Table 3.8. For the non-liquefiable sand layer earth pressure coefficients for a saturated backfill are applied, as presented in Table 3.5. Besides the adjusted earth pressures also a higher water pressure manually applied in the liquefiable layer due to the presence of excess pore pressures. An excess pore pressure ratio of 0.5 is adopted.

Figure 3.13: Schematization of the case with a liquefiable layer halfway over the height of the retaining structure.

Table 3.8: Horizontal seismic coefficients and total seismic horizontal earth pressure coefficient for excess pore pressures within the soil, according to both methods.

<table>
<thead>
<tr>
<th>Event</th>
<th>$k_b$</th>
<th>$K_{AE}$</th>
<th>$K_{PE}$</th>
<th>$K_{AE}$</th>
<th>$K_{PE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static case</td>
<td>0.00</td>
<td>0.31</td>
<td>4.14</td>
<td>0.31</td>
<td>4.14</td>
</tr>
<tr>
<td>Case-100</td>
<td>0.05</td>
<td>0.60</td>
<td>2.00</td>
<td>0.44</td>
<td>3.56</td>
</tr>
<tr>
<td>Case-200</td>
<td>0.10</td>
<td>0.73</td>
<td>1.74</td>
<td>0.65</td>
<td>2.89</td>
</tr>
<tr>
<td>Case-300</td>
<td>0.11</td>
<td>0.76</td>
<td>1.68</td>
<td>0.71</td>
<td>2.74</td>
</tr>
<tr>
<td>Case-600</td>
<td>0.23</td>
<td>\</td>
<td>\</td>
<td>\</td>
<td>\</td>
</tr>
</tbody>
</table>
Results model  Earth pressures over the height of the structure for the static situation and for seismic event Case-100 are presented in Annex A. The maximum bending moments, anchor forces and displacements are summarized in Table 3.9.

The obtained bending moments over the height of the structure are presented in Figure 3.14. There is a difference in moment distribution for the static case between the saturated case and the case with liquefiable layer, blue lines. Because of the liquefiable layer with lower strength. The red lines represent the Case-100 seismic event. The solid line shows the moment distribution where the $\varphi$ is adjusted and the dotted line the distribution for adjusted $\theta$. The bending moment lines according to both methods follow the same trend, however the extreme values according to the $\varphi$ method are somewhat higher.

Table 3.9: Maximum bending moments, anchor forces and displacements for liquefiable layer halfway.

<table>
<thead>
<tr>
<th>Event</th>
<th>Maximum bending moment [kNm]</th>
<th>Maximum anchor force [kN/m']</th>
<th>Maximum displacement [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adjusting $\varphi$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Static case</td>
<td>207</td>
<td>123</td>
<td>24</td>
</tr>
<tr>
<td>Case-100</td>
<td>484</td>
<td>272</td>
<td>58</td>
</tr>
<tr>
<td>Case-200</td>
<td>896</td>
<td>331</td>
<td>123</td>
</tr>
<tr>
<td>Case-300</td>
<td>956</td>
<td>350</td>
<td>133</td>
</tr>
<tr>
<td>Adjusting $\theta$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Static case</td>
<td>207</td>
<td>123</td>
<td>24</td>
</tr>
<tr>
<td>Case-100</td>
<td>431</td>
<td>247</td>
<td>52</td>
</tr>
<tr>
<td>Case-200</td>
<td>854</td>
<td>317</td>
<td>116</td>
</tr>
<tr>
<td>Case-300</td>
<td>928</td>
<td>342</td>
<td>129</td>
</tr>
</tbody>
</table>

Figure 3.14: Resulting bending moments for each event from D-sheet for model with liquefiable layer halfway height of structure for both considered methods taking into account excess pore pressures.
Case 2: structure embedded in liquefiable layer

In the second case the structure is assumed to be embedded in a liquefiable layer. Decrease of passive earth pressure by increasing excess pore pressure may result in stability problems of the structure. In Figure 3.15 a schematization of the case is presented.

Earth pressure coefficients according to both methods are presented in Table 3.8. For the non liquefiable sand layers again the earth pressure coefficients for a saturated backfill are applied. In the liquefiable layer an excess pore pressure ratio of 0.5 is adopted, leading to adjusted earth pressure coefficients and higher water pressures against the structure.

Results model

The earth pressures over the height of the structure for the static situation and for seismic event Case-100 are presented in Annex A. The maximum bending moments, anchor forces and displacements are summarized in Table 3.10.

Bending moments over the height of the structure for different loading events are presented in Figure 3.16. The difference in moment distribution in the static phase between the saturated case and the case with liquefiable layer is explained by the presence of the liquefiable layer with less favourable soil parameters. The red lines represent the Case-100 seismic event, where the solid lines represent the moment for adjusted $\varphi$ methods and the dotted line is the distribution for adjusted $\theta$. Due to decrease of passive earth pressure the maximum bending embedding moment decreases. Again the method with adjusted $\varphi$ leads to a higher extreme bending moment.

<table>
<thead>
<tr>
<th>Event</th>
<th>Maximum bending moment [kNm]</th>
<th>Maximum anchor force [kN/m']</th>
<th>Maximum displacement [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adjusting $\varphi$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Static case</td>
<td>301</td>
<td>154</td>
<td>50</td>
</tr>
<tr>
<td>Case-100</td>
<td>820</td>
<td>301</td>
<td>149</td>
</tr>
<tr>
<td>Case-200</td>
<td>1312</td>
<td>362</td>
<td>247</td>
</tr>
<tr>
<td>Case-300</td>
<td>1399</td>
<td>384</td>
<td>264</td>
</tr>
<tr>
<td>Adjusting $\theta$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Static case</td>
<td>301</td>
<td>154</td>
<td>50</td>
</tr>
<tr>
<td>Case-100</td>
<td>706</td>
<td>279</td>
<td>128</td>
</tr>
<tr>
<td>Case-200</td>
<td>1233</td>
<td>349</td>
<td>231</td>
</tr>
<tr>
<td>Case-300</td>
<td>1338</td>
<td>371</td>
<td>252</td>
</tr>
</tbody>
</table>
Analysing earth pressure coefficients of the liquefiable layer it is shown that active earth pressure coefficients for the $\varphi'$-method are slightly higher than for the $\theta$-method, which leads to higher loading. On the other hand passive earth pressure coefficients according to the $\varphi'$-method is are lower than obtained with the $\theta$-method. In other words this results in higher loading and lower resistance.

Difference in response of structure are larger for the sheet pile embedded in a liquefiable layer. This is explained by the difference in passive earth pressure coefficient according to both methods, leading to change in passive resistance. Active earth pressure coefficients are more in accordance to each other, resulting in smaller differences between both methods in active earth pressures.

Note:
In this analysis the excess pore pressure ratio $r_u$ was not related to the magnitude of the seismic action. An $r_u$ of 0.5 was chosen to introduce a significant effect, but in fact this value has to be predicted according to the soil profile, soil type and seismic action.
3.3. Conclusion

Sensitivity Mononobe-Okabe method
Earth pressure coefficients determined by the Mononobe-Okabe method appear to be very sensitive to input parameters. The magnitude of passive earth pressure coefficients is sensitive to the internal friction angle. This soil property can be determined quite accurately, however introducing a safety factor to this property may lead to a disproportional decrease of the passive (or increase of the active) earth pressures.

Influence uncertainty wall friction angle
Scattering of the earth pressure coefficients increases for increasing wall friction coefficient (δ). Design codes prescribe different values for the magnitude of this wall friction angle, having large influence on especially the passive earth pressure coefficients. Same trend is be observed for the active earth pressure coefficients but these differences are smaller.

Differences approaches excess pore pressures
Differences are observed between approaches that describe the development of earth pressure coefficients with presence of excess pore pressures, especially for the passives soil resistance. Larger excess pore pressure ratios (\(r_u\)) lead to increasing differences, for both active and passive earth pressure coefficients. Generally the \(\varphi\)-method leads to more conservative results. Design codes adopted only one of the two methods.

Influence uncertainty in seismic coefficient
Also uncertainty is present in the relationship between the PGA and the seismic coefficient, leading to a spread in earth pressure coefficients. As shown in calibration of the reference case a reduction factor of 2.0 could be applied on the seismic coefficient for most seismic events, leading to a considerable load reduction. This case was however recalculated and fitting on results was possible. Relationships for seismic coefficient are normally used for design purposes and provide therefore typically upper boundary values. Besides that these relationships are used for each type of structure, from rigid gravity based structures to ductile slender anchored, leading to different behaviour and response. It can therefore not be concluded that a reduction of the seismic coefficient is valid in all cases, however it must be kept in mind that the provided relationships represent upper boundaries.

Limit methods
The limit of the Mononobe-Okabe method can be reached for already low values of the horizontal seismic coefficient. Once the limit is reached, no results are found for the earth pressure coefficients. Advanced dynamic analysis with finite element models may be performed to obtain results beyond the limits of the Mononobe-Okabe method.
Pseudo-static analysis Akita Port

In this chapter two similar anchored quay walls in Akita Port in Japan are considered that were hit by the Nihonkai Chubu Earthquake. One quay wall suffered damage due to this earthquake, related to severe liquefaction. The other did not suffer any visible damage, also no signs of liquefaction were observed.

First the case is introduced and soil profile and properties of the structures are determined. The liquefaction potential of soils in the backfill is evaluated using the a cyclic stress approach method. Once the liquefiable soils are identified, the pseudo-static method by Mononobe-Okabe is applied to both cases. It is investigated to what extent this method is able to reproduce results that correspond to the observed damage.

4.1. Introduction case study Akita Port

In May 1983 the northern part of Japan was hit by the Nihonkai Chubu Earthquake with a magnitude of 7.7. Akita Port is situated around 100 km from the epicentre. The earthquake caused damage to the anchored quay walls in the port. Sand boils were observed next to at least one of these damaged walls (Ohama No.2 Wharf). In the vicinity the damaged quay walls, one quay didn’t suffer visible damage (Ohama No.1 Wharf). The cross section of this non-damaged quay was similar to the quay that was damaged, however at that location no sand boils related to liquefaction were observed. Soil profiles of the backfill behind the quays were different, especially in the degree of compaction of the sand in the backfill. This is an important factor for the liquefaction potential of the sand layer.

The maximum accelerations that were recorded at Akita Port observation station (see Figure 4.2) are: 0.219 g in north-south direction, 0.235 g in east-west direction and 0.054 g in vertical direction. In Figure 4.1 are time records of the Nihonkai Chubu Earthquake presented.

Figure 4.1: Time record of earth quake motion at Akita Port (Iai, S. and Kameoka, T., 1993).
4.1.1. Ohama No.1 Wharf
In Figure 4.3 the cross section of the anchored quay wall at Ohama No.1 Wharf is presented. No visible damage was observed to the quay wall after the earthquake. The sheet pile is of the type FSP VIL, is embedded until a depth of -15.50 meters and has a total length of 17.50 meters, with the top at +2.00 meters. At +0.50 meters the anchor is attached to the sheet pile. A 16.00 meter long semi-high tensile strength tie rod with diameter of 55 millimeters connects the sheet pile with the anchor wall. This anchor wall exists of 9.60 meter long steel tubular piles with a diameter of 750 mm and a wall thickness of 10 mm.
4.1. Introduction case study Akita Port

The soil profile is determined using the SPT-test results and grain size distributions of the soil layers. In Annex B the test results are presented. Not for all layers a grain size distribution is available. There is however a bore hole profile present. Based on the bore hole profile, the grain sizes of the surrounding layers and the corresponding SPT N-values an estimation is made for the soil type of the less documented soil layers.

First the soil is classified using the grain size distribution. Subsequently a first estimation of the soil type, unit weight and other soil properties are made according to Eurocode 7. N-values are normalized for the type of equipment used and the overburden pressure to \((N_1)_{60}\)-values according to the method described in section 2.6.1. An estimation is made of the relative density of the soil layers based on the found \((N_1)_{60}\)-values and soil type according to the theory developed by Skempton (1986). The final soil properties determined using the soil type and the degree of compaction. The soil profile at Ohama No.1 Wharf is presented in Figure 4.4.

4.1.2. Ohama No.2 Wharf

The cross section of the anchored quay wall at Ohama No.2 Wharf is presented in Figure 4.5, together with the damaged cross section as observed after the earthquake. The same type of sheet pile is used as for Ohama No.1 Wharf, the FSP-VIL type. The total length of the wall is 22.50 meters and the bottom of the wall is at -20.50 meters. The anchor is also attached at the sheet pile at height +0.50 meters. The sheet pile is connected to the anchor wall with a high strength steel tie rod with a length of 19.00 meters and a diameter of 55 millimeters. The anchor wall exists of a double steel pipe pile with a diameter of 550 mm and a wall thickness of 12 mm with a length of 15.00 meters. As can be seen in Figure 4.5 the quay wall suffered damage due to the earthquake.

Halfway the retaining height of the sheet pile wall a crack occurred, which indicates that the moment capacity of the sheet pile was reached at that location. Also deformation of the anchor wall is observed, what resulted in forward displacement of the quay wall. The anchors were pulled by the sheet piles in the direction of the sea and accordingly inclined, presumably due to the reduced resistance by the liquefied backfill sand.

The soil profile is determined in the same way as for Ohama No.1 Wharf. Grain size distribution was given for several layers together with SPT N-values for two locations along the quay wall, see Annex B.
Figure 4.5: (Damaged) cross section of anchored quay wall at Ohama Wharf No.1 (Iai, S. and Kameoka, T., 1993).

Figure 4.6: Deformation of the sheet pile wall at Ohama No.2 Wharf for different cross sections (Iai, S. and Kameoka, T., 1993).

Figure 4.7: Soil profile and related soil properties based on SPT-test and grain size distribution at Ohama No.2 Wharf.
4.2. Pseudo-static analysis

In Chapter 3 the performance of the Mononobe-Okabe for different configurations of the backfill was analysed. In this section the pseudo-static method is applied on both quay walls on both considered anchored quay walls in Akita port. It is investigated to what extent the pseudo-static method are capable in predicting the behaviour of the considered quay walls.

The method used to calculate the static earth pressure coefficients is the method of Coulomb for cohesionless soils. The Mononobe-Okabe method, which is an extension of the Coulomb method, is used to predict the total dynamic earth pressures for cohesionless soils. To include the effects of the presence of water in the backfill an equivalent unit weight is determined according to the method described in PIANC, which is used to calculate the seismic inclination angle. These methods were already described in more detail in section 2.7.

4.2.1. Liquefaction potential

Sand boils were observed at Ohama No.2 Wharf indicating that liquefaction occurred within the backfill of the anchored quay wall. These sand boils were not seen at Ohama No.1 Wharf, there was no indication that liquefaction had occurred at this location. These observations also match with the observed damage of the quay walls.

The cyclic stress approach developed by Idriss, I.M. and Boulanger, R.W. (2008) is used as a first method to evaluate the liquefaction potential of the backfill during the considered earthquake. A detailed description of the method was provided in section 2.6.1.

According to Eurocode 8 the soil is liquefiable if the safety factor is less than 1.25. This threshold can however differ for different countries. The method is developed to be used for design purposes and contains some safety margin. In this research a case is recalculated and for that reason no safety factors are applied in the calculations, assuming that a safety factor higher than 1.0 means that no liquefaction occurred.

Besides the method by Idriss, I.M. and Boulanger, R.W. (2008) other cyclic stress methods are available for the determination of the resistance against liquefaction. Within this research only one method is considered, because the main focus of this research is to investigate the influence of excess pore pressures on the dynamic earth pressures.

Ohama No.1 Wharf

In Table 4.1 results of the liquefaction potential of the backfill of Ohama No.1 Wharf are presented. The soil parameters from Figure 4.4 are used as input. For all soil layers the safety factor against liquefaction is higher than 1.0, except for the bottom layer.

Three SPT-test results are available along the Ohama No.1 Wharf see Appendix B. The exact location of the tests is not known and spread of results of the tests is quite large. Applying the minimum measured N-values as input for the analysis of the liquefaction potential results in complete liquefaction of the upper three layers. This is not in line with the observations along the quay.

The method is however sensitive for the input of N-values, where a slight increase of the input N-values already leads to a significant difference in liquefaction potential. The considered method is also developed for designing structures, suggesting that a safety margin is present in the method. In this research however results of a case are reproduced, which means that there is less uncertainty because the event already occurred. For these reasons the average N-values of the three SPT-tests are used as input for the Idriss, I.M. and Boulanger, R.W. (2008) method, leading to a less conservative evaluation. Results of the $r_u$-values over the height of the structure are presented in Table 4.1.
Table 4.1: Results of the liquefaction potential and predicted excess pore pressures in the backfill at Ohama No.1 Wharf.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Depth (m)</th>
<th>$CSR_{M,eq}$</th>
<th>$CRR_M$</th>
<th>FoS B&amp;I, 2008</th>
<th>$r_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand clean &gt;gwl</td>
<td>1.1</td>
<td>0.15</td>
<td>0.22</td>
<td>1.42</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td>2.2</td>
<td>0.21</td>
<td>0.48</td>
<td>2.31</td>
<td>0.03</td>
</tr>
<tr>
<td></td>
<td>3.2</td>
<td>0.23</td>
<td>0.51</td>
<td>2.23</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>4.2</td>
<td>0.24</td>
<td>0.48</td>
<td>1.95</td>
<td>0.06</td>
</tr>
<tr>
<td></td>
<td>5.2</td>
<td>0.25</td>
<td>0.43</td>
<td>1.70</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td>6.1</td>
<td>0.26</td>
<td>1.03</td>
<td>3.98</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>7.5</td>
<td>0.26</td>
<td>0.77</td>
<td>2.92</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>8.1</td>
<td>0.26</td>
<td>0.69</td>
<td>2.60</td>
<td>0.02</td>
</tr>
<tr>
<td>Sand very silty &gt;gwl</td>
<td>9.0</td>
<td>0.26</td>
<td>0.51</td>
<td>1.95</td>
<td>0.06</td>
</tr>
<tr>
<td>Sand slightly silty &gt;gwl</td>
<td>10.1</td>
<td>0.26</td>
<td>0.39</td>
<td>1.49</td>
<td>0.16</td>
</tr>
<tr>
<td></td>
<td>11.1</td>
<td>0.26</td>
<td>0.35</td>
<td>1.35</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>12.2</td>
<td>0.26</td>
<td>0.19</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>13.0</td>
<td>0.25</td>
<td>0.30</td>
<td>1.17</td>
<td>0.38</td>
</tr>
<tr>
<td>Sand slightly silty &gt;gwl</td>
<td>14.1</td>
<td>0.25</td>
<td>0.66</td>
<td>2.64</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>15.0</td>
<td>0.25</td>
<td>0.71</td>
<td>2.88</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>16.1</td>
<td>0.24</td>
<td>0.33</td>
<td>1.34</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>17.4</td>
<td>0.24</td>
<td>9.64</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>18.5</td>
<td>0.24</td>
<td>6.36</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>19.6</td>
<td>0.23</td>
<td>4.40</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>20.5</td>
<td>0.23</td>
<td>3.34</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Clay very sandy &gt;gwl</td>
<td>21.5</td>
<td>0.22</td>
<td>0.11</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>22.4</td>
<td>0.22</td>
<td>0.14</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Sand slightly silty &gt;gwl</td>
<td>23.3</td>
<td>0.22</td>
<td>0.16</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>24.4</td>
<td>0.21</td>
<td>0.18</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>25.3</td>
<td>0.21</td>
<td>0.18</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>26.3</td>
<td>0.21</td>
<td>0.18</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>27.4</td>
<td>0.20</td>
<td>0.21</td>
<td>1.04</td>
<td>0.68</td>
</tr>
</tbody>
</table>

**Ohama No.2 Wharf**

Along Ohama No.2 Wharf large scale liquefaction within the backfill was observed in the field. The loose compaction of the backfill leads to low N-values, which in turn causes the low resistance against liquefaction. As can be seen in Table 4.2 the upper layer liquefies, which is in line with observations. This layer consists of sand that is artificially deposited and hardly compacted. All other layers do have a higher resistance and excess pore pressures are not predicted.

**4.2.2. Seismic loading**

The maximum measured horizontal acceleration during the Nihonkai Chubu Earthquake at Akita Port is 0.235 g. Vertical acceleration doesn’t lead to significant loading because of the slenderness of the structure and is therefore not considered (PIANC, 2001).

The equation by (PIANC (2001) and Turkish Ministry of Public Works and Settlement (2008)) is used in to determine the horizontal seismic coefficient. For an acceleration of 0.235 g this results in a horizontal seismic coefficient of $k_h = 0.206$.

The report ‘Seismic Design Guidelines for Port Structures’ by PIANC contains various case studies, including the considered anchored quay walls in Akita Port. In this report a design value for the horizontal seismic coefficient of $k_h = 0.100$ is documented, which is about two times lower than the calculated seismic coefficient.

Lowering the design value of the horizontal seismic coefficient is in line with findings in chapter 3. Both values for seismic coefficient $k_h = 0.100$ and $k_h = 0.206$ are analysed in order to investigate which coefficient results in the best match with the observed damage to the quay walls.
4.2. Pseudo-static analysis

Table 4.2: Results of the liquefaction potential and predicted excess pore pressures in the backfill at Ohama No.2 Wharf.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Depth [m]</th>
<th>CSR</th>
<th>$CRR_M$</th>
<th>FoS B&amp;I, 2008</th>
<th>$r_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfillsand &lt;gwl</td>
<td>3.1</td>
<td>0.31</td>
<td>0.09</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>5.0</td>
<td>0.31</td>
<td>0.11</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>7.2</td>
<td>0.30</td>
<td>0.12</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>9.2</td>
<td>0.29</td>
<td>0.11</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>11.1</td>
<td>0.29</td>
<td>0.14</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>13.3</td>
<td>0.28</td>
<td>0.11</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Sand slightly silty &gt;gwl</td>
<td>15.3</td>
<td>0.26</td>
<td>1.24</td>
<td>4.71</td>
<td>0.00</td>
</tr>
<tr>
<td>Sand slightly silty &gt;gwl</td>
<td>17.6</td>
<td>0.25</td>
<td>473.4</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>18.1</td>
<td>0.25</td>
<td>3.13</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>18.5</td>
<td>0.25</td>
<td>282.8</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>19.1</td>
<td>0.24</td>
<td>205.7</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>20.2</td>
<td>0.24</td>
<td>0.21</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Clay very sandy &gt;gwl</td>
<td>21.3</td>
<td>0.23</td>
<td>0.17</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>22.0</td>
<td>0.23</td>
<td>0.32</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>23.0</td>
<td>0.23</td>
<td>0.14</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>24.1</td>
<td>0.22</td>
<td>0.15</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Sand slightly silty &gt;gwl</td>
<td>24.9</td>
<td>0.22</td>
<td>0.20</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>25.8</td>
<td>0.22</td>
<td>14.78</td>
<td>5.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>27.0</td>
<td>0.21</td>
<td>0.40</td>
<td>1.89</td>
<td>0.07</td>
</tr>
</tbody>
</table>

4.2.3. Structural properties

In the paper by Iai, S. and Kameoka, T. (1993) geometry of structures and used materials are documented, which is used as a starting point in determining the properties of the structures. First properties of the sheet piles are derived. Secondly the properties of the anchor walls of the both quays are determined, which differ from each other.

Sheet piles

The sheet piles of both anchored quay walls are of the type FSP VIL. Geometrical properties of this type of sheet pile profile are presented in Table 4.3. The used steel class for the sheet piles is not documented. It is however stated that the yield strength ($f_y$) of the steel sheet pile is 300 $N/mm^2$. The maximum elastic moment the sheet pile can resist is determined using $M_{max,el} = W_{el} \cdot f_y$, for the plastic bending moment capacity the $W_{pl}$ is determined. From Eurocode 7 it follows that for a rough surface the wall friction angle $\delta$ is equal to $(2/3)\phi'$ when the calculation is performed for planar sliding planes.

Table 4.3: Properties of FSP VIL type sheet piles.

<table>
<thead>
<tr>
<th>Sheet pile properties</th>
<th>Type of sheet pile: FSP VIL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section area</td>
<td>A [cm/m']</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>W $[cm^3/m']$</td>
</tr>
<tr>
<td>Second moment of area</td>
<td>I $[cm^4/m']$</td>
</tr>
<tr>
<td>Profile width</td>
<td>w [mm]</td>
</tr>
<tr>
<td>Profile height</td>
<td>h [mm]</td>
</tr>
<tr>
<td>Youngs modulus</td>
<td>E $[kN/m^2]$</td>
</tr>
<tr>
<td>Bending stiffness</td>
<td>EI $[kNm^2/m']$</td>
</tr>
<tr>
<td>Yield strength</td>
<td>fy $[N/mm^2]$</td>
</tr>
<tr>
<td>Elastic bending moment capacity</td>
<td>$M_{el,max}$ $[kNm/m']$</td>
</tr>
<tr>
<td>Plastic bending moment capacity</td>
<td>$M_{pl,max}$ $[kNm/m']$</td>
</tr>
</tbody>
</table>
**Anchor walls**

Steel tubular piles are placed around 20 meters behind the sheet pile walls at both locations, which are connected to the sheet piles with semi-high strength steel tie rods. Spacing of the anchors is 2.00 meters and the diameter of the steel tie rods is 55 millimeters. The steel class of the tubular piles is not documented, as a first estimation a steel class S355 is adopted.

The stiffness of the anchor is defined by the amount of deformation required to mobilize a certain anchor force. Since the stiffness of the anchors is not exactly known an iterative calculation is performed aimed at matching the deformation of the sheet pile wall and the anchor wall for a certain anchor force. Both elongation of the tie rods and deformation of the anchor piles contributes to the total deformation of the anchor, see Figure 4.8.

![Figure 4.8: Schematization of displacement anchor wall and lengthening of tie rod leading to total displacement of sheet pile wall.](image)

**Group effects**

Because of the relatively small centre-to-centre distance between the piles compared to the pile diameter, group effects may be expected. Potential group effects are analysed by modelling the anchor piles in D-Pile Group using the Cap layered interaction model. This model accounts for group effects of piles and is used to calculate the capacity of the piles including influence of these group effects. The program is however less suitable for calculation of the the deformations during seismic events, because adjustments to the input soil properties cannot be made according to the Mononobe-Okabe method. Therefore D-Sheet Piling Single Pile is used to analyse the deformations during seismic events.

Results of D-Pile Group model are compared to results of the Single Pile model for the static situation to investigate if group effects influence the performance of the individual piles.

- **Ohama No.1 Wharf**
  Both models predict the same deformation pattern, leading to the conclusion that group-effects are of minor interest for the Ohama No.1 Wharf. See Figure 4.9 for a schematization of the influence areas of the anchor piles. The influence area doesn't influence the horizontal capacity much for Ohama No.1 Wharf.

- **Ohama No.2 Wharf** The anchor wall of Ohama No.2 Wharf exists of two rows of piles behind each other, which are connected at the top by a concrete cap, see Figure 4.5. D-Sheet Piling with Single Pile module is however only capable modelling one single pile, while there are two piles present. Therefore the equivalent pile properties are adopted to match the displacement of the system calculated by D-Pile Group.

In Table 4.4 the properties of the individual anchor piles are presented together with the modified properties to match the single pile behaviour with the pile group behaviour.
4.2. Pseudo-static analysis

Figure 4.9: Planview of the anchor piles including a rough schematization of the influence areas of the piles; on the left for No.1 Wharf and on the right for No.2 Wharf

Table 4.4: Input properties of tubular anchor piles for both anchor walls.

<table>
<thead>
<tr>
<th>Anchor piles properties</th>
<th>Ohama No.1 Wharf</th>
<th>Ohama No.2 Wharf</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>One pile</td>
<td>Modified</td>
</tr>
<tr>
<td>Level top</td>
<td>L_{top} [m]</td>
<td>1.50</td>
</tr>
<tr>
<td>Level bottom</td>
<td>L_{bot} [m]</td>
<td>12.10</td>
</tr>
<tr>
<td>Diameter pile</td>
<td>D [mm]</td>
<td>750</td>
</tr>
<tr>
<td>Thickness wall</td>
<td>t [mm]</td>
<td>10</td>
</tr>
<tr>
<td>Section area</td>
<td>A [mm^2/m']</td>
<td>2.325E+04</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>W [mm^3/m']</td>
<td>4.244E+06</td>
</tr>
<tr>
<td>Second moment of area</td>
<td>I [mm^4/m']</td>
<td>1.592E+09</td>
</tr>
<tr>
<td>Steel class</td>
<td></td>
<td>S355</td>
</tr>
<tr>
<td>Bending stiffness</td>
<td>E [kN/m^2]</td>
<td>3.34E+05</td>
</tr>
<tr>
<td>Maximum elastic moment</td>
<td>M_{el,max} [kNm/m']</td>
<td>1.51E+03</td>
</tr>
</tbody>
</table>

**Tie-rod**  Elongation of the steel tie rod also contributes to the total displacement of the anchor. The tie rod is assumed to be homogeneous with a constant diameter resulting in a linear relationship between anchor force and elongation, presented in equation 4.1. See Table 4.5 for properties of the tie rods.

\[ \Delta L = \frac{L_{tierod} \cdot F_{anch}}{E \cdot A} \]  

(4.1)

Table 4.5: Properties of semi-high strength tie rods.

<table>
<thead>
<tr>
<th>Tie rod properties</th>
<th>Ohama No.1 Wharf</th>
<th>Ohama No.2 Wharf</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
<td>z [m]</td>
<td>1.50</td>
</tr>
<tr>
<td>E-modulus</td>
<td>E [kN/m^2]</td>
<td>2.10E+08</td>
</tr>
<tr>
<td>Section area</td>
<td>A [m^2/m']</td>
<td>1.19E-03</td>
</tr>
<tr>
<td>Length</td>
<td>L_{anch} [m]</td>
<td>16.0</td>
</tr>
<tr>
<td>Angle</td>
<td>\alpha [°]</td>
<td>0</td>
</tr>
<tr>
<td>Steel class</td>
<td></td>
<td>S355</td>
</tr>
<tr>
<td>Yield strength</td>
<td>f_y [N/mm^2]</td>
<td>355</td>
</tr>
<tr>
<td>Yield force</td>
<td>F_{el,max} [kN/m']</td>
<td>843</td>
</tr>
</tbody>
</table>
Stiffness anchors  Anchor force and corresponding total deformation are iteratively found aimed at matching the deformations of the anchor wall and the sheet pile given a certain anchor force.

In Figure 4.10 the steps in determining the stiffness of the anchors are presented, steps were described below:

- **Stiffness static phase**
  An anchor force obtained with D-Sheet Piling given an initially chosen anchor stiffness inserted in the D-Group model, which results in a certain deformation of the anchor wall. Including the elongation of the anchor tie rod this gives in a total displacement. Dividing the anchor force by the total displacement gives an updated anchor stiffness. The updated stiffness is applied in the D-Sheet Piling model to obtain an updated anchor force. This procedure is repeated until deformation of the anchor wall and sheet pile wall matches.

- **Equivalent pile properties**
  Once the anchor force is obtained for the static case, the properties of anchor pile in the Single Pile model are adjusted in such a way that the deformation of this single pile matches the calibrated deformation of the D-Pile Group model for the static situation. (This step is not required for Ohama No.1 Wharf.)

- **Stiffness dynamic phase**
  Lastly these modified pile properties are used in the Single Pile module of DSheet Piling to determine the anchor stiffness for the seismic events including the changed soil properties. Again the stiffness is iteratively found by matching the deformation of the anchor wall and the sheet pile wall updating the anchor stiffness.

Figure 4.10: Schematization of procedure how the properties of the anchor walls are determined using different models.
Ohama No.1 Wharf

The anchor stiffness of the anchored quay wall at Ohama No.1 Wharf are presented in Table 4.6. The stiffness of the anchor decreases with increasing seismic coefficient. Figure 4.11 represents the development of the anchor stiffness. The anchor force against the horizontal displacement for the Ohama No.1 Wharf is presented on the left. 4.11.

A linear approximation of the anchor stiffness a conservative approach. For lower anchor forces is the stiffness higher. However in this type of analysis only the maximum load is considered and therefore one anchor stiffness is adopted for each loading event corresponding to the maximum load.

Table 4.6: Calculated anchor stiffness of Ohama No.1 Wharf for each loading event.

<table>
<thead>
<tr>
<th>Ohama No.1 Wharf</th>
<th>Static</th>
<th>$k_h = 0.100$</th>
<th>$k_h = 0.206$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Representative spring constant [kN/m/m]</td>
<td>3727</td>
<td>2243</td>
<td>745</td>
</tr>
</tbody>
</table>

Ohama No.2 Wharf

The anchor stiffness of the quay wall at Ohama No.2 Wharf decreases much more compared to Ohama No.1 Wharf quay wall. The passive horizontal earth pressure supporting the anchor wall decreases due to the liquefaction of the upper soil layer, leading to a drop of the anchor capacity. In the paper by Iai, S. and Kameoka, T. (1993) displacements of the quay walls after the seismic event were documented. These measured displacements were together with maximum anchor force used to find the representative anchor stiffness.

The development of the anchor force against the horizontal displacement of Ohama No.2 Wharf is presented in Figure 4.11 on the right. The grey area represents the loading stage where the structure became unstable in the model due to progressive displacements. First a maximum anchor force with corresponding displacement are determined. An equivalent anchor stiffness was obtained by dividing the maximum anchor force by the average between the calculated displacement and the maximum measured displacement. Results are presented in Table 4.7.

Table 4.7: Calculated anchor stiffness of Ohama No.2 Wharf for each loading event.

<table>
<thead>
<tr>
<th>Ohama No.2 Wharf</th>
<th>Static</th>
<th>$k_h = 0.100$</th>
<th>$k_h = 0.206$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Representative spring constant [kN/m/m]</td>
<td>3558</td>
<td>156</td>
<td>107</td>
</tr>
</tbody>
</table>

Figure 4.11: Development of anchor stiffness; On the left anchor force against horizontal displacement for Ohama No.1 Wharf, on the right for Ohama No.2 Wharf.
4.2.4. Results

Once all properties of soil and structures are determined the pseudo-static analysis is performed for both quay walls. The Mononobe-Okabe method is applied to determine the total dynamic earth pressures, including effects of the presence of water in the backfill. An equivalent unit weight is determined, which is then used to adjust the seismic inclination angle (see section 2.7). Dynamic water pressures of the free standing water in front of the quay wall are calculated according to Westergaard. In the following sections results of the pseudo-static analysis are presented for both cases in Akita Port.

Ohama No.1 Wharf

The D-Sheet Piling model of the anchored quay wall Ohama No.1 Wharf is presented in Figure 4.12. For every seismic event stiffness of the anchor and soil properties of different layers are adjusted. A distributed load can not be modelled over the height of the quay wall, so every meter a point load is applied, which represent the Westergaard dynamic water pressure.

![Figure 4.12: D-Sheet Piling model of quay wall at Ohama No.1 Wharf for dynamic stage.](image)

The Mononobe-Okabe method is applied and adjusted to account for the presence of water, exclusive excess pore pressures. In Table 4.8 both active and passive earth pressure coefficients for the static case and the dynamic cases are presented.

Extreme values of the performed calculations are presented in Table 4.9. The bending moment distribution and the deformation of the structure are presented respectively in Figure 4.13 and Figure 4.14.

**Table 4.8: Input earth pressure coefficients for soil layers at Ohama No.1 Wharf.**

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Depth [m]</th>
<th>Static situation</th>
<th>$k_h = 0.100$</th>
<th>$k_h = 0.206$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1 (Sand- Med dense)</td>
<td>0.0</td>
<td>$K_A$ $K_Q$ $K_P$</td>
<td>$K_{AE}$ $K_{PE}$</td>
<td>$K_{AE}$ $K_{PE}$</td>
</tr>
<tr>
<td>Layer 2 (Sand - Loose)</td>
<td>8.5</td>
<td>0.33 0.55 8.08</td>
<td>0.38 6.89</td>
<td>0.58 5.56</td>
</tr>
<tr>
<td>Layer 3 (Sand - Med dense)</td>
<td>9.5</td>
<td>0.26 0.46 8.08</td>
<td>0.38 6.93</td>
<td>0.56 5.64</td>
</tr>
<tr>
<td>Layer 4 (Sand - Dense)</td>
<td>13.5</td>
<td>0.23 0.40 12.55</td>
<td>0.33 10.84</td>
<td>0.49 8.97</td>
</tr>
<tr>
<td>Layer 5 (Clay - Firm)</td>
<td>21.0</td>
<td>0.33 0.55 4.75</td>
<td>0.47 3.99</td>
<td>0.71 3.09</td>
</tr>
<tr>
<td>Layer 6 (Sand - Med dense)</td>
<td>22.5</td>
<td>0.24 0.43 9.96</td>
<td>0.35 8.57</td>
<td>0.53 7.04</td>
</tr>
</tbody>
</table>
4.2. Pseudo-static analysis

Table 4.9: Results of maximum bending moments, anchor force and displacements of anchored quay wall at Ohama No.1 Wharf.

<table>
<thead>
<tr>
<th>Event</th>
<th>$M_{max}$ [kNm/m']</th>
<th>$M_{min}$ [kNm/m']</th>
<th>$F_{anch,max}$ [kN/m']</th>
<th>$u_{max}$ [mm]</th>
<th>$u_{anch}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>-378</td>
<td>406</td>
<td>129</td>
<td>52</td>
<td>35</td>
</tr>
<tr>
<td>$k_h = 0.100$</td>
<td>-651</td>
<td>613</td>
<td>202</td>
<td>140</td>
<td>125</td>
</tr>
<tr>
<td>$k_h = 0.206$</td>
<td>-1380</td>
<td>367</td>
<td>360</td>
<td>520</td>
<td>514</td>
</tr>
</tbody>
</table>

Maximum bending moments are at approximately half the retaining height of the wall. For the higher seismic loading the active earth pressure becomes larger and the passive earth pressure decreases. The maximum elastic bending moment capacity of the sheet pile is 1146 kNm/m', which is lower than the calculated 1380 kNm/m' for a seismic coefficient of 0.206.

The passive earth pressure decreased leading to a lower clamping of the wall. The embedded part of the quay wall deformed in the direction of the passive wedge for due to this lower pressure, as is presented in Figure 4.14. A decreased anchor stiffness however also results in larger displacements at the top of the structure. Both mechanism contribute to the increased total displacement of the quay wall, although decreasing of the anchor stiffness contributes more to the increased displacements.

Figure 4.13: Bending moment distribution of anchored quay wall at Ohama No.1 Wharf for both considered seismic coefficients.
Ohama No.2 Wharf

In Figure 4.15 configuration of the model for Ohama No.2 Wharf is shown. The earth pressure coefficients used for each loading stage are presented in Table 4.10. Excess pore pressures are expected in the top layer according to Idriss, I.M. and Boulanger, R.W. (2008), resulting in complete liquefaction of this layer. Decrease of effective stresses in this layer is modelled by manually increasing the water pressure in this layer, representing the excess pore pressures. Finally the effective stresses become even zero in the top layer.

Extreme values of the results are presented in Table 4.11. Both bending moment distribution and deformation distribution over the height of the structure are presented respectively in Figure 4.16 and Figure 4.17.

Table 4.10: Input earth pressure coefficients for soil layers at Ohama No.2 Wharf.

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Depth [m]</th>
<th>$K_A$</th>
<th>$K_0$</th>
<th>$K_P$</th>
<th>$K_{AE}$</th>
<th>$K_{PE}$</th>
<th>$K_{AE}$</th>
<th>$K_{PE}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1 (Sand - (Very) loose)</td>
<td>0.0</td>
<td>0.30</td>
<td>0.50</td>
<td>6.11</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Layer 2 (Sand - loose)</td>
<td>12.5</td>
<td>0.33</td>
<td>0.55</td>
<td>4.75</td>
<td>0.47</td>
<td>3.96</td>
<td>0.73</td>
<td>3.02</td>
</tr>
<tr>
<td>Layer 3 (Sand- dense)</td>
<td>14.5</td>
<td>0.20</td>
<td>0.36</td>
<td>18.72</td>
<td>0.30</td>
<td>16.16</td>
<td>0.46</td>
<td>13.40</td>
</tr>
<tr>
<td>Layer 4 (Clay - firm)</td>
<td>20.0</td>
<td>0.33</td>
<td>0.55</td>
<td>4.75</td>
<td>0.47</td>
<td>3.96</td>
<td>0.73</td>
<td>3.02</td>
</tr>
<tr>
<td>Layer 5 (Sand- Med dense)</td>
<td>22.5</td>
<td>0.26</td>
<td>0.46</td>
<td>8.08</td>
<td>0.38</td>
<td>6.89</td>
<td>0.58</td>
<td>5.55</td>
</tr>
<tr>
<td>Layer 6 (Sand - (Med) dense)</td>
<td>25.5</td>
<td>0.24</td>
<td>0.43</td>
<td>9.96</td>
<td>0.36</td>
<td>8.52</td>
<td>0.54</td>
<td>6.94</td>
</tr>
</tbody>
</table>
4.2. Pseudo-static analysis

Figure 4.15: D-Sheet Piling model of quay wall at Ohama No.2 Wharf for dynamic stage.

Table 4.11: Results of maximum bending moments, anchor force and displacements of anchored quay wall at Ohama No.2 Wharf.

<table>
<thead>
<tr>
<th>Event</th>
<th>$M_{\text{max}}$ [kNm/m']</th>
<th>$M_{\text{min}}$ [kNm/m']</th>
<th>$F_{\text{anch, max}}$ [kN/m']</th>
<th>$u_{\text{max}}$ [mm]</th>
<th>$u_{\text{anch}}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>-364</td>
<td>432</td>
<td>116</td>
<td>52</td>
<td>33</td>
</tr>
<tr>
<td>$k_h = 0.100$</td>
<td>-558</td>
<td>3680</td>
<td>238</td>
<td>1697</td>
<td>1538</td>
</tr>
<tr>
<td>$k_h = 0.206$</td>
<td>-556</td>
<td>4543</td>
<td>256</td>
<td>2661</td>
<td>2435</td>
</tr>
</tbody>
</table>

Differences are found between the two quay walls with regard to the calculated seismic bending moments and displacements. The top soil layer at Ohama No.2 Wharf fully liquefied, which has leads to a drop in stiffness of the anchor. Due to the small resistance of the anchor displacements at the top become much larger compared to the quay wall at Ohama No.1 Wharf. As a result bending moments at the retaining part of the quay wall are only limited despite the fact that the active pressures increase.

The loss of resistance of the anchor combined with relative stiff layers at the embedded part of the structure leads to a bending moment distribution of a clamped beam. This results in high bending moments at the location where the structure is fixed for rotation and a lower field moment.

Observed damage after the structure was hit by the earthquake does to some extent match with the calculated results of the pseudo-static analysis, see Figure 4.6. Cracks were discovered both halfway the retaining height and at the embedded part of the structure. Locations of the calculated maximum and minimum bending moments do match with the locations of these cracks. The maximum bending moment is however too high. On the other hand is the minimum bending moment too low to cause significant damage as was observed. In reality the wall probably was clamped to less extent than in the model and the anchor stiffness higher. This would lead to higher bending moments in the retaining part of the structure and lower bending moments at the embedded part.

Assessing the results according to both seismic coefficients, the bending moment distribution is for both seismic coefficients exaggerated. Resulting displacements for the $k_h$ of 0.100 are although more in line with observations in the field. The model is however known to be not accurate in prediction of deformations, especially close to failure. A seismic coefficient of 0.100 leads thus leads to results that are more in line with observations compared to a $k_h$ of 0.206, although still not accurate.
Figure 4.16: Bending moment distribution of anchored quay wall at Ohama No.1 Wharf for both considered seismic coefficients.

Figure 4.17: Deformation distribution of anchored quay wall at Ohama No.2 Wharf for both considered seismic coefficients.
As was already discussed, the structure behaves like a cantilever beam. In the analysis one value for the wall friction angle ($\delta = (2/3)\phi$) is adopted. Some design codes however prescribe a lower wall friction angle (or even zero) for the passive side, because of the uncertainty of the magnitude of this friction angle.

A lower wall friction angle on the passive side is in most cases a conservative approach, because the structure relies on the passive earth pressure and decreasing it will lead to larger internal forces and larger displacements. In this case the difference in stiffness between the anchor and soil layers where the wall is embedded is however so large that a smaller passive earth pressure will be favourable for the distribution of the bending moments. Lower bending moments at the embedded part and higher bending moments at the retaining part of the structure are expected. On the other hand due to the lower passive earth pressures the displacements will increase.

Table 4.12: Results of maximum bending moments, anchor force and displacements of anchored quay wall at Ohama No.2 Wharf considering a wall friction angle of zero.

<table>
<thead>
<tr>
<th>Event</th>
<th>$M_{max}$ [kNm/m']</th>
<th>$M_{min}$ [kNm/m']</th>
<th>$F_{anch,max}$ [kN/m']</th>
<th>$u_{max}$ [mm]</th>
<th>$u_{anch}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>-364</td>
<td>432</td>
<td>116</td>
<td>52</td>
<td>33</td>
</tr>
<tr>
<td>$k_h = 0.100$</td>
<td>-1853</td>
<td>1358</td>
<td>470</td>
<td>3179</td>
<td>3027</td>
</tr>
<tr>
<td>$k_h = 0.206$</td>
<td>\</td>
<td>\</td>
<td>\</td>
<td>\</td>
<td>\</td>
</tr>
</tbody>
</table>

Figure 4.18: Bending moment distribution and displacement of anchored quay wall at Ohama No.2 Wharf considering a wall friction angle of zero.

Results of the pseudo-static analysis with a wall friction angle of zero for the passive earth pressure coefficients are presented in Table 4.12. In Figure 4.18 the results for zero wall friction angle are compared with the results where $\delta = (2/3)\phi$ is adopted. The found maximum bending moments for delta of zero are more in line with the observed damage and also the distributions matches. However calculated the displacements are overestimated. Only results are found for a seismic coefficient of 0.100, because for a $k_h$ of 0.206 the passive earth pressure decreased too much and the structure is calculated unstable.
4.3. Conclusion

Applying the Mononobe-Okabe method for the evaluation of the anchored quay wall at Ohama No.1 Wharf results in a reasonable fit with observations. Although only for a reduced seismic coefficient. After reducing the leading extreme bending moments remain below the bending moment capacity. The calculated displacements are however not in line with measured residual displacements.

On the other hand the type of method is not capable to calculate bending moments in the anchored quay wall at Ohama No.2 Wharf. Liquefaction occurred in the backfill leading to a significant loss of stiffness of the anchor. Relatively stiff layers are present at the embedded part of the structure, leading to a large difference in stiffness between anchor and the embedment of the structure. The model is not able to cope with this large difference in stiffness, leading to exaggerated bending moments. Decreasing the difference in stiffness lead to a better fit with observations, however displacements are in that case overestimated.

Different seismic coefficients are considered in this analysis of the Akita Port case, leading to different bending moment and displacement distributions for the same earthquake loading. Assessment of an experimental reference case (Higuchi, S. et al., 2012) in previous section already showed that the seismic coefficient could be lowered approximately by a factor two. Applying the design codes for the determination of the seismic coefficient for the anchored quay walls in Akita Port leads to an upper boundary of the seismic coefficient. Too many unknown factors are present in the analysis to conclude that this upper boundary could be lowered. However halving the design value of the seismic coefficient leads in this analysis to results that are more in line with observations in the field.

Plastic soil behaviour not modelled well in this type of analysis. Also the soil structure interaction is not accounted for, since large deformations influence the development of excess pore pressures and corresponding soil behaviour. More sophisticated models are applied in the next chapters to account for these aspects in a dynamic analysis.
In order to gain more insight in the actual soil structure interaction a dynamic analysis is performed using an effective stress finite element model. Constitutive material models have to be adopted in the finite element model that describe the dynamic soil behaviour. Basically these models describe the relation between stress and strain of the soil. Because of the large amount of different soil- and loading types and purposes of the model, many different constitutive models are developed from simple to very complex. Each model has advantages and disadvantages, consequently for each case the most suitable constitutive model has to be chosen.

The user-defined UBC3D-PLM constitutive material model is adopted for modelling the behaviour of liquefiable soils. This model is originally developed for potentially liquefiable sandy soils and able to accumulate plastic strains and pore pressures in case of undrained cyclic loading. Background of the model is described in more detail in Annex C.

Performance of the model depends very much on calibration of the parameters. Model results are not always realistic and must always be reviewed critically. Especially when the static shear ratio increases the model experiences problems (Makra, A., 2013). Most parameters are stress path dependent and have to be calibrated for each specific situation.

In this section the main objectives are:

- Evaluate parameter selection methods for UBC3D-PLM constitutive model
- Evaluate effects of state of the soil on the onset of liquefaction and the performance of the model
- Define a calibration method to find suitable parameters for the UBC3D-PLM model specific for loading conditions around an anchored quay wall using typical laboratory tests.

To achieve this goals, first loading paths around an anchored quay wall are defined corresponding to typical laboratory tests. For two types of soil the liquefaction resistance curve was determined in the laboratory using to cyclic triaxial tests. Element tests (DSS, TC and TE) are reproduced in the soil test facility of PLAXIS 2D, which are calibrated to fit these laboratory results. Two calibration methods are applied to determine the initial model parameters for the UBC3D-PLM model. The fit the model with liquefaction resistance curves from the laboratory is evaluated. Subsequently model parameters are calibrated in order to obtain a reasonable accurate fit with laboratory results.

These tests are all performed under specific loading conditions. Around an anchored quay wall large difference in loading states. These varying stress states have influence on the liquefaction potential of the soil but also influence the model performance. Calibrated model parameter sets are established leading to accurate results for these varying loading conditions.
5.1. Defining loading paths

Constitutive models used for dynamic analysis of a system can be calibrated by fitting cyclic element tests on experimental test results (Finn, W.D. et al. (1995), Marcuson, W.F. et al. (2007), Beaty, M.H. and Perlea, V.G. (2011), Ziotopoulou, K. and Boulanger, R.W. (2015)). It is crucial for the accuracy of the calibration to determine the parameters using laboratory tests that fit properly the loading conditions existing in the field. The behaviour of the soil and the model is stress path dependent and for that reason the loading conditions have to be taken into account in the calibration procedure to acquire accurate results (Petalas, A. and Galavi, V., 2013).

Around an anchored quay walls several zones can be identified that fit the loading conditions corresponding to typical element tests. Model parameters for these zones are calibrated to give accurate results for the typical loading conditions. In Figure 5.1 soil tests corresponding to loading conditions around an anchored quay wall are indicated.

In passive soil wedge of the sheet pile wall the stress path corresponds to a lateral compression (or axial extension in undrained conditions) test, since the horizontal stress increases and is expected to become higher than the vertical stress. At the active side of the anchor wall a lateral extension (or axial compression) test corresponds to the local loading conditions and failure behaviour, since the horizontal earth pressure decreases in the active zone.

In between the anchor wall and the sheet pile wall these zones could also be identified. However because of the close distance between each other these zones will overlap leading to complex stress states, which are not be indicated on beforehand. For that reason in these zones at this moment no tests are specified. Looking at the deformation of the system as a whole the direct simple shear test can be specified between the anchor wall and the sheet pile wall, since the horizontal and vertical effective stresses do not change and the sample is sheared by the seismic loading.

Figure 5.1: Schematization of Ohama No.2 Wharf and relevant laboratory tests to modes of shearing on potential surfaces of sliding planes.

In passive soil wedge of the sheet pile wall the stress path corresponds to a lateral compression (or axial extension in undrained conditions) test, since the horizontal stress increases and is expected to become higher than the vertical stress. At the active side of the anchor wall a lateral extension (or axial compression) test corresponds to the local loading conditions and failure behaviour, since the horizontal earth pressure decreases in the active zone.

In between the anchor wall and the sheet pile wall these zones could also be identified. However because of the close distance between each other these zones will overlap leading to complex stress states, which are not be indicated on beforehand. For that reason in these zones at this moment no tests are specified. Looking at the deformation of the system as a whole the direct simple shear test can be specified between the anchor wall and the sheet pile wall, since the horizontal and vertical effective stresses do not change and the sample is sheared by the seismic loading.
5.2. Model parameters UBC3D-PLM model

Within this research two parameter selection procedures are proposed to determine initial model parameters for the UBC3D-PLM model. In Appendix C the theoretical background is presented of how the soil behaviour is described in the model.

The considered methods for the extraction of the main model parameters are based on fitting curves on experimental element tests. These methods differ to some extent from each other since they are obtained from other experimental tests and other types of sand. The UBC3D-PLM model is however very sensitive for the type of input and in order to obtain accurate results calibration to the stress path and soil type of interest is required. Two parameter selection procedures are considered in order to investigate what the differences between these methods are and to evaluate which method is the best applicable for the soil type that is present in Akita Port.

In Table 5.1 all the model parameters of the UBC3D-PLM model are presented. Relationships for these parameters are elaborated according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) which requires SPT-values as input. A second parameter selection procedure is developed by Souliotis, C. and Gerolymos, N. (2016), which uses the relative density of the soil as input. It must be stated that these methods are meant to obtain initial model parameters. Generally the model has to be calibrated to experimental element tests performed on the soil type of interest.

Table 5.1: Model parameters used in the UBC3D-PLM model (Beaty, M.H. and Byrne, P.M. (2011), Makra, A. (2013), Petalas, A. and Galavi, V. (2013))

<table>
<thead>
<tr>
<th>Corrected SPT blow counts</th>
<th>$(N_{1})_{d0}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction angle at constant volume</td>
<td>$\varphi_{cv}$ [°]</td>
</tr>
<tr>
<td>Peak friction angle</td>
<td>$\varphi_{peak}$ [°]</td>
</tr>
<tr>
<td>Effective cohesion</td>
<td>$c'$ [kPa]</td>
</tr>
<tr>
<td>Elastic shear modulus number</td>
<td>$k_{G}^{e}$ [-]</td>
</tr>
<tr>
<td>Plastic shear modulus number</td>
<td>$k_{G}^{p}$ [-]</td>
</tr>
<tr>
<td>Elastic bulk modulus number</td>
<td>$k_{B}^{e}$ [-]</td>
</tr>
<tr>
<td>Elastic bulk modulus power</td>
<td>$m_{e}$ [-]</td>
</tr>
<tr>
<td>Elastic shear modulus power</td>
<td>$n_{e}$ [-]</td>
</tr>
<tr>
<td>Plastic shear modulus power</td>
<td>$n_{p}$ [-]</td>
</tr>
<tr>
<td>Failure ratio</td>
<td>$R_{f}$ [-]</td>
</tr>
<tr>
<td>Reference stress</td>
<td>$p_{A}$ [kPa]</td>
</tr>
<tr>
<td>Tension cut-off</td>
<td>$\sigma_{t}$ [kPa]</td>
</tr>
<tr>
<td>Fitting parameter to adjust densification rule</td>
<td>$f_{ac_{hard}}$ [-]</td>
</tr>
<tr>
<td>Fitting parameter to adjust post liquefaction behaviour</td>
<td>$f_{ac_{post}}$ [-]</td>
</tr>
</tbody>
</table>
**Beaty & Byrne (2011) and Makra (2013)**

Model parameters are determined by curve fitting experimental data, preferably from cyclic undrained direct simple shear (DSS) tests. Mostly these experimental test results are not available and only in-situ tests like Standard Penetration test (SPT) or Cone Penetration test (CPT) are performed. Beaty, M.H. and Byrne, P.M. (2011) have developed a parameter selection procedure that requires corrected SPT-values as input. The method is developed by fitting experimental tests on Fraser Sand, but is generally also used for other type of sands. Below are the relationships presented (Beaty, M.H. and Byrne, P.M., 2011):

\[
\sin \varphi_{cv} = \frac{\sin \varphi - \sin \psi}{1 - \sin \varphi \sin \psi}
\]  
(5.1)

\[
\varphi_{pi} = \varphi_{cv} + \frac{(N_1)_{60}/10.0}{10.0}
\]  
(5.2)

\[
\varphi_p = \varphi_{pi} + \max \left( 0, \frac{(N_1)_{60} - 15}{5} \right)
\]  
(5.3)

\[
k_G^c = 21.7 \cdot 20.0 \cdot (N_1)_{60}^{0.333}
\]  
(5.4)

\[
k_B^c = k_G^c \cdot 0.7
\]  
(5.5)

\[
k_P^c = k_G^c \cdot (N_1)_{60}^{2} \cdot 0.003 + 100.0
\]  
(5.6)

\[ne = me = 0.5\]  
(5.7)

\[np = 0.4\]  
(5.8)

\[R_f = 1.1 \cdot (N_1)_{60}^{-0.15}\]  
(5.9)

\[f_{ac_{hard}} = 0.45 \text{ or } f_{ac_{hard}} = 1.0\]  
(5.10)

\[f_{ac_{post}} = 0.02 \text{ or } f_{ac_{post}} = 1.0\]  
(5.11)

**Souliotis and Gerolymos (2016)**

The second method is based on a two-step procedure, developed by Souliotis, C. and Gerolymos, N. (2016). It requires relative density and \((N_1)_{60}\) as input. All relationships proposed in the calibration method are presented below.

\[
\sin \varphi_{cv} = \frac{\sin \varphi - \sin \psi}{1 - \sin \varphi \sin \psi}
\]  
(5.12)

\[
\varphi_p - \varphi_{cv} = 3.8 \cdot \left[ D_r \cdot \left( 9 - \ln(p_0) \right) - 0.9 \right] \geq 0
\]  
(5.13)

\[
k_G^c = 1592 \cdot D_r^{2/3}
\]  
(5.14)

\[
k_B^c = 0.7 \cdot k_G^c
\]  
(5.15)

\[
k_P^c = 2.0 \cdot p_a^{-1} \cdot D_r^{3/2} - 3.4 \cdot D_r^{3/2} + 2.1 \cdot p_a \cdot D_r - 0.3 \cdot p_a^2
\]  
(5.16)

\[ne = me = 0.5\]  
(5.17)

\[np = D_r\]  
(5.18)

\[R_f = \frac{1}{(N_1)_{60}^{0.15}}\]  
(5.19)

\[f_{ac_{hard}} = a \cdot \exp(b \cdot (N_1)_{60}) \geq 0.12\]  
(5.20)

With:

\[a = 0.018 \cdot (\sigma_{iv0}/p_a)^{0.42}\]  
(5.21)

\[b = 7.51 \cdot 10^{-4} \cdot (\sigma_{iv0}/p_a)^2 - 1.79 \cdot 10^{-2} \cdot (\sigma_{iv0}/p_a) + 0.19\]  
(5.22)
As shown in Figure 5.2 adjusting the post liquefaction factor \( (f_{ac_{post}}) \) can be used to calibrate the model to the cyclic resistance curve by Idriss, I.M. and Boulanger, R.W. (2008). This is only relevant for very dense soils as the model results differ in that region from the theoretical curve. In this research the more denser sandy soils are not of interest since the considered soils have a maximum SPT-value of approximately 10. Besides these very dense packed soils also have a very low liquefaction potential and are therefore of less interest in this research. Adjusting the \( f_{ac_{post}} \) has therefore no influence on the CSR curve for the considered types of soil.

![Figure 5.2: Effect of post liquefaction factor on the cyclic resistance ratio (Souliotis, C. and Gerolymos, N., 2016)](image)

### 5.3. Liquefaction resistance sands Akita Port

In order to evaluate the liquefaction resistance of the soil layers at Ohama No.1 Wharf and Ohama No.2 Wharf two kind of sands were taken from Akita Port to test in the laboratory. The first soil type was taken from the backfill of Ohama No.2 Wharf (Ohama Sand). Secondly a soil sample was taken from the fill at the Gaiko district. This last sample is considered to be representing for the backfill of Ohama No.1 Wharf and for the lower sand layers at both locations (Iai, S. and Kameoka, T., 1993).

On both soil samples undrained cyclic triaxial tests were performed. In Figure 5.3 the obtained liquefaction curves are presented, with on the horizontal axis the amount of cycles and on the vertical axis the shear stress ratio. Liquefaction of the sample is defined by referring to two different states. Firstly the state of reaching the excess pore water pressure of 100% of the initial confining pressure as indicated by the open squares or circles and secondly reaching the axial strain level of 5% or 10% (in the double amplitude), which is indicated by the closed squares, circles or triangles.

As can been seen in Figure 5.3 liquefaction resistance of the Gaiko sand sample is higher than the Ohama Sand, even for a lower relative density. This indicates that not only the relative density is of importance in determining the resistance against liquefaction. Possible explanation of the difference in resistance could be that shape of the soil particles of the considered soils is different. Besides the shape of the particles the Gaiko Sand also contains more smaller sand particles and has a larger silt content, which generally leads to a higher resistance against liquefaction (Idriss, I.M. and Boulanger, R.W., 2008).
5.4. Performance UBC3D-PLM according to literature

Calibration of the UBC3D-PLM constitutive material model is done by reproducing the cyclic resistance curves obtained in the laboratory. These curves were obtained under certain circumstances, which influence the resistance against liquefaction. Since the stress path has influence on the liquefaction resistance, this also has to be properly modeled and captured by the model.

Makra, A. (2013) investigated effects of critical state parameters on triggering of liquefaction predicted by the model. She suggested possible adjustments to the $f_{\text{achard}}$ and $f_{\text{apos}}$ parameters to improve model performance for certain loading conditions. Subsequently Winde, H.P. (2015) investigated the influence of changing model parameters on outcome of the model, which resulted in a set of parameters that can be adjusted to influence onset of liquefaction predicted by the model. Before modelling the case at Akita Port, first the expected model performance for changing state parameters is investigated based on research by Makra, A. (2013) and Winde, H.P. (2015). By using this earlier research on the performance of the model the obtained model results in this research can be analysed faster and specific improvements can be implemented directly.

Effect of state of the soil

Relative density of soil is one of the most decisive parameters for the resistance against liquefaction. Generally the cyclic resistance at a certain overburden stress increases with an increasing rate when the relative density increases (Idriss, I.M. and Boulanger, R.W., 2008). As was shown by (Makra, A., 2013) the model results are in accordance with this theory. Although calibration to laboratory test results remains crucial to obtain accurate model results.

Loading level

Calibration of the model can be done for several loading levels, depending on the expected loading level in the field. Generally the model is calibrated to fit an average expected CSR. Less accurate results were found for both lower and higher stress ratios. Typically CSR curves predicted by the UBC3D-PLM model have a higher gradient compared to what is observed based on actual laboratory tests.

Effect of overburden pressure

The initial vertical effective stress level also influences the liquefaction resistance of soil. Higher overburden stresses lead to a decrease of liquefaction resistance for constant relative density (Idriss, I.M. and Boulanger, R.W., 2008). Results of the calibrated model by Makra, A. (2013) show that the decrease of cyclic resistance for increasing $\sigma_{v0}'$ is larger than what is obtained with the empirical solution by Idriss, I.M. and Boulanger, R.W. (2008). This leads to a conservative approach of the soil strength at higher overburden stresses. Suggested improvement by Makra, A. (2013) is to adjust the hardening factor ($f_{\text{achard}}$) as the initial overburden stress changes to correct for this underestimation of the soil strength.

Effect of static lateral earth pressure coefficient

Generally the cyclic resistance ratio increases for increasing $K_0$ values. Calibration by Makra, A. (2013) shows that the UBC3D-PLM model does not follow this trend. Resistance against liquefaction of loose sands for isotropic conditions is smaller than resistance predicted by the model for $K_0 = 0.5$. This does not match with theory. For dense sands the cyclic resistance is larger for isotropic conditions compared to a lateral earth pressure coefficient of $K_0 = 0.5$.  

Figure 5.3: Liquefaction resistance curve Ohama Sand and Gaiko Sand at Akita Port (Iai, S. and Kameoka, T., 1993).
5.4. Performance UBC3D-PLM according to literature

Effect of static shear stress
Typical for sloping ground is the presence of initial static shear stress, which affects the cyclic resistance of the soil. For dense sands an increase of the static shear stress ratio ($\alpha$) leads to a higher resistance against liquefaction. On the other hand for loose soil the cyclic resistance ratio decreases for increasing static shear stress ratios (Idriss, I.M. and Boulanger, R.W., 2008).

The UBC3D-PLM model is very sensitive, especially when initial static shear is present. Results of the calibrated model by Makra, A. (2013) show that the cyclic resistance ratio drops drastically when initial static shear is included. The model underestimates the resistance against liquefaction significantly for both loose and dense soils and also for different cyclic stress ratios. Therefore it can be concluded that the model is not able to correctly reflect the effect of static shear on CRR for a range of $\alpha$ values.

This is explained by failure due to flow failure. Liquefaction does in this case not lead to zero effective stress, but failure occurs due to the fact that the shear stress already exceeds the decreasing shear strength of the soil (Makra, A., 2013).

Damping
The actual damping mechanisms are too complex to model individually and therefore the energy dissipation is modelled by an alternative representative damping mechanism. The energy is dissipated by the hysteretic loop due to the cyclic loading. Empirically obtained damping curves depending on the amount of strain are derived by several researchers, see section 6.2.1 for different damping curves. Makra, A. (2013) found that the model produces larger damping compared to the empirical curves and relates this to the elastic unloading behaviour in the UBC3D-PLM model.

Table 5.2: Summary model performance for DSS test (Makra, A., 2013).

<table>
<thead>
<tr>
<th>Cyclic direct simple shear test (DSS) test</th>
<th>State of the soil</th>
<th>Loading level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Higher relative density leads to higher resistance</td>
<td>Calibration on medium loading level leads to:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Overestimation of soil resistance for higher loading levels</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Underestimation of soil resistance for lower loading levels</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Overburden pressure</th>
<th>Higher overburden pressure leads to underestimation of soil resistance</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>(An)isotropic loading</th>
<th>Isotropic loading conditions for:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>- Loose sands: lower liquefaction resistance compared to K0 = 0.50 loading conditions</td>
</tr>
<tr>
<td></td>
<td>- Dense sand: higher liquefaction resistance compared to K0 = 0.50 loading conditions</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial static shear</th>
<th>Underestimation of resistance for higher initial static shear stress ratios</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Damping</th>
<th>Damping ratio is larger in model than in reality</th>
</tr>
</thead>
</table>

All observations by other researchers are based on the simulation of cyclic direct simple shear (DSS) tests, while loading conditions corresponding to cyclic triaxial tests are also relevant around an anchored quay wall. No literature was found about the model performance for cyclic triaxial loading conditions leading to the onset of liquefaction of the soil.
5.5. Initial model parameters

In order to investigate the performance of the UBC3D-PLM liquefaction model for the two types of sand considered in this research for several loading conditions, typical laboratory element tests were simulated in the soil test facility in PLAXIS. Results of these element tests were analysed in order to gain insight in the behaviour of the model. The initial set of model parameters is derived based on correlations proposed by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) and by Souliotis, C. and Gerolymos, N. (2016).

The method by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) gives correlations for the model parameters based on the corrected SPT blow-count \((N1)_{60}\). In Figure 5.3 the corrected blow-counts for both types of sand at Ohama No.2 Wharf are indicated in orange and the corresponding model parameters based on these corrected blow-counts are presented on the left.

Souliotis, C. and Gerolymos, N. (2016) give correlations between the model parameters and the relative density of the soil. The relative density is for both sands measured in the laboratory tests and provided in the paper by Iai, S. and Kameoka, T. (1993). Model parameters are derived based on these measured relative densities (indicated in orange) and are presented on the right in Table 5.3.

Correlations are available that connect the relative density to the corrected SPT blow-count (equation 6.10 in section 6.2.2). There is however much uncertainty in this relationship showed by the wide range of proposed \(C_d\) by several researchers. Both methods are elaborated using the measured property of the soil to exclude the uncertain factor from the correlation between the corrected SPT blow-count and relative density.

<table>
<thead>
<tr>
<th></th>
<th>Byrne &amp; Beaty (2011) and Makra (2013)</th>
<th>Souliotis &amp; Gerolymos (2016)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(N1)_{60}</td>
<td>Ohama Sand Gaiko Sand</td>
<td>Ohama Sand Gaiko Sand</td>
</tr>
<tr>
<td>[blows]</td>
<td>9 10</td>
<td>9 10</td>
</tr>
<tr>
<td>(D_r) [%]</td>
<td>47 50</td>
<td>42 55</td>
</tr>
<tr>
<td>(\varphi_{cv}) [(^\circ)]</td>
<td>30.0 33.0</td>
<td>30.0 33.0</td>
</tr>
<tr>
<td>(\varphi_p) [(^\circ)]</td>
<td>30.9 34.0</td>
<td>33.6 38.8</td>
</tr>
<tr>
<td>(c) kN/m(^2)</td>
<td>0.0 1.0</td>
<td>0.0 1.0</td>
</tr>
<tr>
<td>(k_G) [-]</td>
<td>902.1 934.3</td>
<td>891.2 1069</td>
</tr>
<tr>
<td>(k_B) [-]</td>
<td>319.2 380.3</td>
<td>1301 1593</td>
</tr>
<tr>
<td>(k_T) [-]</td>
<td>631.5 654.0</td>
<td>623.9 748.5</td>
</tr>
<tr>
<td>(R_f) [-]</td>
<td>0.7911 0.7787</td>
<td>0.7310 0.6380</td>
</tr>
<tr>
<td>(p_A) kN/m(^2)</td>
<td>100 100</td>
<td>100 100</td>
</tr>
<tr>
<td>(\sigma_t) kN/m(^2)</td>
<td>0.00 0.00</td>
<td>0.00 0.00</td>
</tr>
<tr>
<td>(f_{ach_{hard}}) [(-)]</td>
<td>0.45 0.45</td>
<td>0.12 0.19</td>
</tr>
<tr>
<td>(f_{ach_{post}}) [(-)]</td>
<td>0.02 0.02</td>
<td>0.02 0.02</td>
</tr>
</tbody>
</table>

Concerning the model parameters obtained by both calibration methods it is observed that there is especially a large difference in the values for the plastic shear modulus \((K_p^p)\) and the hardening factor \((f_{ach_{hard}})\). Minor differences are observed for the other model parameters.
5.6. Undrained cyclic direct simple shear test

In this section results of cyclic direct simple shear (CDSS) tests modelled in the soil test facility of PLAXIS are analysed.

Loading of the soil samples is expressed in a certain cyclic stress ratio (CSR), the CSR is the ratio of cyclic shear stress amplitude ($\tau_{cyc}$) over the effective vertical consolidation stress ($\sigma'_{vc}$) given by equation 5.23.

$$CSR = \frac{\tau_{cyc}}{\sigma'_{vc}}$$  \hspace{1cm} (5.23)

The available laboratory test results of the soils at Akita Port are all based on triaxial tests. These were all are conducted at a consolidation stress level of 98 kPa at isotropic conditions. A normally consolidated soil sample in a one dimensional direct simple shear test will have a $K_0$ value around 0.5, which has large influence on the liquefaction resistance. The obtained resistance from laboratory tests have to be translated to conditions matching with DSS tests. In Figure 5.4 a schematisation of an undrained cyclic direct simple shear test is presented. Initially vertical and horizontal stress levels are applied on the soil sample, subsequently the sample is loaded by the cyclic shear stresses.

Ishihara, K. et al. (1977) performed cyclic torsional shear tests with varying $K_0$ values and investigated the influence on the cyclic resistance ratio for anisotropically consolidated specimens ($K_0 \neq 1$). It was concluded that CRR for these samples could be related to isotropically consolidated specimens ($K_0 = 1$). In the same way the CRR values for cyclic direct simple shear tests could be related to CRR values from cyclic triaxial tests using the following correlation:

$$CRR_{DSS} = \left(\frac{1 + 2(K_0)_{DSS}}{3}\right)CRR_{TX}$$  \hspace{1cm} (5.24)

The found liquefaction resistance curves for both Ohama and Gaiko sands are corrected according to this relationship to be used to fit the cyclic direct simple shear tests, as was shown in the next section.

Figure 5.4: Schematisation of an undrained cyclic direct simple shear test.
5.6.1. Results initial model parameters

The obtained initial model parameters according to both calibration methods were used as input for the model. In Figure 5.5 and Figure 5.6 the results of the liquefaction resistance curves are presented for respectively Ohama sand and Gaiko sand.

In the procedure fitting the laboratory tests and the results from soil tests in PLAXIS, the amount of cycles to liquefaction was determined for increasing CSR values. The conditions do not change during the tests, therefore CRR may assumed to be constant. In fact the amount of cycles to liquefaction is determined for varying loading levels given a constant CRR.

![Figure 5.5: Liquefaction resistance curves by UBC3D-PLM model for Ohama sand based on initial model parameters for cyclic DSS test.](image1)

![Figure 5.6: Liquefaction resistance curves by UBC3D-PLM model for Gaiko sand based on initial model parameters for cyclic DSS test.](image2)

For both initial parameters sets overestimation of the liquefaction resistance for Ohama Sand is obtained. The fit according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) is better than for the parameter set obtained by the method of Souliotis, C. and Gerolymos, N. (2016).

For the Gaiko Sand the difference between both calibration methods is larger. The parameterset by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) leads to an underestimation of the liquefaction resistance, while the parameter set by Souliotis, C. and Gerolymos, N. (2016) overestimates the liquefaction resistance.
5.6.2. Calibration of model

In order to calibrate the liquefaction resistance curves to the experimental results, model parameters are adjusted that have influence on the onset of liquefaction. The physical meaning of these relevant model parameters is investigated before the calibration procedure. In Appendix C the formulation of the UBC3D-PLM model is elaborated and especially the way the development of excess pore pressures is described in the model.

Winde, H.P. (2015) did research to the influence of all model parameters of the UBC3D-PLM model on the moment of onset of liquefaction. His initial model parameters were based on the method by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) and the modelled soil test was a cyclic DSS test with a vertical effective stress level of 100 kPa and a $K_0$ value of 0.5, which corresponds to the conditions in this research. In Table 5.4 the influence on the moment of onset of liquefaction for variation of model parameters is indicated. It is evident that changing the corrected blow-count has the most influence, since all model parameters are based on this input value. Changing this input value it not an option in this case because these values are measured. Besides the focus is on specifying the parameter set for typical conditions with the same measured SPT-value.

<table>
<thead>
<tr>
<th>$\psi_{cv}$</th>
<th>$\psi_p$</th>
<th>$\psi_{cv} - \psi_p$</th>
<th>$k^P_{G}$</th>
<th>$k^P_{G}$</th>
<th>$k^P_{H}$</th>
<th>me</th>
<th>ne</th>
<th>np</th>
<th>$R_f$</th>
<th>$f a c_{hard}$</th>
<th>(N1)$_{90}$</th>
<th>$f a c_{post}$</th>
<th>(N1)$_{90, tot}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>+</td>
<td>-</td>
<td>-</td>
<td>+</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>+</td>
<td>+</td>
<td>None</td>
<td>None</td>
<td>None</td>
<td>++</td>
</tr>
<tr>
<td>Dense</td>
<td>-</td>
<td>+</td>
<td>-</td>
<td>-</td>
<td>+</td>
<td>-</td>
<td>-</td>
<td>None</td>
<td>+</td>
<td>None</td>
<td>None</td>
<td>None</td>
<td>++</td>
</tr>
</tbody>
</table>
++ Highly influential, + Influential, - Hardly influential

Adjusting $K^P_{G}$, $R_f$ or $f a c_{hard}$ has the most influence on the development of excess pore pressures for loose sands (Winde, H.P., 2015). These parameters are the most important input parameters in the described relationships defining the plastic strain increment, confirming this conclusion. According to Winde, H.P. (2015) variation of the constant friction angle $\psi_{cv}$ and $\psi_{peak}$ for loose sands also influences the moment of onset of liquefaction. This can be explained by the fact that for loose sands the mobilised friction angle quickly reaches the yield surface, while for dense sands the yield surface is reached for much higher loading events.

In Appendix C the procedure how in the model excess pore pressures are generated is elaborated. By analysing these relationships model parameters that have influence on the development of excess pore pressures in the soil are identified. This analysis confirms the conclusion by Winde, H.P. (2015) for the parameters that have influence on the onset of liquefaction. The amount of plastic strains generated in the model has a direct link with the development of excess pore pressures. Therefore the development of the plastic shear modulus has significant influence on the development of excess pore pressures. Besides the stiffness of the soil also the tendency to contractive or dilative behaviour of the soil has influence on the development of excess pore pressures. That is why variation of the constant friction angle $\psi_{cv}$ and $\psi_{peak}$ has influence on the onset of liquefaction.

Concerning the initial parameter sets in Figure 5.3, the largest differences between both sets are in the values of the plastic shear modulus ($k^P_{G}$) and hardening factor ($f a c_{hard}$). These parameters do influence on the plastic shear strain increment and thus also on the development of excess pore pressures. The difference in peak friction angle ($\psi_{peak}$) between both parameter sets is also considerable, which has influence of the amount of contractive or dilative behaviour. This has also influence on the development of excess pore pressures, but in another way than the plastic shear stiffness. In order not to mix different effects of the soil behaviour only the plastic shear modulus ($k^P_{G}$) and hardening factor ($f a c_{hard}$) are initially adjusted to calibrate the liquefaction resistance curves. Minor differences are found for the other model parameters, which are therefore initially kept constant.

In Table 5.5 the adjusted model parameter sets are presented. For the Ohama Sand only the hardening factors ($f a c_{hard}$) are lowered for both calibration methods, since the initial parameter sets lead to an overestimation of the liquefaction resistance.
The model parameters for the Gaiko Sand according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) had to be adjusted to a greater extent. Since the model initially underestimated the resistance against liquefaction, the hardening factor was increased to limit the increment of plastic strains. This appeared not to be enough to fit the liquefaction resistance curve by the model with results from laboratory. Therefore also the plastic shear strain modulus itself was increased to increase the stiffness of the soil completely, leading to a reasonable fit for the Gaiko sand. The initial calibration method according to Souliotis, C. and Gerolymos, N. (2016) led to an overestimation of the liquefaction resistance. For that reason the hardening factor is lowered to fit the laboratory results.

The calibrated liquefaction curve for Ohama Sand is shown in Figure 5.7 and for Gaiko Sand in Figure 5.8. The fit of the curves to experimental results was done for the medium occurring stress ratio. For lower CSR values, the liquefaction resistance for both soils is underestimated. Because of the higher model stiffness in the first few cycles cycles less plastic strains are generated causing less pore pressure generation. To compensate for this effect the hardening factor is lowered. For a lower CSR value, more cycles to liquefaction are required and the effect of the initial stiffer cycles is relatively smaller. This effect was already observed by Makra, A. (2013) and explained in section 5.4.

Differences in the fit for lower values of CSR are less relevant since these values are not directly in the range of interest. More important is a proper fit for the average and higher occurring CSR values. Besides the model gives a conservative prediction not leading to unsafe model results.

For the Ohama sand both calibrated parameter sets give similar results and show a similar trend. Although the fit of the parameters set by Souliotis, C. and Gerolymos, N. (2016) is somewhat better for higher CSR values, differences are minor. Concerning the results of the Gaiko sand the parameter set by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) provides a better fit than the set by Souliotis, C. and Gerolymos, N. (2016), especially for lower CSR values.

Comparing the liquefaction resistance curves only gives an indication of the moment of the onset of liquefaction. The stress strain behaviour cannot be analysed from these curves. No experimental results are available about this soil behaviour to liquefaction, so model results cannot be validated. However to analyse the soil behaviour predicted by the model the development of the excess pore pressures, the stress path and the shear strains for both sands is presented.

### Table 5.5: Calibrated model parameters for Ohama sand and Gaiko sand for an undrained cyclic DSS test.

<table>
<thead>
<tr>
<th></th>
<th>Byrne &amp; Beaty (2011) and Makra (2013)</th>
<th>Souliotis &amp; Gerolymos (2016)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ohama Sand Gaiko Sand</td>
<td>Ohama Sand Gaiko Sand</td>
</tr>
<tr>
<td>(N1)_{60}</td>
<td>9 10</td>
<td>9 10</td>
</tr>
<tr>
<td>D_r [%]</td>
<td>47 50</td>
<td>42 55</td>
</tr>
<tr>
<td>\varphi_{cv} [°]</td>
<td>30 0</td>
<td>30 0</td>
</tr>
<tr>
<td>\varphi'_{p} [°]</td>
<td>30.9 34.0</td>
<td>33.6 38.8</td>
</tr>
<tr>
<td>c [kN/m²]</td>
<td>0.0 1.0</td>
<td>0.0 1.0</td>
</tr>
<tr>
<td>k_e [kN/m²]</td>
<td>902.1 934.3</td>
<td>891.2 1069</td>
</tr>
<tr>
<td>k_P [kN/m²]</td>
<td>319.2 1141</td>
<td>1301 1593</td>
</tr>
<tr>
<td>k_B [kN/m²]</td>
<td>631.5 654.0</td>
<td>623.9 748.5</td>
</tr>
<tr>
<td>me [-]</td>
<td>0.50 0.50</td>
<td>0.50 0.50</td>
</tr>
<tr>
<td>ne [-]</td>
<td>0.50 0.50</td>
<td>0.50 0.50</td>
</tr>
<tr>
<td>np [-]</td>
<td>0.40 0.40</td>
<td>0.42 0.55</td>
</tr>
<tr>
<td>R_f [-]</td>
<td>0.791 0.7787</td>
<td>0.731 0.6380</td>
</tr>
<tr>
<td>p_A [kN/m²]</td>
<td>100 100</td>
<td>100 100</td>
</tr>
<tr>
<td>\sigma_t [kN/m²]</td>
<td>0.00 0.00</td>
<td>0.00 0.00</td>
</tr>
<tr>
<td>f_ac [hard</td>
<td>-]</td>
<td>0.30 0.30</td>
</tr>
<tr>
<td>f_ac [post] [-]</td>
<td>0.02 0.02</td>
<td>0.02 0.02</td>
</tr>
</tbody>
</table>
5.6. Undrained cyclic direct simple shear test

The development of the excess pore pressure ratio according to both calibration methods is for the first cycles similar. The decrease of effective vertical stress in the first three cycles is in accordance to the development \( r_u \). After the initial cycles one softer cycle is present after which the desification rule is activated and the stiffness increases again.

The development of pore pressure according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) is slower than for the set obtained by the method of Souliotis, C. and Gerolymos, N. (2016), for both type of soils. From this moment the densification rule is activated. The different trend can be explained by the difference in plastic shear modulus \( (k_p^P) \) and hardening factor \( (f_{achard}) \). The amount of hardening for these loading level is for the set by Souliotis, C. and Gerolymos, N. (2016) is smaller because of the lower \( f_{achard} \) value, leading to less densification and thus more plastic straining and corresponding development of pore pressures.

Once an excess pore pressure ratio \((r_u)\) of 1.0 is reached the amount of shear stains increase significantly because of the loss of strength due to cyclic mobility. This temporary state of the soil occurs only when an isotropic stress state is present. During this state the specimen has the tendency to dilate when loaded, leading to an increase of the vertical effective stress (see both Figure 5.10 and Figure 5.13). While contractive behaviour is observed when unloaded, leading to a decrease of effective vertical stress. The change in effective vertical stress also leads to a variation in shear stiffness (Makra, A., 2013).

---

**Figure 5.7:** Liquefaction resistance curves by UBC3D-PLM model of Ohama sand based on calibrated model parameters for cyclic DSS test.

**Figure 5.8:** Liquefaction resistance curves by UBC3D-PLM model of Gaiko sand based on calibrated model parameters for cyclic DSS test.
In Figure 5.11 the stress strain loop for Ohama sand is presented. The model doesn't allow for further softening once an $r_u$ value of 1.0 is reached. Stiffness remains the same resulting in the repeated loop in Figure 5.11. Post-liquefaction behaviour of the soil is not correctly reflected in the model, because of the locked shear stiffness after reaching liquefaction.

The amount of shear strains in the liquefied state depends on the post liquefaction factor ($f_{ac_{post}}$) and the initial plastic shear modulus ($k_p^G$). The $f_{ac_{post}}$ accounts for the softening of the soil when the peak yield surface is reached. The magnitude of the plastic shear modulus decreases for every cycle until the minimum value is reached defined by $f_{ac_{post}}$. Since the post liquefaction factor is equal for both parameter sets, the difference in shear strains is a direct result of the difference in plastic shear modulus.

Results Ohama Sand

Figure 5.9: Development of excess pore pressure ratio Ohama sand for cyclic undrained DSS test in PLAXIS with CSR = 0.10.

Figure 5.10: Development of stress path Ohama sand for cyclic undrained DSS test in PLAXIS with CSR = 0.10.

Figure 5.11: Development of shear strains Ohama sand for cyclic undrained DSS test in PLAXIS with CSR = 0.10.
5.6. Undrained cyclic direct simple shear test

Results Gaiko sand

Figure 5.12: Development of excess pore pressure ratio Gaiko sand for cyclic undrained DSS test in PLAXIS with CSR = 0.18.

Figure 5.13: Development of stress path Gaiko sand for cyclic undrained DSS test in PLAXIS with CSR = 0.18.

Figure 5.14: Development of shear strains Gaiko sand for cyclic undrained DSS test in PLAXIS with CSR = 0.18.

Evaluation parameter sets

Finally for both types of sand a choice is made for the set of model parameters to be used in the model, based on the fit of the calibrated liquefaction curves and development of the soil behaviour. The choice is also based on characteristics of the the parameterset itself.

The fit of model results with experimental results for Ohama sand according to both methods is comparable. However the parameterset by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) seems a more balanced parameterset than the one obtained by applying the method by Souliotis, C. and Gerolymos, N. (2016). The $k_{GP}$ which follows from calibration method by Souliotis, C. and Gerolymos, N. (2016) is high and has to be compensated with a low value of $f_{ac hard}$, while for the set by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) this is not the case. For that reason the calibrated parameter set according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) is adopted for the Ohama sand. The fit of the parameterset by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) is considerably better and therefore adopted.
5.6.3. Effects of state parameters
Once the model parameters are calibrated effects of varying overburden stress, lateral earth pressure coefficient and initial static shear ratio on the results of cyclic DSS tests are investigated. The resistance against liquefaction depends on these state parameters and it is analysed to what extent the model accurately predicts the resistance against liquefaction for variations in these state parameters.

Effect of initial vertical stress ($\sigma_0'$)
The dependence of the cyclic resistance ratio on the overburden pressure is in empirical liquefaction potential evaluations represented by the $K_\sigma$-factor, which was originally introduced by Seed, H.B. (1983). Generally it is observed that the cyclic resistance ratio decreases for increasing initial vertical effective stresses. Besides the initial overburden stress, other factors like the type of test device and the age of the laboratory specimens have influence on the $K_\sigma$-factor. These factors are in this research not considered. The stress level has influence on the $K_\sigma$-factor, which is defined as follows:

$$K_\sigma = \frac{CRR_{\sigma'_0}}{CRR_{\sigma'_0}=1}$$  \hspace{1cm} (5.25)

The $CRR_{\sigma'_0}$ is the cyclic resistance of a soil under a specific effective consolidation stress and the $CRR_{\sigma'_0}=1$ is the resistance of that same type of soil at a consolidation stress of 100 kPa.

Idriss, I.M. and Boulanger, R.W. (2003) derived correlations for $K_\sigma$ based on the relative density or normalized penetration resistances, as were described in section 2.6.1. Many other theories are available that give correlations for $K_\sigma$, all derived for different types of sand or circumstances. In this research however only the commonly used theory by Idriss, I.M. and Boulanger, R.W. (2003) is adopted.

In Figure 5.15 $K_\sigma$-values for different levels of initial vertical effective pressures are presented. In black the development of $K_\sigma$ according to Idriss, I.M. and Boulanger, R.W. (2003) is shown. The other lines are the $K_\sigma$-values for both Ohama and Gaiko sand obtained from the UBC3D-PLM model.

![Figure 5.15: Development of $K_\sigma$ for Ohama Sand and Gaiko Sand according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for an undrained cyclic DSS test (for $P_{at} = 98$ kPa).](image)

For an overburden pressure lower than the initial atmospheric pressure the resistance of the Ohama sand is underestimated compared to the theory. On the other hand when the overburden pressure is higher than the atmospheric pressure the soil resistance is slightly overestimated. This behaviour could be corrected by adjusting the hardening factor ($f_{achard}$) according to the magnitude of the overburden pressure. In Table 5.6 the suggested corrected hardening factors are presented, leading to fit of the $K_\sigma$ factor with the theory by Idriss, I.M. and Boulanger, R.W. (2003). Since the differences are quite small it is questionable, given other uncertainties, if this slight correction contributes to the overall accuracy of the model.

Deviation of $K_\sigma$ in the UBC3D-PLM model with respect to Idriss, I.M. and Boulanger, R.W. (2003) for Gaiko sand are much smaller and correction is not necessary.
It can be concluded effects of that varying initial vertical effective stresses on the liquefaction potential are relatively well captured by the model. Variation of the vertical effective stress compared to the initial situation is in this case is not so large. For bigger changes of the effective vertical stress the fit of $K_\sigma$ is worse and adjustments of the $f_{ac\text{hard}}$ factor are needed (Makra, A., 2013). These is however of less interest in this case study.

### Table 5.6: Corrected $f_{ac\text{hard}}$ values in UBC3D-PLM model for Ohama Sand to match the theoretical $K_\sigma$ Idriss, I.M. and Boulanger, R.W. (2003) for an undrained cyclic DSS test.

<table>
<thead>
<tr>
<th>Overburden stress $\sigma'_{v0}$ [kPa]</th>
<th>Idriss &amp; Boulanger (2008) $K_\sigma$ [-]</th>
<th>UBC3D-PLM $K_\sigma$ [-]</th>
<th>Corrected $f_{ac\text{hard}}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>1.06</td>
<td>1.03</td>
<td>0.34</td>
</tr>
<tr>
<td>98</td>
<td>1.00</td>
<td>1.00</td>
<td>/</td>
</tr>
<tr>
<td>150</td>
<td>0.96</td>
<td>0.98</td>
<td>0.28</td>
</tr>
</tbody>
</table>

#### Effect of lateral earth pressure coefficient ($K_0$)

The cyclic resistance ratio is strongly influenced by the magnitude of the initial lateral earth pressure coefficient. The dependence of the CRR on the lateral earth pressure coefficient is expressed by the following equation, which is similar to equation 5.24 (Ishihara, K. et al., 1977):

$$\text{CRR}_{K_0 \neq 1} = \left( \frac{1 + 2K_0}{3} \right) \cdot \text{CRR}_{K_0 = 1}$$

As can be seen from this equation, CRR increases for increasing values of $K_0$.

Initially all undrained cyclic DSS tests are conducted with a $K_0$ of 0.5 and calibrated to the laboratory tests. Subsequently the initial $K_0$ is varied in the test to both higher and lower values. In Figures 5.16 and 5.17 the corresponding liquefaction resistance curves are presented for respectively the Ohama sand and Gaiko sand.

The resistance of the Ohama sand for varying $K_0$ values is not in line with earlier described expected behaviour. Even the opposite is observed for Ohama Sand, the liquefaction resistance decreases for increasing $K_0$-values and the other way around the resistance increases for decreasing $K_0$. When the lateral earth pressure coefficient becomes larger than 1.0 the resistance increases significantly with respect to the initial resistance.

For the Gaiko type of sand the resistance increases for increasing $K_0$-values, which is in accordance to theory. Again a value of $K_0$ of 2.0 leads to a significant increase of the liquefaction resistance.

Variation of $K_0$ can be divided into two separate effects on the soil state:

- **Effect on static shear ratio $\alpha$**
  
  An initial $K_0$ value of 1.0 corresponds to isotropic loading conditions. All values $K_0$ other than 1.0 lead to anisotropic loading conditions introducing initial shear stresses ($\alpha \neq 0$) in the specimen. These shear stresses influence the liquefaction resistance of the soil, but also the model performance as was stated in section 5.4.

- **Effect on average effective stress $p'$**
  
  The average effective stress $p'$ is influenced by the initial $K_0$ value, assuming the vertical pressure remains constant. As was shown in previous section this also influences the liquefaction resistance. The model is however able to reasonably reflect the effect of the overburden pressure.

Within the results for varying $K_0$-values some observations are investigated in further detail, since they could not directly be explained by theory. First the model results are analysed for a $K_0$ value of 2.0, since the cyclic resistance ratio increases significantly. Secondly the difference in model behaviour between Ohama sand and Gaiko for $K_0$ of 1.0 is investigated.
In order to investigate soil behaviour in the model initially development of the principal stresses is analysed for $K_0$ values of 0.5, 1.0 and 2.0 (see Figure 5.20 to Figure 5.22).

A $K_0$ value unequal to 1.0 leads to an initial anisotropic initial loading condition. For $K_0$ values lower than 1.0 $\sigma_1$ coincides with $\sigma_{yy}$ and $\sigma_3$ with $\sigma_{xx}$. When $K_0$ is larger than 1.0 the major principal stress is in x direction. The intermediate principal stress ($\sigma_2$) corresponds in both situations with the out of plane loading direction $z$.

As loading continues it is observed that the model tends to an isotropic stress state, where all principal stresses are equal. From this state the effective stresses in all directions decrease with the same rate to a liquefied state. If the initial condition is already isotropic the soil remains in this state and effective stresses decrease until liquefaction is reached.

Converging of the model to an isotropic stress state seems logical since in case of a $r_u$ value of 100 % the soil behaves like a thick fluid which by definition has an isotropic stress state. It is not exactly clear why the model reaches this state so early. Measuring the development of stresses during an experimental undrained cyclic direct simple shear test is however difficult, so exact development of stresses is hard to determine. It is although characteristic for most elastic-plastic constitutive material models (like UBC3D-PLM) to move towards an isotropic stress state during undrained cyclic simple shear loading when no sustained initial static shear ratio is present. This is a reasonable approximation relative to observed experimental behaviour.
5.6. Undrained cyclic direct simple shear test

To reach this isotropic state in the model the minor principal effective stress ($\sigma_3'$) in all cases initially increases, while the $\sigma_1'$ continuously decreases. When $K_0$ is lower than 1.0 this implies that the horizontal effective stress initially decreases and the vertical effective stress continuously increases. On the other hand when $K_0$ is larger than 1.0 the horizontal effective stress continuously decreases, while the vertical effective stress initially increases. It can be concluded that depending on the $K_0$ value three different types of behaviour are observed in the model.

In Figures 5.23 to 5.25 development of the average effective stress ($p'$), excess pore pressures ($p_{uw}$ and total stress ($p_{tot}$) are presented for respectively $K_0$ values of 0.5, 1.0 and 2.0.

As loading starts the average effective stress level decreases for values of $K_0$, despite the fact that principal stress components increase. Even when two principal stresses increase still the average effective stress monotonically decreases. The $p'$ for $K_0 = 1.0$ decreases much faster than for other values of $K_0$, corresponding to observations in the trend of the principal stresses.

For $K_0$ values higher than 1.0 initially positive excess pore pressures are generated (suction) in the model, while for values lower than 1.0, only negative excess pore pressures are generated. In Annex C it is described in detail how the generation of excess pore pressures is defined in the model. During undrained loading in theory there is no change in volume. However in the model excess pore pressures are obtained by generating small strains which are multiplied by the combined bulk modulus of the soil and water.

In a DSS test strain in horizontal direction is restricted given the boundary conditions of the test. Variation of effective stresses in horizontal therefore won't have influence on the development of excess pore pressures since strains cannot be generated in the model. In vertical direction strains are allowed, only a constant stress level is imposed. Variations in effective stress level in this direction will have influence on the excess pore pressure, since the total stress level must be in accordance with the constant stress boundary condition in vertical direction. For a $K_0$ of 2.0 suction is observed in the model, corresponding to an increase of $\sigma_3$ in order to comply to the boundary condition.

The average total stress level converges to the initial vertical effective stress level, independent from the initial $K_0$ value. In vertical direction the total stress level does not change because of the boundary condition in vertical direction. Although in horizontal direction the effective stresses develop differently than the development of excess pore pressure, leading to a variation in total stress level. The model converges to an isotropic stress state, which involves that the total stress level imposed in vertical direction must be met in every direction.

Until now only the development of the soil behaviour in the model is explained. This model behaviour has also influence on the liquefaction resistance of the considered soil. Anisotropic loading in an undrained cyclic DSS test generally leads to more cycles to liquefaction. Two phases in soil behaviour could be distinguished for initially anisotropic loading conditions. First development of the stress state to an isotropic stress condition. Secondly from this isotropic condition to liquefaction.
For initial isotropic conditions this first phase is not present and apparently the development of excess pore pressures is much faster leading to a much shorter trajectory to liquefaction. In case of anisotropic loading several cycles are required to reach the isotropic state, the decrease of effective average stress is in the first phase much smaller. After reaching the isotropic stress state the rate of decrease becomes larger. This first phase due to anisotropic loading thus leads to a higher predicted resistance against liquefaction. Even if the initial average effective stress level is smaller.

The difference in soil behaviour between $K_0$ of 0.5 and 2.0 is attributed to the fact that the difference between the two principal stresses is larger, leading to more cycles required to reach the isotropic stress state. In Figure 5.19 the development of $K_0$ values with varying initial $K_0$ values to the isotropic state is presented. It can be seen that for both incrementally decreasing and increasing initial $K_0$ values, the amount of cycles to the isotropic stress state increases.

![Figure 5.19: Development of $K_0$ for varying initial $K_0$ values](image)

The difference in behaviour between the Ohama sand and Gaiko sand for $K_0$ values of 1.0, which is observed in the liquefaction resistance curves in Figure 5.16 and Figure 5.17 is attributed to the ratio of $k_G^P$ and $f_{a_{c_{hard}}}$ and the amount of cycles to liquefaction. As was described earlier for $K_0 \neq 1$ an anisotropic phase and an isotropic phase could be distinguished, where for a $K_0$ value of 1.0 the model shows a continuous isotropic behaviour.

The rate of decrease of effective stresses in the isotropic state depends on $k_G^P$, so the lower $k_G^P$ the faster liquefaction is reached assuming other model parameters don't vary. The $k_G^P$ of the Ohama sand is much lower compared to the value of the Gaiko sand and the $f_{a_{c_{hard}}}$ are equal, leading to larger increment of excess pore pressures.

Difference between the behaviour could be explained by the fact that for the Ohama sand the phase from anisotropic conditions to isotropic has relatively more influence on the resistance against liquefaction than for the Gaiko sand. The gradient of the decreasing effective stresses in the isotropic is smaller for the Gaiko sand compared to the Ohama sand, which is attributed to the difference in $k_G^P$. Because of the smaller gradient in the second phase, the amount of cycles required in the first phase to reach the isotropic state has relatively less influence on the total amount of cycles to liquefaction. Ultimately the second phase becomes dominant compared to the first phase leading to a higher resistance against liquefaction for a $K_0$ value of 1.0.
5.6. Undrained cyclic direct simple shear test

Figure 5.20: Development of principal effective stresses in an undrained cyclic DSS test with $K_0 = 0.5$ for CSR = 0.10

Figure 5.21: Development of principal effective stresses in an undrained cyclic DSS test with $K_0 = 1.0$ for CSR = 0.10

Figure 5.22: Development of principal effective stresses in an undrained cyclic DSS test with $K_0 = 2.0$ for CSR = 0.10
Figure 5.23: Development of average effective stresses, excess pore pressures and total stresses in an undrained cyclic DSS test with $K_0 = 0.5$

Figure 5.24: Development of average effective stresses, excess pore pressures and total stresses in an undrained cyclic DSS test with $K_0 = 1.0$

Figure 5.25: Development of average effective stresses, excess pore pressures and total stresses in an undrained cyclic DSS test with $K_0 = 2.0$
Effect of initial static shear stress ratio ($\alpha$)

The cyclic resistance ratio is also affected by the presence of static shear. Static shear stresses are present in sloping ground, but can also be present around an anchored quay wall when the structures has deformed as a result of static loading. Seed, H.B. (1983) introduced the factor $K_\alpha$ to correct the cyclic resistance ratio for the effects of initial static shear stresses. This factor is defined as:

$$K_\alpha = \frac{CRR_\alpha}{CRR_{\alpha=0}}$$  \hspace{1cm} (5.27)

In this equation the $\alpha$-factor is the ratio of the static shear stresses over the effective consolidation stress ($\alpha = \tau_{stat}/\sigma'_{vo}$) on the plane of interest, where $CRR_\alpha$ is the resistance of the soil for a specific $\alpha$ and $CRR_{\alpha=0}$ is the resistance when no static shear stresses are present. In section 2.6.1 expressions by Idriss, I.M. and Boulanger, R.W. (2003) are elaborated for determining the $K_\alpha$ value. Depending on the state of the soil the liquefaction resistance either increase or decrease for increasing static shear ratio (Idriss, I.M. and Boulanger, R.W., 2008).

Several other relationships are available for $K_\alpha$, based on other soil types using different element tests. The validity of these expressions for general use is questionable since insufficient empirical field data or laboratory test results are available. It is however important to include the effects of static shear into the liquefaction analysis, since it has significant influence on the cyclic resistance. In this research the relationships proposed by Idriss, I.M. and Boulanger, R.W. (2003) is adopted, since it is commonly used.

In Figure 5.26 the development of the $K_\alpha$ values are presented according to Idriss, I.M. and Boulanger, R.W. (2003) and the UBC3D-PLM model for increasing static shear ratios. The liquefaction resistance according to the UBC3D-PLM model is increasingly underestimated as the static shear ratio ($\alpha$) becomes higher for an undrained cyclic DSS test.

![Figure 5.26: Development of $K_\alpha$ for Ohama Sand and Gaiko Sand according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for DSS test.](image)

Since the presence of initial static shear has major influence on the onset of liquefaction in the model, parameters were calibrated to compensate the underestimation of the soil resistance for initial static shear ratios ($\alpha$) up to 0.20. Again the densification factor and the plastic shear modulus were adjusted to match the $K_\alpha$ value from theory by Idriss, I.M. and Boulanger, R.W. (2003). Since the liquefaction resistance was underestimated for higher initial static shear ratios, the parameters were both increased for increasing $\alpha$ values as can be seen in Table 5.7.
Table 5.7: Corrected $f_{ac,hard}$ and $k_G^P$ values in UBC3D-PLM model to match the theoretical $K_g$, Idriss, I.M. and Boulanger, R.W. (2003) for an undrained cyclic DSS test.

<table>
<thead>
<tr>
<th>Static shear ratio</th>
<th>Ohama Sand</th>
<th>Gaiko Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_a$ [-]</td>
<td>$f_{ac,hard}$ [-]</td>
<td>$K_g$ [-]</td>
</tr>
<tr>
<td>0.05</td>
<td>0.951</td>
<td>0.818</td>
</tr>
<tr>
<td>0.10</td>
<td>0.931</td>
<td>0.453</td>
</tr>
<tr>
<td>0.15</td>
<td>0.925</td>
<td>0.301</td>
</tr>
<tr>
<td>0.20</td>
<td>0.918</td>
<td>0.235</td>
</tr>
</tbody>
</table>

In accordance with observations by Makra, A. (2013), for low $f_{ac,post}$ factors numerical issues occur for the higher initial static shear ratios leading to irregular development excess pore pressures and stress path. Increasing the post liquefaction factor to 1.0 solves this problem and for that reason this factor is adopted for all parameter sets for zones where initial static shear stresses are present. Besides solving the numerical issue adopting a $f_{ac,post}$ value of 1.0 is also considered to represent the soil behaviour well in case of flow failure. In case of loose soils it is expected that the ultimate shear strength is met because of the increase of excess pore pressures already before reaching the peak surface, causing flow failure without further degradation of stiffness (Makra, A., 2013).
5.7. Undrained cyclic triaxial test

Around an anchored quay wall several loading conditions are present, matching different element tests. A triaxial test typically matches stress states where the direction of the principal stresses correspond with the x and y direction where stresses in x direction are decreased or increased with respect to the initial situation. In this section the performance of the UBC3D-PLM model is investigated for stress paths corresponding to undrained triaxial compression and triaxial extension tests.

In PLAXIS no cyclic triaxial test is implemented and therefore the test is simulated in the general soil test environment in the Soil Test Facility in Plaxis.

Goal of simulating the undrained cyclic triaxial tests is to match these tests with the obtained laboratory results of the Ohama and Galko sand and to assess the impact of different stress states. The liquefaction resistance curves from laboratory tests were obtained with cyclic triaxial tests, so the results don’t have to be adapted and could be directly used for calibration of the element test.

The cyclic stress ratio in a triaxial test is defined as follows:

\[ CSR = \frac{q_{\text{cyc}}}{2\sigma_3'} \]  

(5.28)

In this equation \( q_{\text{cyc}}/2 \) is the maximum cyclic shear stress and \( \sigma_3' \) is the isotropic consolidation stress. The laboratory tests were performed with a consolidation stress of 98 kPa and the applied CSR values can be read from test results.

5.7.1. Results initial model parameters

Initially the calibrated model parameters for the DSS tests are also adopted in the simulation of the cyclic triaxial test. The only parameter that was adjusted is the plastic shear modulus \((k_P^G)\). The value for the plastic shear modulus is based on the results of direct simple shear tests (DSS), and thus valid for the stress path of according to DSS test. The stress path in a triaxial test is different and do not involve principle stress rotation, contrary to direct simple shear tests. The derived plastic shear modulus does not necessarily reflect the soil response for triaxial tests and it is required to analyse if the value of \( k_P^G \) had to be adopted (Petalas, A. and Galavi, V., 2013). The following equations were derived by Puebla, H. et al. (1997) to include the effect of rotation of principal stresses in terms on the stiffness parameter:

For \( 0^\circ \leq a_\sigma \leq 45^\circ \), then:

\[ k_P^G = (k_P^G)_0 \cdot (F - (F - 1) \cos(2a_\sigma)) \]  

(5.29)

For \( 45^\circ \leq a_\sigma \leq 90^\circ \), then:

\[ k_P^G = (k_P^G)_0 \cdot F \]  

(5.30)
Where:

- \( a_\sigma \) is the angle between major principle stress direction and vertical axis.
- \((k_G^p)_0 \) is the plastic modulus number corresponding to \( a_\sigma = 0^\circ \) (vertical compression).
- \( F \) is the factor of anisotropic plastic response, which is less than unity (proposed value is 0.317).

These equations were originally developed to adjust the stiffness parameter obtained from experimental results of triaxial tests and use it for direct simple shear tests to account for the principal stress rotation (\( a_\sigma = 45^\circ \)). These relations can however also be used the other way around.

The values of \( k_G^p \) were derived for direct simple shear tests, corresponding to \( a_\sigma = 45^\circ \). The plastic shear modulus corresponding to the triaxial axial test (\( (k_G^p)_0 \)) is obtained as follows:

\[
(k_G^p)_0 = \frac{k_G^p}{F - (F-1)\cos(2a_\sigma)} = \frac{k_G^p}{0.317}
\]

In Table 5.8 the adapted plastic shear strains moduli for both types of sands are presented, together with the other model parameters.

Table 5.8: Initial model parameters for Ohama sand and Gaiko sand for undrained cyclic triaxial test, based on calibration of undrained cyclic DSS tests.

<table>
<thead>
<tr>
<th></th>
<th>Byrne &amp; Beaty (2011) and Makra (2013)</th>
<th>Souliotis &amp; Gerolymos (2016)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ohama Sand</td>
<td>Gaiko Sand</td>
</tr>
<tr>
<td>((N1)_{60}) [blows]</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>(D_r) [%]</td>
<td>47</td>
<td>50</td>
</tr>
<tr>
<td>(\varphi_{CV}) [°]</td>
<td>30.0</td>
<td>33.0</td>
</tr>
<tr>
<td>(\varphi_p) [°]</td>
<td>30.9</td>
<td>34.0</td>
</tr>
<tr>
<td>(c) kN/m²</td>
<td>0.0</td>
<td>1.0</td>
</tr>
<tr>
<td>(k_G^e) [-]</td>
<td>902.1</td>
<td>934.3</td>
</tr>
<tr>
<td>(k_G^p) [-]</td>
<td>1007</td>
<td>1200</td>
</tr>
<tr>
<td>(k_B^e) [-]</td>
<td>631.5</td>
<td>654.0</td>
</tr>
<tr>
<td>(me) [-]</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>(ne) [-]</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>(np) [-]</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td>(R_f) [-]</td>
<td>0.7911</td>
<td>0.7787</td>
</tr>
<tr>
<td>(p_A) kN/m²</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>(\sigma_t) kN/m²</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>(f a c_{hard}) [-]</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>(f a c_{post}) [-]</td>
<td>0.02</td>
<td>0.02</td>
</tr>
</tbody>
</table>

These initial parameter sets based on calibration of cyclic DSS tests lead to the liquefaction curves in Figure 5.28 and Figure 5.29 for respectively Ohama sand and Gaiko sand. In the procedure to find the fit between the laboratory tests and the results from the soil tests in PLAXIS, the amount of cycles to liquefaction was determined for increasing CSR values. The conditions do not change during the tests, therefore CRR may assumed to be constant.

Regarding the Ohama sand, both methods lead to a reasonable approximation of the liquefaction resistance although the resistance is over the full range somewhat overestimated. More significant differences are found in the model results of the liquefaction resistance of the Gaiko sand. The method of Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) lead to an underestimation of the liquefaction resistance, while the method by Souliotis, C. and Gerolymos, N. (2016) gives an overestimation of the resistance. In the next section the model parameters are calibrated in order to fit the laboratory results.
5.7. Undrained cyclic triaxial test

Figure 5.28: Liquefaction resistance curves by UBC3D-PLM model for Ohama sand based on initial model parameters for an undrained cyclic triaxial test.

Figure 5.29: Liquefaction resistance curves by UBC3D-PLM model for Gaiko sand based on initial model parameters for an undrained cyclic triaxial test.

5.7.2. Calibration of model

Since the initial model parameters do not lead to fit with the laboratory results, the model is calibrated. In section 5.6 the model parameters corresponding to cyclic DSS tests were calibrated by adjusting the $k_p^G$ and $f_{ac,hard}$ values. As is elaborated in Annex C these parameters have influence on the development of plastic strains which are linked to the development of excess pore pressures. In calibration of the triaxial tests the same model parameters are calibrated.

In Table 5.9 the calibrated model parameters are presented for both types of sand according to both calibration methods. The fit of the liquefaction resistance curves was done for a medium CSR value. Again the liquefaction resistance is underestimated for the lower CSR values, as was also observed for the cyclic DSS test. As was explained in section 5.6 this has to do with the calibration of the $f_{ac,hard}$ factor on the medium stress level.

Initially both calibration methods overestimated the liquefaction resistance of the Ohama sand. A decrease of the $f_{ac,hard}$ factor in the parameter set by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) leads to a more rapid decrease of the shear stiffness and to more plastic straining and development of excess pore pressures. A reasonable fit was obtained by decreasing the hardening factor. For the parameterset by Souliotis, C. and Gerolymos, N. (2016) both $k_p^G$ and the $f_{ac,hard}$ values were used to fit the liquefaction resistance curve. The resistance had to be lowered, which was initially tried to lower the $f_{ac,hard}$ value. Since this value was in the initial parameterset already very low it could not be lowered much. For that reason the value of the plastic shear modulus for the cyclic DSS test was adopted again and the given this value the $f_{ac,hard}$ was used to fit the curve leading to a reasonable fit.
Table 5.9: Calibrated model parameters of Ohama sand and Gaiko sand for an undrained cyclic triaxial test.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Byrne &amp; Beaty (2011) and Makra (2013)</th>
<th>Souliotis &amp; Gerolymos (2016)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ohama Sand</td>
<td>Gaiko Sand</td>
</tr>
<tr>
<td>(N1)_{60} [blows]</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>D_r [%]</td>
<td>47</td>
<td>50</td>
</tr>
<tr>
<td>( \varphi_{cv} ) [(^\circ)]</td>
<td>30.0</td>
<td>33.0</td>
</tr>
<tr>
<td>( \varphi'_p ) [(^\circ)]</td>
<td>30.9</td>
<td>34.0</td>
</tr>
<tr>
<td>c [kN/m^{2}]</td>
<td>0.0</td>
<td>1.0</td>
</tr>
<tr>
<td>( k_e^p ) [(-)]</td>
<td>902.1</td>
<td>934.3</td>
</tr>
<tr>
<td>( k_p^G ) [(-)]</td>
<td>1007</td>
<td>1521</td>
</tr>
<tr>
<td>( k_B^e ) [(-)]</td>
<td>631.5</td>
<td>654.0</td>
</tr>
<tr>
<td>me [-]</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>ne [-]</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>np [-]</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td>( R_f ) [(-)]</td>
<td>0.7911</td>
<td>0.7787</td>
</tr>
<tr>
<td>( p_A ) [kN/m^{2}]</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>( \sigma_t ) [kN/m^{2}]</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>( f a\text{ch}_{hard} ) [(-)]</td>
<td>0.23</td>
<td>0.60</td>
</tr>
<tr>
<td>( f a\text{ch}_{post} ) [(-)]</td>
<td>0.02</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Figure 5.30: Liquefaction resistance curves by UBC3D-PLM model of Ohama sand based on calibrated model parameters for an undrained cyclic triaxial test.

For the Gaiko sand the parameter set by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) initially lead to an underestimation of the liquefaction resistance. Initially only the \( f a\text{ch}_{hard} \) factor was increased to fit the laboratory tests. It was concluded that adjusting only this parameter did not lead to an acceptable fit. For that reason also the plastic shear modulus was increased to four times the \( k_p^G \) according to the DSS test. Finally the \( f a\text{ch}_{hard} \) factor was calibrated to fit the liquefaction curve. On the other hand the parameter set by Souliotis, C. and Gerolymos, N. (2016) initially overestimated the liquefaction resistance. As was also done for the Ohama type, for the same reason both the \( k_p^p \) and \( f a\text{ch}_{hard} \) were calibrated to fit laboratory results. The \( k_p^p \) found for the cyclic DSS test was adopted and subsequently the \( f a\text{ch}_{hard} \) was used to fit the liquefaction resistance curve.
5.7. Undrained cyclic triaxial test

Analysis soil behaviour

Concerning the Ohama type of sand the rate of the development of excess pore pressures to liquefaction decreases with the amount of cycles as is observed in Figure 5.32. The development according to both calibration methods is similar, leading to the same amount of cycles to liquefaction.

Looking however to the development of the average stress (p’) over the deviatoric stress (q) there is a difference observed between both calibration methods. After the initial three cycles the densification rule is activated and the decrease of average effective stress for each cycle becomes smaller. The different trend is again explained by the difference in $k_p$ and $f_{ac\, hard}$ values.

Once the liquefaction state is reached in the model the amount of strains significantly increase, as was also observed and described in section 5.6 for the cyclic DSS test. The strains continuously increase with increasing amount of cycles once the liquefied state is reached, contrary to the observed strain loop for the undrained cyclic DSS test. This is explained by the difference in failure behaviour between the cyclic DSS test and the cyclic triaxial test. In a cyclic DSS test liquefaction is reached by cyclic mobility, while in a cyclic triaxial test flow failure is observed. The shear strength of the sample is already exceeded before effective stresses become zero. The failure behaviour could therefore be better compared to the failure of a soil sample in a cyclic DSS test with presence of initial static shear.

Again a significant difference in the development of plastic strains after liquefaction is observed between both calibration methods. In the liquefied state shear the post liquefaction factor ($f_{ac\, post}$) adjusts the plastic shear modulus. This factor accounts for the softening of the soil when the peak yield surface is reached. The magnitude of the plastic shear modulus (depending on initial $k_p$ and $f_{ac\, hard}$) itself has also influence since this value is adjusted by the $f_{ac\, post}$ after reaching the peak yield surface. Since the post liquefaction factor is equal for both parameter sets, the difference in shear strains is a direct result of the difference in plastic shear modulus.

In Figure 5.34 and Figure 5.37 developments of the strains against deviatoric stress is presented. Strains remain limited until liquefaction is reached. After reaching liquefaction cyclic behaviour of strains is observed, but moving in positive direction. After 15 cycles a positive strain rate is observed. In physical undrained cyclic triaxial tests large strain amplitudes can be observed once the soil reaches failure. The hammer imposing the cyclic load swings heavily up and down, possibly leading to an increasing strain rate. This could be an explanation for the positive strain observed in the model.
Results Ohama sand

Figure 5.32: Development of excess pore pressure ratio Ohama sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.16.

Figure 5.33: Development of stress path Ohama sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.16.

Figure 5.34: Development of shear strains Ohama sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.16.
5.7. Undrained cyclic triaxial test

Results Gaiko sand

![Graph showing excess pore pressure ratio for Gaiko sand](image1)

Figure 5.35: Development of excess pore pressure ratio Gaiko sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.28.

![Graph showing stress path for Gaiko sand](image2)

Figure 5.36: Development of stress path Gaiko sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.28.

![Graph showing shear strains for Gaiko sand](image3)

Figure 5.37: Development of shear strains Gaiko sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.28.

Evaluation parameter sets

After analysing results for the liquefaction resistance curves and soil behaviour according to both calibration methods, a choice is made for the parameter sets to be adopted.

For the Ohama type of sand both methods lead after calibration to reasonable fit with the laboratory results. It is chosen to adopt the parameter set by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) since there are no significant differences and the other sets adopted in the research were also based on this calibration method.

The calibrated parameter set by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) results in a significantly better fit with the experimental results for Gaiko sand. For that reason this parameter set is adopted.
5.7.3. Effects of state parameters
Once the model parameters for both sands are defined the effects of varying overburden stress, lateral earth pressure coefficient and initial static shear ratio on the liquefaction resistance in an undrained cyclic triaxial test are investigated. The resistance against liquefaction depends on these state parameters and it is analysed to what extent the model accurately predicts the resistance against liquefaction for variations in these state parameters.

**Effect of initial vertical stress \( (\sigma'_{10}) \)**
In section 5.6 the effect of the overburden stress level on the liquefaction potential for a cyclic DSS test was already described. The dependence of the cyclic resistance ratio on the overburden pressure is represented by the \( K_\sigma \)-factor, which was originally introduced by Seed, H.B. (1983). Generally it is observed that the cyclic resistance ratio decreases for increasing initial vertical effective stresses. For the background of this factor reference is made to section 2.6.1 and section 5.6.

Figure 5.38: Development of \( K_\sigma \) for Ohama Sand and Gaiko Sand according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for an undrained cyclic triaxial test.

Differences in \( K_\sigma \) values by the UBC3D-PLM model with respect to the theory by Idriss, I.M. and Boulanger, R.W. (2008) are for both Ohama sand and Gaiko sand small. The liquefaction resistance is somewhat overestimated. Differences are however so small that adjustments in the hardening factor do not lead to a higher accuracy since the amount of cycles to liquefaction are predicted well.

Effects of variations in the initial vertical effective stresses on the liquefaction potential are well captured by the model. Variation of vertical effective is however not so large. For larger differences adjustments of the sachard factor are needed (Makra, A., 2013). This is however of less interest in this case study.
5.7. Undrained cyclic triaxial test

**Effect of lateral earth pressure coefficient ($K_0$)**

The cyclic resistance ratio for conditions other than the isotropic stress state can approximately be determined using the following formula (Ishihara, K. et al., 1977):

$$CRR_{K_0 \neq 1} = \frac{1 + 2K_0}{3} \cdot CRR_{K_0=1}$$  

(5.31)

This equation shows that for $K_0$ values of 1.0 the largest liquefaction resistance was found. Values lower than 1.0 lead to a decrease in resistance. The same behaviour was observed both both type of sands in Figure 5.40 and Figure 5.41.

In a triaxial test anisotropic loading conditions can directly be linked to the presence of static shear stresses. In Figure 5.39 the definition of static shear in a cyclic triaxial test is shown, which is introduced by a difference in magnitude of the principal stresses. Those two phenomena can therefore not be seen independent from each other. A $K_0$ value other than 1.0 can be translated to an $\alpha$ value using the equation presented in Figure 5.39. The model behaviour for varying $\alpha$ values is elaborated in the next section.

According to the presented equation for determining $\alpha$ a $K_0$ value of 0.50 would lead to the same $\alpha$ value as a $K_0$ value of 2.00. However different behaviour is observed in the model, where the resistance against liquefaction for a $K_0$ of 2.00 is smaller than for $K_0$ is 0.50. This can be explained by the difference in $p'$, since the liquefaction resistance decreases for higher overburden pressures.

![Figure 5.39: Definition of static shear on the soil sample in an undrained cyclic triaxial test (X., Wei and J., Yang, 2015).](image)

![Figure 5.40: Liquefaction resistance curves of Ohama sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.](image)
Effect of initial static shear stress ratio ($\alpha$)

The effects of the presence of initial static shear on the cyclic resistance ratio for an undrained cyclic DSS test are already elaborated in section 5.6. Presence of initial static shear also has influence on the cyclic resistance ratio for the stress states corresponding to an undrained cyclic triaxial test. The magnitude of the static shear is again represented by the $\alpha$, which is in a triaxial test defined as the ratio between the initial static shear stress and the normal effective stress on the plane where the maximum shear stress is present. In Figure 5.39 both the equation for $\alpha$ as the definition of the plane with maximum shear stress were presented. As was already mentioned the initial static shear stress on the plane with maximum shear stress is introduced by a difference in magnitude of the principal stresses.

Again the $K_\alpha$ according to Idriss, I.M. and Boulanger, R.W. (2008) is adopted to define the effect of initial static shear on the cyclic resistance ratio. In Figure 5.42 the development of $K_\alpha$ according to the UBC3D-PLM model and Idriss, I.M. and Boulanger, R.W. (2008) is presented for both sands. What is observed is that for small $\alpha$ values the cyclic resistance increases where for larger values the resistance decreases according to the UBC3D-PLM model. Initially the model overestimates the cyclic resistance ratio with respect to the theory where it underestimates the resistance for larger $\alpha$ values.
Like for the undrained cyclic DSS tests, the model parameters were calibrated to fit the $K_\alpha$ value by Idriss, I.M. and Boulanger, R.W. (2003) up to $\alpha$ values of 0.20. Again the densification factor and the plastic shear modulus were adjusted to fit the results. In Table 5.10 the corrected model parameters for both types of sand are presented. Differences between $K_\alpha$ values are smaller than for an undrained cyclic DSS tests so parameters also have to be adjusted to less extent.

Table 5.10: Corrected $fa_{\text{hard}}$ and $k_p^G$ values in UBC3D-PLM model to match the theoretical $K_\alpha$ Idriss, I.M. and Boulanger, R.W. (2003) for an undrained cyclic triaxial test.

<table>
<thead>
<tr>
<th>Ohama Sand</th>
<th>Idriss &amp; Boulanger (2008) $K_\alpha$</th>
<th>UBC3D-PLM $K_\alpha$</th>
<th>Corrected $fa_{\text{hard}}$</th>
<th>Corrected $k_p^G$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial static shear ratio $\alpha$ [-]</td>
<td>$\alpha$ [-]</td>
<td>[-]</td>
<td>[-]</td>
<td>[-]</td>
</tr>
<tr>
<td>0.05</td>
<td>0.947</td>
<td>1.009</td>
<td>0.15</td>
<td>/</td>
</tr>
<tr>
<td>0.10</td>
<td>0.922</td>
<td>0.874</td>
<td>/</td>
<td>/</td>
</tr>
<tr>
<td>0.15</td>
<td>0.911</td>
<td>0.651</td>
<td>0.45</td>
<td>/</td>
</tr>
<tr>
<td>0.20</td>
<td>0.899</td>
<td>0.583</td>
<td>0.90</td>
<td>/</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Gaiko Sand</th>
<th>$K_\alpha$ [-]</th>
<th>UBC3D-PLM $K_\alpha$</th>
<th>Corrected $fa_{\text{hard}}$</th>
<th>Corrected $k_p^G$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial static shear ratio $\alpha$ [-]</td>
<td>$\alpha$ [-]</td>
<td>[-]</td>
<td>[-]</td>
<td>[-]</td>
</tr>
<tr>
<td>0.05</td>
<td>0.958</td>
<td>1.056</td>
<td>0.40</td>
<td>/</td>
</tr>
<tr>
<td>0.10</td>
<td>0.939</td>
<td>0.990</td>
<td>0.50</td>
<td>/</td>
</tr>
<tr>
<td>0.15</td>
<td>0.938</td>
<td>0.877</td>
<td>0.75</td>
<td>/</td>
</tr>
<tr>
<td>0.20</td>
<td>0.936</td>
<td>0.799</td>
<td>1.00</td>
<td>1900</td>
</tr>
</tbody>
</table>

### 5.8. Conclusion

In the previous sections the performance of the UBC3D-PLM constitutive material model were analysed for loading paths corresponding to both undrained cyclic direct simple shear tests as undrained cyclic triaxial tests. It can be concluded that for both element tests parameter sets could be derived that lead to a reasonable fit of the liquefaction curves predicted by the model compared to curves obtained in the laboratory. Calibration on medium cyclic stress ratios lead however in all cases to an underestimation of the liquefaction resistance for the lower CSR values.

**Effects of overburden stress**

It could be concluded that the effects of variation of overburden stress are well captured by the model. The parameters sets for undrained cyclic DSS tests had to be slightly adjusted to fit the theoretical curves, however since differences were small finally no adjustments were proposed.

**Effects of lateral earth pressure coefficient**

In an undrained cyclic DSS test the effects of varying $K_0$ values had major influence on the soil behaviour predicted in the model. Independent from the initial $K_0$, the model always tends to an isotropic stress state. The phase from anisotropic to isotropic conditions takes a few cycles. In most cases this leads to a relatively larger resistance against liquefaction for $K_0 = 1.0$ compared to cases with initially already isotropic state. For undrained cyclic triaxial tests other behaviour is observed. The largest resistance is found for initial isotropic conditions and an incrementally decreasing resistance is found for decreasing $K_0$ values. Anisotropic loading conditions in undrained cyclic triaxial tests could be linked to initial static shear ratios leading to the same failure behaviour and development of liquefaction resistance.

**Effects of initial static shear ratio**

The effects of initial static shear ratio on the liquefaction resistance were not well captured by the model. In an undrained cyclic DSS test a different failure behaviour of the soil sample was observed in case of the presence of initial static shear. This behaviour was not accurately captured using the calibrated parameter sets. Major adjustments to both the $fa_{\text{hard}}$ and $k_p^G$ were required to match the amount of cycles to liquefaction. For the undrained cyclic triaxial tests the results were closer to the theoretical curves, leading to less extensive adjustments to the parameter set. Ultimately a reasonable fit was obtained of the liquefaction resistance curves for the considered $\alpha$ values for both element tests.

No laboratory data is available about the development of strains after liquefaction. It is important to have insight in the strain development, since it has influence on the deformation behaviour of the soil after liquefaction. These deformations impose loads on the anchored quay wall.
In this chapter a dynamic analysis of the anchored quay walls in Akita Port is performed. Akita was hit in 1983 by the Nihonkai Chubu Earthquake. Time motions of this earthquake and reference earthquakes are introduced. With a site response analysis these measured earthquake motions are translated to the the depth of interest. Subsequently dynamic calculations are carried out using the dynamic module in PLAXIS 2D. In Figure 6.1 an overview is given of the followed procedure to set up a dynamic analysis.
In this research the Hardening Soil small strain (HSsmall) constitutive material model is used to describe the soil behaviour for the static loading case and for the cyclic loading for soils not susceptible to liquefaction. For dynamic analysis of Ohama No.1 is therefore the HSsmall model adopted. After calibration of the model parameters and configuration of the model the dynamic calculation is performed and results are analysed.

The user-defined UBC3D-PLM constitutive material model is adopted for liquefiable soil layers. In chapter 5 the UBC3D-PLM model is calibrated to liquefaction resistance curves. Adjustments to parameter sets are suggested to accurately model the liquefaction resistance for varying initial stress states. At Ohama No.2 Wharf liquefiable layers are present to which the UBC3D-PLM model is assigned. Around the structure zones are defined in the soil corresponding to specific stress states. The earlier derived parameter sets are assigned to these zones to obtain accurate results for the separate zones. Ultimately the dynamic calculation of the system is performed and development of excess pore pressures and response of the structure is analysed.

6.1. Time records earthquakes
Several earthquake signals with different characteristics are applied at the base of the dynamic analysis. The response of the soil and structure on the different motions is analysed to investigate the dependency of the response on the characteristics of the earthquake signal.

6.1.1. Nihonkai Chubu Earthquake
In May 1983 Akita Port was hit by the Nihonkai Chubu Earthquake, which had a magnitude of 7.7. The time signal of this earthquake is presented in Figure 6.2. The maximum measured acceleration was in east-west direction and has a maximum of 0.233 g. Original measuring data of the earthquake was corrected for the type of measuring equipment and the higher frequencies are filtered out.

This signal is a free field motion measured at the observation station in Akita Port. The station is located a few kilometres from the considered quay walls on a stiff soil column. At the location of the quay walls the stiffer layers are found at larger depths with softer soil layers on top of it. These softer soil layers modify the characteristics of the motion, which has to be evaluated with a site response analysis.

6.1.2. Reference earthquake records
Besides the Nihonkai Chubu Earthquake also other earthquake motions are considered. Three reference earthquake records are presented in Figures 6.3 to Figure E.9. These signals are also free field motions measured at surface level on a stiff soil column.

The peak accelerations of the considered earthquake signals are not equal. To make proper comparison the time series are scaled to the same PGA (0.233 g) as the Nihonkai Chubu Earthquake. Subsequently all signals are also scaled to a PGA of 0.10 g.

All scaled time series are presented in Annex E.
6.2. Site response analysis

To determine the soil response at the project location a site response analysis of the earthquake motion at bedrock level is performed. This analysis requires an input signal at bedrock level that propagates in upward direction to the surface.

The measured earthquake time series are all free field motions, as indicated in Figure 6.6 with 1. At the top of the soil column a sudden transition between soil to air is present, which is a free end. Behaviour can be compared to a standing wave in a basin, which is fully reflected at a closed end. The amplitudes of both upward- and downward propagating wave are equal and the measured total amplitude is equal to twice the upward motion \(2A_1\).
The bedrock layer reaches at the observation station to the surface as is schematized in Figure 6.6. At the project location this bedrock layer is located at larger depth. Because of the high stiffness of the bedrock layer, the signal is barely altered due to local soil conditions as it travels through this layer. The characteristics of the motion at observation station can therefore be considered similar as the motion in the bedrock at the project location.

In the bedrock layer the total wave motion is a combination of both the upward- and downward propagating wave (within rock motion). Only the upward propagating wave is relevant as input for the site response analysis. The measured earthquake motions are equal to twice the amplitude of the upward propagating wave, so by halving these motions the input for the site response analysis is obtained.

Once the input signal is obtained the seismic waves propagate through the local soil deposit. Wave characteristics are altered due to varying soil properties. These local soil conditions modify the amplitude, duration and frequency content of the motion. In this research two methods for site response analysis are adopted.

Firstly the equivalent linear analysis in the frequency domain is applied to calculate the site response, since the required parameters for this simplified method are relatively easy to derive. These results are then used to validate the results of the full dynamic non-linear calculation performed in PLAXIS. Determination of the model parameters for the non-linear method is complex. Field- and laboratory testing may be required to evaluate these parameters. In absence of these tests, the linear equivalent analysis is used to validate the results of the non-linear method. Besides the equivalent linear analysis is less sensitive to errors compared to the more complex numerical models in PLAXIS.

For the lower loading levels the soil strains remain low and the soil behaviour can be properly described using linear relationships with non-linear soil properties. In this region results by the linear equivalent analysis are expected to be in reasonable agreement with the dynamic calculations in PLAXIS if both models correspond to each other (Kramer, S.L., 1996).

More severe loading events introduce larger strains in the soil leading to a much more non-linear behaviour of the soil, which is not captured by the equivalent linear analysis. For more severe loading events plastic strains are introduced, where the equivalent linear analysis only accounts for elastic strains. The stiffness of non-linear soil also changes over time, which is not accounted for in the equivalent linear analysis. For these type of problems a calibrated PLAXIS model is used to model this non-linear soil behaviour. The non-linear model is formulated in terms of effective stresses, which allow modelling of the development of water pressures over the duration of the earthquake event (Kramer, S.L., 1996).

![Figure 6.6: Schematization of location of the measured earthquake and the input motion (Mejia, L.H. and Dawson, E.M., 2006).](image-url)
6.2.1. Equivalent linear analysis
A simplified way of determining the horizontal acceleration at the depth of interest is done by applying a one-dimensional equivalent linear method. Several software packages are available that are all based on this theory, the most popular are EEER, SHAKE2000 and Strata. The software package Strata is used in this research, which computes the soil response for vertically propagating, horizontally shear waves propagating through a soil body with only horizontal layers (Kottke, A.R. et al., 2013). A short introduction into the equivalent linear analysis is given.

Introduction
In the equivalent linear analysis a one-dimensional soil column is subjected to a horizontal acceleration time motion at bedrock level. By applying a fast Fourier transformation on this motion, the Fourier amplitude spectrum at bedrock level is obtained. Subsequently the response at the layer of interest can be found by multiplying the input Fourier amplitude spectrum with a transfer function for that location. In fact each harmonic component of the signal is multiplied by the transfer function resulting in the output Fourier amplitude spectrum. Subsequently an inverse Fourier transformation can be applied on the output Fourier amplitude spectrum obtaining again the time-dependent acceleration signal at the layer of interest. At each layer the frequency-dependent horizontal motion is given by the sum of the upward \( A_n \) and downward \( B_n \) propagating waves. The top of the soil column is a free end and as already stated the amplitudes of both upward- and downward propagating wave are equal at that location. The total amplitude is thus equal to two times the upward motion \( 2A_n \). This boundary condition together with recursive formulas, the amplitudes of the motions in the layers below can be calculated.

The transfer functions have to account for the non-linear behaviour of the soil. The strain dependent shear modulus and damping ratio are found by iteratively calculating the strain level. The non-linear behaviour of the shear modulus and damping ratio is obtained using a modulus reduction curve and a damping curve that describe the variation of these parameters with the shear strain level. A frequency dependent strain transfer function is introduced to calculate the shear strain level of in the soil layer of interest. The strain level is then inserted to find the updated strain dependent soil properties, according to the defined curves. These updated soil properties are subsequently used again as input in the strain transfer function. Through an iterative process the soil response is found. In this way the non-linear behaviour is taken into account in the linear-equivalent analysis (Kottke, A.R. et al., 2013).

Model configuration

Soil profile
Within Strata soil types and soil profile have to be defined, which follow from soil investigation in Chapter 4.1. The only parameter still to be determined is the shear wave velocity, representing the speed with which shear waves travel through the soil. The shear wave velocity can be found according to:

\[
v_s = \sqrt{\frac{G_0}{\gamma_{sat} g}}
\]  

(6.1)

With:

\[
G_0 = G_0^{ref} \left( \frac{\dot{\varepsilon} \cdot \cos(q') - \sigma' \cdot \sin(q')}{\dot{\varepsilon} \cdot \cos(q) + \sigma' \cdot \sin(q')} \right)^m
\]  

(6.2)

Modulus reduction curve and damping curve
Elastic strain dependent characteristics of the soil have to be defined in the model. Several different theoretical relationships are available for the ratio \( G_s/G_0 \) and the damping ratio \( \zeta \) against the shear strain. In this study the strain dependent relationships by Darendeli & Stokoe (2001) are adopted for both the shear modulus ratio and damping ratio. The damping ratio must be such that it produces the same amount of energy loss during a loading cycle as the actual hysteresis loop. In a later stage these curves are used to calibrate damping curves in the HSsmall PLAXIS 1D model, see Figure 6.7.
Input motion  A time motion has to be specified at the elastic half-space at base of the model, representing bedrock level. In Strata the type of input motion has to be defined, so the model know how to deal with the input motion. Mainly two types of motions are distinguished: a 'Within'-motion or an 'Outcrop'-motion. A 'Within'-motion means that the motion is measured at a location between two layers. The input motion is then a superposition of the downward- and upward propagating wave. However if the motion is measured at a free boundary, the resulting wave is equal to twice times the upward propagating wave. Strata has to half the input motion when applied at the base of the model (Kottke, A.R. et al., 2013).

Since all considered time signals are measured at surface level, all input signals in Strata have to be defined as being an 'Outcrop' motion.

6.2.2. Non-linear site response analysis
A non linear free field site response analysis can be performed by using finite element model PLAXIS 2D. Surface motions more can be determined more accurately for higher loading events, provided that the calibration of the model is executed properly. A short introduction into the method is given and configuration of the model is elaborated.

Introduction
Non-linear constitutive soil models account for non-linear constitutive behaviour of the soil, which is not captured by the equivalent linear analysis. For more severe loading events this non-linear behaviour becomes very relevant, since plastic strains are introduced. In the constitutive material model adopted in PLAXIS a failure criterion is described. The moment this criterion is met a flow rule is introduced describing the amount and direction of plastic strains. These plastic strains influence the stiffness and damping behaviour of the soil as was also described in the equivalent linear analysis. This soil behaviour is not captured in an equivalent linear analysis, since it only accounts for elastic strains.

Many modelling choices have to be made developing a proper finite element model. Configuration of the model is considered in the next section.

Table 6.1: Model parameters used in the HSsmall model (Brinkgreve, R.B.J. et al. (2010), Brinkgreve, R.B.J. et al. (2015))

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight</td>
<td>γ_dry [kN/m³]</td>
</tr>
<tr>
<td>Saturated unit weight</td>
<td>γ_sat [kN/m³]</td>
</tr>
<tr>
<td>Effective cohesion</td>
<td>c' [kPa]</td>
</tr>
<tr>
<td>Effective angle of internal friction</td>
<td>φ [°]</td>
</tr>
<tr>
<td>Angle of dilatancy</td>
<td>ω [°]</td>
</tr>
<tr>
<td>Secant stiffness in standard drained triaxial test</td>
<td>E_50,ref [kPa]</td>
</tr>
<tr>
<td>Tangent stiffness for primary oedometer loading</td>
<td>E_oed,ref [kPa]</td>
</tr>
<tr>
<td>Unloading/reloading stiffness</td>
<td>E_ur,ref [kPa]</td>
</tr>
<tr>
<td>Exponent for stress level dependency of stiffness</td>
<td>m [-]</td>
</tr>
<tr>
<td>Poisson's ratio for unloading and reloading</td>
<td>μ_ur</td>
</tr>
<tr>
<td>Reference stress for stiffness</td>
<td>p_ref [kPa]</td>
</tr>
<tr>
<td>Lateral earth pressure coefficient for normally consolidated soil</td>
<td>k_0,nc [-]</td>
</tr>
<tr>
<td>Failure ratio</td>
<td>R_f [-]</td>
</tr>
<tr>
<td>Initial or very small shear strain modulus</td>
<td>G_0 [kPa]</td>
</tr>
<tr>
<td>Shear strain level where secant shear modulus has decreased to 70% of G0</td>
<td>γ_0.7 [-]</td>
</tr>
</tbody>
</table>
Model parameters HSsmall model
Both static and cyclic behaviour of the soil are modelled using the Hardening Soil small strain (HSsmall) constitutive model, excluding loss of strength due to excess pore pressures. In order to describe the soil behaviour accurately the model requires many input parameters, which are presented in Table 6.1. In Appendix C the background of this constitutive model is elaborated.

Empirical relationships for sands are developed to obtain a first estimate of the model parameters, based on the characteristic property relative density ($D_r$) (Brinkgreve, R.B.J. et al., 2010). The empirical relationships are originally derived for several types sands with different relative densities. The empirical formulas are presented below (Brinkgreve, R.B.J. et al., 2010), where the value for $E_{ur}^{ref}$ is taken slightly lower compared to what was reported by Brinkgreve, R.B.J. et al. (2010):

\[
E_{50}^{ref} = 60,000 \cdot D_r / 100
\]
\[
E_{90}^{ref} = 60,000 \cdot D_r / 100
\]
\[
E_{ur}^{ref} = 180,000 \cdot D_r / 100
\]
\[
m = 0.7 - D_r / 320
\]
\[
R_f = 1 - D_r / 800
\]
\[
G_0^{ref} = 60,000 + 68,000 \cdot D_r / 100
\]
\[
\gamma_{0.7} = (2 - D_r / 100) \cdot 10^{-4}
\]

Besides these empirical relationships the following parameters are also prescribed:

\[
v_{ur} = 0.2 \text{ and } p_{ref} = 100 \text{ kN/m}^2
\]

In correlations by Brinkgreve, R.B.J. et al. (2010) a dilatancy angle is prescribed for sands with a $\varphi$ larger than 30 degree. When shear waves cause deformation of the soil during the dynamic phase, water under pressures could develop in undrained conditions due to dilatant behaviour of the soil sample. Under pressures lead to higher effective stresses and thus resulting in stronger soil behaviour, which is not necessarily realistic. For that reason a dilatancy angle of zero is adopted in undrained calculations.

Since the relative density is the only input parameter, this has to be determined for all soil layers. Often only results from Standard Penetration Tests (SPT) or Cone Penetration Tests (CPT) are available. Relationships are available linking the ($N_{160}$ or $q_n$ values from the in-situ to the relative density. In this research the following relationship is adopted to determine the relative density:

\[
D_r = \sqrt[3]{\frac{(N_{1})_{60}}{C_d}}
\]

Different relationships with large range for $C_d$ are available for the determination of the relative density. A $C_d$-value of 41 was suggested by Meyerhof, G.G. (1957), while research by Skempton, A.W. (1986) of field and laboratory data lead to higher values for normally consolidated natural sands. For fine sands a $C_d$ value of 55 was suggested and for coarse sands a value of 65. Skempton also mentioned that the age of the deposit has significant influence on the $C_d$-value. The younger the soil deposit, the lower the $C_d$-value. Finally Cubrinovski, M. and Ishihara, K. (1999) investigated high quality undisturbed samples and found a $C_d$ value of 51 for clean sands, a value of 26 for silty sands and an average value of 39.

The spread in relative density together with the spread in measured N-values over the soil layer leads to uncertainty in the model parameters. It is decided to use the $C_d$ values by Skempton (1986) for the determination of the relative density, which is relative to the other relationships a conservative approach leading to lower relative densities. The average N-values of the soil layers are used as input to calculate the relative density.
Model configuration

Damping characteristics  The model parameters used in the HSsmall constitutive material model were determined as presented in section 6.2.2 according to (Brinkgreve, R.B.J. et al., 2010). The damping characteristics of the model must however be calibrated, since these depend very much on the local conditions and have large influence on the response of the soil.

In the equivalent linear analysis the relationship of Darendeli & Stokoe (2001) was used to define the elastic strain dependent behaviour of the shear modulus and the damping ratio. Within the HSsmall model these type of curves are also adopted to define the elastic strains in the model. The black lines in Figure 6.7 are the strain dependent damping curves adopted in the HSsmall model. These curves are defined in the models by the values for $G_0$ and $\gamma_{0.7}$. Since these strain dependent characteristics vary with the content of the soil and the soil type thus are not exactly known beforehand the HSsmall damping curves are calibrated to theoretical damping curves. These characteristics can also be derived from laboratory tests, but by absence of these test results these curves are calibrated on theoretical curves. Many researches have developed relationships of the shear modulus ratio ($G_S/G_0$) and damping ratio ($\zeta$) against the shear strain ($\gamma_s$). Maximum stiffness is found for small strain values, and when strain levels increase the stiffness decreases to finally a minimum value $G_{ur}$. In Figure 6.7 several curves are presented, according to different theories (Idriss (1990), Seed & Idriss (1970), Vucetic & Dobry (1991) and Darendeli & Stokoe (2001)). The curve defined in the HSsmall model is calibrated with $\gamma_{0.7}$ to fit the, for this layer most relevant, theoretical curves as much as possible, see Figure 6.7.

![Figure 6.7: Calibrating strain dependent stiffness parameters and damping curves used in the HSsmall PLAXIS model according to theoretical damping curves.](image)

In general after calibration the graphs show a reasonable correspondence to the different reduction curves. Due to the spread of the theoretical curves not perfect match is obtained and an small error is introduced (Besseling, F., 2012).

The calibration according to theoretical curves is based on elastic behaviour of the soil. For lower loading levels only minor plastic strains have been developed. Since the equivalent linear analysis only accounts for elastic soil behaviour results are similar to the calibrated non-linear finite element model (Kramer, S.L., 1996). For larger strain levels the damping behaviour of the soil is however dominated by plastic deformation. Since these plastic strains are not captured by the equivalent linear analysis, results for higher loading events are not accurate. The HSsmall constitutive model does account for these plastic strains and related changing soil stiffness by combination of a failure criterion, hardening rule and flow rule.

To validate the model results including the plastic soil behaviour laboratory tests are required. These tests are not available, so the model has to be validated in another way. By keeping the loading level low, plastic strains are not generated and the equivalent linear analysis may be used to validate the non-linear effective stress constitutive model.
Model parameters

For both anchored quay walls the soil profile was determined according to Appendix B. The relationships presented in section 6.2.2 are used to determine for each soil layer the model parameters for the HSsmall model. Subsequently the damping parameters are calibrated according to the theoretical damping curves and results by Strata. In Table 6.2 and Table 6.3 the calibrated model parameters for the soil layers at both anchored quay walls are presented.

### Table 6.2: Calibrated input parameters HSsmall material model soil layers Ohama No.1 Wharf.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Description</th>
<th>Dr</th>
<th>E_{50,ref}</th>
<th>E_{oed,ref}</th>
<th>E_{ur,ref}</th>
<th>m</th>
<th>K_{0,ref}</th>
<th>R_{f}</th>
<th>G_{0,ref}</th>
<th>γ_{0,7}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand1</td>
<td>Med dense, clean</td>
<td>70</td>
<td>42000</td>
<td>42000</td>
<td>84000</td>
<td>0.481</td>
<td>0.455</td>
<td>0.913</td>
<td>107600</td>
<td>0.000300</td>
</tr>
<tr>
<td>Sand2</td>
<td>Loose / med dense, very silty</td>
<td>65</td>
<td>39000</td>
<td>39000</td>
<td>78000</td>
<td>0.497</td>
<td>0.500</td>
<td>0.919</td>
<td>104200</td>
<td>0.000350</td>
</tr>
<tr>
<td>Sand3</td>
<td>Med dense, slightly silty</td>
<td>70</td>
<td>42000</td>
<td>42000</td>
<td>84000</td>
<td>0.481</td>
<td>0.455</td>
<td>0.913</td>
<td>107600</td>
<td>0.000300</td>
</tr>
<tr>
<td>Sand4</td>
<td>Dense, slightly silty</td>
<td>85</td>
<td>51000</td>
<td>51000</td>
<td>102000</td>
<td>0.434</td>
<td>0.398</td>
<td>0.894</td>
<td>117600</td>
<td>0.000150</td>
</tr>
<tr>
<td>Clay</td>
<td>Firm, very sandy</td>
<td>\</td>
<td>20000</td>
<td>20000</td>
<td>40000</td>
<td>0.550</td>
<td>0.546</td>
<td>0.920</td>
<td>75000</td>
<td>0.000165</td>
</tr>
<tr>
<td>Sand5</td>
<td>Med dense, slightly silty</td>
<td>65</td>
<td>27000</td>
<td>27000</td>
<td>54000</td>
<td>0.559</td>
<td>0.426</td>
<td>0.944</td>
<td>90600</td>
<td>0.000150</td>
</tr>
</tbody>
</table>

### Table 6.3: Calibrated input parameters HSsmall material model soil layers Ohama No.2 Wharf.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Description</th>
<th>Dr</th>
<th>E_{50,ref}</th>
<th>E_{oed,ref}</th>
<th>E_{ur,ref}</th>
<th>m</th>
<th>K_{0,ref}</th>
<th>R_{f}</th>
<th>G_{0,ref}</th>
<th>γ_{0,7}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfill sand</td>
<td>Very Loose/Loose, clean</td>
<td>40</td>
<td>24000</td>
<td>24000</td>
<td>48000</td>
<td>0.575</td>
<td>0.500</td>
<td>0.950</td>
<td>87200</td>
<td>0.000360</td>
</tr>
<tr>
<td>Sand1</td>
<td>Loose, slightly silty</td>
<td>60</td>
<td>36000</td>
<td>36000</td>
<td>72000</td>
<td>0.513</td>
<td>0.500</td>
<td>0.925</td>
<td>100800</td>
<td>0.000160</td>
</tr>
<tr>
<td>Sand2</td>
<td>Dense, slightly silty</td>
<td>85</td>
<td>51000</td>
<td>51000</td>
<td>102000</td>
<td>0.434</td>
<td>0.357</td>
<td>0.894</td>
<td>117600</td>
<td>0.000140</td>
</tr>
<tr>
<td>Clay</td>
<td>Firm, very sandy</td>
<td>\</td>
<td>20000</td>
<td>20000</td>
<td>40000</td>
<td>0.550</td>
<td>0.546</td>
<td>0.920</td>
<td>75000</td>
<td>0.000165</td>
</tr>
<tr>
<td>Sand3</td>
<td>Med dense, slightly silty</td>
<td>65</td>
<td>27000</td>
<td>27000</td>
<td>54000</td>
<td>0.559</td>
<td>0.455</td>
<td>0.944</td>
<td>90600</td>
<td>0.000160</td>
</tr>
<tr>
<td>Sand4</td>
<td>Med dense / dense, slightly silty</td>
<td>70</td>
<td>42000</td>
<td>42000</td>
<td>84000</td>
<td>0.481</td>
<td>0.398</td>
<td>0.913</td>
<td>107600</td>
<td>0.000140</td>
</tr>
</tbody>
</table>

Model boundary conditions

At the bottom of the model a compliant base is applied, a boundary condition that absorbs downward propagating waves. This implies that the reflected waves from upper soil layers can propagate into the deeper soil layers through the boundary with minimum reflection (Brinkgreve, R.B.J. et al., 2015).

Since the earthquake motions are measured at surface level, those could be seen as outcrop motions consisting two times the upward propagating wave. At the base this input motion is applied. The compliant base boundary implies that only upward going waves are present, since the downward going waves are absorbed. For that reason the input motion (which is equal to twice times the upward going wave) at the base of the model has to be divided by two, in order to get the right input motion for the site response analysis.

For the lateral boundaries in the dynamic calculation the so-called tied degree of freedom boundaries are applied. This condition connects the nodes on the same height at the left and right model boundaries, such that they are characterized by the same vertical and horizontal displacement. By connecting both side boundaries a one dimensional soil column to is created, which saves computational time. Results of the 1D column may also be properly compared with the results of the 1D equivalent linear analysis.

Damping

In the Hardening Soil small strain material model in PLAXIS two major damping components are present, representing the energy dissipation while waves propagate through the soil. First the hysteric damping, which is caused by the cyclic loading and the decrease of stiffness for larger strains, see Annex C.1 for background of hysteric damping. However even for small deformations the soil behaviour can be irreversible. These are not captured by the hysteric damping. This damping at smaller strains can be covered in PLAXIS by assigning a Rayleigh damping to soils. This viscous Rayleigh damping component generally has a damping ratio of 0.5% to 2.0%. In literature different methods are suggested to select appropriate parameters for the Rayleigh damping for different target frequencies. In this study the method by Hudson, M. et al. (1994) is adopted.
According to Hudson, M. et al. (1994) the first target frequency is set equal to the fundamental frequency of the soil profile, which is:

\[ f_1 = \frac{v_s}{4H} \]  \hspace{1cm} (6.11)

Where \( v_s \) is the shear wave velocity and \( H \) is the thickness of the considered soil body. The thickness of the whole soil body is used as input, separate soil layers are not accounted for in the calculation of fundamental frequency. An average value for the shear wave velocity of the soil body is determined.

The second target frequency \( (f_2) \) is set as the closest odd number to the ratio of the fundamental frequency of the input frequency at bedrock level over the fundamental frequency of the whole soil layer. The fundamental frequency of each input signal is defined using Strata (Kottke, A.R. et al., 2013), see Annex F for the frequency spectra of all the input signals.

For each soil layer and each earthquake signal the Rayleigh damping parameters are determined. For both target frequencies a damping ratio \( \xi \) of 1.25% is adopted. The target frequencies for both anchored quay walls are presented in Table 6.4.

### Table 6.4: Target frequencies for both anchored quay walls to determine the Rayleigh damping parameters.

<table>
<thead>
<tr>
<th></th>
<th>Ohama No.1 Wharf</th>
<th>Ohama No.2 Wharf</th>
</tr>
</thead>
<tbody>
<tr>
<td>H [m]</td>
<td>33.0</td>
<td>33.0</td>
</tr>
<tr>
<td>( v_s ) [m/s]</td>
<td>208</td>
<td>201</td>
</tr>
<tr>
<td>( f_1 ) [Hz]</td>
<td>1.58</td>
<td>1.53</td>
</tr>
<tr>
<td><strong>Fundamental frequency:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nihonkai Chubu [Hz]</td>
<td>5.6</td>
<td>5.6</td>
</tr>
<tr>
<td>Imperial Valley [Hz]</td>
<td>6.3</td>
<td>6.3</td>
</tr>
<tr>
<td>Landers [Hz]</td>
<td>1.9</td>
<td>1.9</td>
</tr>
<tr>
<td>Kocaeli [Hz]</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>( f_2 ) [Hz]</td>
<td>3.0</td>
<td>3.0</td>
</tr>
</tbody>
</table>

**Mesh generation and time step**  
In dynamic finite element modelling very important aspects are determination of the size of the mesh-elements and the time steps of the calculation, to ensure that the seismic wave propagates properly through the model. This means that the element size may not be too large. On the other hand too small element size result in a very long computation time. For that reason it is beneficial to optimize the element size for each soil layer.

The average element size per layer can be determined according to (Kuhlmeier, R.L. and Lysmer, J., 1973):

\[ \text{AverageElementSize}_{\text{layer}} \leq \lambda = \frac{v_s_{\text{layer}}}{8f_{\text{max}}} \] \hspace{1cm} (6.12)

Where \( v_s \) is the shear wave velocity of the considered layer and \( f_{\text{max}} \) is the maximum frequency component of the input signal. This maximum frequency component can be obtained from the results of the equivalent linear analysis performed in Strata. PLAXIS can also automatically determine the mesh size, but this results in a relative fine mesh for the whole model leading to large computational times.

In PLAXIS also the dynamic time step and the amount of substeps have to be defined. The dynamic time step is set equal to the time step of the input signal. It must however be prevented that waves travel through more than one element within one dynamic time step. This may lead to a tighter time step restriction. In PLAXIS the amount of substeps within one time step can be defined, to fulfil the requirement. The minimum required time step can be determined as follows:

\[ \Delta t = \frac{\text{AverageElementSize}_{\text{layer}}}{v_s_{\text{layer}}} \] \hspace{1cm} (6.13)

In Table 6.5 the required average element sizes and maximum allowable time steps are presented for each layer of both Ohama No.1 Wharf and Ohama No.2 Wharf.
### Table 6.5: Required minimum average mesh size and time step for Ohama No.1 Wharf and Ohama No.2 Wharf.

<table>
<thead>
<tr>
<th>Layer</th>
<th>$f_{\text{max}}$ [Hz]</th>
<th>$v_s$ [m/s]</th>
<th>Avg. element size [m]</th>
<th>Max. time step [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Ohama No.1 Wharf</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 1 (Sand - Med dense)</td>
<td>12.00</td>
<td>159</td>
<td>1.66</td>
<td>0.010</td>
</tr>
<tr>
<td>Layer 2 (Sand - Loose)</td>
<td>12.00</td>
<td>178</td>
<td>1.85</td>
<td>0.010</td>
</tr>
<tr>
<td>Layer 3 (Sand - Med dense)</td>
<td>12.00</td>
<td>187</td>
<td>1.95</td>
<td>0.010</td>
</tr>
<tr>
<td>Layer 4 (Sand - Dense)</td>
<td>12.00</td>
<td>228</td>
<td>2.38</td>
<td>0.010</td>
</tr>
<tr>
<td>Layer 5 (Clay - Firm)</td>
<td>12.00</td>
<td>209</td>
<td>2.18</td>
<td>0.010</td>
</tr>
<tr>
<td>Layer 6 (Sand - Med dense)</td>
<td>12.00</td>
<td>230</td>
<td>2.40</td>
<td>0.010</td>
</tr>
<tr>
<td><strong>Ohama No.2 Wharf</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Layer 1 (Sand - (Very) loose)</td>
<td>12.00</td>
<td>158</td>
<td>1.65</td>
<td>0.010</td>
</tr>
<tr>
<td>Layer 2 (Sand - Loose)</td>
<td>12.00</td>
<td>191</td>
<td>1.99</td>
<td>0.010</td>
</tr>
<tr>
<td>Layer 3 (Sand - Dense)</td>
<td>12.00</td>
<td>225</td>
<td>2.34</td>
<td>0.010</td>
</tr>
<tr>
<td>Layer 4 (Clay - Firm)</td>
<td>12.00</td>
<td>204</td>
<td>2.13</td>
<td>0.010</td>
</tr>
<tr>
<td>Layer 5 (Sand - Med dense)</td>
<td>12.00</td>
<td>225</td>
<td>2.34</td>
<td>0.010</td>
</tr>
<tr>
<td>Layer 6 (Sand - (Med) dense)</td>
<td>12.00</td>
<td>240</td>
<td>2.50</td>
<td>0.010</td>
</tr>
</tbody>
</table>

It is also possible to obtain the required amount of substeps automatically, based on compression wave velocity. This however often leads to a large amount of required substeps due to the high stiffness at small strains which are present due to small numerical errors resulting in high compression wave velocities. For this reason it is strongly advised to determine the amount of required substeps manually, to reduce the computational time significantly. Also because the interest is in shear wave velocities and not to compression wave velocities. PLAXIS provides even higher amount of required substeps for undrained calculations, which is explained by the fact that compression wave velocities tend to go to infinity for a Poisson's ratio that approaches 0.5 (Besseling, F., 2012).
6.2.3. Results site response analysis

In this section the results of the free field site response analysis performed with equivalent linear analysis are compared with results with the 1D finite element model in PLAXIS. For all considered earthquakes both time series and the Fourier amplitude spectrum at surface level are determined. In this section for each signal the time series are shown, only the Fourier amplitude spectrum for the Nihonkai Chubu Earthquake is presented. In Annex F all results of the site response analysis are presented.

The damping parameters were first calibrated according to available theoretical damping curves. Subsequently results using these calibrated model parameters were obtained and differences between PLAXIS output and Strata output were analysed. The damping parameters were then again adjusted to match the results between PLAXIS and Strata as much as possible. The development of the acceleration was analysed over the height of the soil profile in order to analyse the influence of each soil layer on the response.

**Ohama No.1 Wharf**  
**Nihonkai Chubu Earthquake** - Time series and frequency amplitude spectrum

![Figure 6.8: Calculated time series at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g.](image)

![Figure 6.9: Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g.](image)

**Imperial Valley Earthquake** - Time series

![Figure 6.10: Calculated time series at the surface at Ohama No.1 Wharf of the Imperial Valley Earthquake scaled to 0.10 g.](image)
6.2. Site response analysis

**Landers Earthquake** - Time series

Figure 6.11: Calculated time series at the surface at Ohama No.1 Wharf of the Landers Earthquake scaled to 0.10 g.

**Kocaeli Earthquake** - Time series

Figure 6.12: Calculated time series at the surface at Ohama No.1 Wharf of the Kocaeli Earthquake scaled to 0.10 g.

**Ohama No.2 Wharf**

**Nihonkai Chubu Earthquake** - Time series and frequency amplitude spectrum

Figure 6.13: Calculated time series at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g.

Figure 6.14: Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g.
**Imperial Valley Earthquake** - Time series

![Figure 6.15: Calculated time series at the surface at Ohama No.2 Wharf of the Imperial Valley Earthquake scaled to 0.10 g.](image)

**Landers Earthquake** - Time series

![Figure 6.16: Calculated time series at the surface at Ohama No.2 Wharf of the Landers Earthquake scaled to 0.10 g.](image)

**Kocaeli Earthquake** - Time series

![Figure 6.17: Calculated time series at the surface at Ohama No.2 Wharf of the Kocaeli Earthquake scaled to 0.10 g.](image)

**Conclusion**

Considering the results of the equivalent linear analysis and the dynamic finite element analysis a reasonable fit was found between the results. Both the time series and the frequency amplitude spectra of both analysis are compared, which both show reasonable fit.

From analysing the time series it can be observed that the maximum peak accelerations obtained by the PLAXIS model are for all time signals somewhat lower compared to the Strata results. This effect is much smaller looking at the differences between the results halfway the soil column. As the wave propagates higher in the soil column the peaks in the PLAXIS model are smaller leading to the conclusion that the damping in the finite element model is higher than in the Strata model. The lower peaks do fit much better compared to the maximum accelerations. In the upper part of the soil column looser layers are present. An explanation for the smaller peaks in PLAXIS could be that in these layers already some plastic strains have developed, leading to larger damping. This is not captured by Strata, since the equivalent linear analysis assumes elastic soil behaviour. This explanation fits also with the observation that for the smaller peaks the fit is much better.
6.3. Dynamic analysis Ohama No.1 Wharf

In this section results of the dynamic analysis of the anchored quay wall at Ohama No.1 Wharf are presented. After validation of the site response and corresponding calibration of the soil parameters the structure is also modelled and a dynamic analysis is performed for the anchored quay wall.

6.3.1. Model configuration

The soil profile and soil parameters that are used in the calibration of the 1D soil column in section 6.2.2 are adopted in the dynamic model. Also the amount of Rayleigh damping, mesh size and time step size as discussed in section 6.2.2 are applied.

The sheet pile wall and anchor wall are modelled as plates using elastic material model, both elements also have end bearing in order to prevent settlement due to its own weight. The tie-rod is modelled as a node-to-node anchor connecting the sheet pile wall with the anchor wall and has a centre-to-centre distance of 2.0 meters. Properties of the structural elements in the model are already determined in section 4.2.3 and are also applied in the PLAXIS model.

Besides applying damping to the soil, also some Rayleigh damping is added to the structures as it is observed that transient response remained in the structures response indicating some resonance occurred due to high frequency components.

At both sides of the model free field boundary conditions are applied, in order to prevent waves from reflecting back into the model. After modelling the static stage an extra stage is implemented in the model where at both side boundaries drained layers are applied. This is done in order to prevent complete loss of strength at the boundaries which would lead to a loss of support at the boundaries resulting in large deformations. At the bottom boundary again a compliant base boundary is applied as was already explained in section 6.2. To prevent the numerical model boundaries from having influence on the model results the model has to be wide enough. A test run is applied where a pseudo-static analysis is performed in the PLAXIS model. The deformation pattern of the soil is analysed to see to how far the active and passive wedge extend. A distance of approximately 50.0 meters of the model boundary from the deformed soil wedges leads to negligible numerical influence of the boundary on the results. Ultimately this resulted in a model width of 180 meters.

In the model first the construction stages are included, to realistically model the stress states present in the soil before starting the dynamic phase. First the sheet pile wall and anchor wall are installed, subsequently the soil is excavated in layers in front of the wall to the required depth.

![Figure 6.18: Layout of PLAXIS 2D model, with drained boundaries at both sides.](image)

6.3.2. Results dynamic analysis

In Table 6.6 and Figure 6.19 extreme values are presented of the structure's response after dynamic loading. The spread in the response of the structure for the different earthquake motions is significant, indicating that the peak ground acceleration is not the only relevant parameter specifying the loading event. The Kocaeli earthquake results in larger displacements and larger extreme bending moments. This earthquake contains more energy in the lower frequency range compared to the other earthquake motions as can been seen in the site response analysis in Appendix F. The system is more prone to these lower frequency contents. The response of the system to the Imperial Valley Earthquake and the Landers Earthquake is comparable. Higher deformations and internal forces are found for the Nihonkai Chubu Earthquake.
Although the spread of the results for the different seismic events is large, however all extreme values for bending moments remain below the bending moment capacity of the sheet pile wall. This is in line with observations at the site, where no structural failure was observed at the structure post the earthquake. The bending moment at the retaining part of the wall (field moment) increases during all seismic events compared to the static situation, while the maxima at the embedded part (fixed moment) generally decrease (Figure 6.19). This can be explained by the fact that the earth pressure at the active side increases while at the same moment the passive earth pressure decreases in the dynamic phase.

After modelling of the construction stages and the static loading stage, the displacement of the maximum wall is about 3.0 to 4.0 centimetres. The displacement was reset in the model since this is the starting point of the dynamic phase. The presented displacement are therefore the resultant displacements only from the corresponding dynamic phase. Comparing the results of the displacements with the fixed bending moments leads to the conclusion that a higher passive resistance leads to smaller deformations. The more the wall is clamped at the bottom, the higher the fixed moment corresponding to smaller deformations. At the same time a larger displacement corresponds to a larger anchor force. In Figure 6.20 the horizontal deformation of the soil after the Nihonkai Chubu Earthquake is presented. It can be seen that the largest deformations are also found at the location where the bending moment in the wall is the highest. In the field after the Nihonkai Chubu Earthquake deformations of 5.0 to 10.0 centimetres of the top of the anchored quay wall are observed. The model predicts a displacement of approximately 17.0 centimeters, which is considered to be acceptable accurate since the complete behaviour of the structure is in accordance with observations.

Table 6.6: Output of the dynamic PLAXIS model.

<table>
<thead>
<tr>
<th>Event</th>
<th>Minimum bending moment [kNm/m']</th>
<th>Maximum bending moment [kNm/m']</th>
<th>Maximum anchor force [kN/m']</th>
<th>Maximum displacement [m]</th>
<th>Displacement anchor [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>-222</td>
<td>222</td>
<td>114</td>
<td>0.038</td>
<td>0.031</td>
</tr>
<tr>
<td>Nihonkai Chubu</td>
<td>-654</td>
<td>117</td>
<td>184</td>
<td>0.18</td>
<td>0.17</td>
</tr>
<tr>
<td>Imperial Valley</td>
<td>-411</td>
<td>238</td>
<td>156</td>
<td>0.057</td>
<td>0.051</td>
</tr>
<tr>
<td>Landers</td>
<td>-456</td>
<td>208</td>
<td>165</td>
<td>0.074</td>
<td>0.068</td>
</tr>
<tr>
<td>Kocaeli</td>
<td>-768</td>
<td>88.1</td>
<td>210</td>
<td>0.29</td>
<td>0.29</td>
</tr>
</tbody>
</table>

Figure 6.19: Bending moment distribution and displacement over the height of the structure for considered earthquake motions.
6.3. Comparison with pseudo-static analysis

Results of the pseudo-static analysis and dynamic analysis of for the Nihonkai Chubu Earthquake compared to each other. In Table 6.7 extreme values are presented and ratio between the extreme values. In Figure 6.21 the bending moment distribution and displacements are shown.

Table 6.7: Comparison of the extreme bending moments, anchor forces and displacements between dynamic analysis and pseudo-static analysis for Nihonkai Chubu Earthquake

<table>
<thead>
<tr>
<th></th>
<th>Minimum bending moment [kNm/m']</th>
<th>Maximum bending moment [kNm/m']</th>
<th>Maximum anchor force [kN/m']</th>
<th>Maximum displacement [m]</th>
<th>Displacement anchor [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLAXIS - Static</td>
<td>-222</td>
<td>222</td>
<td>114</td>
<td>0.038</td>
<td>0.031</td>
</tr>
<tr>
<td>Dsheet - Static</td>
<td>-378</td>
<td>406</td>
<td>129</td>
<td>0.052</td>
<td>0.035</td>
</tr>
<tr>
<td>Ratio (Pseudo-static / Dynamic)</td>
<td>1.71</td>
<td>1.83</td>
<td>1.14</td>
<td>1.35</td>
<td>1.14</td>
</tr>
<tr>
<td>PLAXIS - Nihonkai Chubu</td>
<td>-654</td>
<td>117</td>
<td>184</td>
<td>0.18</td>
<td>0.17</td>
</tr>
<tr>
<td>Dsheet - (k_h = 0.100)</td>
<td>-651</td>
<td>613</td>
<td>202</td>
<td>0.14</td>
<td>0.13</td>
</tr>
<tr>
<td>Ratio (Pseudo-static / Dynamic)</td>
<td>1.00</td>
<td>5.25</td>
<td>1.10</td>
<td>0.80</td>
<td>0.73</td>
</tr>
<tr>
<td>PLAXIS - Nihonkai Chubu</td>
<td>-654</td>
<td>117</td>
<td>184</td>
<td>0.18</td>
<td>0.17</td>
</tr>
<tr>
<td>Dsheet - (k_h = 0.206)</td>
<td>-1380</td>
<td>367</td>
<td>361</td>
<td>0.52</td>
<td>0.51</td>
</tr>
<tr>
<td>Ratio (Pseudo-static / Dynamic)</td>
<td>2.11</td>
<td>3.14</td>
<td>1.96</td>
<td>2.97</td>
<td>3.01</td>
</tr>
</tbody>
</table>

In the pseudo-static analysis two loading levels are applied, one based on design codes (\(k_h = 0.206\)) and one reduced horizontal seismic coefficient (\(k_h = 0.100\)). The reduced seismic coefficient leads to a good match for the minimum bending moment, which is mainly due to an overestimated passive resistance. Application of the original horizontal seismic coefficient leads to a decrease of the passive resistance compared to the reduced value, resulting in a significant increase of the field moment reaching the bending moment capacity of the wall. In the Mononobe-Okabe method planar sliding planes are adopted, while curved sliding planes are more realistic. This leads to an altered shape of the soil wedge. Besides that is the stiffness of the soil in PLAXIS stress dependent and the gradient of the passive resistance lower.
Concerning the displacements of the wall, the pseudo-static method with reduced seismic coefficient underestimates the occurring displacements. This is again explained by an overestimation of the passive resistance in the pseudo-static analysis. Applying the seismic coefficient from design codes results in an overestimation. The same conclusions can be drawn regarding the anchor force, larger passive resistance leads to a smaller anchor force.

All of the above observations can be confirmed by analysing the earth pressures from the pseudo-static analysis and the dynamic analysis. In Table 6.8 horizontal effective stresses at certain points next to the wall are presented. Ratios between the results of the pseudo-static analysis and the dynamic analysis give an indication of the relative difference.

Figure 6.21: Comparison bending moment distribution and displacement obtained from dynamic analysis and pseudo-static analysis for Nihonkai Chubu Earthquake.

Figure 6.22: Development of $K_{AE}$ and $K_{PE}$ according to dynamic analysis and pseudo-static analysis.
Point L corresponds to a location at the active side of the wall, see Figure 6.20. In line with the theory, the horizontal stress increases during the seismic event. A good correspondence of the horizontal stress between pseudo-static and dynamic analysis is found when the reduced seismic coefficient is adopted while for the original seismic coefficient the horizontal pressure is 50% higher.

The same is done for a location at the passive side of the wall, point M. The passive earth pressure decreases as can be expected. However for both the reduced as the original seismic coefficient the pseudo-static analysis predicts significantly higher horizontal stresses leading to again the conclusion that the passive resistance is overestimated. This is attributed to the fact that in the Mononobe-Okabe method planar sliding planes are adopted and the gradient of development of the passive resistance is too high.

In Figure 6.22 values for $K_{AE}$ and $K_{PE}$ over the height of the active and passive soil layer are shown. The values from the dynamic analysis are represented by the solid lines and from the pseudo-static analysis by the dashed lines. Results for $K_{AE}$ are much more in correspondence to each other than results for $K_{PE}$ confirming the earlier observations. At the top of the wall $K_{AE}$ become higher. This is the result of deformation of the wall in the direction of the soil, leading locally to passive earth pressure coefficients.

### 6.3.4. Conclusion

Comparing results of the dynamic analysis with the pseudo-static analysis generally the pseudo-static analysis leads to a conservative outcome. The occurring bending moments and displacements are considered to be too high. Regarding this case study especially the passive resistance is overestimated during the dynamic phase, which has although a favourable influence on the maximum field moment. Applying the horizontal seismic coefficient according to the design codes the extreme values are overestimated. Using the reduced value as was proposed in section 4.2.2 the extreme values are better predicted, however especially due to an overestimation of the passive resistance.

Analysing the results of the reference earthquake motions, reducing the value for the seismic coefficient is not valid for all seismic motions. Since the response of the structure for the Kocaeli Earthquake is larger compared the the Nihonkai Chubu Earthquake this would lead to an underestimation of the extreme values.

In the pseudo-static analysis the only input parameter for the seismic motion is the peak ground acceleration (PGA). As Habets, C.J.W. (2015) already concluded the applicability of the PGA as the predominant input parameter is limited, since other aspects as duration and frequency content of the earthquake are not taken into account. This conclusion is confirmed by findings in this research. It is observed that the response of soil and structure can be significantly different for different type of earthquake motions with the same peak ground acceleration. For that reason it is not possible to suggest a general reduction factor for the seismic coefficient based on this analysis. To be able to apply the pseudo-static analysis independent from the characteristics of
the earthquake motion an upper boundary value has to be applied, resulting in most cases in a conservative approach. Application of the pseudo-static analysis in performance-based-design gives no insight. In this design philosophy maximum allowable deformations of the structure are the limiting requirements, and the pseudo-static analysis is not well able to predict the development of these deformations.

The pseudo-static methods are however able to give a first indication of failure or non-failure of the structure, based on occurring internal forces. Non-failure of the structure according to the pseudo-static methods is a safe approximation, but could lead to over-dimensioning. Failure according to the pseudo-static analysis does not necessarily mean that the structure fails, however more sophisticated analysis is required to assess the safety of the structure.

Some remarks have to be made to the comparison between results of the dynamic analysis and the pseudo-static analysis:

- **Interpretation soil investigation and soil parameters**
  The found maximum displacement according to the PLAXIS 2D model is somewhat higher compared to observations from the field. This can be partially explained by the fact that the SPT-tests are conservatively interpreted, because the exact location of the tests is not known. It is possible that at the cross section of interest the SPT-values are somewhat higher, leading to more favourable soil conditions. Besides that the soil parameters are determined based on design codes, which provide rather conservative values for soil properties. Both aspects may lead to an underestimation of the soil strength, resulting in somewhat larger displacements.

- **No dynamic water pressure in dynamic analysis**
  In the pseudo-static analysis a dynamic water pressure according to Westergaard was included, which is unfavourable for the distribution of the internal forces in the wall. Within the dynamic analysis this dynamic water pressures are not included resulting in a slight underestimation of the loading during the dynamic phase.

- **Other parameter set pseudo-static analysis and dynamic analysis**
  The SPT-tests performed at Ohama No.1 Wharf are used as input for the determination of the parameter sets for both type of analyses. Parameters were subsequently obtained from commonly used correlations. Both analyses require however different input parameters, leading possibly to a slightly different interpretation of the soil behaviour. It is not said that this is the case, it is an uncertain factor and could partially explain the large difference in passive resistance.

- **Effects of water**
  In the dynamic analysis in PLAXIS effects of dynamic water pressures are not included. This could be solved by manually lower the water level in front of the quay wall according to theory by Westergaard. Also effects of movement of water with shaking of the structure, added mass, is not accounted for in the calculation.

- **Liquefaction not yet included in this analysis**
  Conclusions drawn about both the dynamic analysis and the pseudo-static analysis are based on a case where liquefaction did not occur. Observations about the performance of the analyses are therefore not necessarily also valid for a case where liquefaction occurs. More research into the performance of the dynamic analysis including effects of liquefaction is therefore needed.
6.4. Dynamic analysis Ohama No.2 Wharf

Once calibration of the UBC3D-PLM model for the element tests is finished, the whole system will be considered. A dynamic analysis of the anchored quay wall at Ohama No.2 Wharf is performed including the liquefaction potential of the soil. The structure is modelled in PLAXIS 2D and the constitutive material models HSsmall and UBC3D-PLM are adopted to model the soil behaviour. Since the quay wall at Ohama No.2 Wharf is failed due to occurrence of liquefaction the UBC3D-PLM model is used to account for effects of liquefaction. The HSsmall model is assigned to the deeper soil layers not vulnerable to liquefaction and the UBC3D-PLM model is assigned to the shallow liquefiable layers.

6.4.1. Static analysis

Before performing a dynamic calculation first a static calculation is performed for the Ohama No.2 Wharf to determine the initial stress state in the soil. This significant influence on the liquefaction behaviour of the soil as is elaborated in section 5. It is however also observed that these initial conditions have large influence on the model performance. Since adjustments to the model parameters have been suggested to improve performance of the model for certain stress states, it is crucial to have insight in these stress states prior to the dynamic loading event.

It is shown in section 5 that effects of the initial overburden pressures on the liquefaction potential is well captured by the model. For that reason adjustments are not made in the calibration procedure of the model parameters. On the other hand the effects of both initial lateral earth pressure coefficient ($K_0$) and the initial static shear stress ratio ($\alpha$) are not necessarily well captured by the model and adjustments to the calibrated parameter sets are suggested. It is for that reason important to have insight in the development of $K_0$ and $\alpha$ around the anchored quay wall to be able to suggest effective adjustments to the parameterset for specific zones around the structure.

In Figure 6.23 an overview is given of the initial lateral earth pressure coefficient around the anchored quay wall after construction for the upper two soil layers. The neutral $K_0$ value is around 0.50, while in the passive soil wedge of the sheet pile wall $K_0$ values of 2.0 and higher are present. At the active side of the sheet pile wall the $K_0$ value has decreased to 0.40. Similar behaviour could be observed around the anchor pile, where in the passive soil wedge $K_0$ values of 0.65 are present and on the active side $K_0$ values of 0.45. At the top of the sheet pile higher $K_0$ values are present, which is a result of the pulling force of the anchor keeping the wall at its place. The passive resistance above the anchor is mobilized due the deformation of sheet pile which moves above the anchor in the direction of the soil.

![Figure 6.23: Overview of lateral earth pressure coefficients ($K_0$) around anchored quay wall at Ohama No.2 Wharf after construction.](image)
In Figure 6.24 an overview of the distribution of the static shear ratio around the anchored quay wall after construction is presented. Only the upper two soil layers are considered, since these are vulnerable to liquefaction. As is already mentioned changes in $K_0$ and $\alpha$ could not be seen independent from each other. At locations where the $K_0$ changes a lot with respect to initial state, also significant shear stresses are found corresponding to rotation of the principal stress directions.

Once the distribution of $K_0$ and $\alpha$ around the structure are, zones around the structure are specified corresponding to certain stress states. Distinction is made between zones where the $K_0$ has increased with respect to the initial neutral value (indicated with P, passive) and zones where $K_0$ decreased (indicated A, active). Zones where the $K_0$ did not changed are indicated as neutral (N). Besides the zones are also classified on the presence of static shear stresses ($\alpha$) or not. In Figure 6.25 the identified zones are presented.
6.4.2. Parameter selection

Defining zones around structure

In this section zones are linked to typical element tests with corresponding initial conditions.

For the neutral zones the lateral stress does not change and only cyclic shear stresses load the sample, which corresponds to loading conditions corresponding to cyclic DSS tests. In the active zones the lateral stress decreases, corresponding to a lateral extension test. In undrained conditions a lateral extension test shows similar behaviour as an axial compression test. This last test is considered and calibrated in the earlier sections. At last in the passive the lateral stresses increase, which corresponds to a lateral compression test. Again in undrained conditions the loading conditions for a lateral compression test are similar to an axial extension test, which is calibrated in the previous section. In Figure 6.26 the adopted element tests including conditions for the specified zones are indicated.

The adopted zones are specified based on static analysis. As a result of the dynamic loading it is expected that the shape of these zones will change, active soil wedges become larger and passive soil wedges smaller. An iterative procedure is required to analyse the variation of the shapes of the zones. It is however assumed that the static analysis gives a reasonable approximation of the shape of the zones.

Figure 6.26: Indication of zones corresponding to typical element tests with corresponding conditions.

Model parameter selection

In chapter 5 calibrated parametersets for both Ohama and Gaiko sand are derived for undrained cyclic DSS tests and cyclic triaxial tests and adjustments to these parametersets are suggested to account for varying conditions. In Figure 6.26 for each zone a corresponding element test is adopted including conditions and an calibrated parameterset is adopted specifically for the prevailing conditions. In Appendix G all calibrated parameter sets adopted in the dynamic analysis of Ohama No.2 Wharf are presented.

Initially a value of 0.02 is adopted for the \( f_{ac_{post}} \) factor for stress states without initial static shear. When these values are however adopted the model experienced numerical issues, no equilibrium within the set error margin is found. These numerical issues are also seen in calibration of the element tests with the presence of static shear stresses. For that reason a \( f_{ac_{post}} \) factor of 1.0 is adopted for all zones, solving the numerical errors.

In Figure 6.27 the different zones defined in the numerical model are presented. As can be seen the zones are only specified for the upper two layers, since these are vulnerable to liquefaction.
6.4.3. Model configuration
In this section several modelling aspects are elaborated, which are required for properly modelling the system.

• **Model configuration** The anchor of the sheet pile wall at Ohama No.2 Wharf consists of two anchor piles close behind to each other. Initially these piles are both modelled in PLAXIS corresponding to the actual situation. It is however observed that the soil behaviour in the small space between those piles is not modelled well. Extreme excess pore pressures are observed corresponding to large deformations and stress levels. It is therefore decided to model one equivalent pile exclude the small space between both anchor piles in the model. This is also done in section 4.2 for the application of the pseudo-static analysis. It is considered to be a reasonable schematization since the pile just behind the front pile does not experience much soil resistance since it they are placed so close to behind each other.

For the upper two layers the UBC3D-PLM model is adopted and model parameters are derived in last section. The deeper layers are not vulnerable to liquefaction and for these layers the H5small constitutive material model is adopted. The soil profile and model parameters derived in section 6.2.2 are also adopted in this model.

Sheet pile wall and anchor wall are modelled as plates using elastic material model, both elements also have end bearing in order to prevent settlement due to it's own weight. The tie-rod is modelled as a node-to-node anchor connecting the sheet pile wall with the anchor wall and has a centre-to-centre distance of 2.0 meters. Properties of the structural elements in the model are already determined in section 4.2.3 and are also applied in the PLAXIS model. Again Rayleigh damping is added to the structures.

• **Boundary conditions, mesh size and time steps**
At both sides of the model free field boundary conditions are applied, in order to prevent waves from reflecting back into the model. After modelling all the construction stages an extra stage is implemented in the model where at both side boundaries drained layers are applied to prevent complete loss of support at the boundaries. At the bottom boundary again a compliant base boundary is applied as is already explained in section 6.2. To prevent the numerical model boundaries from having influence on the model results a model width of 160 meters is adopted.

In the model first the construction stages are included, to realistically model the stress states present in the soil before starting the dynamic phase. First the sheet pile wall and anchor wall are installed, subsequently the soil is excavated in layers in front of the wall to the required depth. In Figure 6.28 the total model is presented. For more background on model properties reference is made to section 6.2.2 and section 6.3.1.
6.4. Dynamic analysis Ohama No.2 Wharf

6.4.4. Results soil deformation

Before considering the response of the structure first soil behaviour predicted by the model is analysed. Same deformation pattern is observed as for the Ohama No.1 Wharf, where liquefaction not occurred. Deformations are however larger and the deformed soil wedge extends deeper, since the structure deformed more. Largest soil deformations are found at the top of the sheet pile wall and anchor wall, see Figure 6.29. At locations of maximum soil displacement also maximum displacement of structure is found.

Figure 6.29: Horizontal deformation pattern of soil after Nihonkai Chubu Earthquake.

Figure 6.30 shows distribution of excess pore pressure ratio $r_u$ around the anchored quay wall after the Nihonkai Chubu Earthquake. The upper layer (Ohama Sand) almost completely liquefied. This is in correspondence with results of method by Idriss, I.M. and Boulanger, R.W. (2008) performed for the soil profile of Ohama No.2 Wharf in section 4.2.1. The layer containing Gaiko Sand did not liquefy according to the method by Idriss, I.M. and Boulanger, R.W. (2008), although the margin to liquefaction is small. Contrary to this the UBC3D-PLM model predicts complete liquefaction of this layer over a large area between the anchored quay wall and anchor wall. Behind the anchor wall soil just below the Ohama sand liquefies, but $r_u$ values decreases with depth and distance from the anchor wall.

In the active soil wedge of both the sheet pile wall and the anchor wall, zones are present where the excess pore pressure ratio $r_u$ does not reach to 100%. Locally even slightly water under pressures are predicted by the model.
In Figure 6.31 and Figure 6.32 development of the excess pore pressures over time is presented. The location K in blue corresponds to a location in the upper half of the backfill and location L is located in the lower half. At both locations excess pore pressures initially increase approximately with the same rate up to an $r_u$ of 100% at about 12 seconds. From that moment the soil at location K is fully liquefied and the excess pore pressure remains approximately the same. At location L whole different behaviour is observed, the excess pore pressures start decreasing after 12 seconds and even become slightly positive.
Displacements over time the sheet pile wall are presented in Figure 6.33. Initially displacements remain small, which could be explained by the fact that liquefaction not yet occurred at any location. From seven seconds displacements start to develop and at 12 seconds a large increment is observed. This the effect of the occurrence of liquefaction and the associated decrease of stiffness of the soil. Once displacements of the structure incrementally increase the excess pore pressures at location L decrease.

The decrease of excess pores pressures at location L is linked to the development volumetric strains during the earthquake. In the zones where liquefaction did not fully develop, positive volumetric strains are observed corresponding to extension see Figure 6.34. Due to the increase of volume water pressure could not fully develop to reach liquefaction, effective stresses increased in these zones.

Figure 6.32: Development excess pore pressures at two locations behind sheet pile wall for Nihonkai Chubu Earthquake.

Figure 6.33: Displacement of anchored quay wall over time for Nihonkai Chubu Earthquake.
Due to excessive deformation of the structure excess pore pressures locally decreased and as a result of that the effective stresses increased again. In other words the soil strength increased again during the earthquake due to large deformations. It must be noted that significant deformation is required to initiate this effect, so it is most likely that the structure already suffered damage.

Dakoulas, P. and Gazetas, G. (2008) did research to the development of seismic earth- and water pressures against caisson quay walls using the Pastor-Zienkiewicz elastoplastic constitutive model. In this research also a zone with positive excess pore pressures is observed just behind the structure, triggered by the outward movement of the wall. These observations are supported by the research by Lee, C.J. (2005) where during a centrifuge negative pore pressures are measured just behind a quay wall during dynamic loading.

![Figure 6.34: Overview of volumetric strains ε_v around the anchored quay wall after Nihonkai Chubu Earthquake.](image)

The development of horizontal earth pressures at three locations around the quay wall are presented in Figures 6.35 to 6.37. Points K and L are located in the active soil wedge of the structure and N is within the passive soil wedge. See Figure 6.30 for location of these points around the structure.

At location K the excess pore pressure ratio becomes 100 % after approximately 10 seconds. The effective horizontal stresses decrease to about zero, while the total horizontal pressure increases with the same rate as the excess pore pressures increase. Different behaviour is observed at location L. Again until about 10 seconds the effective stresses decrease because of an increase of excess pore pressures. From that moment on the excess pore pressures decrease again and effective stresses increase. Because of cyclic degradation effective stresses increase to more extent than the excess pore pressures decrease, leading to an increase of total stresses.

The effective and total stress decrease during the seismic event, while excess pore pressures hardly develop at that location. Decrease of the stresses is therefore mainly the effect to cyclic degradation of the soil.
6.4. Dynamic analysis Ohama No.2 Wharf

Figure 6.35: Development effective and total horizontal pressure at location K

Figure 6.36: Development effective and total horizontal pressure at location L

Figure 6.37: Development effective and total horizontal pressure at location L
6.4.5. Response structure

In Table 6.9 and Figure 6.38 extreme values are presented of the structure's response after dynamic loading. The spread in bending moment distribution resulting from different earthquake motions is relatively small to the spread in displacements.

During the earthquake an increase of the earth pressure at the active side and at the same time a decrease of the passive earth pressure is observed. This leads to an increase of maximum field moment, while the embedding generally decrease. The resistance of the soil decreases due to e.g. development of excess pore pressures and cyclic degradation of the soil.

Calculated extreme bending moment just exceeds the elastic bending moment capacity (1146 kNm) of the sheet pile profile, although the plastic bending moment capacity is not exceeded. In the calculation no safety factors or degradation effects due to corrosion are included. It could therefore be that the capacity of the structure is lower than calculated and the capacity is just exceeded.

In Figure 6.38 shows that the resulting displacement of the wall after the Nihonkai Chubu Earthquake is approximately 1.40 meters. In the field displacements of about 1.50 meters are observed, so good fit is found for the amount of displacements.

Table 6.9: Output dynamic analysis Ohama No.2.

<table>
<thead>
<tr>
<th></th>
<th>Minimum bending moment [kNm/m']</th>
<th>Maximum bending moment [kNm/m']</th>
<th>Maximum anchor force [kN/m']</th>
<th>Maximum displacement [m]</th>
<th>Displacement anchor [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>-294</td>
<td>292</td>
<td>279</td>
<td>0.052</td>
<td>0.038</td>
</tr>
<tr>
<td>Nihonkai Chubu</td>
<td>-1056</td>
<td>1189</td>
<td>658</td>
<td>1.40</td>
<td>1.33</td>
</tr>
<tr>
<td>Imperial Valley</td>
<td>-1135</td>
<td>879</td>
<td>665</td>
<td>0.25</td>
<td>0.22</td>
</tr>
<tr>
<td>Landers</td>
<td>-1214</td>
<td>1072</td>
<td>708</td>
<td>0.82</td>
<td>0.79</td>
</tr>
<tr>
<td>Kocaeli</td>
<td>-1119</td>
<td>1039</td>
<td>662</td>
<td>0.84</td>
<td>0.82</td>
</tr>
</tbody>
</table>

Figure 6.38: Output bending moment distribution and displacements of anchored quay wall at Ohama No.2 Wharf.
Comparison with pseudo-static methods

Comparison of results of pseudo-static and dynamic analysis are presented in Figure 6.39 and Table 6.10. In these figures the results of the dynamic analysis and pseudo-static analysis for the Nihonkai Chubu Earthquake are and compared.

In the pseudo-static analysis two loading levels are applied, one based on design codes ($k_h = 0.206$) and one reduced horizontal seismic coefficient ($k_h = 0.100$). Differences in the extreme bending moments between the dynamic analysis and the pseudo-static analysis are considerable. The pseudo-static analysis results in significantly larger fixed bending moment, which is explained by the overestimation of the passive resistance in the pseudo-static analysis. This is also observed in the dynamic analysis of Ohama No.1 Wharf.

Another reason why the fixed bending moment is so large is because the stiffness of the anchor in the pseudo-static analysis is much lower compared to the stiffness of the soil the sheet pile is embedded in. As can be seen in Figure 6.10 the anchor forces in the dynamic analysis are significantly larger indicating a higher stiffness.

Since the stiffness of the anchors in the pseudo-static analysis are calibrated on the amount of observed deformation, the maximum deformations according to both methods show a reasonable fit.

The reduced horizontal seismic coefficient $k_h$ resulted in a better fit with the dynamic analysis than the $k_h$ based on design codes, which is also the case at Ohama No.1 Wharf.

Table 6.10: Comparison between extreme outcomes of dynamic analysis and pseudo-static analysis for Nihonkai Chubu Earthquake.

<table>
<thead>
<tr>
<th></th>
<th>Minimum bending moment [kNm/m']</th>
<th>Maximum bending moment [kNm/m']</th>
<th>Maximum anchor force [kN/m']</th>
<th>Maximum displacement anchor [m]</th>
<th>Displacement anchor [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLAXIS - Static</td>
<td>-294</td>
<td>292</td>
<td>279</td>
<td>0.052</td>
<td>0.038</td>
</tr>
<tr>
<td>Dsheet - Static</td>
<td>-378</td>
<td>406</td>
<td>129</td>
<td>0.052</td>
<td>0.035</td>
</tr>
<tr>
<td>Ratio (Pseudo-static / Dynamic)</td>
<td>1.29</td>
<td>1.39</td>
<td>0.46</td>
<td>1.00</td>
<td>0.92</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Minimum bending moment [kNm/m']</th>
<th>Maximum bending moment [kNm/m']</th>
<th>Maximum anchor force [kN/m']</th>
<th>Maximum displacement anchor [m]</th>
<th>Displacement anchor [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLAXIS - Nihonkai Chubu</td>
<td>-1056</td>
<td>1189</td>
<td>658</td>
<td>1.40</td>
<td>1.33</td>
</tr>
<tr>
<td>Dsheet - $k_h = 0.100$</td>
<td>-558</td>
<td>3680</td>
<td>238</td>
<td>1.70</td>
<td>1.54</td>
</tr>
<tr>
<td>Ratio (Pseudo-static / Dynamic)</td>
<td>0.53</td>
<td>3.10</td>
<td>0.36</td>
<td>1.21</td>
<td>1.15</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Minimum bending moment [kNm/m']</th>
<th>Maximum bending moment [kNm/m']</th>
<th>Maximum anchor force [kN/m']</th>
<th>Maximum displacement anchor [m]</th>
<th>Displacement anchor [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLAXIS - Nihonkai Chubu</td>
<td>-1056</td>
<td>1189</td>
<td>658</td>
<td>1.40</td>
<td>1.33</td>
</tr>
<tr>
<td>Dsheet - $k_h = 0.206$</td>
<td>-556</td>
<td>4543</td>
<td>256</td>
<td>2.66</td>
<td>2.44</td>
</tr>
<tr>
<td>Ratio (Pseudo-static / Dynamic)</td>
<td>0.53</td>
<td>3.82</td>
<td>0.39</td>
<td>1.90</td>
<td>1.83</td>
</tr>
</tbody>
</table>

$M_{yield,plastic} = 1.146 \text{ kNm/m'}$ Observed displacement 1.50 m

$M_{plastic} = 1.357 \text{ kNm/m'}$
136

6. Dynamic analysis Akita Port

Figure 6.39: Comparison bending moment distribution and displacements obtained from dynamic analysis and pseudo-static analysis for Nihonkai Chubu Earthquake

6.4.6. Conclusion

Main goal of the dynamic analysis of the Ohama No.2 Wharf is accurately predicting the onset of liquefaction in the upper layers and corresponding effects on the dynamic behaviour of the structure. By defining the zones corresponding to typical stress states and assign calibrated parameter sets accurate results for the independent zones are obtained. In this way limitation of the model to predict liquefaction resistance for varying stress states is solved. Accurate results for each zone individually must lead to more accurate results of the total system.

The calibrated dynamic model predicted in large areas of the backfill liquefaction, which is in correspondence with observations during the earthquake. The exact locations where liquefaction occurred in the field are not known, although large scale liquefaction is documented. Model results also largely correspond to the results obtained with the method by Idriss, I.M. and Boulanger, R.W. (2008) to determine the factor of safety against liquefaction.

The dynamic analysis using UBC3D-PLM provided insight in the development of excess pore pressures over the whole soil system and especially behind the anchored quay wall. It is observed that just behind the anchored quay wall and the anchor pile zones are present where the excess pore pressures developed, but as a result of large deformation of the structure these excess pore pressure dissipated again. Due to large deformations a restoring effect occurs in the active soil wedge, which has a favourable influence on the lateral earth pressures.

The failure behaviour of the structure is well predicted by the model. Loss of resistance of the anchor wall lead to excessive displacements of both the anchor and sheet pile wall. The observed displacements of about 1.50 meters are in line with the maximum calculated displacement of 1.40 meters.
Comparison with the pseudo-static methods lead to the conclusion that bending moment distribution according to the pseudo-static methods is not realistic. Regarding this case study especially the passive resistance is overestimated during the dynamic phase. The stiffness of the anchor wall is underestimated in the pseudo-static method, leading to even larger stiffness differences between embedment and anchor. Applying the horizontal seismic coefficient according to the design codes the extreme values are overestimated. Using the reduced value as is proposed in section 4.2.2 the extreme values are better predicted, however still an overestimation of the passive resistance is found.

It can be concluded that the calibrated dynamic analysis including the UBC3D-PLM model for the liquefiable layers lead to results that show good correspondence with observations in the field. Contrary to the satisfying results of the dynamic analysis application of the pseudo-static to exaggerated results. Design codes like the Eurocode provide tools to include the excess pore pressure ratio $r_{u}$ in development of earth pressures according to pseudo-static analysis. Application of this methods however lead to unrealistic results. When the anchor stiffness drops because of liquefaction in the backfill, the structure behaves like a clamped beam because of overestimated passive earth pressures. The pseudo-static method could be fit on the amount of allowable displacements, leading to an overestimation of the extreme the bending moments. On the other hand the displacements are underestimated when the fit is done on the amount of allowable bending moments. It is concluded that pseudo-static methods are therefore not suitable for performance based design if excess pore pressures are expected.

Some remarks on the dynamic analysis have to be made:

- **Effect of $f_{ac\text{post}}$**
  For all parametersets of the UBC3D-PLM model a post liquefaction factor of 1.0 is adopted, since numerical errors are experienced for initially suggested $f_{ac\text{post}}$ factor of 0.02. This factor however has large influence on the amount of strains developed after liquefaction is reached in the model. The maximum displacements in the model are predicted well, but it worth while to analyse what the influence of the $f_{ac\text{post}}$ on the maximum displacements is.

- **No dissipation of water pressures**
  In PLAXIS dissipation of water pressures in time is not taken into account. The duration of the earthquakes varies from 25 seconds to 50 seconds so already some dissipation may be expected during the earthquake. Results found for the development of excess pore pressures may therefore be conservative since dissipation is not accounted for in the model.

- **Conclusions only valid for this case**
  It must be stated that the conclusions done about the application and accuracy of the UBC3D-PLM are only valid for this case. Since the model is very sensitive the conclusions are not necessarily true for other cases. Following the same calibration procedure as done in this research for other cases will give more insight in the performance of the model for the specific case.
Procedure dynamic analysis Eemshaven

In this chapter link is made between the performed research to the case study in Akita Port and applicability of the followed procedure for the assessment of an anchored quay wall in the Eemshaven in Groningen.

In the problem description of this research it was already mentioned that the seismic activity in Groningen due to the gas production lead to seismic loading of quay wall structures in the Eemshaven. Infrastructure in the Eemshaven was originally not designed for these dynamic loads. Since a trend of increasing seismicity is observed it is advisable to account for in future design and for the assessment of the safety of existing structures.

7.1. Introduction

Recently research was done by Haan, de, F.S. (2016) to dynamic loading of existing quay structures in the Eemshaven by induced earthquakes. A safety assessment was performed of an existing quay structure in the Juliana basin to indicate the effect of an induced earthquake on the structure. The most critical part of the structure was the capacity of the anchors. Resistance of the grout anchor reduced with about 25% during an earthquake. The factor of safety of the analysed structure is determined to be 0.98 during the design earthquake, considering a combination of soil and water load, design surface load and design mooring load. This is however an extreme loading combination deterministically determined, therefore a probabilistic approach is suggested to obtain the decisive load combination (Haan, de, F.S., 2016).

Liquefaction was not considered in dynamic analysis of the anchored quay wall. In this research it is however shown that occurrence of liquefaction has large influence on the performance of the structure. Inclusion of liquefaction potential is therefore considered to be essential in dynamic analysis of an anchored quay wall.

In research by Haan, de, F.S. (2016) liquefaction potential of the backfill of the anchored quay wall at the Julianahaven was determined according to the method by Robertson (Robertson, P.K. and Cabal, K.L., 2015), see Figure 7.1. From 6.0 m to 10.0 m below NAP the FoS against liquefaction varies between 1.5 and 1.25, indicating that excess pore pressures could be expected (Nederlands Normalisatie-instituut, 2015). The layer is present halfway the retaining part of the quay wall and has therefore effects on performance of the structure. Possibly the anchor capacity is also affected by excess pore pressures in this layer. It is therefore strongly advised to consider development of excess pore pressures and effects on the structure in more detail. Excluding liquefaction in the dynamic analysis leads possibly to an underestimation of the loading and an overestimation of the resistance.

In the Eurocode and NPR 9998 pseudo-static methods are prescribed where effects of excess pore pressures could be included. It is however shown in this research that application of this method including liquefaction effects lead to unrealistic outcomes. To prevent conservative outcomes it is advised to perform a dynamic analysis including liquefaction potential to obtain insight in the development of excess pore pressures and dynamic behaviour of the structure.
7.2. Wilhelminahaven

Several basins located in the Eemshaven, as indicated in Figure 7.2. One of these basins is the Wilhelminahaven, located at the eastern part of the port. In this basin an anchored quay wall with relieving platform is constructed, which is considered in this section. Seismic loading is not considered in design. Therefore in this chapter a procedure is prescribed how to perform a proper dynamic analysis including liquefaction potential.

The retaining structure consists of a combined wall, with on top a concrete L-shaped relieving platform. This structure is supported by both combi wall and by vibropiles specially installed for supporting the concrete structure. Finally MV-piles are installed which function as anchor to hold the structure in place. In Figure 7.3 a cross section of the structure is presented. It is assumed that the soil profile of the backfill of the anchored quay wall at the Wilhelminahaven is similar to the soil profile at the Julianahaven.
7.3. Description of procedure

In order to assess the dynamic behaviour of the anchored quay wall in the Wilhelminahaven a dynamic analysis using an effective stress finite element model provides the best insight in the actual soil structure interaction. In this section the procedure is described how to set up proper calibrated dynamic analysis including liquefaction potential using the UBC3D-PLM material model. Important aspects and lessons learned from the performed dynamic analysis on the Akita Port case are included in this procedure.

In Figure 6.1 steps in the procedure to set up a dynamic calculation are presented in a flowchart.

- **Determine soil profile and liquefaction potential of layers.**
  Initially soil profile and properties are defined by performing CPT or SPT. A first estimation of the liquefaction potential of soil layers can be made by applying cyclic stress approach methods (for example Idriss & Boulanger (2008) or Robertson (2015)). The UBC3D-PLM material model is assigned to the potential liquefiable layers in the dynamic calculations.

- **Perform static calculation in PLAXIS 2D.**
  The Hardening Soil small strain constitutive material model is adopted for all layers in static calculations, since UBC3D-PLM is not suitable for static calculations (Petalas, A. and Galavi, V., 2013). Initial model parameters of HSsmall are based on correlations by Brinkgreve, R.B.J. et al. (2010), which requires the relative density as input. After modelling the structure and defining the soil profile, model parameters, model boundaries and construction stages the static calculation can be performed.

- **Site response analysis**
  In Groningen earthquakes are triggered at about 3 km depth. To obtain the earthquake time motion at the depth of interest a site response analysis has to be performed. This analysis is location specific and requires an input signal.

The KNMI has provided results of the Probabilistic Seismic Hazard Assessment (PSHA) for the induced seismicity in Groningen using version 2 of the Ground Motion Prediction Equation (GMPE) for Groningen. In Figure 7.4 a schematization of the methodology is presented.
Accelerations are provided at two vertical levels:
- **350 meters depth** - the GMPE V2 model provides acceleration at the base of the Upper North Sea formation.
- **Surface level** - By a location dependent amplification factor effects of the shallow geology on the acceleration are included, leading to the peak ground acceleration.

Seismic motions of induced earthquakes in Groningen have other characteristics compared to tectonic earthquakes. Typically earthquakes in Groningen have large peaks and have relatively short time duration. Measured time motions in Groningen have to be scaled to the design acceleration and can subsequently be adopted in the dynamic analysis, as long as requirements to the signals stated in the NPR 9998 are met (Nederlands Normalisatie-instituut, 2015).

Figure 7.4: Illustration of methodology in the hazard V2 model (Spetzler, J. and Dost, B., 2016)

For the dynamic analysis a motion has to be defined at the base of the model (tens of meters depth). The depth of this layer must be a logical choice based on local deep bore holes. At this moment the NPR 9998 advises to define the base at depth where the shear wave velocity $v_s$ is equal to 300 m/s, when no clear rock layer is present (Nederlands Normalisatie-instituut, 2015). It is important to model at least one meter of this base to properly initiate the signal in the model.

The motion provided by the KNMI has to be translated to the depth of the base layer. Either from -350 meters to the base of the model or by deconvolution, translating the surface motion to the depth of interest. This requires specific knowledge of the local soil deposit, which could be obtained with soil investigation. One option is to perform a site response analysis and determine the motion at depth of interest. The second option is to wait for the committee responsible for the NPR 9998 to define a reference motion to be used for a dynamic analysis.

Generally two different methods are distinguished for location specific site response analysis. First the equivalent linear and secondly the more complex non linear site response analysis. Reference is made to section 6.2 for background of these methods.

- **Perform laboratory tests to determine liquefaction resistance of soils for relevant loading conditions.** Stress states around the structure can be determined based on the static calculation. Since the initial ($K_0$) and the static shear ratio ($\alpha$) have significant influence on both liquefaction resistance of soil and performance of the UBC3D-PLM model, it is advised to calibrate the model on stress states that are present in the field. Loading levels are obtained from the site response analysis.

Liquefaction resistance of the local soils can be determined by performing laboratory element tests. Initial conditions of these laboratory tests have to be fit to stress states around the structure and loading levels according to the site response analysis, to obtain the most accurate liquefaction resistance of the actual soil layers.
Soil behaviour around the structure can be simulated by element tests. Depending on the location soil behaviour corresponds the best to either an undrained cyclic triaxial test or to an undrained cyclic direct simple shear test. It is advised to perform both element tests to obtain the most accurate results. Correlations between test results are available, but introduce extra uncertainty in the obtained liquefaction resistance.

- **Define regions corresponding to stress states**
  Zones corresponding to specific stress state can be distinguished, based on the static analysis. In section 6.4.2 the same was done for the Akita Port case. Soil behaviour in these zones corresponds to a behaviour in specific element test, depending on the loading condition. The UBC3D-PLM method is not well able to account for varying stress states, and has to be calibrated for one condition. For that reason the model is calibrated for each stress state on corresponding laboratory tests, resulting in a unique parameter set for each zone.

- **Calibrate model parameters to results laboratory tests for varying initial conditions**
  Initial model parameters for the UBC3D-PLM constitutive material model could be determined applying either the method by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) or the procedure by Souliotis, C. and Gerolymos, N. (2016).

  The method by Beaty, M.H. and Byrne, P.M. (2011) is based on corrected SPT values. In the Netherlands are mostly CPT performed, which have to be translated to corresponding normalized blowcount with available correlations. The procedure by Souliotis, C. and Gerolymos, N. (2016) requires both normalized blowcount and relative density as input.

  The model is very sensitive to the input parameters and procedures only provide initial estimation of the model parameters. Therefore the model has to be calibrated to results of the laboratory tests. For each initial stress state and loading condition, the initial parameter sets are calibrated. Finally for each zone a calibrated parameter sets is obtained accurately predicting the onset of liquefaction in that zone.

- **Analyse model results**
  Once the model parameters, input signal and model properties are defined the dynamic analysis can be performed. Results have to be analysed if these are consistent and to what extent they are in line with expectations. Important results are development of displacement of the structure, excess pore pressures and stresses and strains in the soil.

  Insight is provided in the behaviour of the structure during seismic event. Results can be used to assess if mitigation measures are required, and if so to prescribe the most effective measures.
Conclusions and recommendations

8.1. Conclusions
In this research main objective was to obtain more insight in effects of liquefaction on dynamic earth pressures on anchored quay walls and corresponding dynamic behaviour of this structure. The performance of different evaluation methods is analysed in order to investigate the accuracy of these methods and applicability for seismic design of anchored quay walls. Several sub conclusions could be distinguished corresponding to the sub questions, ultimately leading to the final conclusion of this research.

How do pseudo-static design methods account for effects of excess pore pressures on dynamic (earth) pressures?
Basis for determination of dynamic earth pressures on retaining structures using pseudo-static methods is the Mononobe-Okabe method. Accounting for effects of excess pore pressures can be done according to two approaches. Either by adapting the seismic inclination angle $\theta$ (increased loading) or by adapting the internal friction angle $\varphi$ (decreased resistance). Both approaches lead to different trends of the dynamic earth pressure coefficient for increasing seismic coefficient $k_h$, especially for the passive earth pressure coefficient.

To what extent are pseudo-static methods capable of realistically predicting dynamic earth pressures on anchored quay walls, with and without effects of development of excess pore pressures?
The anchored quay walls in Akita Port are assessed by applying a pseudo-static method. It is concluded that the Mononobe-Okabe method for the evaluation of the anchored quay wall without liquefaction results in a reasonable fit with the observations adopting a reduced seismic coefficient. The maximum bending moments in the sheet pile wall are reasonably approximated, however displacements are exaggerated. Generally the passive earth pressure is overestimated, leading to an altered bending moment distribution. Although limitations of the analysis, the method is suitable to determine if failure is expected.

When effects of liquefaction are included in the pseudo-static method this leads to a strong conservative approximation. Main problem is the difference in stiffness between embedding and anchor in case of liquefaction in the backfill, leading to an exaggerated bending moment distribution. Passive earth pressures are overestimated, while the stiffness of the anchor drops. Since Mononobe-Okabe is a limit equilibrium method, it is not well able to capture progressive failure. Application of the pseudo-static analysis for anchored quay walls in liquefiable layers does not lead to satisfying results.

How can the HSsmall and UBC3D-PLM constitutive material models be calibrated to realistically model the soil behaviour around an anchored quay wall under seismic loading?
For soils not vulnerable to liquefaction the HSsmall constitutive material model is adopted, which has the advantage that it accounts for small strain stiffness. Initial model parameters are obtained by proposed relationships (Brinkgreve, R.B.J. et al., 2010) based on relative density of the soil, subsequently damping parameters are fit to theoretical damping curves representing the considered soil type the best.
The user defined UBC3D-PLM constitutive material model is adopted for modelling behaviour of potential liquefiable soils. Two parameter selection procedures (first procedure by Beaty, M.H. and Byrne, P.M. (2011) & Makra, A. (2013) and secondly the procedure by (Souliotis, C. and Gerolymos, N., 2016)) are applied and calibrated to liquefaction resistance curves from laboratory of local sands. Plastic stiffness parameters in the model control the plastic strain rate and mainly responsible for development of excess pore pressures. These are the most effective parameters to be adjusted to fit the liquefaction resistance curves.

Around an anchored quay wall stress states can be defined that correspond to loading paths according to an undrained cyclic direct simple shear test or undrained cyclic triaxial test. In the UBC3D-PLM model these element tests are reproduced, which show after calibration reasonable fits with laboratory results. Variation of the initial stress states in have influence on the liquefaction potential but also on the model performance. Liquefaction potential is underestimated by the model for presence of initial static shear. Model parameters are calibrated to match empirical curves for the considered initial static shear ratios. By calibrating model parameters for specific stress states, reasonable fits for the amount of cycles to liquefaction are obtained.

To what extent is the UBC3D-PLM model capable predicting the development of excess pore pressures together with the loss of strength of the soil and the corresponding dynamic behaviour of the anchored quay wall?

A calibrated dynamic calculation using the HSsmall constitutive material model lead to reasonable accurate for the case without liquefaction. Bending moment distribution and displacements are in line with observations. Different earthquake motions lead to relatively large spread in displacement of the structure, less differences are found for bending moment distribution.

In dynamic analysis of Ohama No.2 Wharf the onset of liquefaction in the upper layers and corresponding effects on the dynamic behaviour of the structure are with reasonable accuracy predicted with the UBC3D-PLM constitutive material model. By defining zones with typical stress states in the soil and assigning to each zone a calibrated parameter sets, accurate results for the independent zones are obtained. The limitation of the model to deal with different stress states is solved by defining zones with calibrated parameters for that specific stress state. The calibrated dynamic model predicts liquefaction in large part of the backfill, which is in correspondence with observations during the earthquake and with results obtained with the empirical method by Idriss, I.M. and Boulanger, R.W. (2008).

It is observed that just behind the quay wall and the anchor piles zones are present where excess pore pressures developed, but as a result of large deformation of the structure these excess pore pressure dissipated again. It is concluded that due to large deformations and corresponding extension of the soil, pore pressures decrease leading to a restoring effect.

The failure behaviour of the structure is well predicted by the model. Loss of resistance of the anchor piles lead to excessive displacements of both the anchor and sheet pile wall. The predicted bending moments and maximum displacement are reasonable accurately predicted well in the model. It can be concluded that the calibrated dynamic analysis including the UBC3D-PLM model for the liquefiable layers lead satisfying results for the Akita Port case.

Which procedure has to be followed in the structural assessment of the anchored quay wall with relieving platform in the Wilhelminahaven in Groningen subjected to an induced earthquake?

The procedure described for the Akita Port case can be followed for calibrating and performing a dynamic analysis including liquefaction potential for the Wilhelminahaven. Typically characteristics of the input motion in the Wilhelminahaven are different to the Akita Port case. In addition the site response analysis requires extra attention, since the motion has to be translated to the base of the model and the bedrock layer necessarily a clear separate layer.

Ultimately answer is given to the research question. Dynamic active earth pressures increase and passive earth pressures decrease due to seismic loading. Occurrence of (partial)liquefaction leads to an extra increase of the total active pressures, and on the other hand to an extra decrease of passive earth pressures. Main remarks about the performance of different analysis methods are presented in Table 8.1.
Table 8.1: Summary performance pseudo-static and dynamic analysis

<table>
<thead>
<tr>
<th></th>
<th>Pseudo-static analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>No liquefaction</strong></td>
<td>Mononobe-Okabe</td>
</tr>
<tr>
<td></td>
<td>+ Simple method for indication failure/non-failure</td>
</tr>
<tr>
<td></td>
<td>- Overestimation passive earth pressures</td>
</tr>
<tr>
<td></td>
<td>- Conservative extreme values bending moments and displacements</td>
</tr>
<tr>
<td></td>
<td>- Soil structure interaction not accounted for</td>
</tr>
<tr>
<td></td>
<td>- Influence characteristics earthquake motion not accounted for</td>
</tr>
<tr>
<td><strong>Liquefaction</strong></td>
<td>Mononobe-Okabe, adapted:</td>
</tr>
<tr>
<td></td>
<td>- Overestimation passive earth pressures</td>
</tr>
<tr>
<td></td>
<td>- Seismic inclination angle (θ)</td>
</tr>
<tr>
<td></td>
<td>- Exaggerated extreme values bending moments and displacements</td>
</tr>
<tr>
<td></td>
<td>- Internal friction angle (φ)</td>
</tr>
<tr>
<td></td>
<td>- Not suitable to capture progressive failure</td>
</tr>
<tr>
<td></td>
<td>- Soil structure interaction not accounted for</td>
</tr>
<tr>
<td></td>
<td>- Influence characteristics earthquake motion not accounted for</td>
</tr>
<tr>
<td></td>
<td>Dynamic analysis</td>
</tr>
<tr>
<td><strong>No liquefaction</strong></td>
<td>PLAXIS - HSsmall model</td>
</tr>
<tr>
<td></td>
<td>+ Reasonable accurate bending moment distribution and deformation pattern</td>
</tr>
<tr>
<td></td>
<td>+ Insight in soil structure interaction</td>
</tr>
<tr>
<td></td>
<td>+ Influence characteristics earthquake motion</td>
</tr>
<tr>
<td><strong>Liquefaction</strong></td>
<td>PLAXIS - UBC3D-PLM model for liquefiable layers</td>
</tr>
<tr>
<td></td>
<td>+ Reasonably accurate development $p_{excess}$</td>
</tr>
<tr>
<td></td>
<td>+ Indication of bending moment distribution and deformation pattern</td>
</tr>
<tr>
<td></td>
<td>+ Insight in soil structure interaction</td>
</tr>
<tr>
<td></td>
<td>+ Influence characteristics earthquake motion</td>
</tr>
<tr>
<td></td>
<td>- Complex calibration procedure</td>
</tr>
<tr>
<td></td>
<td>- Laboratory tests required for calibration model</td>
</tr>
<tr>
<td></td>
<td>- Post-liquefaction behaviour not well captured</td>
</tr>
</tbody>
</table>

8.2. Recommendations

Applicability of model for Groningen case
In this research only case studies in Akita Port are considered and conclusions are drawn based on findings for these cases. Tectonic earthquake signals are adopted to investigate the dynamic behaviour of soils and structures. In Groningen the induced earthquakes however have different characteristics. Also local conditions in Groningen could have significant influence on the development of these signals and on the resistance of liquefaction. It is therefore recommended to perform similar research for an anchored quay wall in Groningen where soils are present that are vulnerable to liquefaction.

Other configuration of the structure
The considered anchored quay walls in Akita Port both contain an anchor wall. Conclusions on model performance are drawn based on this type of structure. The configuration of this structure is such that it behaves like a cofferdam, because of the relatively small distance between sheet pile and anchor wall. It is recommended to investigate the model performance for flexible sheet pile with other types of anchorage. And analyse the development of excess pore pressures and the performance of considered design methods.
Perform undrained cyclic laboratory tests
As was emphasized in this research experimental element tests are crucial for calibration of the model of the whole system. If the UBC3D-PLM model is used to assess the liquefaction potential of soils around a structure it is recommended to perform different undrained cyclic element tests of local sands using conditions present around the considered structure. By calibrating the model to very local conditions, reasonable model results could be obtained for the onset of liquefaction around the structure. Since less is known about the development of strains after liquefaction it is also recommended to measure strains in the laboratory after liquefaction has occurred, since this has influence on the loading of your structure and settlement of the backfill.

Consider other methods for determination of liquefaction potential
In this research only the cyclic stress approach by Idriss, I.M. and Boulanger, R.W. (2008) was adopted to determine the liquefaction potential in the pseudo-static analysis. Other methods to determine the safety factor against liquefaction could give other outcome leading to other loading conditions around the considered structure.

More extensive research to parameter selection procedure for UBC3D-PLM model
The considered parameter selection procedures to obtain initial model parameters for the UBC3D-PLM model do only depend on the corrected SPT-blowcount or the relative density. Concerning the liquefaction resistance of Ohama sand and the Gaiko sand it is observed that for the Gaiko sand has a higher liquefaction resistance for than the Ohama sand, even for lower relative density. This implies that the relative density is not the only parameter defining the liquefaction resistance, which is for example approved by the method of Idriss, I.M. and Boulanger, R.W. (2008). Although it is expected to be difficult it would be valuable to account for other aspects in selecting the model parameters besides only the corrected blowcount or relative density.

Application of different constitutive models finite element models
In this research only the UBC3D-PLM constitutive material model in PLAXIS 2D was applied. Although the model was calibrated, the model has got its limitations. Several other constitutive models are available, which perform differently compared to the model applied in this research. In order to gain more insight in the performance of the UBC3D-PLM model compared to others and the actual soil behaviour around the anchored quay wall it is recommended also apply other constitutive models and/or finite element models.

At the moment PLAXIS is working on implementation of the PM4SAND constitutive model in PLAXIS. The ability of the model to approximate the stress-strain response at liquefaction is improved compared to the UBC3D-PLM model, which provides a better representation of the soil behaviour.


PIANC (2001). Seismic design guidelines for port structures. report of working group no. 34 of the maritime navigation commission. Technical report, PIANC.


List of Figures

1.1 On the left the path of seismic waves from source to structure leading to local site effects; on the right the focus area of this research. ............................................................... 2

1.2 Flowchart of research .......................................................... 4

2.1 (left) Global distribution of earthquake occurrence associated with plate boundaries (Bosboom, J., 2015). (right) Earthquake density in province of Groningen in 2014-2015 together with the faults in subsoil indicated (TNO, 2015) .......................................................................................... 5

2.2 Different type of seismic waves; body waves and surface waves (source: http://earthquake.usgs.gov/learn/glossary/?term=seismic\%20wave) .................................................................................................................... 7

2.3 Schematization of seismic waves reaching the port structure including the possible hazards (PIANC, 2001) .................................................................................................................. 8

2.4 Failure modes of anchored sheet pile quay wall (PIANC, 2001) ................................................................. 10

2.5 Parameters related to displacements (a) and stresses (b) used for specifying the damage criteria for an anchored quay wall (PIANC, 2001) ................................................................. 11

2.6 Schematization of occurrence of liquefaction and consequences in saturated cohesionless soil (PIANC, 2001) .................................................................................................. 13

2.7 On the left values for $C_E$, $C_S$ and $C_B$ developed by Skempton, A.W. (1986) and on the right energy ratio correction factors by Clayton, C.R.I. (1990) ................................................. 14

2.8 Liquefaction triggering curves for $M = 7.5$ and $\sigma'_v = 1$ atm with a database of case histories processed according to different liquefaction evaluation methods; left SPT-based and right CPT-based (Idriss, I.M. and Boulanger, R.W., 2008). ................................................................. 16

2.9 Excess pore pressure ratio against the factor of safety against liquefaction for $FoS > 1$ (Marcuson, W. et al., 1991). .................................................................................................................. 17

2.10 Possible failure mechanisms related to the occurrence of a liquefiable layer. (a). Failure of the anchor because of decreased soil strength. (b). Structural failure of the sheet pile wall due to load increase in the (partly)liquefied layer. (c). Loss of stability of sheet pile wall because of decrease of passive soil strength ................................................................. 18

2.11 Suggested basic strategy for liquefaction remediation by PIANC (PIANC, 2001) .................................................. 19

2.12 Triangular active soil wedge (left) and triangular passive soil wedge (right) according to Coulomb (Kramer, S.L., 1996) ................................................................. 21

2.13 Active soil wedge (left) and passive soil wedge (right) defined by Mononobe-Okabe (Kramer, S.L., 1996). (Angle of structure with horizontal is indicated with $\theta$, this must be replaced by $\Psi$). ................................................................. 22

2.14 Partially saturated backfill, determination of $\gamma_{eq}$ (Kramer, S.L., 1996) ................................................................. 24

2.15 Contributions of change unit weight soil and water and earth pressure coefficients to total horizontal pressures by increasing excess pore pressure ratio $r_u$ for adjusting $\theta$ approach. ................................................................. 26

2.16 Contributions of change unit weight soil and water and earth pressure coefficients to total horizontal pressures by increasing excess pore pressure ratio $r_u$ for adjusting $\Psi$ approach. ................................................................. 26

2.17 Comparison horizontal seismic coefficients against horizontal acceleration according to different design codes. .................................................................................................................. 28

3.1 Total seismic active earth pressure coefficient (left) and total seismic passive earth pressure coefficient (right) against the horizontal seismic coefficient for varying $\varphi$ and $\delta$ for a dry backfill. ................................................................. 32

3.2 Total seismic active earth pressure coefficient (left) and total seismic passive earth pressure coefficient (right) against the horizontal seismic coefficient for varying $\varphi$ and $\delta$ for a saturated backfill. .................................................................................................................. 33

3.3 Total seismic active earth pressure coefficient (left) and total seismic passive earth pressure coefficient (right) against the horizontal seismic coefficient for varying $\varphi$ and $\delta$ values with $r_u$ varying from 0.25 to 0.75. .................................................................................................................. 34
4.1 Conceptual test setup of the reference experimental centrifuge test case (left) together with the subsequent seismic events (right) (Higuchi, S. et al., 2012). .......................................................... 36
4.2 Maximum bending moments over height of the experimental reference case sheet pile wall for each event combined in one figure. .............................................................................. 37
4.3 Seismic coefficients for case histories of retaining walls at non-liquefied waterfront sites (PIANC, 2001) .......................................................... .......................................................... 
4.4 Schematization of the water pressures and earth pressures including resultant forces. .......................................................... .......................................................... 
4.5 Schematization of the reference case modelled in D-Sheet Piling. ................................................................................... 41
4.6 Deformation of the sheet pile wall at Ohama No.2 Wharf for different cross sections (Iai, S. and Kameoka, T., 1993) .......................................................... 44
4.7 Soil profile and related soil properties based on SPT-test and grain size distribution at Ohama No.1 Wharf. .......................................................... 53
4.8 (Damaged) cross section of anchored quay wall at Ohama Wharf No.1 (Iai, S. and Kameoka, T., 1993) .......................................................... 54
4.9 Soil profile and related soil properties based on SPT-test and grain size distribution at Ohama No.2 Wharf. .......................................................... 54
4.10 Deformation of the sheet pile wall at Ohama No.2 Wharf for different cross sections (Iai, S. and Kameoka, T., 1993) .......................................................... 54
4.11 Soil profile and related soil properties based on SPT-test and grain size distribution at Ohama No.2 Wharf. .......................................................... 54
4.12 Plan of view of the anchor piles including a rough schematization of the influence areas of the piles; on the left for No.1 Wharf and on the right for No.2 Wharf (Iai, S. and Kameoka, T., 1993) .......................................................... 59
5.1 Schematization of the water pressures and earth pressures including resultant forces. .......................................................... .......................................................... 
5.2 Deformation distribution of anchored quay wall at Ohama No.1 Wharf for both considered seismic coefficients. .......................................................... 63
5.3 Deformation distribution of anchored quay wall at Ohama No.1 Wharf for both considered seismic coefficients. .......................................................... 63
5.4 Deformation distribution of anchored quay wall at Ohama No.2 Wharf for both considered seismic coefficients. .......................................................... 66
5.5 Deformation distribution of anchored quay wall at Ohama No.2 Wharf for both considered seismic coefficients. .......................................................... 66
5.6 Deformation distribution of anchored quay wall at Ohama No.2 Wharf considering a wall friction angle of zero. .......................................................... 67
5.1 Schematization of Ohama No.2 Wharf and relevant laboratory tests to modes of shearing on potential surfaces of sliding planes. ................................................................. 70
5.2 Effect of post liquefaction factor on the cyclic resistance ratio (Souliotis, C. and Gerolymos, N., 2016) ................................................................. 73
5.3 Liquefaction resistance curve Ohama Sand and Gaiko Sand at Akita Port (Iai, S. and Kameoka, T., 1993). ................................................................. 74
5.4 Schematisation of an undrained cyclic direct simple shear test. ......................... 77
5.5 Liquefaction resistance curves by UBC3D-PLM model for Ohama sand based on initial model parameters for cyclic DSS test. ......................... 78
5.6 Liquefaction resistance curves by UBC3D-PLM model for Gaiko sand based on initial model parameters for cyclic DSS test. ......................... 78
5.7 Liquefaction resistance curves by UBC3D-PLM model of Ohama sand based on calibrated model parameters for cyclic DSS test. ..................... 81
5.8 Liquefaction resistance curves by UBC3D-PLM model of Gaiko sand based on calibrated model parameters for cyclic DSS test. ..................... 81
5.9 Development of excess pore pressure ratio Ohama sand for cyclic undrained DSS test in PLAXIS with CSR = 0.10. ................................................................. 82
5.10 Development of stress path Ohama sand for cyclic undrained DSS test in PLAXIS with CSR = 0.10. ................................................................. 82
5.11 Development of shear strains Ohama sand for cyclic undrained DSS test in PLAXIS with CSR = 0.10. ................................................................. 82
5.12 Development of excess pore pressure ratio Gaiko sand for cyclic undrained DSS test in PLAXIS with CSR = 0.18. ................................................................. 83
5.13 Development of stress path Gaiko sand for cyclic undrained DSS test in PLAXIS with CSR = 0.18. ................................................................. 83
5.14 Development of shear strains Gaiko sand for cyclic undrained DSS test in PLAXIS with CSR = 0.18. ................................................................. 83
5.15 Development of $K_0$ for Ohama Sand and Gaiko Sand according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for an undrained cyclic DSS test (for $P_0 = 98$ kPa). ................................................................. 84
5.16 Liquefaction resistance curves of Ohama sand according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for different $K_0$-values for undrained cyclic DSS test. ................................................................. 86
5.17 Liquefaction resistance curves of Gaiko sand according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for different $K_0$-values for undrained cyclic DSS test. ................................................................. 86
5.18 Definition of cartesian stress directions for a 3D situation ................................................................. 87
5.19 Development of $K_0$ for varying initial $K_0$ values ................................................................. 88
5.20 Development of principal effective stresses in an undrained cyclic DSS test with $K_0 = 0.5$ for CSR $= 0.10$. ................................................................. 89
5.21 Development of principal effective stresses in an undrained cyclic DSS test with $K_0 = 1.0$ for CSR $= 0.10$. ................................................................. 89
5.22 Development of principal effective stresses in an undrained cyclic DSS test with $K_0 = 2.0$ for CSR $= 0.10$. ................................................................. 89
5.23 Development of average effective stresses, excess pore pressures and total stresses in an undrained cyclic DSS test with $K_0 = 0.5$. ................................................................. 90
5.24 Development of average effective stresses, excess pore pressures and total stresses in an undrained cyclic DSS test with $K_0 = 1.0$. ................................................................. 90
5.25 Development of average effective stresses, excess pore pressures and total stresses in an undrained cyclic DSS test with $K_0 = 2.0$. ................................................................. 90
5.26 Development of $K_0$ for Ohama Sand and Gaiko Sand according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for DSS test. ................................................................. 91
5.27 Schematisation of an undrained cyclic triaxial test for $K_0 = 1.0$ with in first phase the consolidation stress applied and in second phase the axial cyclic loading. ................................................................. 93
5.28 Liquefaction resistance curves by UBC3D-PLM model for Ohama sand based on initial model parameters for an undrained cyclic triaxial test. ................................................................. 95
5.29 Liquefaction resistance curves by UBC3D-PLM model for Gaiko sand based on initial model parameters for an undrained cyclic triaxial test. ................................................................. 95
5.30 Liquefaction resistance curves by UBC3D-PLM model of Ohama sand based on calibrated model parameters for an undrained cyclic triaxial test. ................................................................. 96
5.31 Liquefaction resistance curves by UBC3D-PLM model of Gaiko sand based on calibrated model parameters for an undrained cyclic triaxial test. ................................................................. 97
5.32 Development of excess pore pressure ratio Ohama sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.16. ................................................................. 98
5.33 Development of stress path Ohama sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.16. ................................................................. 98
5.34 Development of shear strains Ohama sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.16. ................................................................. 98
5.35 Development of excess pore pressure ratio Gaiko sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.28. ................................................................. 99
5.36 Development of stress path Gaiko sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.28. ................................................................. 99
5.37 Development of shear strains Gaiko sand for undrained cyclic triaxial test in PLAXIS with CSR = 0.28. ................................................................. 99
5.38 Development of $K_p$ for Ohama Sand and Gaiko Sand according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for an undrained cyclic triaxial test. ................................................................. 100
5.39 Definition of static shear on the soil sample in an undrained cyclic triaxial test (X., Wei and J., Yang, 2015). ................................................................. 101
5.40 Liquefaction resistance curves of Ohama sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test. ................................................................. 101
5.41 Liquefaction resistance curves of Gaiko sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test. ................................................................. 102
5.42 Development of $K_n$ for Ohama Sand and Gaiko Sand according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for an undrained cyclic triaxial test. ................................................................. 102
6.1 Flowchart dynamic analysis ...................................................................................... 105
6.2 Corrected and filtered time signal of the horizontal acceleration of the Nihonkai Chubu Earthquake at Akita Port. ................................................................. 106
6.3 Measured time series of the horizontal acceleration of the Imperial Valley Earthquake. ................................................................. 107
6.4 Measured time series of the horizontal acceleration of the Landers Earthquake. ................................................................. 107
6.5 Measured time series of the horizontal acceleration of the Kocaeli Earthquake. ................................................................. 107
6.6 Schematization of location of the measured earthquake and the input motion (Mejia, L.H. and Dawson, E.M., 2006). ................................................................. 108
6.7 Calibrating strain dependent stiffness parameters and damping curves used in the HSSmall PLAXIS model according to theoretical damping curves. ................................................................. 112
6.8 Calculated time series at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g. ................................................................. 116
6.9 Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g. ................................................................. 116
6.10 Calculated time series at the surface at Ohama No.1 Wharf of the Imperial Valley Earthquake scaled to 0.10 g. ................................................................. 116
6.11 Calculated time series at the surface at Ohama No.1 Wharf of the Landers Earthquake scaled to 0.10 g. ................................................................. 117
6.12 Calculated time series at the surface at Ohama No.1 Wharf of the Kocaeli Earthquake scaled to 0.10 g. ................................................................. 117
6.13 Calculated time series at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g. ................................................................. 117
6.14 Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g. ................................................................. 117
6.15 Calculated time series at the surface at Ohama No.2 Wharf of the Imperial Valley Earthquake scaled to 0.10 g. ................................................................. 117
6.16 Calculated time series at the surface at Ohama No.2 Wharf of the Landers Earthquake scaled to 0.10 g. ................................................................. 118
6.17 Calculated time series at the surface at Ohama No.2 Wharf of the Kocaeli Earthquake scaled to 0.10 g. ................................................................. 118
6.18 Layout of PLAXIS 2D model, with drained boundaries at both sides. ................................................................. 119
List of Figures

6.19 Bending moment distribution and displacement over the height of the structure for considered earthquake motions. ................................. 120
6.20 Deformation pattern soil after Nihonkai Chubu Earthquake. ................................................................................................. 121
6.21 Comparison bending moment distribution and displacement obtained from dynamic analysis and pseudo-static analysis for Nihonkai Chubu Earthquake. ................................................................................................. 122
6.22 Development of $K_{AE}$ and $K_{PE}$ according to dynamic analysis and pseudo-static analysis. ......................................................... 122
6.23 Overview of lateral earth pressure coefficients ($K_v$) around anchored quay wall at Ohama No.2 Wharf after construction. ................................................................................................. 125
6.24 Overview of shear stress ratios ($\alpha$) around anchored quay wall at Ohama No.2 Wharf after construction. ............................................. 126
6.25 Indication of zones corresponding to typical stress states around anchored quay wall. Magnitude of lateral earth pressure indicated with: N - Neutral, A - Active, P - Passive. Static shear stress: $\alpha$ - static shear stress present, 0 - no static shear stress present. ............................................. 126
6.26 Indication of zones corresponding to typical element tests with corresponding conditions. ......................................................... 127
6.27 Overview of zones defined in the finite element model corresponding to different model parameter sets. ................................................................................................. 128
6.28 Overview of PLAXIS 2D model, with drained layers at both side boundaries. ................................................................................................. 129
6.29 Horizontal deformation pattern of soil after Nihonkai Chubu Earthquake. ................................................................................................. 129
6.30 Distribution of $\epsilon_\text{v}$ value around anchored quay wall at Ohama No.2 Wharf after Nihonkai Chubu Earthquake. ................................................................................................. 130
6.31 Development excess pore pressures at two locations behind sheet pile wall for Nihonkai Chubu Earthquake. ................................................................................................. 130
6.32 Development excess pore pressures at two locations behind sheet pile wall for Nihonkai Chubu Earthquake. ................................................................................................. 131
6.33 Displacement of anchored quay wall over time for Nihonkai Chubu Earthquake. ................................................................................................. 131
6.34 Overview of volumetric strains $\epsilon_\text{v}$ around the anchored quay wall after Nihonkai Chubu Earthquake. ................................................................................................. 132
6.35 Development effective and total horizontal pressure at location K ................................................................................................. 133
6.36 Development effective and total horizontal pressure at location L ................................................................................................. 133
6.37 Development effective and total horizontal pressure at location L ................................................................................................. 133
6.38 Output bending moment distribution and displacements of anchored quay wall at Ohama No.2 Wharf. ................................................................................................. 134
6.39 Comparison bending moment distribution and displacements obtained from dynamic analysis and pseudo-static analysis for Nihonkai Chubu Earthquake ................................................................................................. 136
7.1 Factor of safety against liquefaction of soil profile at Julianahaven (Haan, de, F.S., 2016). ................................................................................................. 140
7.2 Overview of Eemshaven with different basins. ................................................................................................. 140
7.3 Cross section quay wall Wilhelmshaven (Haan, de, F.S., 2016). ................................................................................................. 141
7.4 Illustration of methodology in the hazard V2 model (Spetzler, J. and Dost, B., 2016). ................................................................................................. 142
A.1 Earth pressure distribution for dry backfill with on the left the earth pressures for the static situation and on the right for the Case100 seismic event. ................................................................................................. 164
A.2 Resulting bending moments and deformation for each seismic event of D-sheet model of the reference case. ................................................................................................. 164
A.3 Earth pressure distribution for a partly saturated backfill; on the left the earth pressures for the static situation and on the right for the Case100 seismic event. ................................................................................................. 165
A.4 Resulting bending moments and deformation for each seismic event from D-sheet for model with a partly saturated backfill. ................................................................................................. 165
A.5 Earth pressure distribution for backfill with equivalent unit weight; on the left the earth pressures for the static situation and on the right for the Case100 seismic event. ................................................................................................. 166
A.6 Resulting bending moments and displacements for each seismic event obtained by D-sheet for model with a backfill with an equivalent unit weight. ................................................................................................. 166
A.7 On the left the water pressures for the static situation and on the right the effective horizontal earth pressures for the static situation. ................................................................................................. 167
A.8 On the left the water pressures for the Case100 event including excess pore pressures and on the right the effective horizontal earth pressures for the Case100 seismic event determined with adjusted $\phi'$ method. .................................................. 167
A.9 On the left the water pressures for the Case100 event including excess pore pressures and on the right the effective horizontal earth pressures for the Case100 seismic event determined with adjusted $\theta$ method. .................................................. 168
A.10 Resulting bending moments and displacements for each event from D-sheet for model with liquefiable layer halfway height of structure for both considered methods taking into account excess pore pressures. .................................................. 168
A.11 On the left the water pressures for the static situation and on the right the effective horizontal earth pressures for the static situation. .................................................. 169
A.12 On the left the water pressures for the Case100 event including excess pore pressures and on the right the effective horizontal earth pressures for the Case100 seismic event determined with adjusted $\phi'$ method. .................................................. 169
A.13 On the left the water pressures for the Case100 event including excess pore pressures and on the right the effective horizontal earth pressures for the Case100 seismic event determined with adjusted $\theta$ method. .................................................. 170
A.14 Resulting bending moments and displacements for each event from D-sheet for model with liquefiable layer at embedded part of structure for both considered methods taking into account excess pore pressures. .................................................. 170

B.1 Interpretation of results of SPT-test performed at Ohama No.1 Wharf (Iai, S. and Kameoka, T., 1993). .................................................. 172
B.3 Soil profile and related soil properties based on SPT-test and grain size distribution at Ohama No.1 Wharf. .................................................. 173
B.4 Interpretation of results of SPT-test performed at Ohama No.2 Wharf (Iai, S. and Kameoka, T., 1993). .................................................. 174
B.5 Grainsize distribution at Ohama No.2 Wharf (Iai, S. and Kameoka, T., 1993). .................................................. 174
B.6 Soil profile and related soil properties based on SPT-test and grain size distribution at Ohama No.2 Wharf. .................................................. 175

C.1 Yield surface of HSsmall model, including yield cap (Brinkgreve, R.B.J. et al., 2015). .................................................. 177
C.2 Left: Expansion of the yield surface due to shear strains towards the Mohr Coulomb failure line. Right: Expansion of the cap due to primary compression towards the yield cap. .................................................. 178
C.4 Mohr-Coulomb failure contour used in the UBC3D-PLM model (Petelas, A. and Galavi, V., 2013). .................................................. 180
C.5 Failure surface and yield surfaces of UBC3D-PLM model including the post liquefaction behaviour (Greef, de, J., 2015). .................................................. 181

D.1 Liquefaction resistance curves of Ohama sand at different overburden stresses according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test. .................................................. 186
D.2 Liquefaction resistance curves of Gaiko sand at different overburden stresses according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test. .................................................. 186
D.3 Development of $K_r$ of both Ohama sand and Gaiko sand according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for an undrained cyclic DSS test. .................................................. 186
D.4 Liquefaction resistance curves of Ohama sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test. .................................................. 187
D.5 Liquefaction resistance curves of Gaiko sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test. .................................................. 187
D.6 Liquefaction resistance curves of Ohama sand for different initial static shear ratios according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test. .................................................. 188
D.7 Liquefaction resistance curves of Gaiko sand for different initial static shear ratios according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test. .................................................. 188
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>D.1</td>
<td>Corrected and filtered time series of the horizontal acceleration at bedrock level of the Nihonkai Chubu Earthquake at Akita Port.</td>
</tr>
<tr>
<td>D.2</td>
<td>Scaled time series of the horizontal acceleration at bedrock level of the Nihonkai Chubu Earthquake at Akita Port to a PGA of 0.10 g.</td>
</tr>
<tr>
<td>D.3</td>
<td>Original time series of the horizontal acceleration at bedrock level of the Imperial Valley Earthquake.</td>
</tr>
<tr>
<td>D.4</td>
<td>Scaled time series of the horizontal acceleration at bedrock level of the Imperial Valley Earthquake to a PGA of 0.23 g.</td>
</tr>
<tr>
<td>D.5</td>
<td>Scaled time series of the horizontal acceleration at bedrock level of the Imperial Valley Earthquake to a PGA of 0.10 g.</td>
</tr>
<tr>
<td>D.6</td>
<td>Original time series of the horizontal acceleration at bedrock level of the Landers Earthquake.</td>
</tr>
<tr>
<td>D.7</td>
<td>Scaled time series of the horizontal acceleration at bedrock level of the Landers Earthquake to a PGA of 0.23 g.</td>
</tr>
<tr>
<td>D.9</td>
<td>Liquefaction resistance curves of Ohama sand at different overburden stresses according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.</td>
</tr>
<tr>
<td>D.10</td>
<td>Liquefaction resistance curves of Gaiko sand at different overburden stresses according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.</td>
</tr>
<tr>
<td>D.12</td>
<td>Liquefaction resistance curves of Ohama sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.</td>
</tr>
<tr>
<td>D.13</td>
<td>Liquefaction resistance curves of Gaiko sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.</td>
</tr>
<tr>
<td>D.14</td>
<td>Liquefaction resistance curves of Ohama sand for different initial static shear ratios according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.</td>
</tr>
<tr>
<td>D.15</td>
<td>Liquefaction resistance curves of Gaiko sand for different initial static shear ratios according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.</td>
</tr>
<tr>
<td>E.1</td>
<td>Calculated time series at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g.</td>
</tr>
<tr>
<td>E.2</td>
<td>Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g.</td>
</tr>
<tr>
<td>E.3</td>
<td>Calculated time series at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.23 g.</td>
</tr>
<tr>
<td>E.4</td>
<td>Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.23 g.</td>
</tr>
<tr>
<td>E.5</td>
<td>Calculated time series at the surface at Ohama No.1 Wharf of the Imperial Valley Earthquake scaled to 0.10 g.</td>
</tr>
<tr>
<td>E.6</td>
<td>Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Imperial Valley Earthquake scaled to 0.10 g.</td>
</tr>
<tr>
<td>E.7</td>
<td>Calculated time series at the surface at Ohama No.1 Wharf of the Imperial Valley Earthquake scaled to 0.23 g.</td>
</tr>
<tr>
<td>E.8</td>
<td>Scaled time series of the horizontal acceleration at bedrock level of the Landers Earthquake to a PGA of 0.23 g.</td>
</tr>
<tr>
<td>E.9</td>
<td>Original time series of the horizontal acceleration at bedrock level of the Landers Earthquake.</td>
</tr>
<tr>
<td>E.10</td>
<td>Scaled time series of the horizontal acceleration at bedrock level of the Landers Earthquake to a PGA of 0.10 g.</td>
</tr>
<tr>
<td>E.11</td>
<td>Scaled time series of the horizontal acceleration at bedrock level of the Kocaeli Earthquake to a PGA of 0.10 g.</td>
</tr>
<tr>
<td>E.13</td>
<td>Liquefaction resistance curves of Ohama sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.</td>
</tr>
<tr>
<td>E.14</td>
<td>Liquefaction resistance curves of Gaiko sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.</td>
</tr>
<tr>
<td>E.15</td>
<td>Liquefaction resistance curves of Ohama sand for different initial static shear ratios according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.</td>
</tr>
<tr>
<td>E.16</td>
<td>Liquefaction resistance curves of Gaiko sand for different initial static shear ratios according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.</td>
</tr>
</tbody>
</table>
F.8 Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Imperial Valley Earthquake scaled to 0.23 g. ................................. 201
F.9 Calculated time series at the surface at Ohama No.1 Wharf of the Landers Earthquake scaled to 0.10 g. .................................................. 202
F.10 Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Landers Earthquake scaled to 0.10 g. ........................................ 202
F.11 Calculated time series at the surface at Ohama No.1 Wharf of the Landers Earthquake scaled to 0.23 g. ................................. 202
F.12 Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Landers Earthquake scaled to 0.10 g. ........................................ 202
F.13 Calculated time series at the surface at Ohama No.1 Wharf of the Kocaeli Earthquake scaled to 0.10 g. ................................. 203
F.14 Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Kocaeli Earthquake scaled to 0.23 g. ................................. 202
F.15 Calculated time series at the surface at Ohama No.1 Wharf of the Kocaeli Earthquake scaled to 0.10 g. ................................. 203
F.16 Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Kocaeli Earthquake scaled to 0.23 g. ................................. 203
F.17 Calculated time series at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g. .................................................. 204
F.18 Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g. .................................................. 204
F.19 Calculated time series at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.23 g. .................................................. 204
F.20 Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.23 g. .................................................. 204
F.21 Calculated time series at the surface at Ohama No.2 Wharf of the Imperial Valley Earthquake scaled to 0.10 g. .................................................. 205
F.22 Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Imperial Valley Earthquake scaled to 0.10 g. .................................................. 205
F.23 Calculated time series at the surface at Ohama No.2 Wharf of the Imperial Valley Earthquake scaled to 0.23 g. .................................................. 205
F.24 Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Imperial Valley Earthquake scaled to 0.23 g. .................................................. 205
F.25 Calculated time series at the surface at Ohama No.2 Wharf of the Landers Earthquake scaled to 0.10 g. .................................................. 206
F.26 Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Landers Earthquake scaled to 0.10 g. .................................................. 206
F.27 Calculated time series at the surface at Ohama No.2 Wharf of the Landers Earthquake scaled to 0.23 g. .................................................. 206
F.28 Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Landers Earthquake scaled to 0.23 g. .................................................. 206
F.29 Calculated time series at the surface at Ohama No.2 Wharf of the Kocaeli Earthquake scaled to 0.10 g. .................................................. 207
F.30 Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Kocaeli Earthquake scaled to 0.10 g. .................................................. 207
F.31 Calculated time series at the surface at Ohama No.2 Wharf of the Kocaeli Earthquake scaled to 0.23 g. .................................................. 207
F.32 Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Kocaeli Earthquake scaled to 0.23 g. .................................................. 207
List of Tables

2.1 Relationships for horizontal seismic coefficients according to different design codes ....... 27

3.1 Results for maximum bending moment, anchor tensile force and displacement for different seismic events (Higuchi, S. et al., 2012) ................................................................. 37

3.2 Properties used in the model of the experimental case derived from research by Higuchi, S. et al. (2012) and Habets, C.J.W. (2015) ................................................................. 39

3.3 Horizontal seismic coefficient and total seismic horizontal earthpressure coefficients according to original Mononobe-Okabe method ......................................................... 41

3.4 Calibrated horizontal seismic coefficients and total seismic horizontal earth pressure coefficients for the reference case ................................................................. 43

3.5 Horizontal seismic coefficients with total seismic horizontal earthpressure coefficients for a partly saturated backfill ..................................................... 44

3.6 Maximum bending moment, anchor force and displacement during each seismic event for a partly saturated backfill ..................................................... 45

3.7 Soil properties liquefiable layer ........................................................................ 46

3.8 Horizontal seismic coefficients and total seismic horizontal earth pressure coefficient for excess pore pressures within the soil, according to both methods ..................................................... 46

3.9 Maximum bending moments, anchor forces and displacements for liquefiable layer halfway ..................................................... 47

3.10 Maximum bending moments, anchor forces and displacements for liquefiable layer at embedded part ......................................................... 48

4.1 Results of the liquefaction potential and predicted excess pore pressures in the backfill at Ohama No.1 Wharf ......................................................... 56

4.2 Results of the liquefaction potential and predicted excess pore pressures in the backfill at Ohama No.2 Wharf ......................................................... 57

4.3 Properties of FSP VIL type sheet piles .................................................................... 57

4.4 Input properties of tubular anchor piles for both anchor walls ......................................... 59

4.5 Properties of semi-high strength tie rods ................................................................... 59

4.6 Calculated anchor stiffness of Ohama No.1 Wharf for each loading event ................................ 61

4.7 Calculated anchor stiffness of Ohama No.2 Wharf for each loading event ................................ 61

4.8 Input earth pressure coefficients for soil layers at Ohama No.1 Wharf ......................................... 62

4.9 Results of maximum bending moments, anchor force and displacements of anchored quay wall at Ohama No.1 Wharf ......................................................... 63

4.10 Input earth pressure coefficients for soil layers at Ohama No.2 Wharf ......................................... 64

4.11 Results of maximum bending moments, anchor force and displacements of anchored quay wall at Ohama No.2 Wharf ......................................................... 65

4.12 Results of maximum bending moments, anchor force and displacements of anchored quay wall at Ohama No.2 Wharf considering a wall friction angle of zero ......................................................... 67

5.1 Model parameters used in the UBC3D-PLM model (Beaty, M.H. and Byrne, P.M. (2011), Makra, A. (2013), Petalas, A. and Galavi, V. (2013)) ................................................................. 71

5.2 Summary model performance for DSS test (Makra, A., 2013) ......................................................... 75

5.3 Initial model parameters for Ohama sand and Gaiko sand based on methods by Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) and Souliotis, C. and Gerolymos, N. (2016) ......................................................... 76

5.4 Influence of model parameters on onset of liquefaction (Winde, H.P., 2015) ......................................................... 79

5.5 Calibrated model parameters for Ohama sand and Gaiko sand for an undrained cyclic DSS test ......................................................... 80

5.6 Corrected $f_{ach,hard}$ values in UBC3D-PLM model for Ohama Sand to match the theoretical $K_d$ Idriss, I.M. and Boulanger, R.W. (2003) for an undrained cyclic DSS test ......................................................... 85
5.7 Corrected $f_{a,\text{hard}}$ and $k_{p}^{G}$ values in UBC3D-PLM model to match the theoretical $K_{G}$ Idriss, I.M. and Boulanger, R.W. (2003) for an undrained cyclic DSS test. ........................................... 92

5.8 Initial model parameters for Ohama sand and Gaiko sand for undrained cyclic triaxial test, based on calibration of undrained cyclic DSS tests. ........................................... 94

5.9 Calibrated model parameters of Ohama sand and Gaiko sand for an undrained cyclic triaxial test. 96

5.10 Corrected $f_{a,\text{hard}}$ and $k_{p}^{G}$ values in UBC3D-PLM model to match the theoretical $K_{G}$ Idriss, I.M. and Boulanger, R.W. (2003) for an undrained cyclic triaxial test. ........................................... 103

6.1 Model parameters used in the HSsmall model (Brinkgreve, R.B.J. et al. (2010), Brinkgreve, R.B.J. et al. (2015)) ................................................................................................................. 110

6.2 Calibrated input parameters HSsmall material model soil layers Ohama No.1 Wharf. .......................... 113

6.3 Calibrated input parameters HSsmall material model soil layers Ohama No.2 Wharf. .......................... 113

6.4 Target frequencies for both anchored quay walls to determine the Rayleigh damping parameters. 114

6.5 Required minimum average mesh size and time step for Ohama No.1 Wharf and Ohama No.2 Wharf. .................................................................................................................. 115

6.6 Output of the dynamic PLAXIS model. ......................................................................................... 120

6.7 Comparison of the extreme bending moments, anchor forces and displacements between dynamic analysis and pseudo-static analysis for Nihonkai Chubu Earthquake ........................................... 121

6.8 Comparison horizontal effective earth pressure obtained from dynamic analysis and pseudo-static analysis for Nihonkai Chubu Earthquake. .............................................................................. 123

6.9 Output dynamic analysis Ohama No.2. .......................................................................................... 134

6.10 Comparison between extreme outcomes of dynamic analysis and pseudo-static analysis for Nihonkai Chubu Earthquake. ......................................................................................... 135

8.1 Summary performance pseudo-static and dynamic analysis ..................................................... 147

B.1 Representative N-values and normalized N-values of soil layers at Ohama No.1 Wharf. ............... 173

B.2 Representative N-values and normalized N-values of soil layers at Ohama No.2 Wharf. ............... 175

D.1 Calibrated model parameters for Ohama sand and Gaiko sand for an undrained cyclic DSS test. 185

D.2 Calibrated model parameters of Ohama sand and Gaiko sand for an undrained cyclic triaxial test. 189

G.1 Calibrated parameters UBC3D-PLM model Ohama sand for stress paths corresponding to undrained cyclic DSS tests. .......................................................................................... 210

G.2 Calibrated parameters UBC3D-PLM model Ohama sand for stress paths corresponding to undrained cyclic triaxial tests. .......................................................................................... 210

G.3 Calibrated parameters UBC3D-PLM model Gaiko sand for stress paths corresponding to undrained cyclic DSS tests. .......................................................................................... 211

G.4 Calibrated parameters UBC3D-PLM model Gaiko sand for stress paths corresponding to undrained cyclic triaxial tests. .......................................................................................... 211
Several characteristic case studies are elaborated in order to analyse the performance of the Mononobe-Okabe method and compare results of prescribed modifications to the original method. In this appendix results of the different cases and corresponding D-Sheet models is shown.

A.1. Introduction into model
In the D-Sheet model construction stages have to be defined in order to determine the right stress states in the soil and response of the structure. Distinction is made between the following construction stages:

- Stage 1: Initial static case - installation sheet pile and anchors
  In this stage the surface level is equal at both sides of the structure and the sheet pile and anchor properties are specified.

- Stage 2: Static case - deepening / excavating
  In this stage soil was excavated in front of the sheet pile wall, resulting in the static loading condition.

- Stage 3: Dynamic phase
  In this stage the earth pressure coefficients of the soil were adjusted in order to account for the extra dynamic load.

These construction stages are modelled for each case. The arching effect of the anchors is modelled by removing the upper 1.50 meters of the soil profile, which is the soil above the anchors.

A.2. Results models
The following characteristic cases are elaborated:

- Reference case; experiment
- Partially saturated backfill
- Backfill containing liquefiable layer

For each modelled case results for the earth pressure distribution, bending moment distribution and displacements over the height of the structure are presented.
A.2.1. Reference case

Figure A.1: Earth pressure distribution for dry backfill with on the left the earth pressures for the static situation and on the right for the Case100 seismic event.

Figure A.2: Resulting bending moments and deformation for each seismic event of D-sheet model of the reference case.
A.2. Results models

A.2.2. Partially saturated backfill

Separate layers

Figure A.3: Earth pressure distribution for a partly saturated backfill; on the left the earth pressures for the static situation and on the right for the Case100 seismic event.

Figure A.4: Resulting bending moments and deformation for each seismic event from D-sheet for model with a partly saturated backfill.
Equivalent unit weight

Figure A.5: Earth pressure distribution for backfill with equivalent unit weight; on the left the earth pressures for the static situation and on the right for the Case100 seismic event.

Figure A.6: Resulting bending moments and displacements for each seismic event obtained by D-sheet for model with a backfill with an equivalent unit weight.
A.2.3. Backfill with liquefiable layer
Liquefiable layer within backfill

Figure A.7: On the left the water pressures for the static situation and on the right the effective horizontal earth pressures for the static situation.

Figure A.8: On the left the water pressures for the Case100 event including excess pore pressures and on the right the effective horizontal earth pressures for the Case100 seismic event determined with adjusted $\gamma'$ method.
Figure A.9: On the left the water pressures for the Case100 event including excess pore pressures and on the right the effective horizontal earth pressures for the Case100 seismic event determined with adjusted $\mu$ method.

Figure A.10: Resulting bending moments and displacements for each event from D-sheet for model with liquefiable layer halfway height of structure for both considered methods taking into account excess pore pressures.
Retaining structure embedded in liquefiable layer

Figure A.11: On the left the water pressures for the static situation and on the right the effective horizontal earth pressures for the static situation.

Figure A.12: On the left the water pressures for the Case100 event including excess pore pressures and on the right the effective horizontal earth pressures for the Case100 seismic event determined with adjusted \( \phi' \) method.
Figure A.13: On the left the water pressures for the Case100 event including excess pore pressures and on the right the effective horizontal earth pressures for the Case100 seismic event determined with adjusted $\mu$ method.

Figure A.14: Resulting bending moments and displacements for each event from D-sheet for model with liquefiable layer at embedded part of structure for both considered methods taking into account excess pore pressures.
Interpretation soil investigation

At Akita Port several in-situ site tests have been performed. Behind both quay walls soil samples were taken to determine the grainsize distribution and SPT-tests were performed. Results of these tests are used for the determination of the soil profile and the corresponding soil parameters.

The following steps are performed in order to determine the soil profile:

- **Determine soil type**
  Based on bore hole tests and corresponding grain size distribution the type of soil are determined, which is important for the interpretation of the SPT-test results. The soil is classified based on the grain size distribution. Using this classification a first estimation of the unit weight is made based on Eurocode 7 (Nederlands Normalisatie-instituut, 2012).

- **Interpretation of SPT-values**
  The amount of blow counts required depends not only on the soil type but also on the stress state and type of equipment used for the SPT-test. In section 2.6.1 is described how the normalized SPT-values ($N_{160}$) can be calculated based on the observed SPT-values ($N_{SPT}$).

- **Determine relative density**
  Based on the type of soil and the normalized N-values relationships are available that give an indication of the relative density of the soil. As is described in section 6.2.2 there is a large spread in the different relationships between normalized blow count and relative density, which introduces some uncertainty in the estimation of the relative density.

- **Determine soil parameters**
  Finally once the soil type and the compaction are known the soil parameters of the different soil types can be determined. This is done using Eurocode 7, which provides a table with representative soil parameters for each type of soil.

In the following two sections interpretation of the soil tests at both anchored quay walls is presented. First SPT-test results and grain size distributions are shown. In the SPT test results at the left side the borehole profile is presented, which gives an indication of the type of soil. Together with the provided grainsize distributions a first classification of the soil is obtained. Subsequently for each soil layer an representative SPT-value is determined, which is indicated per layer with a red line. These values are used to calculate the normalized SPT values for all soil layers. The normalized values are obtained with the method presented in section 2.6.1, where it is explained how to correct the measured SPT-values type of equipment and execution and for the stress state. Lastly the obtained soil profile including soil parameters are presented.

It must be stated that the location of the bore holes and the SPT-tests is not exactly known. From the results it seems that the location of the SPT-tests and the bore holes is somewhat different, since they do not fully correspond to each other. However approximately the same soil profile is obtained, only the heights differ to some extend.
B.1. Ohama No.1 Wharf

Results soil tests

Figure B.1: Interpretation of results of SPT-test performed at Ohama No.1 Wharf (Iai, S. and Kameoka, T., 1993).

Figure B.2: Grainsize distribution at Ohama No.1 Wharf (Iai, S. and Kameoka, T., 1993).
Table B.1: Representative N-values and normalized N-values of soil layers at Ohama No.1 Wharf.

<table>
<thead>
<tr>
<th>Layer</th>
<th>z [m]</th>
<th>N</th>
<th>C_R</th>
<th>C_ER</th>
<th>C_B</th>
<th>C_S</th>
<th>N_{60}</th>
<th>γ [kN/m³]</th>
<th>p_{w} [kPa]</th>
<th>p_{e} [kPa]</th>
<th>m</th>
<th>C_n</th>
<th>(N1)_{60}</th>
<th>Δ(N1)_{60}</th>
<th>(N1)_{60,cs}</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.2</td>
<td>17</td>
<td>0.87</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>20</td>
<td>19</td>
<td>80</td>
<td>31</td>
<td>49</td>
<td>0.39</td>
<td>1.32</td>
<td>26.2</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>9.0</td>
<td>17</td>
<td>0.94</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>21</td>
<td>20</td>
<td>172</td>
<td>79</td>
<td>93</td>
<td>0.38</td>
<td>1.03</td>
<td>22.0</td>
<td>5.57</td>
</tr>
<tr>
<td>3</td>
<td>11.1</td>
<td>20</td>
<td>0.95</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>25</td>
<td>21</td>
<td>216</td>
<td>100</td>
<td>116</td>
<td>0.37</td>
<td>0.95</td>
<td>24.1</td>
<td>4.48</td>
</tr>
<tr>
<td>4</td>
<td>15.0</td>
<td>30</td>
<td>0.97</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>39</td>
<td>21</td>
<td>298</td>
<td>139</td>
<td>159</td>
<td>0.33</td>
<td>0.86</td>
<td>33.1</td>
<td>1.15</td>
</tr>
<tr>
<td>5</td>
<td>22.4</td>
<td>18</td>
<td>0.98</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>23</td>
<td>20</td>
<td>451</td>
<td>213</td>
<td>243</td>
<td>0.48</td>
<td>0.65</td>
<td>15.3</td>
<td>0.00</td>
</tr>
<tr>
<td>6</td>
<td>25.3</td>
<td>24</td>
<td>0.98</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>31</td>
<td>21</td>
<td>512</td>
<td>242</td>
<td>278</td>
<td>0.43</td>
<td>0.64</td>
<td>20.2</td>
<td>1.15</td>
</tr>
</tbody>
</table>

Figure B.3: Soil profile and related soil properties based on SPT-test and grain size distribution at Ohama No.1 Wharf.
B.2. Ohama No.2 Wharf

Results soil tests

Figure B.4: Interpretation of results of SPT-test performed at Ohama No.2 Wharf (Iai, S. and Kameoka, T., 1993).

Figure B.5: Grainsize distribution at Ohama No.2 Wharf (Iai, S. and Kameoka, T., 1993).
Table B.2: Representative N-values and normalized N-values of soil layers at Ohama No.2 Wharf.

<table>
<thead>
<tr>
<th>Layer</th>
<th>z [m]</th>
<th>N [%]</th>
<th>C (_{FR}) [%]</th>
<th>C (_{FR}) [%]</th>
<th>C (_{S}) [%]</th>
<th>N(_{60}) [%]</th>
<th>(\gamma) [kN/m(^3)]</th>
<th>(\sigma_{\phi}) [kPa]</th>
<th>(\sigma_{\phi}) [kPa]</th>
<th>m [%]</th>
<th>C(_{n}) [%]</th>
<th>(N(<em>{1})(</em>{60}))</th>
<th>(N(<em>{1})(</em>{60,cs}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.0</td>
<td>5</td>
<td>0.89</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>6</td>
<td>19</td>
<td>89</td>
<td>47</td>
<td>42</td>
<td>0.55</td>
<td>1.60</td>
</tr>
<tr>
<td>2</td>
<td>11.1</td>
<td>8</td>
<td>0.95</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>10</td>
<td>19</td>
<td>205</td>
<td>108</td>
<td>97</td>
<td>0.54</td>
<td>1.02</td>
</tr>
<tr>
<td>3</td>
<td>17.6</td>
<td>41</td>
<td>0.97</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>50</td>
<td>21</td>
<td>337</td>
<td>173</td>
<td>164</td>
<td>0.24</td>
<td>0.89</td>
</tr>
<tr>
<td>4</td>
<td>20.2</td>
<td>17</td>
<td>0.97</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>22</td>
<td>21</td>
<td>391</td>
<td>199</td>
<td>193</td>
<td>0.42</td>
<td>0.76</td>
</tr>
<tr>
<td>5</td>
<td>22.0</td>
<td>20</td>
<td>0.98</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>26</td>
<td>20</td>
<td>427</td>
<td>217</td>
<td>211</td>
<td>0.40</td>
<td>0.74</td>
</tr>
<tr>
<td>6</td>
<td>26.0</td>
<td>28</td>
<td>0.98</td>
<td>1.33</td>
<td>1.00</td>
<td>1.00</td>
<td>36</td>
<td>21</td>
<td>530</td>
<td>267</td>
<td>264</td>
<td>0.36</td>
<td>0.71</td>
</tr>
</tbody>
</table>

Figure B.6: Soil profile and related soil properties based on SPT-test and grain size distribution at Ohama No.2 Wharf.
Constitutive material models

A commonly used finite element program in geotechnical engineering is PLAXIS. In this research the two-dimensional version PLAXIS 2D is used in combination with the dynamics module, to analyse earthquake motions and their effects on the soil-structure interaction including effects of liquefaction. In this appendix an introduction is given into the material models that are used in this research.

PLAXIS itself is not able to describe the soil behaviour. It uses constitutive material models that give a qualitative representation of the soil behaviour, based on model parameters that have to be derived from local soil properties. Basically these models describe the relation between stresses and strains in the soil based on material properties and state variables. Many material models are available each with possibilities and limitations for certain soil types and applications. Reference is made to the PLAXIS Material Models Manual (Brinkgreve, R.B.J. et al., 2015), where all material models are described in detail. In this research two constitutive material models are applied in the calculations. First the background of the Hardening Soil model including small-strain stiffness (HSsmall-model) will be elaborated. This model is used in static calculations and in dynamic calculations for soil layers not vulnerable to liquefaction. Secondly the user-defined UBC3D-PLM model is be treated, which is able to accumulate pore pressures in case of cyclic loading and thus used for liquefiable layers.

C.1. HSSmall model

The Hardening Soil model is a more advanced material model for the simulation of the soil behaviour. In this paragraph the most important properties of the model are elaborated. For the full mathematical background of the Hardening Soil small strain constitutive model reference is made to the PLAXIS material model manual (2015) or to the dissertation by Benz, T (2007).

Figure C.1: Yield surface of HSsmall model, including yield cap (Brinkgreve, R.B.J. et al., 2015).

The failure behaviour of the soil is based on the Mohr-Coulomb yield criterion, however the yield surface of this model is not fixed in the principle stress space as is the case in the Mohr-Coulomb model. It can expand because of plastic straining due to two different types of hardening, shear hardening and compression hardening.
The failure surface involves two strength parameters $c$ and $\varphi_p$ and is limited by a yield cap as shown in Figure C.1. The shape of the failure surface is not fixed but depends on the amount of plastic straining, defined by the amount of shear hardening and compaction hardening. As soon as the failure criterion is satisfied, plastic yielding occurs defined by yield functions depending on the loading type (Schanz, T. et al., 1999). Shear hardening is used in the model to describe the amount of irreversible strains due to primary deviatoric loading. In Figure C.2 in the left graph, the expansion of the yield surface for increasing shear strains towards the Mohr Coulomb failure line are presented. Secondly, compression hardening defines the irreversible strains that are developed in case of primary compression in oedometer loading and isotropic loading. In Figure C.2 on the right the expansion of the cap for larger primary compression stresses are shown.

![Figure C.2: Left: Expansion of the yield surface due to shear strains towards the Mohr Coulomb failure line. Right: Expansion of the cap due to primary compression towards the yield cap.](image)

The relationship between axial strain and deviatoric stress in primary loading in the Hardening Soil model is described by the Duncan-Chang model (1970), also known as the hyperbolic model. This model describes a hyperbolic stress-strain relationship according to a power law. In case of unloading or reloading the model predicts elastic soil behaviour (Schanz, T. et al., 1999).

Within the model three stiffness parameters are included corresponding to three typical loading types, all depending on the stress and strain state of the soil. By defining three different stiffness parameters, the soil behaviour can be described accurately for different loading stages. The following stiffness parameters can be distinguished (Schanz, T. et al., 1999):

- $E^{ref}_{\text{sec}}$, **secant stiffness in standard drained triaxial test** [kN/m$^2$]
  This modulus defines the amount of plastic straining due to primary deviatoric loading and controls the development of the yield surface, Figure C.2 (left).

- $E^{ref}_{\text{oed}}$, **tangent stiffness for primary oedometer loading** [kN/m$^2$]
  Plastic straining due to primary compression is controlled by the value of $E^{ref}_{\text{oed}}$, represented by the cap of the yield surface, Figure C.2 (right).

- $E^{ref}_{\text{ur}}$, **unloading/reloading stiffness** [kN/m$^2$]
  Unloading and reloading is described elastically in the model within the yield cone. The associated stiffness is defined by the value of $E^{ref}_{\text{ur}}$.

All stiffness parameters are reference values related a certain reference stress (default value $p_{\text{ref}} = 100$ kPa). These reference values are used to calculate the stress-dependent stiffness for other stress states.

**Small-strain stiffness**

All properties described above are from the original Hardening Soil model. An extension of this model is the Hardening Soil small strain model that takes into account an increased soil stiffness for small strains (Benz, T. (2007), Brinkgreve, R.B.J. et al. (2015)).
The original Hardening Soil model considers elastic soil behaviour for unloading and reloading. In reality the soil shear stiffness decreases for increasing strain levels, leading to the shear modulus curve presented in the left in Figure C.3. The Hardening Soil small strain model accounts for the very small-strain soil stiffness and the non-linear dependency on the strain amplitude. Therefore two additional parameters are required:

- Initial shear modulus $G_0 \ [kN/m^2]$
- Shear strain level $\gamma_{0.7}$ at which the secant shear modulus $G_s$ is reduced to 72.2% of the initial $G_0$

In the HSSmall-model the relationship proposed by Hardin & Drnevich (1972) and updated by Santos & Correia (2001) is adopted to describe the modulus reduction against the shear strain.

\[
\frac{G_s}{G_0} = \frac{1}{1 + 0.385 |\gamma_{0.7}|} \quad \text{(C.1)}
\]

\[
\frac{G_t}{G_0} = \frac{1}{(1 + 0.385 |\gamma_{0.7}|)^2} \quad \text{(C.2)}
\]

The reduction curve is bound by a lower limit, to prevent that the curve reaches far too into the plastic material domain through strain hardening. The cut-off shear modulus is defined by the unloading / reloading stiffness $G_{ur}$ at the cut-off shear strain $\gamma_{cut-off}$.

Figure C.3: **Left:** Example of a modulus reduction curve (Brinkgreve, R.B.J. et al., 2015). **Right:** Hysteretic material behaviour (Laera, A. and R.B.J. Brinkgreve, 2015)

The decrease of soil stiffness for increasing strains described by the modulus reduction curve, until a certain boundary value. Once the loading direction is reversed, the initial stiffness ($G_0$) is regained. If the loading continues in that direction, stiffness decreases again according to the described shear modulus reduction curve. This recovery of soil stiffness with changing loading direction introduces hysteretic damping. Especially in dynamic calculations the when load direction changes much a typical loop is obtained showing the hysteretic behaviour of the model, which is presented on the right side in Figure C.3. The total area within the loop is equivalent to the energy that is dissipated during one cycle.

This hysteretic damping is negligible small for small strains/motions, because the material behaves more or less elastically. Small motions during an earthquake result may therefore unrealistic stiff soil behaviour. For that reason it is recommended to apply an additional Rayleigh damping in the model to filter these small vibrations (Brinkgreve, R.B.J. et al., 2007). On the contrary for large shear strains the hysteretic damping tends to overestimate the actual damping for clayey materials. A solution to this could be applying $G_0$ closer to $G_{ur}$. 
C.2. UBC3D-PLM model

The UBC3D-PLM model is almost similar to the original UBCSAND constitutive model that was developed at University of British Columbia by Beaty, M.H. and Byrne, P.M. (2011), but also compatible with PLAXIS 3D. It is developed originally for potentially liquefiable sandy soils and able to accumulate plastic strains and pore pressures in case of undrained cyclic loading. Again the most important properties of the model are elaborated, for a full mathematical background of the model reference is made to Beaty, M.H. and Byrne, P.M. (2011), Tsegaye, A.B. (2010) and Petalas, A. and Galavi, V. (2013).

The model decomposes stains into two components: elastic- and plastic strains. The elastic behaviour is non-linearly described and expressed in terms of an elastic bulk- ($K_e^B$) and shear modulus ($K_e^G$).

The failure criterion is just as the Hardening Soil model based on the Mohr-Coulomb theory. It contains a corresponding non-associated plastic potential function according to Drucker-Prager's criterion that describes the direction of the plastic strains. The UBC3D-PLM model is an effective stress model and thus based on the classical plasticity theory with a hyperbolic hardening rule. This rule relates the mobilised friction angle to the amount of plastic shear strain at a given stress level. The plastic potential function is also able to take into account negative mobilised dilatancy angles $\omega_m$ and thus able to include contractive behaviour (Petalas, A. and Galavi, V. (2013) and Greef, de, J. (2015)).

![Mohr-Coulmb failure contour](image)

Figure C.4: Mohr-Coulomb failure contour used in the UBC3D-PLM model (Petalas, A. and Galavi, V. (2013)).

The UBC3D-PLM model uses a failure surface and two yield surfaces:

- Mohr-Coulomb failure surface; indicated by the Mohr-Coulomb failure line.
- Primary shear yield surface for isotropic hardening; remains at maximum mobilised friction angle $\varphi_{m,max}$.
- Secondary shear yield surface for kinematic hardening; friction angle demobilised when unloading.

The behaviour of the model through the several loading cycles is visualised by Greef, de, J. (2015) in Figure C.5. As shown in the figure the unloading behaviour is always elastic. During the first loading cycle the primary loading shear modulus is adopted and the primary yield surface is expanded. During unloading the friction angle is demobilised but the primary yield surface remains constant. During reloading the secondary yield surface is activated again and the secondary shear modulus is adopted. The value of the secondary shear modulus can be modified in the model with the densification factor $f_{\alpha_{hard}}$.

As long as the mobilised friction angle remains below the maximum mobilised friction angle, only the secondary yield surface expands in reloading. Higher loading results again in expanding of the primary yield surface, until it reaches the failure line. Effective stresses decrease due to increasing excess pore pressures leading to lower capacity of the soil. At the moment the failure line is reached, behaviour of the soil changes. A lower shear modulus is adopted for loading, which can be adjusted in the model by the post-liquefaction factor $f_{\alpha_{post}}$. 


A densification rule is implemented increasing the secondary plastic shear modulus after each loading cycle and a simplified kinematic hardening rule is used for the secondary yield surface. Because of the lowering of the secondary surface at unloading plastic strains occur both at loading and reloading. Therefore the model is able to accumulate plastic strains during cyclic loading and consequently the accumulation of pore pressures in undrained loading. The change in excess pore water pressure $p_w$ in undrained calculations are related to the change in volumetric strain $\epsilon_v$, according to (Petalas, A. and Galavi, V., 2013):

$$dp_w = \frac{K_w}{n} d\epsilon_v$$

(C.3)

An important issue in modelling of cyclic liquefaction in sands is the volumetric locking. After the stress path reaches the yield surface that is defined by the peak friction angle, the evolution of volumetric strains becomes constant because of the formulation of the flow rule. This limitation is solved by implementing an equation that gradually decreases the plastic shear modulus $K_p$. The relationship includes the $f_ae_{post}$-factor that determines the minimum shear stiffness.

The UBC3D-PLM has some known limitations, which are important before interpreting the results. The following limitations can have major influence on the performance of the model:

- **No compaction hardening is included**
  The model does not contain, in contrary to the HSsmall model, a so-called cap on the yield contour that introduces plastic strains in case of large isotropic stresses and low shear stresses. Therefore no compaction hardening is included in the model and it is in theory possible to have large elastic strains at large principle stresses, whereas in reality also plastic strains occur. For earthquake this is of less influence on the results, because significant shear stresses occur (Winde, H.P., 2015).

- **Overestimation of damping in dynamic calculations**
  Non-conservative may occur, where the amount of total energy in the model is lower than in reality. This happens because the damping in the model is overestimated compared to the real dissipation of energy. The amount of damping in the model can be adjusted according to the results.

- **Dissipation of excess pore pressures not included in dynamic calculations**
  In PLAXIS no dissipation of excess pore pressures is included in dynamic calculations, leading to an overestimation of the pore pressures after some time.

- **Effects of static shear**
  The value of the static shear stress ratio has large influence on the performance of the model. Makra, A. (2013) did research to the performance of the model for increasing static shear stress ratios and concluded that for larger ratios the model tends to underestimate the cyclic resistance ratio significantly. Also for relative low values of the cyclic stress ratio relative to the static shear stress ratio, numerical
instabilities were observed. Makra has overcome this problem by adjusting both the densification- and the post-liquefaction factor to 1.0. By using a post-liquefaction factor of 1.0 the stiffness degradation after liquefaction is not accounted for. The research by Makra, A. (2013) was focussed on sloping ground where this assumption proved to be reasonable since the decisive failure mechanism was flow liquefaction, which occurs before the excess pore pressures ratio becomes equal to 1.0. It is therefore not necessarily the right solution in analysing an anchored quay wall.

C.2.1. Development excess pore pressures
Since the UBC3D-PLM model is used to model the development of excess pore pressures in liquefiable soils, the way the model calculates the excess pore pressures is analysed. The relationships in the model that collectively describe the development of excess pore pressure are elaborated. Goal is to understand the working of the model and consequences of adjustments to certain parameters on the soil behaviour and development of pore pressures. The description of the UBC3D-PLM model by Petalas, A. and Galavi, V. (2013) is used to analyse the relationships that describe the development of excess pore pressures in the model.

In an undrained calculation the increment of pore water pressure is defined in the model by the relatively simple relationship:

$$dp_w = \frac{K_w}{n}d\varepsilon_v$$  \hspace{1cm} (C.4)

In this equation $K_w$ is the bulk modulus of the pore fluid (combined water and soil skeleton), $n$ is the soil porosity and $d\varepsilon_v$ is the volumetric strain of the fluid. Volumetric compatibility in undrained conditions requires that the equivalent fluid volumetric strain must be equal to the volumetric strain of the soil skeleton. Resulting in the following correlations:

$$dp = dp' + dp_w$$  \hspace{1cm} (C.5)

$$\frac{K_w}{n} = (K_u - K')$$  \hspace{1cm} (C.6)

Where $K_u$ is the bulk modulus of the undrained soil and $K'$ is the drained bulk modulus for the soil skeleton.

For undrained conditions the Poisson’s ratio ($\nu$) is set by default in the model to 0.495, which is close to the upper limit of 0.5 since water is almost incompressible. Note that a $\nu$ value of 0.5 would lead to numerical errors because of dividing by zero, for that reason a value close to 0.5 is adopted. The undrained bulk modulus is then defined as follows:

$$K_u = \frac{2Ge(1 + \nu_u)}{3(1 - 2\nu_u)}$$  \hspace{1cm} (C.7)

In this equation $G^e$ is the elastic shear modulus.

The drained bulk modulus of the soil skeleton $K'$ is determined in the same way, however in stead of using the undrained Poisson's ratio the drained Poisson's ratio is applied that based on the stress dependent stress moduli:

$$\nu' = \frac{3K^e - 2G^e}{6K^e + 2G^e}$$  \hspace{1cm} (C.8)

Once the bulk modulus of water is calculated also the development of the plastic volumetric strains in the model have to be analysed. The flow rule defines the plastic strain increment in the UBC3D-PLM model once the failure line is reached and is defined as follows:

$$d\varepsilon^p_v = sin\psi_m \cdot d\gamma^p$$  \hspace{1cm} (C.9)

With:

$$sin\psi_m = sin\varphi_{mob} - sin\varphi_{cv}$$  \hspace{1cm} (C.10)

In these equations $d\varepsilon^p_v$ is the plastic strain increment, $\varphi_{cv}$ is the constant volume friction angle and $\varphi_{mob}$ is the mobilized friction angle.
A unique stress ratio is defined by $\varphi_{cv}$ for which plastic shear strains do not cause plastic volumetric strains. Stress ratios below $\sin\varphi_{cv}$ lead to contractive behaviour in the model, while stress ratios above $\sin\varphi_{cv}$ imply dilative behaviour. The larger the difference in stress state, the larger the contractive or dilative behaviour. The mobilized friction angle derived from the Mohr-Coulomb yield criterion is defined as follows:

$$\sin\varphi_{mob} = \frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'} = \frac{t_{mob}}{s'}$$  \hspace{1cm} (C.11)

Where the mobilized shear stress is represented by $t_{mob}$ and $s$ is the mean effective stress.

The plastic strain increment is formulated as:

$$dy^p = \left(\frac{1}{G^*}\right) d\sin\varphi_{mob}$$  \hspace{1cm} (C.12)

Where:

$$G^* = k_p^p \left(\frac{p'}{P_A}\right)^{n_p} \left[1 - \left(\frac{\sin\varphi_{mob}}{\sin\varphi_{peak}}\right) R_F\right]^2$$  \hspace{1cm} (C.13)

$$d\sin\varphi_{mob} = 1.5 k_p^p \left(\frac{p}{P_A}\right)^{n_p} \frac{p_A}{p_m} \left[1 - \left(\frac{\sin\varphi_{mob}}{\sin\varphi_{peak}}\right) R_F\right]^2 d\lambda$$  \hspace{1cm} (C.14)

For cyclic loading in the model distinction is made between primary and secondary loading. Initial loading leads to an increase of the primary yield surface, where the plastic shear modulus is equal to the input value for $k_p^p$ and used in the hardening rule. If for next loading cycles the mobilized friction angle ($\sin\varphi_{em}^0$) is smaller than the memorized maximum mobilized friction angle ($\sin\varphi_{em}^0$), the primary yield surface is not reached and less plastic strains are generated. The plastic shear modulus $k_p^p$ for secondary loading is then defined as follows:

$$k_p^p = k_p^p \cdot \left(4 + \frac{n_{rev}}{2}\right) \cdot hard \cdot fac_{hard}$$  \hspace{1cm} (C.15)

Where $n_{rev}$ is the number of shear stress reversals from loading to unloading, hard is a factor that corrects the densification rule for loose sands and $fac_{hard}$ is a multiplier to adjust the densification rule. The correction factor hard is defined by the following relationship:

$$hard = min(1, max(0.5, 0.1 \cdot (N_1)_{60}))$$  \hspace{1cm} (C.16)

In equation C.4 the formulation for the increment of excess pore pressures for the increment of volumetric strains is given. The development of excess pore pressures can be changed by adjusting the bulk modulus of the water or by varying the plastic strain increment. The bulk modulus of the water can be adjusted in the model by changing the elastic shear modulus $K_e^p$. Although there can be some uncertainty in this parameter, varying only this parameter would not be realistic since the elastic behaviour is not responsible for the development of high excess pore pressures.

Adjusting the plastic strain increment is more complicated. There is however also more uncertainty present since it depends on the type and state of the soil which is not necessarily well captured by only one input value (SPT-blowcount or relative density). It makes therefore more sense to calibrate these values since they contain much more uncertainty and have a significant contribution to the development of excess pore pressures.
Performance UBC3D-PLM model

D.1. Uncrained cyclic direct simple shear test

In Table D.1 calibrated model parameters are presented for simulation of an undrained cyclic DSS test with the UBC3D-PLM model. Application of these model parameters leads to a reasonable fit with laboratory results for initial stress state of 98 kPa and a $K_0$ value of 0.5.

In the next sections effects of state parameters on the liquefaction resistance curves are presented. Respectively effects of varying overburden stress $a_{vo}$, lateral earth pressure coefficient $K_0$ and initial static shear ratio $\alpha$ are shown. The liquefaction resistance curve obtained in the laboratory is presented together with the curves predicted by the model.

In chapter 5 the results are discussed in more detail.

Table D.1: Calibrated model parameters for Ohama sand and Gaiko sand for an undrained cyclic DSS test.

<table>
<thead>
<tr>
<th></th>
<th>Byrne &amp; Beaty (2011) and Makra (2013)</th>
<th>Souliotis &amp; Gerolymos (2016)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ohama Sand</td>
<td>Gaiko Sand</td>
</tr>
<tr>
<td>$(N1)_{100}$</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>$D_r$</td>
<td>47</td>
<td>50</td>
</tr>
<tr>
<td>$\varphi_{cv}$</td>
<td>30.0</td>
<td>33.0</td>
</tr>
<tr>
<td>$\varphi_p$</td>
<td>30.9</td>
<td>34.0</td>
</tr>
<tr>
<td>$c$</td>
<td>kN/m²</td>
<td>0.0</td>
</tr>
<tr>
<td>$k'_G$</td>
<td>kN/m²</td>
<td>902.1</td>
</tr>
<tr>
<td>$k''_G$</td>
<td>kN/m²</td>
<td>319.2</td>
</tr>
<tr>
<td>$k'_B$</td>
<td>[-]</td>
<td>631.5</td>
</tr>
<tr>
<td>$k''_B$</td>
<td>[-]</td>
<td>0.50</td>
</tr>
<tr>
<td>me</td>
<td>[-]</td>
<td>0.50</td>
</tr>
<tr>
<td>ne</td>
<td>[-]</td>
<td>0.50</td>
</tr>
<tr>
<td>np</td>
<td>[-]</td>
<td>0.40</td>
</tr>
<tr>
<td>np</td>
<td>[-]</td>
<td>0.40</td>
</tr>
<tr>
<td>$R_f$</td>
<td>[-]</td>
<td>0.7911</td>
</tr>
<tr>
<td>$p_A$</td>
<td>[kN/m²]</td>
<td>100</td>
</tr>
<tr>
<td>$\sigma_i$</td>
<td>[kN/m²]</td>
<td>0.00</td>
</tr>
<tr>
<td>$f_{a\text{c,hard}}$</td>
<td>[-]</td>
<td>0.30</td>
</tr>
<tr>
<td>$f_{a\text{c,post}}$</td>
<td>[-]</td>
<td>0.02</td>
</tr>
</tbody>
</table>
D.1.1. Effects of state parameters

Effect of initial vertical stress ($\sigma'_v$)

Figure D.1: Liquefaction resistance curves of Ohama sand at different overburden stresses according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test.

Figure D.2: Liquefaction resistance curves of Gaiko sand at different overburden stresses according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test.

Figure D.3: Development of $K_p$ of both Ohama sand and Gaiko sand according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for an undrained cyclic DSS test.
Effect of lateral earth pressure coefficient ($K_0$)

Figure D.4: Liquefaction resistance curves of Ohama sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test.

Figure D.5: Liquefaction resistance curves of Gaiko sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test.
Effect of initial static shear stress ratio ($\alpha$)

Figure D.6: Liquefaction resistance curves of Ohama sand for different initial static shear ratios according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test.

Figure D.7: Liquefaction resistance curves of Gaiko sand for different initial static shear ratios according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic DSS test.

Figure D.8: Development of $K_0$ of Ohama sand and Gaiko sand according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for an undrained cyclic DSS test.
D.2. Undrained cyclic triaxial test

In Table D.2 calibrated model parameters are presented for simulation of an undrained cyclic triaxial test with the UBC3D-PLM model. Application of these model parameters leads to a reasonable fit with laboratory results for initial cell pressure of 98 kPa.

In the next sections effects of state parameters on the liquefaction resistance curves are presented. Respectively effects of varying overburden stress $\sigma_v$, lateral earth pressure coefficient $K_0$ and initial static shear ratio $\alpha$ are shown. The liquefaction resistance curve obtained in the laboratory is presented together with the curves predicted by the model.

In chapter 5 the results are discussed in more detail.

Table D.2: Calibrated model parameters of Ohama sand and Gaiko sand for an undrained cyclic triaxial test.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Byrne &amp; Beaty (2011) and Makra (2013)</th>
<th>Souliotis &amp; Gerolymos (2016)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ohama Sand</td>
<td>Gaiko Sand</td>
</tr>
<tr>
<td>$(N1)_{60}$ [blows]</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>$D_r$ [%]</td>
<td>47</td>
<td>50</td>
</tr>
<tr>
<td>$\varphi_{cv}$ [°]</td>
<td>30.0</td>
<td>33.0</td>
</tr>
<tr>
<td>$\varphi'$ [°]</td>
<td>30.9</td>
<td>34.0</td>
</tr>
<tr>
<td>$c$ [kN/m²]</td>
<td>0.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$k_G^c$ [-]</td>
<td>902.1</td>
<td>934.3</td>
</tr>
<tr>
<td>$k_G^p$ [-]</td>
<td>1007</td>
<td>1521</td>
</tr>
<tr>
<td>$k_B^c$ [-]</td>
<td>631.5</td>
<td>654.0</td>
</tr>
<tr>
<td>me [-]</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>ne [-]</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>np [-]</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td>$R_f$ [-]</td>
<td>0.7911</td>
<td>0.7787</td>
</tr>
<tr>
<td>$p_A$ [kN/m²]</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$\sigma_f$ [kN/m²]</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$f_{ac_hard}$ [-]</td>
<td>0.23</td>
<td>0.60</td>
</tr>
<tr>
<td>$f_{ac_post}$ [-]</td>
<td>0.02</td>
<td>0.02</td>
</tr>
</tbody>
</table>
D.2.1. Effects of state parameters
Effect of initial vertical stress ($\sigma_{v0}$)

Figure D.9: Liquefaction resistance curves of Ohama sand at different overburden stresses according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.

Figure D.10: Liquefaction resistance curves of Gaiko sand at different overburden stresses according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.

Figure D.11: Development of $K_0$ according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for an undrained cyclic triaxial test.
D.2. Undrained cyclic triaxial test

Effect of lateral earth pressure coefficient ($K_0$)

Figure D.12: Liquefaction resistance curves of Ohama sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.

Figure D.13: Liquefaction resistance curves of Gaiko sand for different lateral earth pressure coefficients according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.
Effect of initial static shear stress ratio ($\alpha$)

Figure D.14: Liquefaction resistance curves of Ohama sand for different initial static shear ratios according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.

Figure D.15: Liquefaction resistance curves of Gaiko sand for different initial static shear ratios according to Beaty, M.H. and Byrne, P.M. (2011) and Makra, A. (2013) for an undrained cyclic triaxial test.

Figure D.16: Development of $K_\alpha$ according to UBC3D-PLM model and theory by Idriss, I.M. and Boulanger, R.W. (2008) for an undrained cyclic triaxial test.
Earthquake Motions

In this appendix earthquake motions are presented that are used as input for the dynamic analysis of both anchored quay walls in Akita Port.

First time serie of the Nihonkai Chubu Earthquake is presented, which caused damage to the considered anchored quay walls in Akita Port. Subsequently three other earthquake motions are presented, each with different characteristics. The following reference earthquake motions were adopted in the research:

- Imperial Valley Earthquake, USA, M = 6.6, October 1979
- Landers Earthquake, United States of America, M = 7.3, June 1992
- Kocaeli Earthquake, Turkey, M = 7.6, August 1999

The PGA of these reference earthquake motions are not exactly equal to each other. To make a comparison between results of different earthquake motions, all motions are scaled to a PGA of 0.23 g. This PGA corresponds to the PGA of the Nihonkai Chubu Earthquake.

At last all motions are also scaled to a PGA of 0.10 g. These downscaled motions are used in the site response analysis to compare results of the linear equivalent analysis with results obtained with a 1D finite element model.

Original measured time series together with the scaled earthquake motions are presented in the next sections.
E.1. Nihonkai Chubu Earthquake, Japan, M = 7.7, May 1983

Figure E.1: Corrected and filtered time series of the horizontal acceleration at bedrock level of the Nihonkai Chubu Earthquake at Akita Port.

Figure E.2: Scaled time series of the horizontal acceleration at bedrock level of the Nihonkai Chubu Earthquake at Akita Port to a PGA of 0.10 g.
E.2. Imperial Valley Earthquake, USA, M = 6.6, October 1979

Figure E.3: Original time series of the horizontal acceleration at bedrock level of the Imperial Valley Earthquake.

Figure E.4: Scaled time series of the horizontal acceleration at bedrock level of the Imperial Valley Earthquake to a PGA of 0.23 g.

Figure E.5: Scaled time series of the horizontal acceleration at bedrock level of the Imperial Valley Earthquake to a PGA of 0.10 g.
E.3. Landers Earthquake, United States of America, $M = 7.3$, June 1992

Figure E.6: Original time series of the horizontal acceleration at bedrock level of the Landers Earthquake.

Figure E.7: Scaled time series of the horizontal acceleration at bedrock level of the Landers Earthquake to a PGA of 0.23 g.

Figure E.8: Scaled time series of the horizontal acceleration at bedrock level of the Landers Earthquake to a PGA of 0.10 g.
Figure E.9: Original time series of the horizontal acceleration at bedrock level of the Kocaeli Earthquake.

Figure E.10: Scaled time series of the horizontal acceleration at bedrock level of the Kocaeli Earthquake to a PGA of 0.23 g.

Figure E.11: Scaled time series of the horizontal acceleration at bedrock level of the Kocaeli Earthquake to a PGA of 0.10 g.
Results site response analysis

In this appendix all results of the site response analysis are presented. Input for this site response analysis are earthquake motions presented in appendix E. Since the response of the soil at surface level not only depends on the input signal but also on the characteristics of local soil deposits a site response analysis has to be performed for each location. Results of the performed site response analysis for both Ohama No.1 Wharf and Ohama No.2 Wharf are presented in the next sections.

In this research two methods used for site response analysis are adopted. First the equivalent linear analysis in the frequency domain is applied to calculate the site response. The results of this analysis are then used to validate the results of the full dynamic non-linear calculation performed in PLAXIS. Determination of the model parameters describing the constitutive material model of the non linear method is complex and field and laboratory testing may be required to evaluate these parameters. In absence of these tests, the linear equivalent analysis is used to validate the results of the non-linear method, since the required parameters for this simplified method are relatively easy to determine.

For the lower loading levels the soil strains remain low and the soil behaviour can be properly described using linear relationships with non-linear soil properties. In this region results by the equivalent linear analysis are expected to be in reasonable agreement with the dynamic calculations in PLAXIS if both models correspond to each other (Kramer, S.L., 1996). The results by equivalent linear analysis are less sensitive to errors compared to the more complex numerical models in PLAXIS. Therefore these results are used to validate the chosen model parameters in the PLAXIS model and to exclude that numerical modelling issues affect the results.

More severe loading events introduce larger strains in the soil leading to a much more non-linear behaviour of the soil, which is not captured by the equivalent linear analysis. This type of analysis only accounts for elastic strains. For more severe loading events also plastic strains are introduced, leading to different soil behaviour. Also the stiffness of non-linear soil changes over the duration of the event, which is not accounted for in the equivalent linear analysis. For these type of problems a calibrated PLAXIS model is used to model this non-linear soil behaviour. The non-linear model is formulated in terms of effective stresses, which allow modelling of the development of water pressures during the earthquake event (Kramer, S.L., 1996).

In the next sections results of the site response analysis at both Ohama No.1 Wharf and Ohama No.2 Wharf are presented. The calculated time series and the frequency amplitude spectra for all earthquake motions are shown. Since both results help analysing the accuracy of the model outcome.

Results for the earthquake motions with PGA of 0.10 g according to the equivalent linear analysis and the dynamic non-linear analysis are presented in one figure to compare both methods. For the earthquake motions with a PGA of 0.23 only the results of the dynamic non-linear method are presented, since the equivalent linear analysis is not applied because strongly non-linear behaviour is expected.
E.1. Ohama No.1 Wharf
Nihonkai Chubu Earthquake, Japan, M = 7.7, May 1983

Figure E1: Calculated time series at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g.

Figure E2: Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g.

Figure E3: Calculated time series at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.23 g.

Figure E4: Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Nihonkai Chubu Earthquake scaled to 0.23 g.
Imperial Valley Earthquake, USA, $M = 6.6$, October 1979

Figure F.5: Calculated time series at the surface at Ohama No.1 Wharf of the Imperial Valley Earthquake scaled to 0.10 g.

Figure F.6: Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Imperial Valley Earthquake scaled to 0.10 g.

Figure F.7: Calculated time series at the surface at Ohama No.1 Wharf of the Imperial Valley Earthquake scaled to 0.23 g.

Figure F.8: Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Imperial Valley Earthquake scaled to 0.23 g.
Landers Earthquake, United States of America, $M = 7.3$, June 1992

Figure F.9: Calculated time series at the surface at Ohama No.1 Wharf of the Landers Earthquake scaled to 0.10 g.

Figure F.10: Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Landers Earthquake scaled to 0.10 g.

Figure F.11: Calculated time series at the surface at Ohama No.1 Wharf of the Landers Earthquake scaled to 0.23 g.

Figure F.12: Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Landers Earthquake scaled to 0.23 g.
Kocaeli Earthquake, Turkey, $M = 7.6$, August 1999

Figure F.13: Calculated time series at the surface at Ohama No.1 Wharf of the Kocaeli Earthquake scaled to 0.10 g.

Figure F.14: Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Kocaeli Earthquake scaled to 0.10 g.

Figure F.15: Calculated time series at the surface at Ohama No.1 Wharf of the Kocaeli Earthquake scaled to 0.23 g.

Figure F.16: Obtained frequency amplitude spectrum at the surface at Ohama No.1 Wharf of the Kocaeli Earthquake scaled to 0.23 g.
E2. Ohama No.2 Wharf
Nihonkai Chubu Earthquake, Japan, $M = 7.7$, May 1983

Figure E17: Calculated time series at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g.

Figure E18: Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.10 g.

Figure E19: Calculated time series at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.23 g.

Figure E20: Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Nihonkai Chubu Earthquake scaled to 0.23 g.
Imperial Valley Earthquake, USA, M = 6.6, October 1979

Figure F.21: Calculated time series at the surface at Ohama No.2 Wharf of the Imperial Valley Earthquake scaled to 0.10 g.

Figure F.22: Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Imperial Valley Earthquake scaled to 0.10 g.

Figure F.23: Calculated time series at the surface at Ohama No.2 Wharf of the Imperial Valley Earthquake scaled to 0.23 g.

Figure F.24: Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Imperial Valley Earthquake scaled to 0.23 g.
Landers Earthquake, United States of America, $M = 7.3$, June 1992

Figure E25: Calculated time series at the surface at Ohama No.2 Wharf of the Landers Earthquake scaled to 0.10 g.

Figure E26: Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Landers Earthquake scaled to 0.10 g.

Figure E27: Calculated time series at the surface at Ohama No.2 Wharf of the Landers Earthquake scaled to 0.23 g.

Figure E28: Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Landers Earthquake scaled to 0.23 g.
Kocaeli Earthquake, Turkey, $M = 7.6$, August 1999

Figure E29: Calculated time series at the surface at Ohama No.2 Wharf of the Kocaeli Earthquake scaled to 0.10 g.

Figure E30: Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Kocaeli Earthquake scaled to 0.10 g.

Figure E31: Calculated time series at the surface at Ohama No.2 Wharf of the Kocaeli Earthquake scaled to 0.23 g.

Figure E32: Obtained frequency amplitude spectrum at the surface at Ohama No.2 Wharf of the Kocaeli Earthquake scaled to 0.23 g.
Model parameters dynamic analysis
Ohama No.2 Wharf

In this appendix calibrated model parameters for the UBC3D-PLM model are presented. These parameter sets are calibrated to fit the liquefaction resistance of the considered soils for different element tests with varying initial conditions. Around the structure zones are identified with similar stress states as in these element tests. The calibrated parameter sets are assigned these zones corresponding to a specific element tests.

See section 6.4 for the elaboration of the model and results of the dynamic analysis of Ohama No.2 Wharf.
Table G.1: Calibrated parameters UBC3D-PLM model Ohama sand for stress paths corresponding to undrained cyclic DSS tests.

<table>
<thead>
<tr>
<th>Ohama sand</th>
<th>DSS</th>
<th>DSS</th>
<th>DSS</th>
<th>DSS</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_0$ [-]</td>
<td>0.40</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>$\alpha$ [-]</td>
<td>0.00</td>
<td>0.00</td>
<td>0.05</td>
<td>0.10</td>
</tr>
<tr>
<td>$(N1)_{60}$ [blows]</td>
<td>9</td>
<td>9</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>$\varphi_{cv}$ [°]</td>
<td>30.0</td>
<td>30.0</td>
<td>30.0</td>
<td>30.0</td>
</tr>
<tr>
<td>$\varphi_p'$ [°]</td>
<td>30.9</td>
<td>30.9</td>
<td>30.9</td>
<td>30.9</td>
</tr>
<tr>
<td>$c$ kN/m²</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$k_G^c$ [-]</td>
<td>902.1</td>
<td>902.1</td>
<td>902.1</td>
<td>902.1</td>
</tr>
<tr>
<td>$k_G^p$ [-]</td>
<td>319.2</td>
<td>319.2</td>
<td>319.2</td>
<td>319.2</td>
</tr>
<tr>
<td>$k_B^c$ [-]</td>
<td>631.5</td>
<td>631.5</td>
<td>631.5</td>
<td>631.5</td>
</tr>
<tr>
<td>$k_B^p$ [-]</td>
<td>631.5</td>
<td>631.5</td>
<td>631.5</td>
<td>631.5</td>
</tr>
<tr>
<td>me [-]</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>ne [-]</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>np [-]</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td>$R_f$ [-]</td>
<td>0.7911</td>
<td>0.7911</td>
<td>0.7911</td>
<td>0.7911</td>
</tr>
<tr>
<td>$p_A$ kN/m²</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$\sigma_f$ kN/m²</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$f_{ac_{hard}}$ [-]</td>
<td>0.30</td>
<td>0.30</td>
<td>0.45</td>
<td>1.00</td>
</tr>
<tr>
<td>$f_{ac_{post}}$ [-]</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Table G.2: Calibrated parameters UBC3D-PLM model Ohama sand for stress paths corresponding to undrained cyclic triaxial tests.

<table>
<thead>
<tr>
<th>Ohama sand</th>
<th>TC</th>
<th>TC</th>
<th>TE</th>
<th>TE</th>
<th>TE</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_0$ [-]</td>
<td>0.40</td>
<td>0.45</td>
<td>0.65</td>
<td>0.65</td>
<td>0.75</td>
</tr>
<tr>
<td>$\alpha$ [-]</td>
<td>0.10</td>
<td>0.10</td>
<td>0.00</td>
<td>0.15</td>
<td>0.20</td>
</tr>
<tr>
<td>$(N1)_{60}$ [blows]</td>
<td>9</td>
<td>9</td>
<td>9</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>$\varphi_{cv}$ [°]</td>
<td>30.0</td>
<td>30.0</td>
<td>30.0</td>
<td>30.0</td>
<td>30.0</td>
</tr>
<tr>
<td>$\varphi_p'$ [°]</td>
<td>30.9</td>
<td>30.9</td>
<td>30.9</td>
<td>30.9</td>
<td>30.9</td>
</tr>
<tr>
<td>$c$ kN/m²</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$k_G^c$ [-]</td>
<td>902.1</td>
<td>902.1</td>
<td>902.1</td>
<td>902.1</td>
<td>902.1</td>
</tr>
<tr>
<td>$k_G^p$ [-]</td>
<td>1007</td>
<td>1007</td>
<td>1007</td>
<td>1007</td>
<td>1007</td>
</tr>
<tr>
<td>$k_B^c$ [-]</td>
<td>631.5</td>
<td>631.5</td>
<td>631.5</td>
<td>631.5</td>
<td>631.5</td>
</tr>
<tr>
<td>$k_B^p$ [-]</td>
<td>631.5</td>
<td>631.5</td>
<td>631.5</td>
<td>631.5</td>
<td>631.5</td>
</tr>
<tr>
<td>me [-]</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>ne [-]</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>np [-]</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td>$R_f$ [-]</td>
<td>0.7911</td>
<td>0.7911</td>
<td>0.7911</td>
<td>0.7911</td>
<td>0.7911</td>
</tr>
<tr>
<td>$p_A$ kN/m²</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>$\sigma_f$ kN/m²</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$f_{ac_{hard}}$ [-]</td>
<td>0.23</td>
<td>0.23</td>
<td>0.23</td>
<td>0.45</td>
<td>0.90</td>
</tr>
<tr>
<td>$f_{ac_{post}}$ [-]</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Table G.3: Calibrated parameters UBC3D-PLM model Gaiko sand for stress paths corresponding to undrained cyclic DSS tests.

<table>
<thead>
<tr>
<th>Gaiko sand</th>
<th>DSS</th>
<th>DSS</th>
<th>DSS</th>
<th>DSS</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_0$ [-]</td>
<td>0.40</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>$\alpha$ [-]</td>
<td>0.00</td>
<td>0.00</td>
<td>0.05</td>
<td>0.10</td>
</tr>
<tr>
<td>$(N1)_{60}$ [blows]</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>$\varphi_{cv}$ [°]</td>
<td>33.0</td>
<td>33.0</td>
<td>33.0</td>
<td>33.0</td>
</tr>
<tr>
<td>$\varphi_p$ [°]</td>
<td>34.0</td>
<td>34.0</td>
<td>34.0</td>
<td>34.0</td>
</tr>
<tr>
<td>$c$ kN/m$^2$</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$k_e^G$ [-]</td>
<td>934.5</td>
<td>934.5</td>
<td>934.5</td>
<td>934.5</td>
</tr>
<tr>
<td>$k_p^G$ [-]</td>
<td>1141</td>
<td>1141</td>
<td>1141</td>
<td>1141</td>
</tr>
<tr>
<td>$k_B^G$ [-]</td>
<td>654</td>
<td>654</td>
<td>654</td>
<td>654</td>
</tr>
<tr>
<td>$m_e$ [-]</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>$n_e$ [-]</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>$n_p$ [-]</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td>$R_f$ [-]</td>
<td>0.7787</td>
<td>0.7787</td>
<td>0.7787</td>
<td>0.7787</td>
</tr>
<tr>
<td>$p_A$ kN/m$^2$</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$\sigma_t$ kN/m$^2$</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$f_{ac_{hard}}$ [-]</td>
<td>0.30</td>
<td>0.30</td>
<td>0.40</td>
<td>0.65</td>
</tr>
<tr>
<td>$f_{ac_{post}}$ [-]</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Table G.4: Calibrated parameters UBC3D-PLM model Gaiko sand for stress paths corresponding to undrained cyclic triaxial tests.

<table>
<thead>
<tr>
<th>Gaiko sand</th>
<th>TC</th>
<th>TC</th>
<th>TE</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_0$ [-]</td>
<td>0.40</td>
<td>0.45</td>
<td>2.00</td>
</tr>
<tr>
<td>$\alpha$ [-]</td>
<td>0.10</td>
<td>0.10</td>
<td>0.20</td>
</tr>
<tr>
<td>$(N1)_{60}$ [blows]</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>$\varphi_{cv}$ [°]</td>
<td>33.0</td>
<td>33.0</td>
<td>33.0</td>
</tr>
<tr>
<td>$\varphi_p$ [°]</td>
<td>34.0</td>
<td>34.0</td>
<td>34.0</td>
</tr>
<tr>
<td>$c$ kN/m$^2$</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$k_e^G$ [-]</td>
<td>934.5</td>
<td>934.5</td>
<td>934.5</td>
</tr>
<tr>
<td>$k_p^G$ [-]</td>
<td>1521</td>
<td>1521</td>
<td>1900</td>
</tr>
<tr>
<td>$k_B^G$ [-]</td>
<td>654</td>
<td>654</td>
<td>654</td>
</tr>
<tr>
<td>$m_e$ [-]</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>$n_e$ [-]</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>$n_p$ [-]</td>
<td>0.40</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td>$R_f$ [-]</td>
<td>0.7787</td>
<td>0.7787</td>
<td>0.7787</td>
</tr>
<tr>
<td>$p_A$ kN/m$^2$</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$\sigma_t$ kN/m$^2$</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$f_{ac_{hard}}$ [-]</td>
<td>0.50</td>
<td>0.50</td>
<td>1.00</td>
</tr>
<tr>
<td>$f_{ac_{post}}$ [-]</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>