Deltares

Stress state under the track bed

Literature review and examples of sites



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Author(s)
Alexander van Duinen

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Client	Allianz Fahrweg Normalspur
Contact	Dr. James Fern
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Author(s)		
	Alexander van Duinen	

Summary

A small but important part of the Swiss railway network is founded on soft soils. These are old railway lines. The use of these lines with modern railway traffic must be verified. Large deformations and strength and stiffness losses are major geotechnical risks for these railway lines. These issues may cause significant track maintenance and operational cost as well as being a safety issue. To better identify critical sites, the railway-induced stress state needs to better understood. Allianz Fahrweg Normalspur has contacted Deltares for assistance in the determination of the stress-state.

Objective
The main objective of this study is to understand the influence of the track bed on the in-situ stress state. This includes:
- Understanding of the influence of the track bed (train, track-bed, foundation) on the in-situ stress state, both for static and cyclic conditions.
- Determination of the stress state under the track bed of railways.
- Give an overview of the state-of-the-art methods and theories to determine the stress state, both for static and cyclic conditions.
- Build up confidence in CPT interpretation to determine the stress state.
- Evaluation of four example sites based on CPT interpretation. The four sites have different soil conditions: soft clayey layers, liquifiable layers, sensitive lake chalk layers and soft peaty layers.

Main conclusions
To get a proper understanding of the influence of the track bed on the in-situ stress state, both for static and cyclic conditions, it is important to get good insight in the subsoil, soil type, pore water pressures, etcetera. Finite Element Models (FEM) with for example Hardening Soil Small Strain Model as a constitutive model can be applied to get a detailed insight in the influence of the track bed on the in-situ stress state. To get accurate results with using these models reliable input parameters of the subsoil and the soil conditions are required. CPTs can be used to retrieve insight in the subsoil. However, CPT interpretation is not straightforward. Because very different soil conditions can be encountered standard CPT correlations from literature can only be used as a first screening method. Cross checks with other field tests and laboratory testing are required to get reliable results.

Conclusions concerning the stress state at static conditions
The stress state under the track bed at static conditions depends on many factors. These are:
- Thickness and weight of the track bed and the railway embankment.
- Stratigraphy of the subsoil with different soil types having different bulk unit weights.
- Stress history of the subsoil related to the geological background (sedimentary environment) of the subsoil and also including soil forming processes (ripening, cementation) and creep (post-sedimentary processes or ageing).
- Pore water pressures in the railway embankment and the subsoil under different geohydrologic conditions, including seasonal variations.

The influence of many of these factors on the stress state is not straightforward:
- The thickness of the track bed and the railway embankment and the stratigraphy of the subsoil can vary considerably which result in variations in the effective stresses. These variations can occur on relatively short distances.
- The stress history of the subsoil and the railway embankment can also vary because there are many factors which influence the stress history, such as the geological

background of the soil, building of the embankment and influence of the train loading. So, at different sites different aspects of the stress history can be dominant.
- The pore water pressures can be very different at different locations due to the geological and geohydrological conditions (artesian groundwater conditions or underdrainage conditions) or the construction of the railway embankment and also vary considerably in time due to seasonal effects. This also affects the stress state.

Careful site investigation is required to quantify the effect of all these aspects on the stress state of the track bed.

Conclusions concerning the stress state at cyclic conditions

The main findings concerning the stress state at cyclic conditions are:
- From the available literature on laboratory tests follows that significant pore pressure build-up after a train passage seems unlikely in case of already cyclic preloaded soil (with intermediate drainage) if the cyclic shear stress amplitude is not enlarged (no change in axle loads and speeds). An increasing pore pressure build-up by subsequent train passages is also not reported in literature on the few available field measurements. This is confirmed by pore pressure measurements at the Delft – Schiedam railway line in The Netherlands. This measurement campaign shows a small increase of pore pressure due to an individual train passage with short peaks. Around 98% of the pore pressure peaks dissipate in 100 seconds or less.
- A strong attenuation of the train induced vertical displacements and excess pore water pressure with depth can be expected. This follows from dynamic 3D FEM analyses with moving loads which are performed for the Delft – Schiedam railway line. The largest displacements (0.30 mm) are only visible in the embankment, especially in the top part of the embankment above the phreatic surface which exhibits more compression. However, at 6 m below the embankment still some train induced deformations occur. The pattern of the attenuation with depth will depend on the configuration of the track bed and the embankment and the level of the phreatic surface.
- CPTs conducted in the railway embankment of the Delft – Schiedam railway line show a strong decrease of the density of the sand in the embankment. Within about 4 m below the top of the embankment the cone resistances reduce to low values.
- Assuming quasi-static conditions (ignoring dynamic amplification of soil displacements and stresses) under the track bed is allowed as long as the train speed is below approximately 60% of the local critical train speed.
- When train speed, axle load or train frequency are going to increase one can expect an increase of the vertical displacements and excess pore pressures. These effects become stronger towards the critical train speed. How these effects attenuate with depths will depend on the configuration of the track bed and the embankment and the level of the phreatic surface.

Conclusions concerning the application of CPTs to determine the stress state for field conditions

CPTs are a powerful tool to investigate the conditions of railway embankments and the subsoil and to determine the stress state below the track bed. This concerns especially the static conditions of the stress state of soft soils. Some aspects need attention:
- The methods to estimate bulk unit weight based on CPTs are not accurate. This introduces an error in the stress state. In two of the examined examples the error in the effective stress was between 4 and 10%. This influences also OCR and K_0.
- The stationary pore water pressure in the subsoil is often difficult to deduce from CPTs. This is the case when there are no permeable soil layers within the depth of the CPT measurements. In one of the examined examples different assumptions on the pore water pressure gradient result in an uncertainty in the effective stress and therefore also an uncertainty in OCR of around a factor 1.5.

- In most CPTs which are analysed to derive the stress state for the four examples the reliability of the measured pore water pressure u_2 seems to be poor. This gives a limitation in the interpretation of the CPT concerning the soil classification.
- The methods to interpret CPT measurements are developed for certain soil types. For other soil types the interpretation methods may be less accurate. To increase the accuracy of the determination of the stress state cross checks with field vane tests or high-quality laboratory tests are essential. This is also the case for the interpretation of the seismic cone.
- It is not possible to derive an accurate estimation of the horizontal stress from CPT measurements. When the horizontal stress is important pressure meter tests or dilatometer tests are required.
- A CPT does not give an accurate estimation of the sensitivity of soil layers, such as lake chalk. For sensitive soils additional measurements with field vane tests are required.
- To assess the state of sand and silt various interpretation methods for CPTs are available in literature. To assure that the CPT interpretation is correct, especially for partially drained soil conditions during CPT penetration, additional laboratory testing is required.
- For the interpretation of CPTs in partially saturated soils additional information on water content, degree of saturation and capillary suction is needed as the capillary suction influences the CPT measurements.

Approach

To give an overview of the state-of-the-art methods and theories to determine the stress state under the track bed a literature study is performed. In this literature study the focus was on situations that are similar to the Swiss railway foundations.

The first step in the literature review is the further definition of the stress-state. The rest of the literature study focussed on field measurement techniques, spatial variability and the application of the various CPT interpretation methods to four examples locations.

The scope of the literature review is aligned with the client.

The client shared data from four examples of sites in Switzerland with different geological conditions. An exploratory analysis of the provided data of the four examples of sites is conducted. This analysis is based on the findings of the literature review and Deltares experience. For each of the examples one or two CPTs are analysed to derive the stress state of the railway embankment and the subsoil. The other available geotechnical data such as boreholes and index tests are used to be able to interpret the CPTs.

Zusammenfassung

Ein kleiner, aber wichtiger Teil des Schweizer Eisenbahnnetzes ist auf weichen Böden gegründet. Dabei handelt es sich um alte Bahnlinien. Die Nutzung dieser Strecken mit modernem Eisenbahnverkehr muss überprüft werden. Große Verformungen sowie Festigkeits- und Steifigkeitsverluste sind große geotechnische Risiken für diese Bahnlinien. Diese potentiellen Probleme können erhebliche Kosten für die Instandhaltung und den Betrieb der Strecken verursachen und stellen auch ein Sicherheitsproblem dar. Um kritische Stellen besser identifizieren zu können, muss der eisenbahninduzierte Spannungszustand besser verstanden werden. Die Allianz Fahrweg Normalspur hat sich an Deltares gewandt, um Unterstützung bei der Bestimmung des Spannungszustandes zu erhalten.

Zielsetzung
Das Hauptziel dieser Studie ist es, den Einfluss des Gleisbetts auf den Spannungszustand in situ zu verstehen. Dies beinhaltet:

- Verständnis des Einflusses des Gleisbetts auf den Spannungszustand in situ, sowohl für statische als auch für zyklische Bedingungen.
- Bestimmung des Spannungszustandes unter dem Gleisbett von Schienenwegen.
- Überblick über die modernsten Methoden und Theorien zur Bestimmung des Spannungszustandes, sowohl für statische als auch für zyklische Bedingungen.
- Aufbau von Vertrauen in die CPT-Interpretation zur Bestimmung des Spannungszustandes.
- Bewertung von vier Beispielstandorten auf der Grundlage der CPT-Interpretation. Die vier Standorte weisen unterschiedliche Bodenbedingungen auf: weiche tonige Schichten, verflüssigbare Schichten, empfindliche Seekreideschichten und weiche torfige Schichten.

Wichtigste Schlussfolgerungen
Um den Einfluss des Gleisbetts auf den Spannungszustand in situ sowohl für statische als auch für zyklische Bedingungen richtig verstehen zu können, ist es wichtig, einen guten Einblick in den Untergrund, die Bodenart, die Porenwasserdrücke usw. zu erhalten. Finite-Elemente-Modelle (FEM), z. B. mit dem Hardening Soil Small Strain Model als konstitutives Modell, können angewendet werden, um einen detaillierten Einblick in den Einfluss des Gleisbetts auf den Spannungszustand in-situ zu erhalten. Um mit diesen Modellen genaue Ergebnisse zu erhalten, sind zuverlässige Eingabeparameter für den Untergrund und die Bodenbedingungen erforderlich. CPTs können verwendet werden, um einen Einblick in den Untergrund zu erhalten. Die Interpretation von CPTs ist jedoch nicht einfach. Da sehr unterschiedliche Bodenbedingungen angetroffen werden können, können Standard-CPT-Korrelationen aus der Literatur nur als erste Screening-Methode verwendet werden. Um zuverlässige Ergebnisse zu erhalten, sind Gegenkontrollen mit anderen Feldtests und Laboruntersuchungen erforderlich.

Schlussfolgerungen in Bezug auf den Spannungszustand unter statischen Bedingungen
Der Spannungszustand unter dem Gleisbett unter statischen Bedingungen hängt von vielen Faktoren ab. Diese sind:

- Mächtigkeit und Gewicht des Gleisbetts und des Bahndamms.
- Stratigraphie des Untergrunds mit verschiedenen Bodentypen und unterschiedlichen Schüttgewichten.
- Spannungsgeschichte des Untergrunds in Verbindung mit dem geologischen Hintergrund (sedimentäres Umfeld) des Untergrunds sowie Bodenbildungsprozesse (Reifung, Zementierung) und Kriechvorgänge (post-sedimentäre Prozesse oder Alterung).

- Porenwasserdrücke im Bahndamm und im Untergrund unter verschiedenen geohydrologischen Bedingungen, einschließlich jahreszeitlicher Schwankungen.

Der Einfluss vieler dieser Faktoren auf den Spannungszustand ist nicht einfach zu bestimmen:

- Die Mächtigkeit des Gleisbettes und des Bahndammes sowie die Stratigraphie des Untergrundes können erheblich variieren, was zu Schwankungen der effektiven Spannungen führt. Diese Schwankungen können auf relativ kurzen Strecken auftreten.
- Der Spannungsverlauf des Untergrunds und des Bahndamms kann ebenfalls variieren, da es viele Faktoren gibt, die den Spannungsverlauf beeinflussen, wie z.B. der geologische Hintergrund des Bodens, der Aufbau des Dammes und der Einfluss der Zugbelastung. So können an verschiedenen Standorten unterschiedliche Aspekte des Spannungsverlaufs vorherrschen.
- Die Porenwasserdrücke können aufgrund der geologischen und geohydrologischen Verhältnisse (artesische Grundwasserverhältnisse oder Unterspülungsverhältnisse) oder der Bauweise des Bahndamms an verschiedenen Stellen sehr unterschiedlich sein und auch zeitlich aufgrund von jahreszeitlichen Effekten stark schwanken. Dies wirkt sich auch auf den Spannungszustand aus.

Um die Auswirkungen all dieser Aspekte auf den Spannungszustand des Gleisbetts zu quantifizieren, ist eine sorgfältige Untersuchung des Standorts erforderlich.

Schlussfolgerungen bezüglich des Spannungszustands unter zyklischen Bedingungen
Die wichtigsten Erkenntnisse über den Spannungszustand unter zyklischen Bedingungen sind:

- Aus der verfügbaren Literatur über Laborversuche geht hervor, dass ein signifikanter Porendruckaufbau nach einer Zugüberfahrt bei bereits zyklisch vorbelastetem Boden (mit Drainage zwischen zwei Zugpassagen) unwahrscheinlich erscheint, wenn die zyklische Schubspannungsamplitude nicht vergrößert wird (keine Änderung der Achslasten und Geschwindigkeiten). Ein zunehmender Porendruckaufbau durch nachfolgende Zugüberfahrten wird auch in der Literatur bei den wenigen verfügbaren Feldmessungen nicht berichtet. Dies wird durch Porendruckmessungen an der Bahnstrecke Delft - Schiedam in den Niederlanden bestätigt. Diese Messkampagne zeigt einen geringen Anstieg des Porendrucks aufgrund einer einzelnen Zugdurchfahrt mit kurzen Spitzen. Etwa 98% der Porendruckspitzen bauen sich in 100 Sekunden oder weniger ab.
- Es ist eine starke Abschwächung der durch den Zug verursachten vertikalen Verschiebungen und des überschüssigen Porenwasserdrucks mit der Tiefe zu erwarten. Dies ergibt sich aus dynamischen 3D-FEM-Analysen mit beweglichen Lasten, die für die Bahnstrecke Delft - Schiedam durchgeführt wurden. Die größten Verschiebungen (0,30 mm) sind nur im Damm sichtbar, vor allem im oberen Teil des Bahndammes oberhalb der phreatischen Oberfläche, die eine stärkere Kompression aufweist. In 6 m Tiefe unter dem Damm treten jedoch noch einige zugbedingte Verformungen auf. Das Muster der Abschwächung mit der Tiefe hängt von der Konfiguration des Gleisbetts und des Dammes sowie von der Höhe der Grundwasseroberfläche ab.
- CPTs, die im Bahndamm der Bahnlinie Delft - Schiedam durchgeführt wurden, zeigen eine starke Abnahme der Dichte des Sandes im Damm. Innerhalb von etwa 4 m unterhalb der Böschungsoberkante sinken die CPT-Konuswiderstände auf niedrige Werte.
- Die Annahme quasistatischer Bedingungen (ohne dynamische Verstärkung der Bodenverschiebungen und -spannungen) unter dem Gleisbett ist zulässig, solange die Zuggeschwindigkeit unter etwa 60 % der lokalen kritischen Zuggeschwindigkeit liegt.

Wenn die Zuggeschwindigkeit, die Achslast oder die Zugfrequenz zunehmen, ist mit einem Anstieg der vertikalen Verschiebungen und der Porenwasserüberdrücke zu rechnen.

Diese Effekte werden in Richtung der kritischen Zuggeschwindigkeit stärker. Wie sich diese Effekte mit zunehmender Tiefe abschwächen, hängt von der Konfiguration des Gleisbetts und des Bahndammes sowie von der Höhe der Grundwasseroberfläche ab.

Schlussfolgerungen bezüglich der Anwendung von CPTs zur Bestimmung des Spannungszustands

CPTs sind ein leistungsfähiges Instrument zur Untersuchung des Zustands von Bahndämmen und des Untergrunds sowie zur Ermittlung des Spannungszustands des Gleisbetts. Dies gilt insbesondere für die statischen Bedingungen des Spannungszustands weicher Böden. Einige Aspekte bedürfen der Aufmerksamkeit:

- Die Methoden zur Schätzung des Schüttgewichts auf der Grundlage von CPTs sind nicht genau. Dies führt zu einem Fehler im Spannungszustand. In zwei der untersuchten Beispiele lag der Fehler bei der effektiven Spannung zwischen 4 und 10%. Dies wirkt sich auch auf OCR und K_0 aus.
- Der stationäre Porenwasserdruck im Untergrund lässt sich oft nur schwer aus CPTs ableiten. Dies ist dann der Fall, wenn es innerhalb der Tiefe der CPT-Messungen keine durchlässigen Bodenschichten gibt. In einem der untersuchten Beispiele führen unterschiedliche Annahmen über den Porenwasserdruckgradienten zu einer Unsicherheit bei der effektiven Spannung und damit auch zu einer Unsicherheit bei der OCR von etwa einem Faktor 1,5.
- In den meisten CPTs, die zur Ableitung des Spannungszustandes für die vier Beispiele analysiert wurden, scheint die Zuverlässigkeit des gemessenen Porenwasserdrucks u_2 schlecht zu sein. Daraus ergibt sich eine Einschränkung bei der Interpretation der CPT hinsichtlich der Bodenklassifizierung.
- Die Methoden zur Interpretation von CPT-Messungen wurden für bestimmte Bodentypen entwickelt. Für andere Bodentypen können die Interpretationsmethoden weniger genau sein. Um die Genauigkeit der Bestimmung des Spannungszustands zu erhöhen, sind Gegenkontrollen mit Feldfahnenversuchen oder hochwertigen Laborversuchen unerlässlich. Dies gilt auch für die Interpretation des seismischen Kegels.
- Es ist nicht möglich, aus CPT-Messungen eine genaue Schätzung der horizontalen Spannung abzuleiten. Wenn die horizontale Spannung von Bedeutung ist, sind Druckmessertests oder Dilatometertests erforderlich.
- Eine CPT liefert keine genaue Einschätzung der Empfindlichkeit $S_t$von Bodenschichten, wie z. B. Seekreide. Bei empfindlichen Böden sind zusätzliche Messungen mit Flügelsondierungsversuchen im Feld erforderlich.
- Zur Beurteilung des Zustands von Sand und Schluff sind in der Literatur verschiedene Interpretationsmethoden für CPTs verfügbar. Um sicherzustellen, dass die CPT-Interpretation korrekt ist, insbesondere bei teilweise entwässerten Böden während der CPT-Eindringung, sind zusätzliche Laboruntersuchungen erforderlich.
- Für die Interpretation von CPTs in teilgesättigten Böden sind zusätzliche Informationen über den Wassergehalt, den Sättigungsgrad und den Kapillarsog erforderlich, da der Kapillarsog die CPT-Messungen beeinflusst.

Ansatz
Um einen Überblick über den Stand der Technik und die Theorien zur Ermittlung des Spannungszustandes unter dem Gleisbett zu geben, wird eine Literaturstudie durchgeführt. In dieser Literaturstudie wird der Fokus auf Situationen gelegt, die den schweizerischen Bahnfundamenten ähnlich sind.

Der erste Schritt der Literaturstudie ist die weitere Definition des Spannungszustandes. Die übrige Literaturstudie konzentriert sich auf Feldmessverfahren, räumliche Variabilität und die Anwendung der verschiedenen CPT-Interpretationsmethoden an vier Beispielstandorten.

Der Umfang dieser Literaturstudie ist mit dem Kunden abgestimmt.

Der Kunde stellte Daten von vier Beispielstandorten in der Schweiz mit unterschiedlichen geologischen Bedingungen zur Verfügung. Es wird eine orientierende Analyse der bereitgestellten Daten der vier Beispielstandorte durchgeführt. Diese Analyse basiert auf den Erkenntnissen aus der Literaturrecherche und den Erfahrungen von Deltares. Für jedes der Beispiele werden ein oder zwei CPTs analysiert, um den Spannungszustand des Bahndammes und des Untergrundes abzuleiten. Die anderen verfügbaren geotechnischen Daten wie Bohrungen und Indexversuche werden verwendet, um die CPTs interpretieren zu können.

Résumé

Une petite mais importante partie du réseau ferroviaire suisse repose sur des sols mous. Il s'agit d'anciennes lignes ferroviaires. L'utilisation de ces lignes avec le trafic ferroviaire moderne doit être vérifiée. Les grandes déformations et les pertes de résistance et de rigidité constituent des risques géotechniques majeurs pour ces lignes ferroviaires. Ces problèmes peuvent entraîner des coûts importants d'entretien et d'exploitation de la voie ferrée et constituer un problème de sécurité. Pour mieux identifier les sites critiques, il est nécessaire de mieux comprendre l'état de contrainte induit par les chemins de fer. Allianz Fahrweg Normalspur a contacté Deltares pour obtenir de l'aide dans la détermination de l'état de contrainte.

Objectif de l'étude
L'objectif principal de cette étude est de comprendre l'influence de la plate-forme de la voie sur l'état de contrainte in situ. Ceci comprend:

- La compréhension de l'influence de l'assise de la voie sur l'état de contrainte in situ, à la fois pour des conditions statiques et cycliques.
- Détermination de l'état de contrainte sous la plate-forme des voies ferrées.
- Donner une vue d'ensemble des méthodes et théories de pointe pour déterminer l'état de contrainte, à la fois pour des conditions statiques et cycliques.
- Renforcer la confiance dans l'interprétation des essais CPT pour déterminer l'état de contrainte.
- Évaluation de quatre sites d'exemple basés sur l'interprétation CPT. Les quatre sites ont des conditions stratigraphiques différentes: couches argileuses molles, couches liquéfiables, couches de craie lacustre sensibles et couches tourbeuses molles.

Conclusions principales
Pour bien comprendre l'influence de la plate-forme de la voie sur l'état de contrainte in situ, à la fois pour les conditions statiques et cycliques, il est important d'avoir une bonne connaissance du sous-sol, du type de sol, des pressions de l'eau interstitielle, etc. Les modèles d'éléments finis (FEM) avec, par exemple ‘Hardening Soil Small Strain Model’ comme modèle constitutif, peuvent être appliqués pour obtenir un aperçu détaillé de l'influence de la plate-forme de la voie sur l'état de contrainte in-situ. Pour obtenir des résultats précis à l'aide de ces modèles, des paramètres d'entrée fiables du sous-sol et des conditions du sol sont nécessaires. Les essais CPT peuvent être utilisés pour obtenir des informations sur le sous-sol. Cependant, l'interprétation des essais CPT n'est pas simple. Étant donné que des conditions de sol très différentes peuvent être rencontrées, les corrélations CPT standards issues de la littérature ne peuvent être utilisées seulement comme première méthode de sélection. Des vérifications croisées avec d'autres tests sur le terrain et en laboratoire sont nécessaires pour obtenir des résultats fiables.

Conclusions concernant l'état de contrainte en conditions statiques
L'état de contrainte sous la plate-forme de la voie en conditions statiques dépend de nombreux facteurs. Ces facteurs sont les suivants :

- L'épaisseur et le poids de la plate-forme de la voie et du remblai de la voie ferrée.
- Stratigraphie du sous-sol avec différents types de sol ayant différents poids volumiques. L'historique des contraintes du sous-sol lié au contexte géologique (environnement sédimentaire) du sous-sol, comprenant également les processus de formation du sol (maturation, cimentation) et le fluage (processus post-sédimentaires ou vieillissement).
- Pressions de l'eau interstitielle dans le remblai ferroviaire et le sous-sol dans différentes conditions géohydrologiques, y compris les variations saisonnières.

L'influence de plusieurs de ces facteurs sur l'état de contrainte n'est pas simple:

- L'épaisseur de la plate-forme et du remblai ferroviaire, ainsi que la stratigraphie du sous-sol, peuvent varier considérablement, ce qui entraîne des variations des contraintes effectives. Ces variations peuvent se produire sur des distances relativement courtes.
- L'historique des contraintes du sous-sol et du remblai ferroviaire peut également varier en raison des nombreux facteurs qui influencent l'historique des contraintes, tels que le contexte géologique du sol, la construction du remblai et l'influence de la charge du train. Ainsi, sur différents sites, différents aspects de l'historique des contraintes peuvent être dominants.
- Les pressions de l'eau interstitielle peuvent être très différentes d'un endroit à l'autre en raison des conditions géologiques et géohydrologiques (conditions de la nappe phréatique artésienne ou conditions de drainage souterrain) ou de la construction du remblai de la voie ferrée et varient également considérablement dans le temps en raison des effets saisonniers. Cela affecte également l'état de contrainte.

Une étude minutieuse du site est nécessaire pour quantifier l'effet de tous ces aspects sur l'état de contrainte de la plate-forme de la voie.

Conclusions concernant l'état de contrainte aux conditions cycliques

Les principales conclusions concernant l'état de contrainte dans des conditions cycliques sont les suivantes:

- La littérature disponible sur les essais en laboratoire montre qu'une augmentation significative de la pression interstitielle après le passage d'un train semble improbable dans le cas d'un sol déjà préchargé cycliquement (avec un drainage intermédiaire) si l'amplitude de la contrainte de cisaillement cyclique n'est pas augmentée (pas de changement dans les charges par essieu, ni de changement de vitesse). L'augmentation de la pression interstitielle lors des passages ultérieurs des trains n'est pas non plus rapportée dans la littérature sur les quelques mesures de terrain disponibles. Ceci est confirmé par des mesures de la pression interstitielle sur la ligne de chemin de fer Delft - Schiedam aux Pays-Bas. Cette campagne de mesures montre une faible augmentation de la pression interstitielle due au passage d'un train individuel avec des pics courts. Environ 98 % des pics de pression interstitielle se dissipent en 100 secondes ou moins.
- On peut s'attendre à une forte atténuation des déplacements verticaux induits par le train et de l'excès de pression de l'eau interstitielle avec la profondeur. Ceci découle des analyses FEM 3D dynamiques avec des charges mobiles qui ont été réalisées pour la ligne de chemin de fer Delft - Schiedam. Les déplacements les plus importants (0,30 mm) ne sont visibles que dans le remblai, en particulier dans la partie supérieure du remblai au-dessus de la surface phréatique qui présente une compression plus importante. Cependant, à 6 m sous le talus, on observe encore des déformations induites par le train. Le profil de l'atténuation en fonction de la profondeur dépendra de la configuration de la plate-forme de la voie et du remblai, ainsi que du niveau de la surface phréatique.
- Les essais CPT effectués dans le remblai de la ligne de chemin de fer Delft - Schiedam montrent une forte diminution de la densité du sable dans le remblai. Dans un rayon d'environ 4 m sous le sommet du remblai, les résistances des cônes sont réduites à de faibles valeurs.
- L'hypothèse de conditions quasi-statiques (ignorant l'amplification dynamique des déplacements et des contraintes du sol) sous la plate-forme de la voie est autorisée tant que la vitesse du train est inférieure à environ 60 % de la vitesse critique locale.
- Lorsque la vitesse du train, la charge par essieu ou la fréquence du train augmentent, on peut s'attendre à une augmentation des déplacements verticaux et des pressions interstitielles. Ces effets s'accentuent à mesure que l'on s'approche de la vitesse critique du train. L'atténuation de ces effets en fonction de la profondeur dépend de la configuration de la plate-forme de la voie et du remblai, ainsi que du niveau de la surface phréatique.

Conclusions concernant l'application des essais CPT pour déterminer l'état de contrainte

Les essais CPT sont un outil puissant pour étudier les conditions des remblais ferroviaires et du sous-sol et pour déterminer l'état de contrainte sous la plate-forme. Cela concerne en particulier les conditions statiques de l'état de contrainte des sols mous. Certains aspects nécessitent une attention particulière :

- Les méthodes d'estimation du poids unitaire en vrac basées sur les essais CPT ne sont pas précises. Cela introduit une erreur dans l'état de contrainte. Dans deux des exemples examinés, l'erreur dans la contrainte effective était comprise entre 4 et 10%. Cela influe également sur OCR et K_0.
- La pression stationnaire de l'eau interstitielle dans le sous-sol est souvent difficile à déduire des essais CPT. C'est le cas lorsqu'il n'y a pas de couches de sol perméables à l'intérieur de la profondeur des mesures CPT. Dans l'un des exemples examinés, différentes hypothèses sur le gradient de pression de l'eau interstitielle entraînent une incertitude sur la contrainte effective et, par conséquent, une incertitude sur la OCR d'un facteur d'environ 1,5.
- Dans la plupart des CPT qui sont analysés pour dériver l'état de contrainte pour les quatre exemples, la fiabilité de la pression d'eau interstitielle mesurée u_2 semble être faible. Cela limite l'interprétation des CPT en ce qui concerne la classification des sols.
- Les méthodes d'interprétation des mesures CPT sont développées pour certains types de sol. Pour d'autres types de sol, les méthodes d'interprétation peuvent être moins précises. Pour augmenter la précision de la détermination de l'état de contrainte, il est essentiel de procéder à des vérifications croisées avec des girouettes pour essais sur le terrain ou des essais en laboratoire de haute qualité. Il en va de même pour l'interprétation du cône sismique.
- Il n'est pas possible d'obtenir une estimation précise de la contrainte horizontale à partir des mesures CPT. Lorsque la contrainte horizontale est importante, des essais au pressiomètre ou au dilatomètre sont nécessaires.
- Un CPT ne donne pas une estimation précise de la sensibilité des couches de sol, telles que la craie lacustre. Pour les sols sensibles, des mesures supplémentaires avec des tests de girouette sur le terrain sont nécessaires.
- Pour évaluer l'état des sables et des limons, il existe différentes méthodes d'interprétation des CPT dans la littérature. Pour s'assurer que l'interprétation des CPT soit correcte, en particulier pour les sols partiellement drainés lors de la pénétration des CPT, des tests supplémentaires en laboratoire sont nécessaires.
- Pour l'interprétation des CPT dans les sols partiellement saturés, des informations supplémentaires sur la teneur en eau, le degré de saturation et la succion capillaire sont nécessaires car la succion capillaire influence les mesures CPT.

Approche

Afin de donner une vue d'ensemble des méthodes et théories les plus récentes pour déterminer l'état de contrainte sous la plate-forme de la voie, une étude bibliographique a été réalisée. Dans cette étude, l'accent a été mis sur des situations similaires aux fondations des chemins de fer suisses.

La première étape de l'étude bibliographique est la définition de l'état de contrainte. Le reste de l'étude bibliographique s'est concentré sur les techniques de mesure sur le terrain, la variabilité spatiale et l'application des différentes méthodes d'interprétation CPT à quatre exemples de sites.

Le champ d'application de l'étude documentaire est aligné sur celui du client.

Le client a partagé des données provenant de quatre exemples de sites en Suisse présentant des conditions géologiques différentes.

Une analyse exploratoire des données fournies pour les quatre exemples de sites est réalisée. Cette analyse est basée sur les résultats de l'analyse documentaire et sur l'expérience de Deltares. Pour chacun des exemples, un ou deux essais CPT sont analysés afin de dériver l'état de contrainte du remblai ferroviaire et du sous-sol. Les autres données géotechniques disponibles, telles que les sondages et les essais d'indexation, sont utilisées pour pouvoir interpréter les essais CPT.

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1 Introduction

1.1 Background

The Swiss railway network was designed from the 1850s and prioritized lines along the valley floors where much soft soil is present. Whilst this was satisfactory in the late 19th century, the use of these old lines with modern railway traffic must be verified. In order to better identify critical sites, the railway-induced stress state of the track bed and the subsoil below the railway needs to be better understood.

The Swiss Federal Railways (SBB) operates approximately 60% of the Swiss network. A small part of the network is founded on soft soils but parts of these are of strategic importance. Large deformations and strength and stiffness losses are major geotechnical risks for railway companies and cause significant track maintenance and operational cost as well as being a safety issue. Figure 1-1 gives a map of Switzerland with the locations of soft soils.



Figure 1-1 Areas where the Swiss railway network is founded on soft soils (courtesy SBB).

Although one can estimate to some extent the field strength and stiffness properties, any engineering verification requires knowing the stress state under and around the track bed in order to perform even simple engineering verification. This stress state is currently poorly understood.

There is limited experience with the determination of the stress state for field conditions, especially for railway foundations (directly on ground level or on an embankment).

Allianz Fahrweg Normalspur, through SBB, has contacted Deltares for assistance in the determination of the stress-state, with as a first step a literature review. Allianz is a joint venture of several railway companies, including SBB.

1.2 Objective of the project

Stress state is a wide topic. Primarily this encompasses the total stresses and effective stresses, both in vertical and horizontal directions, caused by the weight of the railway foundation and subsoil layers as well as other influences such as the cyclic loading due to trains.

Horizontal effective stresses are mostly estimated based on the vertical effective stresses. However, the relationship is not straightforward. The horizontal stress in longitudinal direction of the foundation (intermediate principal stress) will differ from the horizontal stress in cross direction of the foundation.

An important component of the stress state is the effect of yield stress due to historical loading (pre-overburden pressure), creep and ripening. The yield stress can be much higher than the actual effective stress.

Furthermore, there are the principal stress axis rotations below the foundation due to the construction of railway embankments. Principal stress axis rotations in the subsoil are possible due to shearing stresses and deformations during the formation of the subsoil, especially for older (Pleistocene) deposits.

Lastly Switzerland has a glacial geology and many soils, even young, have been affected by the glacial retreat and dead ice trapped in the soil. The glacial retreats – there have been several – can have strongly influenced the state of the subsoil, and this will have to be studied.

The main objective of this study is to understand the influence of the track bed on the in-situ stress state (inclusive of railway cyclic loading). This includes:

- Understanding of the influence of the track bed on the in-situ stress state.
- Determination of the stress state under the track bed of railways.
- Give an overview of the state-of-the-art methods and theories to determine the stress state, both for static and cyclic conditions.

To determine the stress state for field conditions the following is also relevant:

- Build up confidence in CPT interpretation.
- CPT interpretation sensitive for stress state.
- Stress state is a 3D problem.
- CPTs performed next to the track, so translation to the situation below the track is needed.

The relevance of all these aspects may vary per site, as they are related to the geotechnical mechanisms which may play a role at a railway foundation at a particular site. The type of geotechnical models to be applied also determines the relevance of all these aspects. The focus is on situations that are similar to the Swiss railway foundations.

1.3 Examples of sites

SBB shared data from four examples of sites for this project for guidance purposes. The four sites are meant as illustrations and are representative locations. The actual sites and underlying reports are known to the client and the authors. Figure 1-2 gives a schematic cross section of the landscape of Switzerland with the typical geological situation at the locations of the sites.

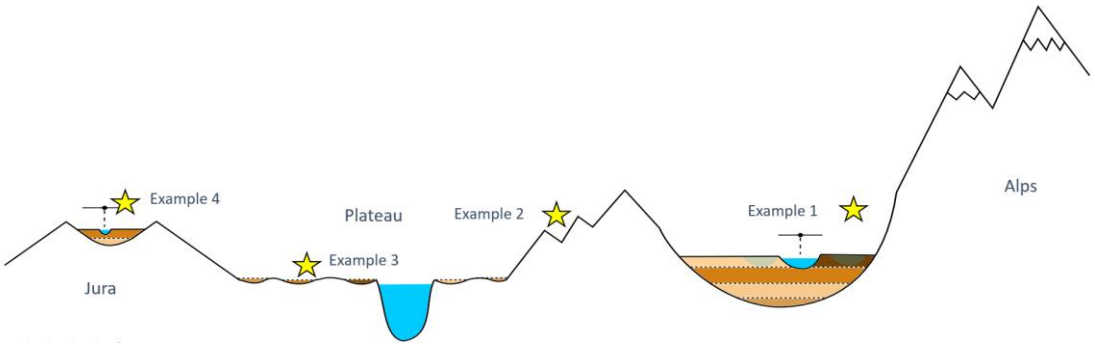


Figure 1-2 Schematic cross profile of the Swiss landscape with typical geological situation of the examples of sites (courtesy SBB).

The examples of the sites are:

- Example 1 of a railway embankment on a flood plain.
- Example 2 of a railway line on a slope.
- Example 3 of a railway line on an embankment.
- Example 4 of a railway line on soft spots with challenging soil.

1.4 Approach

A literature review is conducted and global analyses of the stress state at the four examples are performed. Various Deltares experts are briefly involved in the several topics.

The first step in the literature review is the further definition of the stress-state. The rest of the literature study will focus on the following points:

- Methods to determine the in-situ stress state for field conditions.
- Generic recommendations to derive a representative stress state.
- Indications of spatial variability.
- Literature values of the stress state for various conditions.

The scope of the literature review is aligned with SBB.

In this report of the review the various aspects of the stress state and the in-situ measurement techniques are related to the soil types, configurations of the railway foundations and geotechnical mechanisms where they are relevant.

1.5 Scope

The scope of this project is to give a general overview of state-of-the-art of applicable knowledge of the stress state based on literature and Deltares experience. Mainly aimed and identifying knowledge that is already being applied or is close to being applicable. Not developing new things. Not aimed at exhaustive literature reviews, limited to workable approaches while keeping an overview on the whole range of literature. It is aimed to identify knowledge gaps. Aimed at identifying various influences that determine the stress state with respect to slope stability, deformation and settlement of the railway tracks.

1.6 Outline of the report

In Chapter 2 the examples of sites are presented. For each site a brief overview of the situation of the railway line, the stratigraphy of the subsoil, the geohydrology and the available geotechnical data is given.

In Chapter 3 an overview of the main components of the stress state is described. This chapter mentions the definition of the stress state as used in this project. Furthermore, soil classification, bulk unit weight, pore water pressures, relation between vertical and horizontal stress, yield stress and overconsolidation, cyclic effects from train passages and stress distributions below the track bed are discussed.

In Chapter 4 the most common geotechnical field measurement techniques to derive the stress state are briefly discussed. The focus in this chapter is on the cone penetration test (CPT), including pore water pressure sensor (CPTu) and seismic sensor (SCPTu). For the CPT, CPTu and SCPTu methods to classify soils are given. For clay and peat, sand and silt, intermediate soils and partially saturated soils methods to derive the stress state are discussed. This chapter is partly based on a literature review. Some findings from literature are applied on examples from The Netherlands, where CPTus or SCPTus are available together with laboratory tests results for verification of the SCPTu interpretation.

Chapter 5 gives some examples of laboratory tests to derive information on the stress state.

Chapter 6 focuses on spatial variability. The discussion is about variability in stratigraphy, variability of soil parameters, averaging of uncertainty and statistical analysis with CPT data. This chapter is mainly based on experiences with variability of soft soils in The Netherlands.

In Chapter 7 the interpretation of CPTu and SCPTu of the four examples of sites as presented in Chapter 2 are discussed. The CPTus and SCPTus are interpreted in order to derive the stratigraphy of the subsoil and to obtain the stress state at the four sites. The interpretation of the CPTus and SCPTus is based on the findings in Chapter 4.

Chapter 8 gives the main conclusions of the literature review. In Chapter 9 the recommendations are given.

2 Examples of sites

2.1 Example 1: railway embankment on a flood plain

This example is a site at a railway line in Western Switzerland. Figure 2-1 gives an impression of the situation of the railway line at this site.

The section of the railway line of this example is located in the alluvial plain of a river. Within this section there are some tributaries of the river. The subsoil at this site is therefore characterized by the fluvial deposits of the river plain. The Holocene alluvium of the plain is made up of fine-grained soils and palustrine and/or peaty deposits with the presence of organic matter. These peaty deposits are remains from historic marshy areas. The Holocene deposits are up to 12 m thick.



Figure 2-1 Impression of the situation of the railway line of example 1 (Photo courtesy SBB).

The water content of the clayey material is 24 to 42%. Plasticity index is about 10%. The loss of ignition of the peaty material is between 5 and 67% and water content between 28 and 283%. The peaty material contains a lot of silt and sand, such that the specific gravity is mostly between 2.18 and 2.76

In the northern part of this section of the railway line landslide deposits are located. These are non-coherent gravels, sands and silts with a horizon of decimetric limestone blocks.

The phreatic surface is 1.4 to 2.1 m below surface level. The level of the river is approximately 3 m below the tracks. It appears that the average seasonal rise of the groundwater table is between 0.5 and 1 m. Due to the length of the route and the heterogeneity of the deposits, this beat is not constant and varies in the different zones.

CPTs, SCPTus, boreholes and classification tests are performed. The site investigation is performed in a zone adjacent to the railway embankment.

2.2 Example 2: railway line on a slope

This example is located on a slope. Figure 2-2 gives an overview of the area around the railway line. Figure 2-3 gives an impression of the situation of the railway line of this example.

The railway line in this stretch runs on the northwestern edge of the valley floor. The railway line lies on an embankment for long stretches. The railway embankment has a maximum height of 7 m. As part of the double-track expansion in 2004, the existing railway embankments were widened. In some cases, steep embankment slopes with earth-reinforcement systems were made. Since the subsoil partly contains organic matter, care was taken not to drain the subsoil as part of the double-track expansion.



Figure 2-2 Overview of the area around the railway line at example 2 (Courtesy SBB).



Figure 2-3 Impression of the situation of the railway line of example 2 (Photo courtesy SBB).

In the past and present, deformations/damage to railway infrastructure (tracks, railway embankments, platforms, catenary masts, cable ducts, etc.) have been observed, which can be attributed to lateral and horizontal deformations. This track is very maintenance intensive.

At the section of the railway line, the rock is expected to be around 20-50 m below surface level (exact location not known). The deepest deposits consist of sand with layers of gravel. These gravel layers were probably dumped by the streams from the side valleys. The upper part of the deposits consists of younger clayey silt sediments. In the silty deposits there are layers with organic matter of up to a few meters. Plasticity index of the upper part of the deposits is mostly 4 to 10% and sometimes up to 20%. Organic content is mostly around 2 to 6% and sometimes up to 20%. The consistency index is very low; around 0%. The depositional history of the sediment is locally very complex because influences from the side valleys/slopes are also present within the silty deposits. Locally outcrops of the deep sand deposits are present at the surface. The young silty and sandy sediments are assessed as potentially liquifiable. The material is liquifiable, because it is loose due to high water level and low effective stress in the embankments. Liquefaction is risky because it is a fast mechanism.

The groundwater table is around the level of the streams and therefore only slightly below the terrain surface. The groundwater table is believed to be non-horizontal with some groundwater flow.

CPT, SCPTu, DP, boreholes and classification tests are performed.

2.3 Example 3: railway line on an embankment

This paragraph gives a brief description of the railway track of example 3 with a railway line on an embankment. The railway line is largely on a slope, so that the height of the embankment on the northwest side is higher than on the southwest side. First one track is built in 1857. Later the second track is built in 1920. Figure 2-4 gives an impression of the situation of the railway line of example 3.



Figure 2-4 Impression of the situation of the railway line of example 3 (Photo courtesy SBB).

On the surface fine-grained layers with high organic content were formed. Some artificial fillings containing foreign substances were encountered. Below this are very heterogeneously composed silty sediments, up to several meters thick, consisting of clayey to fine sandy silts with a variable proportion of organic material, interspersed with peaty material and lake chalk-containing lenses and intermediate layers, whereby the transitions are often gradually.

Locally, layers of clean to silty-clayey limestone occur. The material is very soft to medium stiff (very loose to loosely packed). Below this follows a transition to clean to very silty and clayey fluvial sands and gravels, which may still contain occasional organic material and are described as loose to locally very dense (fine-grained intermediate layers, medium stiff to hard). These are underlain by thick lake deposits, which are composed of silt and (fine) sands, locally clayey. They are soft to stiff or stored loosely to medium density.

Lake chalk is sensitive. This is a challenging material, as this clay is sometimes collapsible. However, the lake chalk is not so sensitive as quick clay. Many clay is collapsed and is now in a remoulded state. There is also some intact clay available with an open structure due to precipitation in acidic environment. Collapsible soils are risky because it is a fast mechanism.

The groundwater level is around 435 (+/- 2) meter above sea level according to the geotechnical length profile.

CPTu, SCPTu, DP, SPT and boreholes are available at this site.

2.4 Example 4: railway line on soft spots with challenging soils

This example 4 is located at a railway line on soft spots with challenging soil. Figure 2-5 gives an impression of the situation of the railway line of example 4.



Figure 2-5 Impression of the situation of the railway line of example 4 (Photo courtesy SBB).

The tertiary layers at this site are covered by recent (Quaternary) and loose deposits. These are:

- Screes composed mainly of silty-clay gravels. Locally silty-clay-gravelly moraine deposits dating from the Würmian Quaternary glaciation can be found.
- Lacustrine, marshy and border runoff deposits. The lacustrine deposits are made up of organic silts (locally chalky) as well as clayey silts and more or less silty clays including locally patches of silty sand. These deposits have a consistency ranging from soft to firm.
- The marshy deposits are made up of organic silts, peaty silts and peats. The peat is described as free peat with a fibrous structure of multimetric thickness and locally medium soft consistency.

The peaty deposits at this site are the deepest peat bogs of Switzerland: 15 meters of peat and organic silt. A lot of settlement occurs at this site.

So, the maintenance costs are high. Peat in Switzerland is less organic than in other countries. The peat contains a lot of minerals from the continuous erosion of the Alps. Figure 2-6 gives a schematic cross section of the valley with the peat and gyttja deposits and the backfill for the railway tracks at example 4.

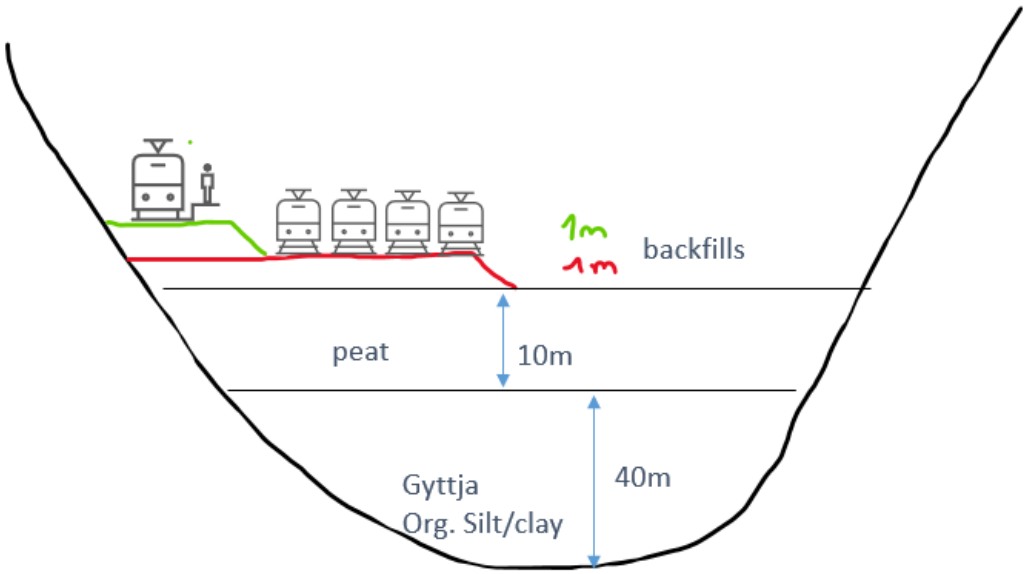


Figure 2-6 Schematic cross section of the valley with the peat and gyttja deposits and the backfill for the railway tracks at example 4 (Courtesy SBB).

Measurements of the pore water pressures at two locations indicate a piezometric head about 2 m below surface level. The measurements indicate a movement of the water table of a maximum of 90 cm over the observation period (2021-12-13 to 2022-02-11).

Boreholes, CPT, DP and pressiometer tests are performed. Index tests such as water contents, densities, Atterberg limits, particle size distributions, and organic matter content are also available. Laboratory tests, such as direct shear tests, oedometer tests and triaxial compression tests (CU) are also performed.

3 Main components of the stress state

3.1 Definition of stress state

The stress state is defined in this study as the stresses in the soil below and adjacent to the railway track, with and without the load effects of trains. Components that determine this stress state are:

- Thickness and weight of the track bed and when applicable the railway embankment.
- Stratigraphy of the subsoil with different soil types having different bulk unit weights.
- Stress history of the subsoil related to the geological background (sedimentary environment) of the subsoil and also including soil forming processes (ripening, cementation) and creep (post-sedimentary processes or ageing).
- Pore water pressures in the railway embankment and the subsoil under different geohydrologic conditions.
- Static and dynamic effects of the train load (dynamic amplification, dynamic pore water pressure built-up, etc).

The stress state involves different stresses and parameters, such as the total stress, pore water pressure, vertical effective stress, horizontal effective stress, earth pressure coefficient K_0, yield stress, overconsolidation ratio OCR, pre overburden pressure POP etc. In this report it is assumed that the reader is familiar with these stresses and parameters.

In this chapter the focus is on issues that may be relevant for the four cases which are presented in Chapter 2. So, in the current chapter the focus is on the geotechnical phenomena which may play a role concerning the soft soils at these sites and on the components of the stress state which are relevant to analyse these phenomena.

3.2 Soil stratigraphy and bulk unit weight

The vertical effective stress is often an important stress in geotechnical analyses. The vertical total stress is a result of the configuration of the track bed and the railway embankment and the stratigraphy of the subsoil, so the thickness and unit weight of the soil layers. The thickness and the weight of the track bed and the railway embankment and the stratigraphy can be determined with boreholes and other in-situ tests (see Chapter 4). To estimate the vertical effective stress accurately based on the vertical total stress also the pore water pressure is important; see Paragraph 3.3.

The thickness of the track bed and the railway embankment can vary considerable due to variations in the natural surface level or due to settlement of soft soils below the railway embankment. The stratigraphy of the subsoil can also vary considerably due to variations in the geological environment. These variations can occur on relatively short distances; see also chapter 6. When conducting a site investigation with to much distance between the boreholes and other field tests variations in stratigraphy can be easily missed. To be able to determine the vertical effective stress accurately a sufficiently dense site investigation is required.

3.3 Pore water pressures

Pore water pressures influence the stress state in a railway embankment and the subsoil largely. The pore water pressures are determined by the phreatic surface and the piezometric head in deeper soil layers and the pore pressure gradient in low-permeable soil layers.

Often a hydrostatic pore water pressure gradient below the phreatic surface is assumed. In a situation with a low-permeable soil layer (aquiclude) on top of a permeable soil layer (aquifer) different geohydrological conditions can occur:

- Artesian groundwater condition with high pore water pressure in a deep aquifer below a low permeable stratum of soft soils (aquiclude) where the aquifer is in hydraulic continuity with neighbouring high grounds causing a pore water pressure gradient in the aquiclude greater than hydrostatic and therefore upward groundwater flow in the aquiclude. This can result in zero vertical effective stress in the aquiclude.
- Underdrainage condition with a phreatic surface higher than the piezometric head in the aquifer below the aquiclude causing a pore water pressure gradient in the aquiclude lower than hydrostatic and therefore downward groundwater flow in the aquiclude.

The deviation of the pore water pressure gradient for artesian or underdrainage groundwater conditions with respect to the hydrostatic groundwater condition can result in a difference in pore water pressure up to tens of kilopascal at some depth in the subsoil. This difference in pore water pressure directly affects the effective stress and overconsolidation ratio or pre overburden pressure.

Seasonal fluctuations

Both the phreatic surface and the piezometric head can vary heavily due to seasonal fluctuations. These seasonal fluctuations occur due to rainfall periods and groundwater flow from higher grounds. These seasonal fluctuations can also cause fluctuations of the artesian and underdrainage conditions, so fluctuations in the pore water pressure gradient in the low permeable soil.

Pore water pressures in the railway embankment

The construction of the railway embankment and the drainage conditions in the embankment are also important for the pore wate pressures. An example comes from the railway embankment near the station of Gouda Goverwelle in The Netherlands (GeoDelft, 2001). This is a site where loose sand and poor drainage of water from the sand cunette occurred.

The railway line between Gouda and Woerden is built on a very soft subsoil of 10 to 11 meter thick with peat and clay. After widening of the railway embankment in order two build two additional tracks at each side of the existing tracks a lot of deformation occurred. As a result of the design but also caused by the settlements the bottom of the cunette of the railway embankment is about 3 meters below the adjacent surface level. Groundwater is trapped in the sand embankment due to the low permeable peat and clay layers around the cunette. In wet periods the phreatic surface is only 0.5 m below surface level. The fluctuation of the phreatic surface is about 0.5 m. Due to large deformations of the embankment the cone resistances in the sand embankment are low. The large deformations of the embankment occurred due to the low strength of the embankment and the high phreatic surface. The large deformations especially occurred in wet periods. The pore water pressure response in the sand embankment due to train passages was also measured. This pore water response was about 1 kPa. In Paragraph 3.6 the pore water pressures which can be generated by cyclic effects from train passages are discussed in more detail.

Partially saturated zone

Above the phreatic surface the subsoil and the railway embankment can be partially saturated. In this partially saturated zone capillary suction occurs. The magnitude of the capillary suction depends on the water content (degree of saturation) in the soil and the soil type. Due to seasonal variations (periods of precipitation and evaporation) the level of the phreatic surface and the magnitude of capillary suction fluctuate. Capillary suction has an influence on the effective stress and the soil strength in the partially saturated zone and will therefore affect cone penetration tests and other in-situ tests.

As the capillary suction fluctuates during the seasons the effective stress and shear strength also fluctuate during the seasons. Therefore, a soil layers which is very stiff in summer can be weak during winter. This topic will be discussed further in Paragraph 4.6.

Pore water pressure measurements
Commonly pore water pressures are measured in the field using standpipes or piezometers. When a site investigation is conducted with boreholes and cone penetration tests and the groundwater regime is not straightforward, one can consider performing cone penetration tests with some dissipation tests at several depths. When a dissipation test is performed long enough the stationary pore water pressure at that depth can be determined. In low permeable soils the required time to get the stationary pore water pressure may be quite long. However, this information about the groundwater regime will be very important to deduce the stress state and for the interpretation of the cone penetration tests.

3.4 Yield stress and overconsolidation

As mentioned in Paragraph 3.1 the stress history of the subsoil is one of the aspects which influences the stress state. The stress history is related to the geological background (sedimentary environment) of the subsoil and to soil forming processes (ripening or desiccation, cementation) and creep (post-sedimentary processes or ageing). Fluctuations of pore water pressures in the subsoil may also influence the stress history. Erosion of soil layers (overburden) and the melting of ice sheets after glaciation can play a role as well. For railway embankments the passing trains are relevant for the stress history of the railway embankment too.

The stress history is often expressed in the pre-consolidation stress, overconsolidation ratio OCR or pre overburden pressure POP. Sometimes yield stress and yield stress ratio YSR are used, as overconsolidation ratio suggests an overconsolidation of the subsoil by geological means (Burland, 1990).

Profile of the pre-consolidation stress or yield stress with depth
The profile of the pre-consolidation stress or yield stress with depth relative to the profile of the vertical effective stress can be different. This depends on the stress history of the subsoil (Chen et al., 1994):

- When the stress history is determined by creep or pore pressure fluctuations the deviation between the yield stress profile and vertical effective stress profile increases with depth. This deviation can be expressed with a constant yield stress ratio YSR.
- When the stress history of the subsoil is determined by erosion or glacial action the deviation between the yield stress profile and vertical effective stress profile is more or less constant with depth. This deviation can be expressed with a constant pre overburden pressure POP.
- When the stress history of the subsoil is determined by ripening or desiccation the yield stress profile is more or less constant with depth. This situation can be expressed with a decreasing yield stress ratio with depth YSR.

Many combinations are possible in-situ; for example, a subsoil of which the stress history is determined by creep with a desiccated crust as a top layer.

For the Holocene subsoil in The Netherlands data about the stress history from many sites is collected (Deltares, 2022). Based on this data it is found that the pre overburden pressure in the subsoil below dikes and in the subsoil adjacent to dikes have more or less the same values. POP is about 10 kPa for peat to 25 kPa for clay. This means that OCR is about 2.0 to 3.5 for an effective stress level of 10 kPa (K_0 is 0.8 to 2.2). OCR is about 1.1 to 1.25 for a stress level of 100 kPa (K_0 is 0.3 to 0.6).

Gain of overconsolidation in time below an embankment

After construction of an embankment the overconsolidation ratio in the subsoil below the embankment will increase over time due to creep. In theory this gain of OCR can be calculated by e.g. the isotachen model (Den Haan, 1994; Leroueil, 2006). At two sites in The Netherlands site investigations are conducted to determine the gain of OCR over time (Deltares, 2024). At these sites at Bergambacht and Papendrecht a new dike was built 23 years ago. Based on the site investigation it is found that the OCR values are in the same ranges as the values mentioned above. It is concluded that the values of OCR expected from the isotachen model are much higher than the values derived from the laboratory tests.

Overconsolidation due to train loading

The train loads on a railway embankment also have an influence on the stress history. In Figure 3-1 it can be seen that the cone resistance of a cone penetration test in a railway embankment decreases considerably (Deltares, 2024a). The sand of the top of the railway embankment is very dense. However, the sand of the bottom of the railway embankment is loose. This decrease of the cone resistance is attributed to the effect of the train loads.

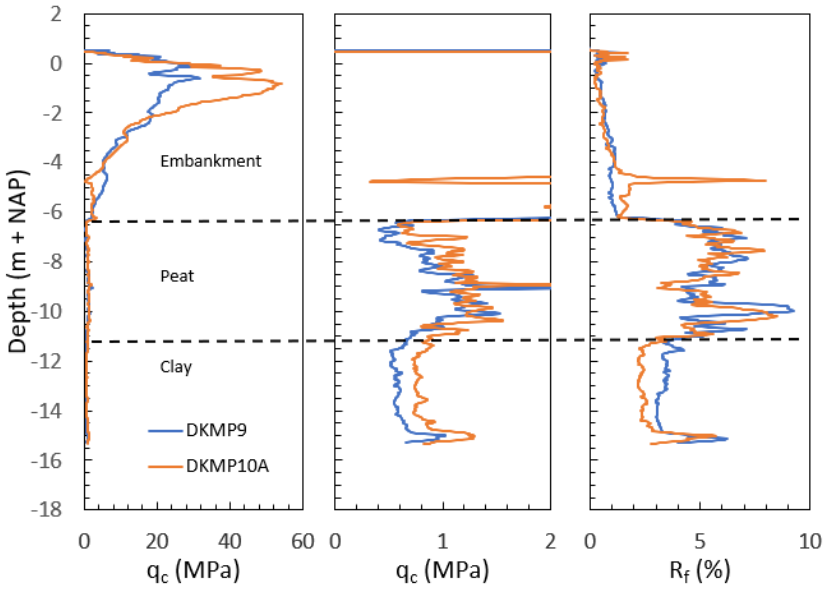


Figure 3-1 Two CPTs in railway line Delft Schiedam, The Netherlands, with decreasing cone resistance in the embankment (Deltares, 2024a).

3.5 Relation between vertical stress and horizontal stress

Usually, the horizontal effective stress is taken as a fraction of the vertical effective stress. This fraction is determined by the earth pressure at rest coefficient K_0. Different aspects affect the earth pressure at rest coefficient K_0:

- $K_{0,nc}$ for normally consolidated conditions depends on soil type. Following the well-known Jaky equation $K_{0,nc}$ depends on the friction angle. Based on a series of $K_0 - CRS$ tests from different sites in The Netherlands it is found that $K_{0,nc}$ increases with increasing bulk unit weight; see Figure 3-2. For peat $K_{0,nc}$ is between 0.2 and 0.4. These low values can be attributed to the high friction angle of peat due to the contribution of the tensile strength of the fibers in peat to the shear strength (Den Haan et al., 2006).
- $K_{0,oc}$ for in-situ conditions (overconsolidated soils) is influenced by the yield stress ratio or overconsolidation ratio; see Figure 3-3. The yield stress ratio or overconsolidation ratio reflects the stress history of the soil (caused by overburden, erosion, creep, ripening) as discussed in Paragraph 3.4.

Train loading may influence the stress history of railway embankment (example decreasing cone resistance at Delft – Schiedam; see Paragraph 3.4). Higher overconsolidation ratio leads to higher $K_{0,oc}$.

- Intermediate principal stress: usually a vertical and a horizontal stress are considered. However, in the three-dimensional space stresses in three directions act on a soil volume. The horizontal stresses can be different in two directions. The stress in the direction of the track bed is the intermediate principal stress. For a railway embankment this intermediate principal stress will be mostly higher ($\sim K_{0,oc}$) than the horizontal stress in the direction perpendicular to the track bed (active earth pressure K_a). This means that the three-dimensional mean stress is somewhat higher than it would be according to the common assumption based on two equal horizontal stresses. The effect of the intermediate principal stress causes a higher shear strength (Matsuoka et al, 1974; Wroth, 1984; Teunissen, 2022).
- Principal stress rotation: Building an embankment will lead to increased horizontal stresses in the subsoil around the toe of the embankment. During the heightening stages of the embankment the principal stresses rotate from vertical to nearly horizontal (Smith, 1984); see Figure 3-4. K_0 changes from around 0.5 to about 2.0 (passive earth pressure K_p). Due to this stress rotation the mobilizable undrained shear strength of the subsoil near the toe of the embankment increases.
- In pushed soil deposits the horizontal stress can be the main principal stress. For Peelo clay in The Netherlands Gareau et al. (2004) performed oedometer tests in a lateral stress oedometer device. The clay of the Peelo Formation is affected by glacier action. The applied oedometer device was able to measure horizontal stresses in three directions. Anisotropy in soil stiffness was found for the Peelo clay, which could be related to historic glacier flow direction.

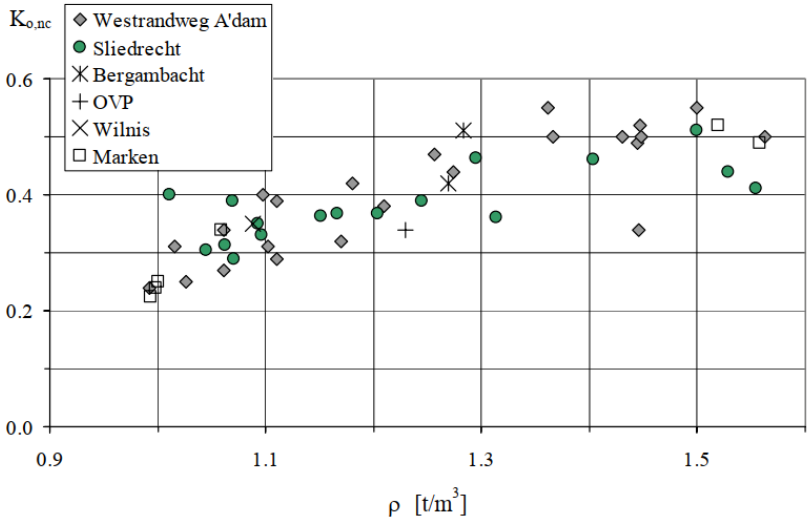


Figure 3-2 Correlation of $K_{0,nc}$ with bulk density (Den Haan et al., 2006).

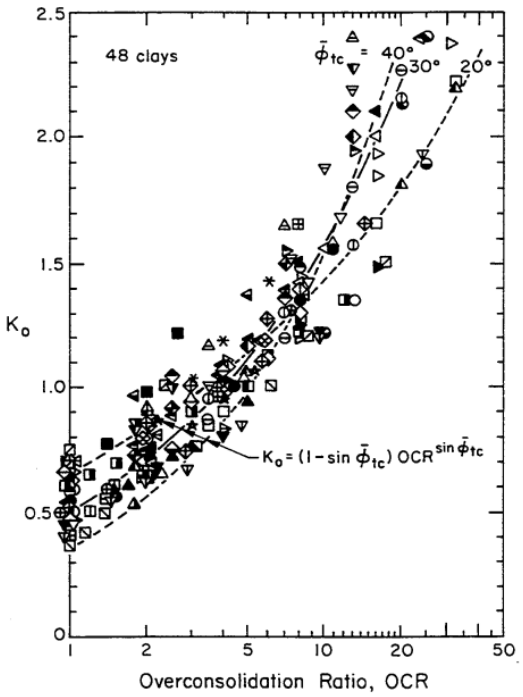


Figure 3-3 Relationship between OCR and $K_{0,oc}$ (Kulhawy & Mayne, 1990).

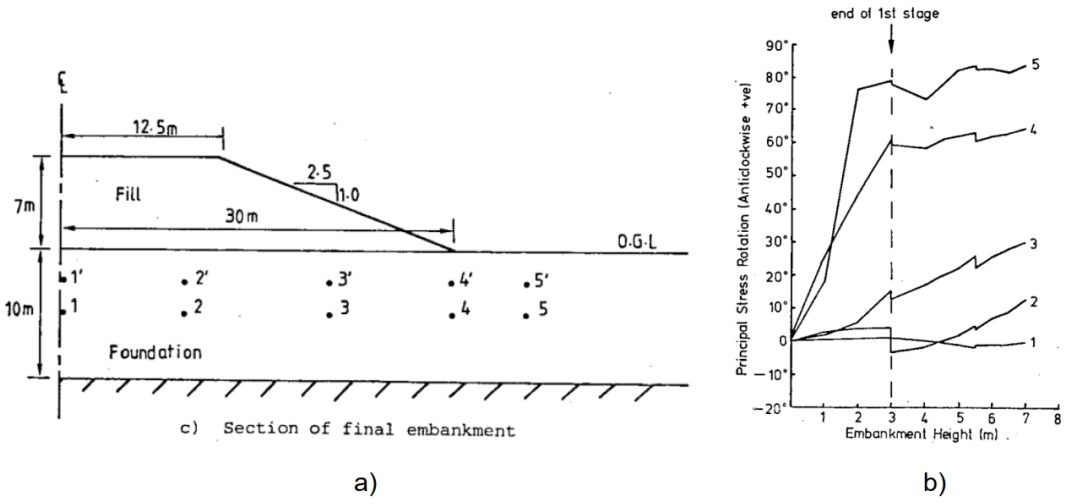


Figure 3-4 FEM analysis of multi-stage embankment, with computed rotation of the major principal stress after each building stage (Smith, 1984).

3.6 Cyclic effects from train passages

A recent literature review by Deltares (2022a) gives an overview of relevant effects from train passages on the stresses and stability of the subsoil. The following is based on this literature review. From the available literature on laboratory tests follows that significant pore-pressure build-up after a train passage seems unlikely in case of already cyclic preloaded soil (with intermediate drainage) if the cyclic shear stress amplitude is not enlarged (no change in axle loads and speeds). An increasing pore pressure build-up by subsequent train passages is also not observed in the few available field measurements.

If the train speed, the axle loads and the axle configuration do not change, then an influence of the train length and the interval between train passages on embankment stability is only possible if cyclic loading by passing axles has an unfavourable effect on pore pressures accumulation or on visco-plastic (permanent) deformations of the soft soil underneath the embankment.

An increase in speed to values above approximately 0.6 times the critical velocity will significantly amplify the oscillating deformations, stresses and pore pressures in the soil caused by passing axles. This amplification may negatively affect the stability or may cause derailment by the amplified deformations. Quasi-static analysis without considering dynamic amplification in the subsoil seems allowed, as long as the train velocity is below the aforementioned threshold value. Accurate determination of the critical velocity requires usage of Rayleigh dispersion curves (Rayleigh wave velocity versus frequency) from measurements (seismic, optical fibre) and/or numerical simulation.

An increase in speed will furthermore also increase the dynamic variation/amplification of axle loads. For slope stability analysis, there seems to be no need to consider this axle load amplification, because the alternating part of different axle loads will not be in phase with each other and will also not be in phase with the oscillations caused in the subsoil by subsequently passing axle loads. The load duration is of influence for the time-dependent instability behaviour.

The influence of train speed is especially important for thick, soft soil deposits (amplification). Also important are the conditions of rails and wheels surfaces and the conditions of railway constructions.

An increase in axle load to magnitudes that were not experienced before can obviously have a negative effect on embankment stability. In case of a static train load with a certain duration, the influence of increasing axle loads and axle configurations on stability can be assessed most accurately with the help of 3D finite element analysis (FEA), using proper constitutive models. If needed, these models can include the favourable time-dependency of soft soil failure (in combination with pore pressure flow). An increase in the amplitude of alternating soil stresses by a moving train may additionally cause pore pressure build-up.

Quasi-static stability analysis (ignoring dynamic amplification of soil displacements and stresses) is allowed as long as the train speed is below approximately 60% of the local critical speed. This observation from literature aligns with the geotechnical stability design and assessment practice in most countries, namely that the case of moving trains is only checked in terms of critical velocities while the stability assessment only considers static conditions and no dynamic effects.

For new railway constructions or changes in traffic accumulation of excess pore water pressure due to cyclic traffic loading may occur. However, dependent on the degree of compaction of the embankment material this effect will be mitigated.

Three-dimensional dynamic finite element analysis of embankments
To study the influence of train speed on displacements and stresses that are generated in the subsoil due to passenger trains numerical analyses have been performed on a section of the railway track between Delft and Schiedam (cross section 5), The Netherlands (Deltares, 2022b); see Figure 3-5. The numerical model used for the analysis consists of the 3D Finite Element Model (FEM) Plaxis 3D. The constitutive model adopted to simulate the non-linear behaviour of the soil is the Hardening Soil Small Strain Model. The dynamic analyses are performed under undrained conditions for the layers below the phreatic line since the rate of loading is much higher than the rate of pore water pressure dissipation.

The layers above the phreatic line are modelled under drained conditions, therefore exhibit more compressibility during dynamic loading. To guarantee elastic soil behaviour during dynamic analysis the OCR of the soil layers was assumed to be 10. A double-decker intercity train is modelled with moving loads of constant magnitude and different speeds (speed constant during an analysis).

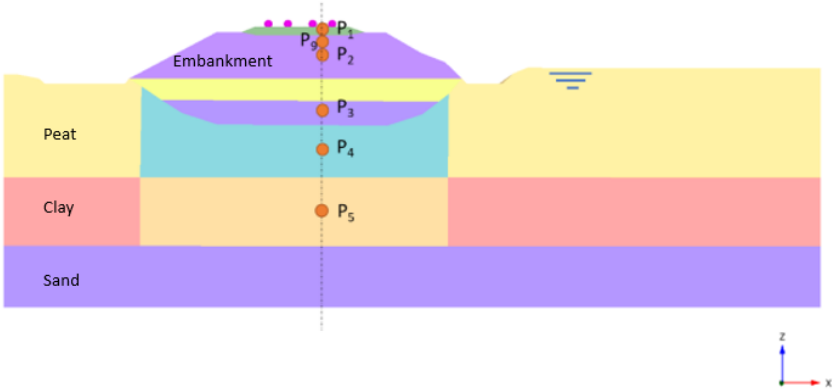


Figure 3-5 Subsoil model and embankment of cross section 5 of the railway track between Delft and Schiedam which is used for dynamic analyses with 3D Finite Element Model (Deltares, 2022b).

The numerical analyses (Deltares, 2022b) have shown that the train speed influences in particular the amplitude of the maximum vertical displacement that occurs during the train passage. The vertical displacement amplitude exhibits a linear relation with the travelling speed, for travelling speeds up to approximately 350 km/h. For a travelling speed of 400 km/h there is a stronger increase of the displacement amplitude, which is again reduced for a travelling speed of 500 km/h. This shows that the critical train speed fits within the range 319-501 km/h, which is computed based on the track dispersion and soil dispersion. There is a strong attenuation of the displacement with depth (Figure 3-6a).

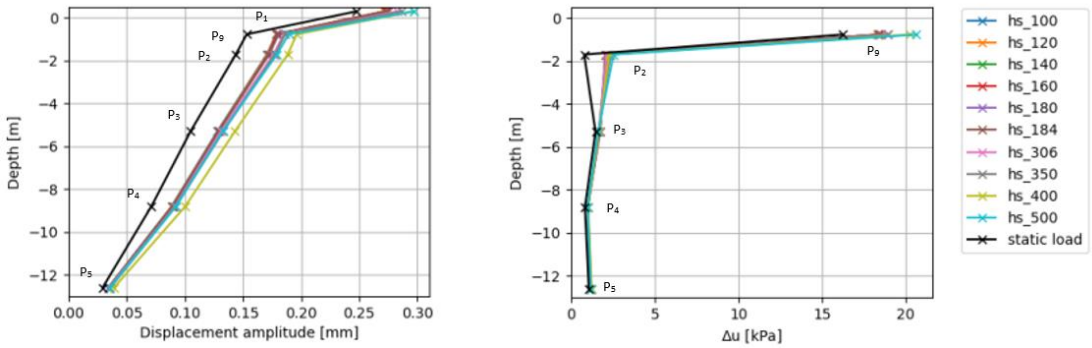


Figure 3-6 Maximum values along depth during the passage of a bogie, at different travelling speeds: (a) vertical displacement and (b) excess pore water pressure Δu.(Deltares, 2022b).

A bi-linear behaviour for the attenuation with depth is identified: a strong attenuation of the displacement at the ballast and embankment, followed by a linear attenuation with depth for the soil layers. This bi-linearity is related to the fact that the top part of the embankment is above the phreatic line and therefore exhibits more compression. The dynamic displacement is found to be greater than the static displacement. The dynamic amplification in the vicinity of the critical train speed is found to be limited.

This could be related to the large thickness of the embankment, but it could also be related to the fact that the numerical simulations are performed at discrete train speed, and that a greater peak occurs at the exact critical train speed. Alternatively, it could be related to numerical issues.

The numerical results do not show a clear relation between travelling speed and excess pore water pressure Δu. The increase in excess pore water pressure due to dynamic loading is only visible in the embankment, see Figure 3-6b (increase between 13 and 25%). The deviatoric shear stress is found to vary linearly with the travelling speed. However, this increase is very limited, and only noticeable in the embankment. The dynamic shear stress is in the embankment greater than the static shear stress. For the deeper subsoil layers, it is found that the deviatoric shear stress is approximately the same as the static one.

Pore pressure response at railway line Delft – Schiedam, The Netherlands
At three sites along the Delft -Schiedam railway line the pore pressures are being measured (Deltares, 2024b). There are a total of 36 pore water pressure transducers installed. The sensors are installed at different depth and are thus embedded in different soil layers. Apart from the pore water pressure transducers, trigger sensors were installed to measure the actual train passages. Data from the rail owner is used to connect train passages with specific pore pressure data.

The measurement data clearly contains fluctuations in pore pressures due to various sources or triggers. There is a seasonal pattern due to wetting and drying of the embankment and the surrounding soil, and there are increases due to precipitation. The observed increase of pore pressure due to an individual train passage is small with short peaks of on average 0.5 kPa and maximum ranging up to 3 kPa in the sensor with the highest response, compared to fluctuations caused by other effects such as precipitation and installation, and most prominently seasonal variations from 5 kPa to 10 kPa.

From the analysis and observations there is no detectable trend indicating a (structural) pore pressure build-up due to cyclic train loading within the pattern of fluctuations described above. That implies that the current operation in terms of composition, frequency and speed of trains does seem not cause long term increase of pore water pressures relevant to embankment stability (i.e. the probability of instability is not increased significantly).

The dissipation time is, broadly speaking, the time the pore pressures go back to normal after being triggered by a passing train. For all sensors the median dissipation time ranges from 20 to 60 seconds, while around 98% of the peaks dissipate in 100 seconds or less (counted from the moment that the train passes).

4 Field measurements / monitoring and interpretation to derive the stress state

4.1 In situ measurement techniques

Widely used geotechnical in-situ tests include the Standard Penetration Test (SPT), Cone Penetration Test (CPT), Flat Plate Dilatometer (DMT), Field Vane Test (FVT) and Pressuremeter Test (PMT) (Schnaid, 2009; Mayne et al., 2009; Robertson & Cabal, 2022). A CPT can be equipped with an additional porewater pressure sensor; the CPTu. An additional sensor to measure the shear wave velocity can also be added; this is a SCPTu.

Typical field measurement techniques in Switzerland are the Dynamic Probing (DP) test, SPT and (S)CPT(u). The DP can be performed using different specifications of the hammer and the falling distance of the hammer. A special DP from Switzerland is the “Von Moos” apparatus.

For each of these test devices a different measurement on one or more aspects of soil behaviour can be derived, depending upon the method of insertion, direction of loading, strain rate, device geometry, duration of testing, and so on (Mayne et al., 2009). Based on historical precedent and local geology each country tends to have a preferred in-situ test (Robertson & Cabal, 2022).

4.1.1 Standard Penetration Test (SPT)

The SPT is a very popular and widely used technique. This is due to the advantages of the simple test equipment and test procedure and the low cost of the test. With the SPT disturbed soil samples can be obtained. The test can be performed in most soil types, also in gravel layers, which are mostly a restriction for a CPT. A lot of experience and correlations are based on the SPT. Therefore, a wide range of soil parameters can be derived from the SPT.

An important disadvantage of this test type is that it produces only one number, the blow count. This blow count is affected by borehole preparation and size, sampler details, rod length and energy efficiency of the hammer-anvil-operator system (Mayne et al., 2009; Robertson & Cabal, 2022). Because of the dynamic nature of the test, interpretation of the SPT to obtain geotechnical parameters is generally restricted to clean cohesionless soils where penetration takes place under drained conditions (Robertson & Cabal, 2022). In practice the SPT is also applied in soft soils. Robertson & Cabal (2022) state that the applicability of the SPT for deriving soil parameters is moderate to low both for sand and clay. Experience with the local soil conditions is required to be able to deduce proper soil parameters from the SPT results.

4.1.2 Dynamic Probing (DP) test

The test equipment and test procedure of the Dynamic Probing (DP) test is comparable with the SPT. An important difference is that the DP uses a cone instead of a sampler. Another difference is that the DP is hammered into the subsoil from the surface, whereas the SPT is hammered into the subsoil in a borehole.

ISO 22476-2 recognizes four procedures for the DP, covering a wide range of specific work per blow:

- Dynamic probing light (DPL): test representing the lower end of the mass range of dynamic equipment.

- Dynamic probing medium (DPM): test representing the medium mass range of dynamic equipment.
- Dynamic probing heavy (DPH): test representing the medium to very heavy mass range of dynamic equipment.
- Dynamic probing super heavy (DPSH): test representing the upper end of the mass range of dynamic equipment.

The “Von Moos” apparatus is a special form of a DPL, with higher weight of the hammer and lower falling distance of the hammer, resulting in a bit higher hammer energy compared to the DPL (Geoprofile, 2024).

The DPSH uses a hammer with the same weight and drop height as the SPT. Due to the similarity of the SPT and DP the measurement results of both tests can be correlated. However, the differences between the test procedures cause noise in the correlation.

The applicability to derive accurate parameters for soft soils is comparable with the SPT, as the DP also gives only the blow count, and the nature of the DP is also dynamic.

4.1.3 Cone Penetration Test (CPT)

The Cone Penetration Test (CPT) has extensive applications in a wide range of soils. However, the CPT is limited primarily to softer soils. Penetration can be restricted in gravel and cemented layers, at least with the standard 10 cm^2 and 15 cm^2 cones. Larger diameter cones can be used in gravely soils. A CPT gives fast and continuous profiling of the subsoil and repeatable and reliable data (not operator-dependent). Another advantage is the strong theoretical basis for interpretation (Robertson & Cabal, 2022).

The applicability of CPT for deriving soil parameters in clay, such as overconsolidation ratio, undrained shear strength and sensitivity is high to moderate (Robertson & Cabal, 2022). To obtain parameters in sand the applicability of the CPT is a bit lower compared to clay. However, the quality of the parameters of sand derived with a CPT are higher than the quality of the parameters derived with the SPT (Robertson & Cabal, 2022).

As mentioned above a CPT can be equipped with a porewater pressure sensor (CPTu). An additional sensor to measure the shear wave velocity can also be added to the CPT. A cone with pressuremeter module is also possible. The applicability of the CPT to derive soil parameters improves when additional sensors are added. In the eighties of the past century some experimental work with a horizontal stress CPT is performed (Jefferies & Been, 2016). At present this device is still very much an experimental device.

The accuracy of the porewater pressure is not obvious. A lot of aspects influence the accuracy of the porewater pressure measurements, such as surface roughness of a filter element, shape and volume of the internal space between the pressure sensor and the outside perimeter area of the filter element, filter material types, filter pore sizes and fluid types for initial saturation of the measuring system (Campanella & Howie, 2008; Peuchen & Terwindt, 2015) and the preparation of the measuring system. During CPT penetration loss of saturation of the measurement system may occur in situations with free gas or in situations with partially saturated soils. Loss of saturation leads to sluggish pore pressure response (see Test 2 in Figure 4-1). Loss of saturation of the pore pressure measuring system may be mitigated by an additional CPT, preceded by pre-drilling or pre-pushing to below the soil zone causing loss of saturation (Peuchen & Terwindt, 2015).

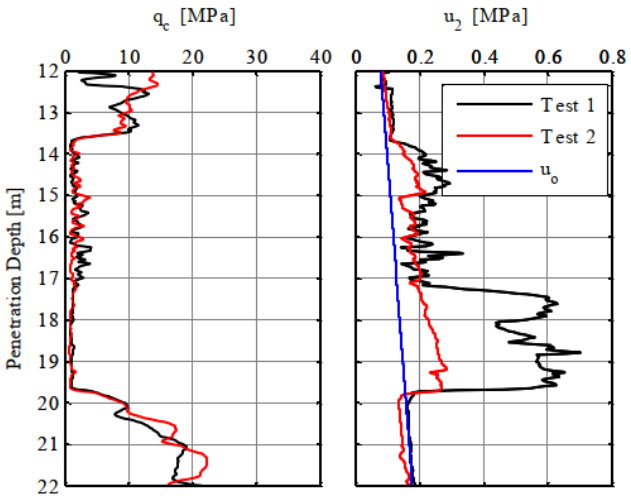


Figure 4-1 Pore pressure response for two closely spaced CPTs (Peuchen & Terwindt, 2015), with loss of saturation leading to sluggish pore pressure response in Test 2.

4.1.4 Flat Plate Dilatometer (DMT)

The DMT is an interesting apparatus as the membrane is pushed in horizontal direction into the subsoil. So, the DMT is testing the soil in another direction than the SPT and CPT do. The apparatus has proved to be fairly accurate, reliable and reproducible (Schnaid, 2009). A DMT takes measurements every 200 mm. An additional sensor to measure the shear wave velocity can be added to the DMT; this is a SDMT.

A DMT can be used in clay, silt and sand. In peat a DMT seem not to perform well. A DMT can be used for soil classification. In clay the overconsolidation ratio and coefficient of earth pressure at rest K_0 can be determined. According to Yu (2006) the correlation to derive the earth pressure coefficient K_0 is more reliable than the correlation to derive the overconsolidation ratio. Material specific constants are required to fit the theoretical solution to the data. In sand the friction angle and state parameter can be derived. To derive the friction angle using a DMT the soil stiffness has to be known (Yu, 2006). This is possible when using a SDMT.

Considering data from different sources Kulhawy & Mayne (1990) gave a general equation to derive K_0:

$$K_0 = (K_D/\beta_k)^{0.47} - 0.6$$

in which K_D is the horizontal stress index measured by the DMT and β_k depends upon soil type and geologic origin. Kulhawy & Mayne (1990) suggest local calibration of the DMT where possible. Figure 4-2 shows the relationship between K_D and K_0 based on data from different sites. Figure 4-3 shows a comparison between K_D from DMT and K_0 from a Self Boring Pressuremeter (SBPMT) for different clays (Kulhawy & Mayne, 1990).

Lunne et al. (1989, 1990) showed that accurate K_0 values in young soft clays could be determined with:

$$K_0 = 0.34(K_D)^{0.54}$$

GeoDelft (2007) found no differences in horizontal stress index K_D in parallel and perpendicular direction near the toe of flood embankments at three sites.

These DMT measurements are performed in soft organic clay and peat. May be the accuracy of the DMT is insufficient in these very soft soils.

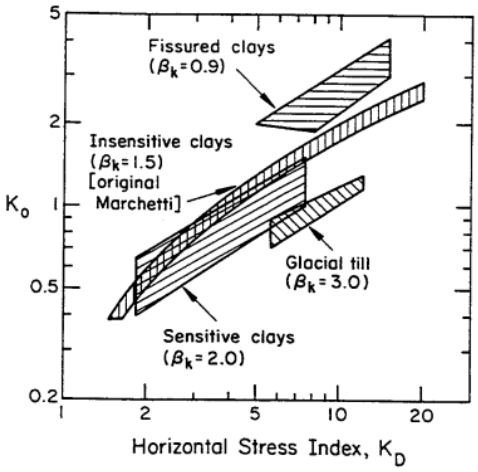


Figure 4-2 Relationship between K_D and K_0 based on data from different sites (Kulhawy & Mayne, 1990).

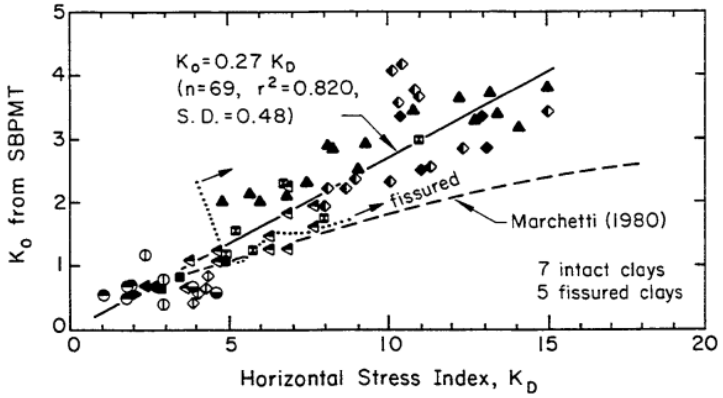


Figure 4-3 Comparison between K_D from DMT and K_0 from a Self Boring Pressuremeter (SBPMT) (Kulhawy & Mayne, 1990).

4.1.5 Pressuremeter Test

The pressuremeter test can be applied as a Self Boring Pressuremeter (SBPMT) or as a pressuremeter mounted on a CPT cone.

The pressure-expansion curve can be interpreted to give an estimate of the stress-strain-strength response of the ground. The state parameter can also directly be retrieved from the pressuremeter test.

Schnaid (2009) and Jefferies & Been (2016) states that the SBPMT appears to be the most appropriate technique for determining the in situ horizontal stress.

Advantage of the SBPMT is the undisturbed soil conditions according to Yu (2006), however interpretation problems can arise due to contrary trends of the arms of the pressuremeter caused by borehole imperfections and tilting of the pressuremeter during inflation (Jardine & Hight, 1987).

Yu (2006) gives interpretation for cone pressuremeter. In very soft soils the corrections for membrane stiffness can have a large influence (35 – 60% of the measured pressuremeter pressure) on the interpretation (GeoDelft, 2007).

Figure 4-4 shows a relationship between OCR deduced from oedometer tests with K_0 measured with the SBPMT for intact clays and fissured clays (Kulhawy & Mayne, 1990).

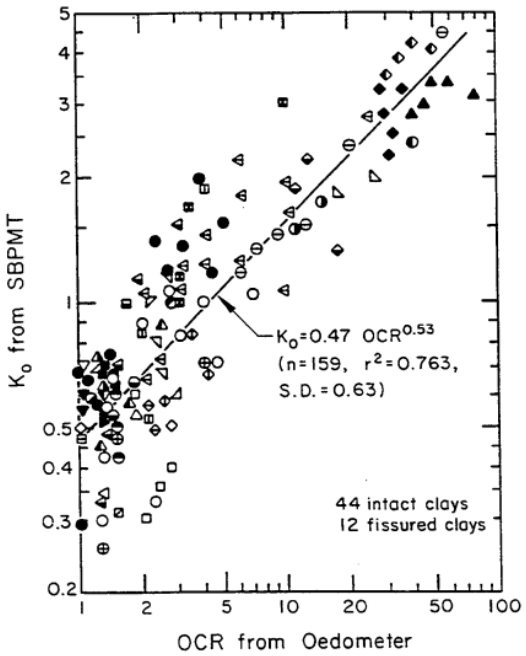


Figure 4-4 Relationship between OCR deduced from oedometer tests with K_0 measured with the SBPMT for intact clays and fissured clays (Kulhawy & Mayne, 1990).

4.1.6 Field Vane Test (FVT)

With a FVT the undrained shear strength and sensitivity can be derived. The undrained shear strength can be used to obtain OCR with use of the SHANSEP-model (Ladd & Foot, 1974). It is important to ensure that the rate of rotation of the vane maintains undrained conditions. The standard rate of rotation produces undrained conditions in most clays. In silty or sandy clays, the FVT can produce unrealistically high values of undrained strength.

Robertson & Cabal (2022) recommend the FVT to be carried out after and adjacent to a CPT sounding. They mention that the CPT can provide a detailed stratigraphic profile and identify the depth and thicknesses of the various clay layers where FVT should be performed. When the FVT results are plotted on top of the estimated values of the undrained shear strength based on the CPT results the FVT results can be used to ensure consistency and to guide the application of a suitable cone factor N_{kt} for the CPT interpretation.

4.2 Soil classification

4.2.1 Classification

Soil classification using geotechnical field techniques is mostly based on CPT, CPTu or SCPTu. In literature a lot of soil classification methods can be found. Well known soil classifications are from Robertson (1986, 2010, 2016). The versions from 2010 and 2016 are based on the normalized cone tip resistance Q_t and normalized sleeve friction F_r. Robertson uses the classification parameters I_c and I_B. The parameter I_B is introduced by Robertson to recognize transitional soils (see Paragraph 4.5). Robertson et al. (1995) also developed a classification chart based on the normalized dimensionless cone tip resistance and the ratio of the small strain stiffness and the cone tip resistance G_0/q_t (with $G_0 = \rho\, V_s^2$, in which ρ is the mass density of the soil and V_s is the shear wave velocity).

Jefferies & Davies (1991) proposed a classification chart with the dimensionless parameter group $Q_t(1 - B_q)$ versus the normalized friction ratio F_r. In this chart, the drainage conditions around the cone during penetration are incorporated, which is helpful for mixtures such as silty sand and clayey sand. The use of this chart requires the knowledge of the pore pressure u_2. B_q is the normalized excess pore pressure. Been & Jefferies (1992) modified the classification chart of Jefferies and Davies (1991) and uses the dimensionless parameter group $Q_t(1 - B_q) + 1$ versus the normalized friction ratio F_r. The +1 term is introduced to deal with loose silt. The Schneider et al. (2008) classification charts based on pore water pressure (u_2/σ'_{v0} and B_q) are also often used. Schneider et al. developed soil classification charts to aid in separating whether cone penetration is drained, undrained, or partially drained. Concerning the classification methods based on u_2 the accuracy of these measurements is a point of attention, as mentioned in Paragraph 4.1.3.

Note that the various classification charts are based on normalized parameters (Q_t and F_r). To derive these parameters the effective stress is important. To calculate the effective stress the bulk unit weight and pore water pressure has to be known. At low effective stress conditions assumptions of bulk unit weight and pore water pressure can importantly influence the normalized parameters.

To calculate total stress the SBTs can be applied with best estimates of bulk unit weight. Estimations of bulk unit weight can be based on measured CPT parameters, see Paragraph 4.2.2. The SBT is also sensitive for the estimated steady state pore water pressure. The uncertainty in pore water pressure can influence soil classification because the use of normalized parameters Q_t, F_r and B_q where effective stress plays a role.

In the Dutch engineering practice, the Begemann (1965) classification is still widely used; see Figure 4-5. According to the Deltares experience the Begemann classification is well suited to classify organic soils and peat. Figure 4-6 and Figure 4-7 show an example at Kinderdijk, The Netherlands. Figure 4-6 gives several classification parameters of this site. In this figure bulk unit weight increases and plasticity index I_p and organic content OC decrease. Figure 4-7 gives the comparison of the classification for this site using the Robertson I_c parameter, the friction ratio F_r, according to Robertson and the Begemann classification parameter R_f. The Begemann classification in Figure 4-7 with the generally decreasing friction ratio R_f indicates less plastic and less organic material with larger depth. This compares better with the trends in Figure 4-6 then the Robertson classification parameter I_c. The F_r parameter makes also more sense to distinguish organic soils and peat than I_c.

Lengkeek (2024) developed a classification chart based on the Robertson (2010, 2016) classification charts and added boundaries to distinguish organic clay and peat. For these boundaries for organic clay and peat F_r is the discriminating parameter.

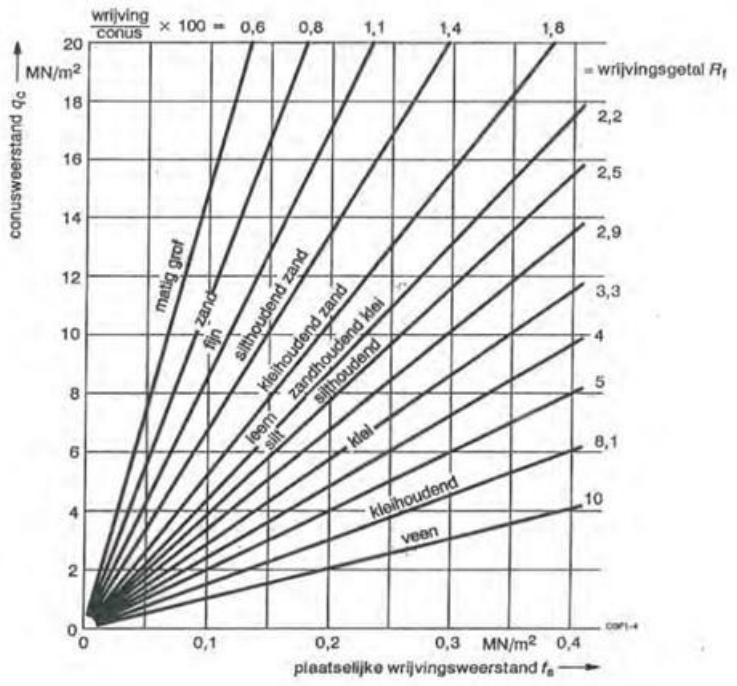


Figure 4-5 Begemann (1965) classification chart with the sleeve friction f_s at the x-axis, cone resistance q_c at the y-axis and areas with soil types indicated by the friction ratio R_f.

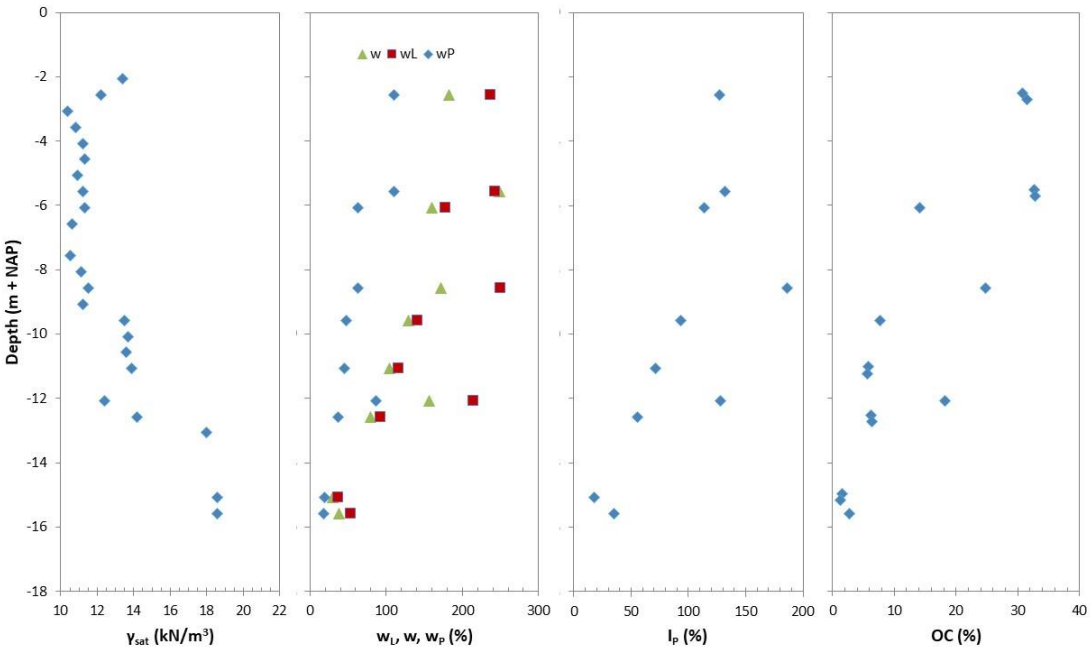


Figure 4-6 Soil classification parameters at Kinderdijk, The Netherlands.

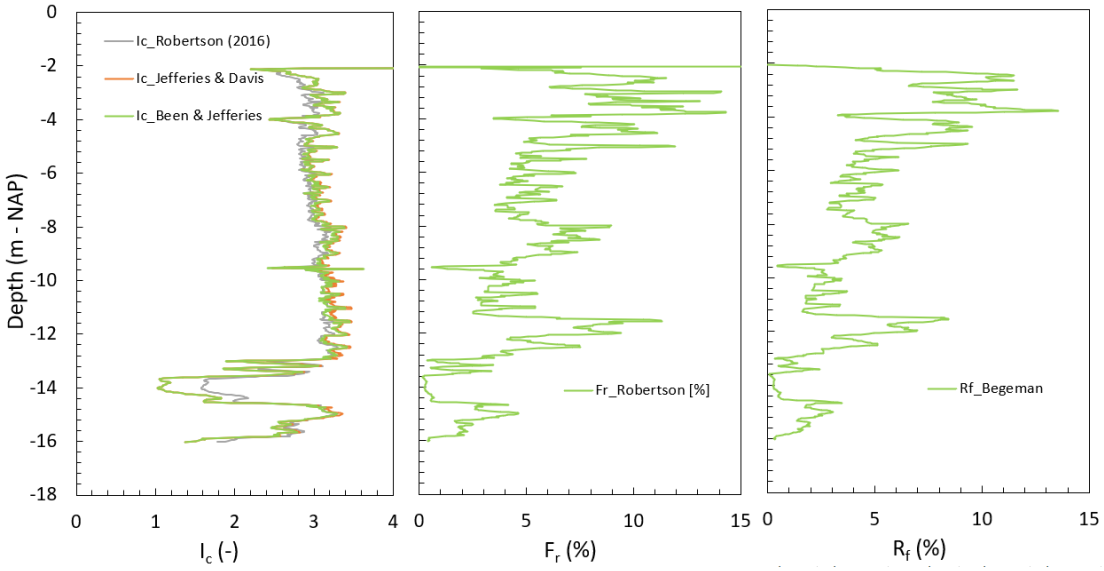


Figure 4-7 Soil classification based on CPT with I_c, F_r and R_f for Kinderdijk, The Netherlands.

4.2.2 Bulk unit weight

To estimate the bulk unit weight γ also several methods based on CPT, CPTu and SCPTu are available in literature. Some of these methods are bases on cone tip resistance q_c or friction ratio R_f (Mayne & Peuchen, 2013; Robertson & Cabal, 2022; Lengkeek, 2022) and others are based on shear wave velocity V_s (Mayne, 2007 and Mayne et al., 2009). Lengkeek developed methods to estimate bulk unit weight for all soil types (Figure 4-8) and also a method especially for peat (Figure 4-9). Lengkeek also found a correlation between friction ratio R_f and specific gravity of the soil particles G_s (Figure 4-10). G_s derived from this correlation can be used in the Robertson method to get better estimations of the bulk unit weight. The methods based on shear wave velocity V_s are presented in Figure 4-11 and Figure 4-12.

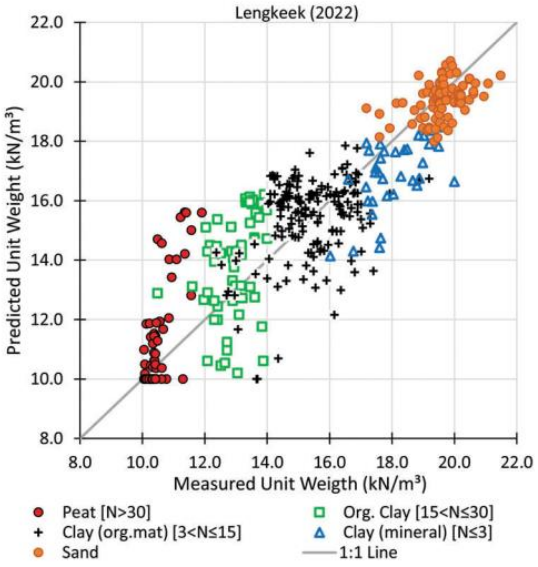


Figure 4-8 Measured versus predicted bulk unit weight with the Lengkeek (2022) method.

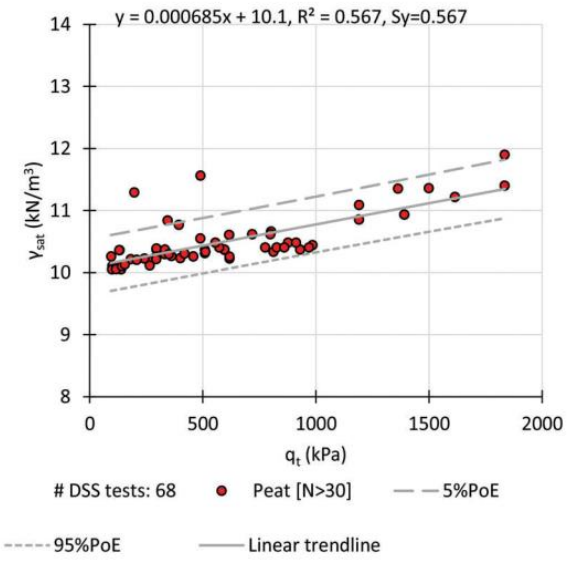


Figure 4-9 Correlation between cone resistance and saturated bulk unit weight for peat (Lengkeek, 2022).

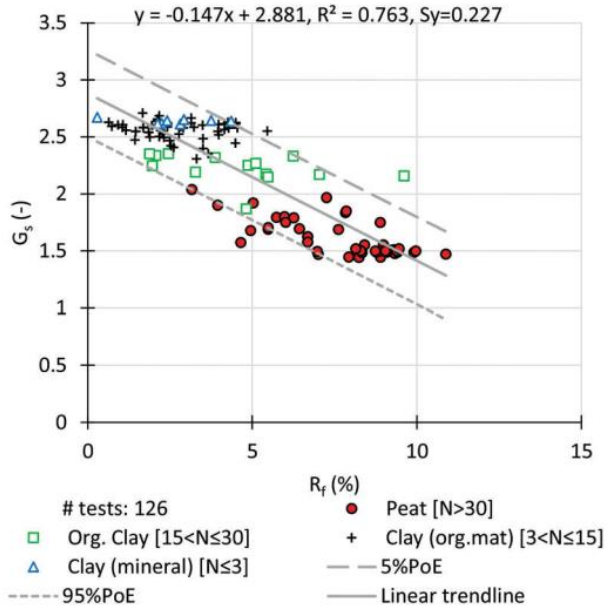


Figure 4-10 Correlation between friction ratio R_f and specific gravity G_s for organic to mineral soils (Lengkeek, 2022).

These methods to deduce bulk unit weight are all based on directly measured CPT parameters, without corrections. This has the advantage that the estimation of the bulk unit weight can be done before using the soil classification methods. So, the soil classification charts can be used with a reasonable estimate of the bulk unit weight.

The effect of the uncertainty of the bulk unit weight estimations on q_{net} and Q_t may sometimes be relatively small, but the effect of this uncertainty on the calculation of overconsolidation ratio OCR and pre overburden pressure POP is mostly very relevant.

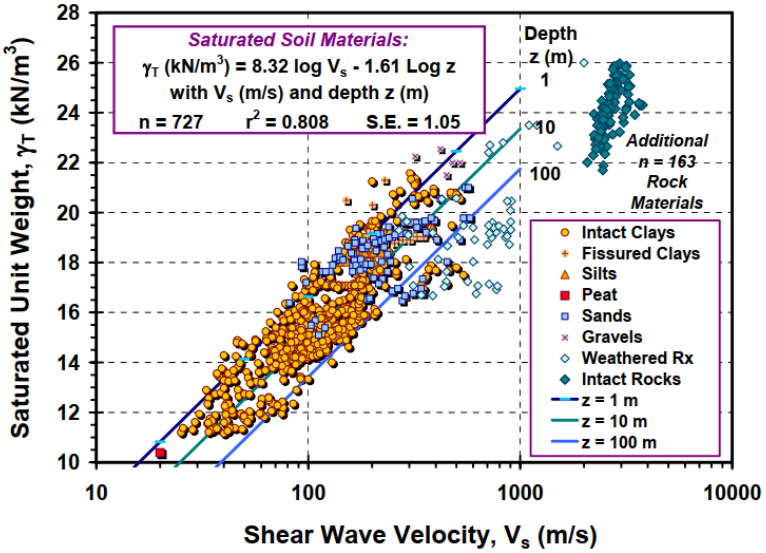


Figure 4-11 Correlation between shear wave velocity V_s and depth and saturated bulk unit weight (Mayne, 2007).

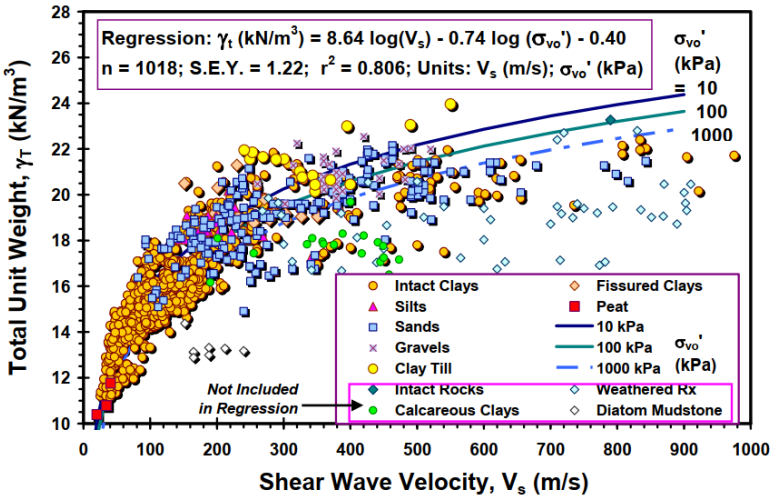


Figure 4-12 Correlation between shear wave velocity and vertical effective stress and saturated bulk unit weight (Mayne et al., 2009).

The correlations to derive the bulk unit weight as mentioned before are applied on the case Kinderdijk, The Netherlands. The bulk unit weight derived from the correlations are compared with bulk unit weights derived from field measurements on tubes of boreholes and laboratory measurements (Figure 4-13). The Robertson and Lengkeek correlations give reasonable results in comparison with the field and laboratory measurements. For peat the Robertson and Lengkeek methods are sometimes not accurate enough; see around -2 to -5 (estimations to low) and -6 to -8 (estimations to high). The Lengkeek correlation for peat is for the top layers at the low side. The Robertson correlation and Lengkeek correlation give comparable results when the Robertson correlation is applied using the estimated G_s with the Lengkeek correlation (Figure 4-10). The Mayne et al. correlation performs not good.

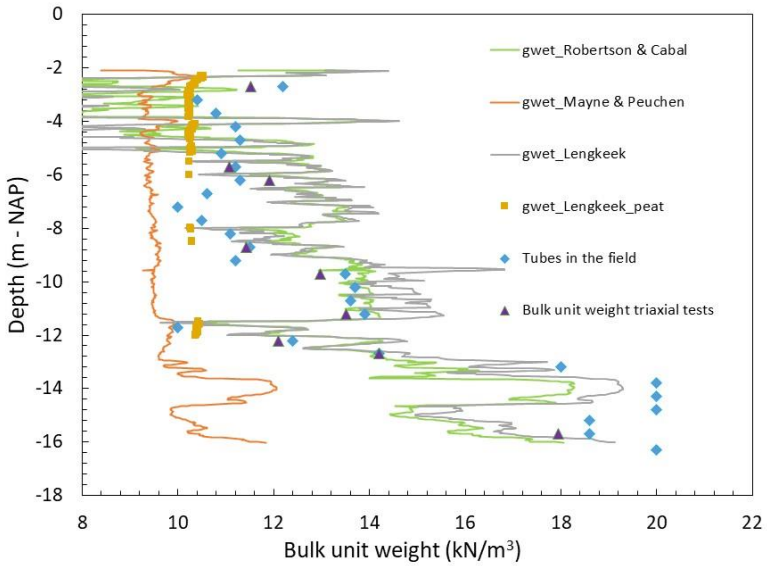


Figure 4-13 Estimation of saturated bulk unit weight for Kinderdijk, The Netherlands, using different CPT correlations and cross check with measured bulk unit weight.

In Figure 4-14 the same correlations to estimate the bulk unit weight are applied to the case Streefkerk, The Netherlands. At this site SCPTu are performed. Using the shear wave velocity data the bulk unit weight for this case is also estimated with the Mayne correlation. For this site the correlations result in reasonable estimates for clays with bulk unit weight above 12 - 13 kN/m^3. Most correlations overestimate the bulk unit weight for organic clay and peat. For the Streefkerk site the Lengkeek correlation for peat gives a good estimation for peat.

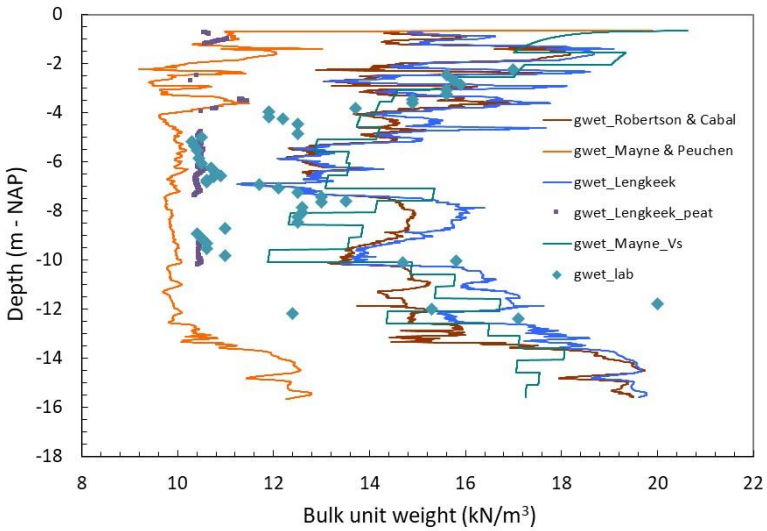


Figure 4-14 Estimation of saturated bulk unit weight for Streefkerk, The Netherlands, using different SCPT correlations and cross check with measured bulk unit weight.

4.2.3 Fines content

In some cases, it might be helpful to have an estimation of the fines content of soil layers, for instance for sandy and silty layers where the liquifiability has to be assessed. Robertson & Wride (1998), Boulanger & Idriss (2016) and Hudson et al (2024) give correlations to get rough estimations of the fines content based on CPT data. Figure 4-15 gives a comparison of these correlations.

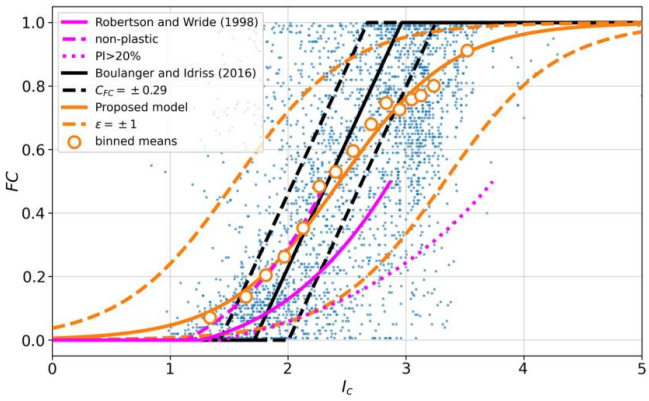


Figure 4-15 Different correlations between material index Ic and fines content (FC) (Hudson et al., 2024).

4.2.4 Plasticity index

Based on CPTu data also a rough estimation of the plasticity index can be made, according to Ramsey et al. (2023). Such an estimation may also be helpful to recognize non clean sand. Figure 4-16 shows a comparison between the plasticity index as determined in the laboratory with the plasticity index estimated from CPTu data.

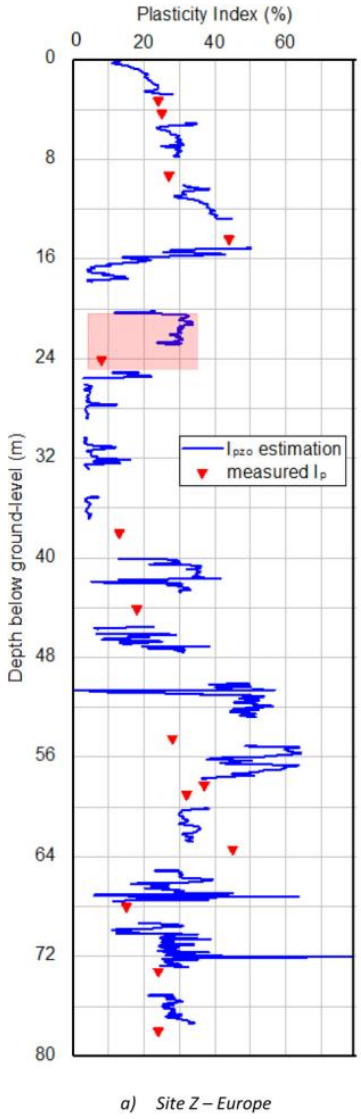


Figure 4-16 Correlation between CPTu data and plasticity index I_p (Ramsey et al., 2023).

4.3 Stress state of clay and peat

4.3.1 Overconsolidation

To get insight in the stress state of clay and peat, the yield stress σ'_{vy} or overconsolidation ratio OCR are important parameters. Different approaches can be applied to derive yield stress or OCR from CPT, CPTu or SCPTu:

- Empirical relations with an empirical cone factor to derive yield stress σ'_{vy} or OCR directly from the net cone resistance q_{net} (Mayne et al., 1995; Mesri, 2001; Demers et al., 2002; Mayne et al., 2009, Robertson, 2009).
- Theoretical relations to derive OCR from normalized cone resistance Q_t based on cavity expansion theory and critical state soil mechanics (Mayne, 1991 and Mo et al., 2023).
- Empirical relations with empirical cone factor N_{kt} and using the SHANSEP model (Ladd & Foot, 1974) to derive OCR via the undrained shear strength s_u from the net cone resistance q_{net}.
- Theoretical solutions to determine the cone factor N_{kt} and using the SHANSEP model (Ladd & Foot, 1974) to derive OCR via the undrained shear strength s_u from net cone resistance q_{net}.

The theoretical solutions to determine the theoretical cone factor N_{kt} account for one or more of the aspects of the soil behaviour, such as the rigidity index, surface roughness of the cone face and the anisotropy of the in-situ stress state around the cone. These solutions are from Teh et al. (1988), Van den Berg (1994), Lu et al. (2004), Yu (2006) and others.

- Empirical relations based on shear wave velocity V_s, directly via yield stress or overconsolidation ratio (Mayne et al., 1998; Mayne, 2007).
- Empirical relations based on shear wave velocity V_s, indirectly via undrained shear strength (Levesque et al., 2007; Agaiby et al., 2015; Agaiby, 2018; L'Heureux et al., 2017) with use of the SHANSEP model.
- Analytical model to derive OCR from normalized cone resistance Q_t and rigidity index I_R ($= G_0/s_u$ with $G_0 = \rho\, V_s$), based on cavity expansion theory and critical state soil mechanics (Mayne, 1991).

Lunne et al. (1997) pointed out that cone penetration is a complex phenomenon and that the theoretical solutions are simplifications of the real soil behaviour around a cone. The used models have restrictions to describe the real soil behaviour at the different circumstances of stress history, anisotropy, sensitivity, ageing and structure. Lunne et al. prefer the empirical correlations, which have to be tuned on the local experience and local soil conditions.

Direct derivation of OCR from cone resistance with empirical relations

The overconsolidation ratio can directly be derived from the net cone resistance with empirical relations, using an empirical cone factor k (Mayne et al., 1995; Mesri, 2001; Demers et al., 2002; and others):

$$\sigma'_{vy} = k(q_t - \sigma_{v0}) \text{ and } OCR = \frac{\sigma'_{vy}}{\sigma'_{v0}}$$

The empirical cone factor k ranges from 0.1 to 0.5. For organic clays Demers et al. (2002) found values between 0.1 and 0.2 and values between 0.14 and 0.5 for non-organic clays.

Robertson (2009) developed a similar equation using the normalized cone factor Q_{tn} and with k in the same range as aforementioned:

$$OCR = k\left(\frac{q_t - \sigma_{v0}}{\sigma'_{v0}}\right) = k\, Q_{tn}$$

Mayne et al. (2009) found an equation using a soil dependent exponent m ranging from 0.7 for sand to 1.0 for intact clay (with the factor 0.33 fixed):

$$\sigma'_{vy} = 0.33(q_t - \sigma_{v0})^m$$

Theoretical relations to derive OCR from normalized cone resistance

To derive OCR Mayne (1991) developed a theoretical relationship based on cavity expansion theory and critical state soil mechanics:

$$OCR = 2\left[\frac{1}{1.95M + 1}\left(\frac{q_t - u_2}{\sigma'_{v0}}\right)\right]^{1/m}$$

With $M = 6\sin\varphi' / (3 - \sin\varphi')$ and m the strength gain exponent. The friction angle φ' ranges from 30º for silty clay to above 60º for fibrous peat for triaxial compression conditions.

Recently Mo et al. (2023) also derived a relationship based on cavity expansion theory and critical state soil mechanics. This relationship is validated against two series of advanced numerical simulations and calibration chamber tests of the CPTu in fine-grained soils.

$$\ln OCR = \ln\left[r^*\left(\frac{2}{M\,N_{kE}}\right)^{1/m}\right] + \frac{1}{m}\ln Q_E$$

With $N_{kE} = \left(\frac{2}{M} + \frac{4}{3}\right)\left(1 + \sqrt{3}\tan\delta_c\right)$ and $Q_E = Q_t\left(1 - B_q\right) + 1$. The spacing ratio r^* = 2.0. δ_c is the friction angle of the cone-soil interface = 0.5 to 0.8 φ'_{cs}.

Derivation of OCR from net cone resistance with SHANSEP model

As the cone penetration resistance is primarily dominated by the undrained shear strength s_u, it is obvious to estimate s_u from the cone penetration resistance first and after that to calculate the overconsolidation ratio OCR. OCR can be derived from the cone resistance with empirical relations and using the SHANSEP model (Ladd & Foot, 1974):

$$s_u = \left(\frac{q_t - \sigma_{v0}}{N_{kt}}\right) \text{ and } OCR = \left(\frac{s_u}{\sigma'_{v0}\,S}\right)^{1/m}$$

S is the normally consolidated undrained shear strength ratio, which ranges from 0.3 for silty clay to 0.6 for peat for triaxial compression mode of failure. For the direct simple shear mode of failure S ranges from 0.21 for silty clay to 0.4 for peat. m is the strength increase exponent, which is about 0.7 to 0.9.

There are different approaches to estimate the empirical cone factor N_{kt}. Robertson (2012) suggested a relationship of N_{kt} with the friction ratio F_r to derive the peak undrained shear strength for direct simple shear mode of failure:

$$N_{kt} = 10.5 + 7\log\left(F_r\right)$$

Mayne & Peuchen (2018) developed a relationship to estimate N_{kt} based on the pore pressure parameter B_q, see Figure 4-17. Note that estimation of N_{kt} based on B_q requires accurate u_2 measurements; see Paragraph 4.1. This relationship gives an estimation of N_{kt} to derive the peak undrained shear strength for triaxial compression mode of failure.

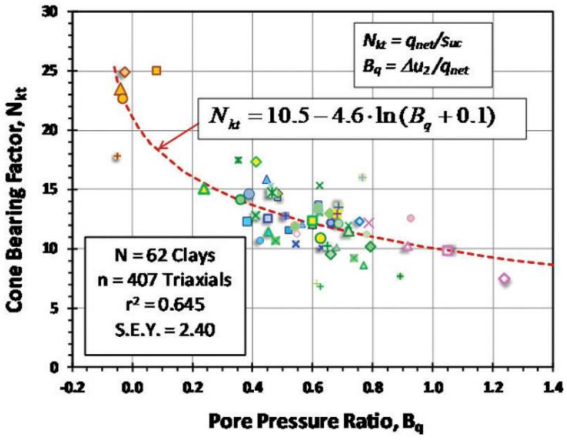


Figure 4-17 Correlation between pore water pressure parameter B_q and cone factor N_{kt} (Mayne & Peuchen, 2018).

To estimate the s_u of peat based on the net cone resistance q_{net} Hayashi & Yamanashi (2018) found a relationship between N_{kt} and loss of ignition LOI; see Figure 4-18. The reference strength is derived from K_0 consolidated-undrained triaxial compression tests (K_0CUC tests). Note that compared to the Robertson (2012) and Mayne & Peuchen (2018) estimations of N_{kt} the N_{kt} of peat is relatively high. Zwanenburg & Jardine (2016) reported a N_{kt} of 15.3 with the field vane tests as the reference strength.

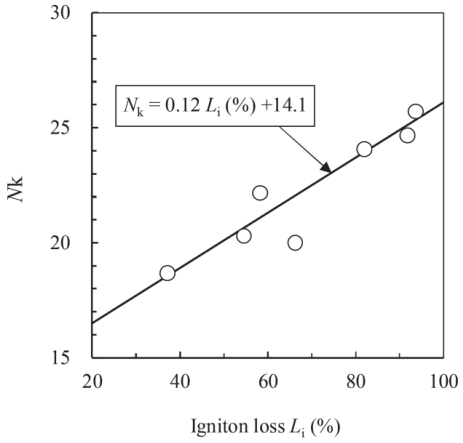


Figure 4-18 Relationship between N_{kt} and loss of ignition LOI to estimate the s_u of peat (Hayashi & Yamanashi, 2018).

Example Kinderdijk, The Netherlands

For a site at Kinderdijk, The Netherlands, some of the previous discussed methods to derive the stress state are applied. The subsoil at Kinderdijk consists of organic clays and peat. Figure 4-19 shows the derived undrained shear strength from CPTs using estimated N_{kt} based on Robertson (2012) and Mayne et al. (2018) for site Kinderdijk, with comparison with undrained shear strength from field vane tests (FVT) and triaxial compression tests (TXC).

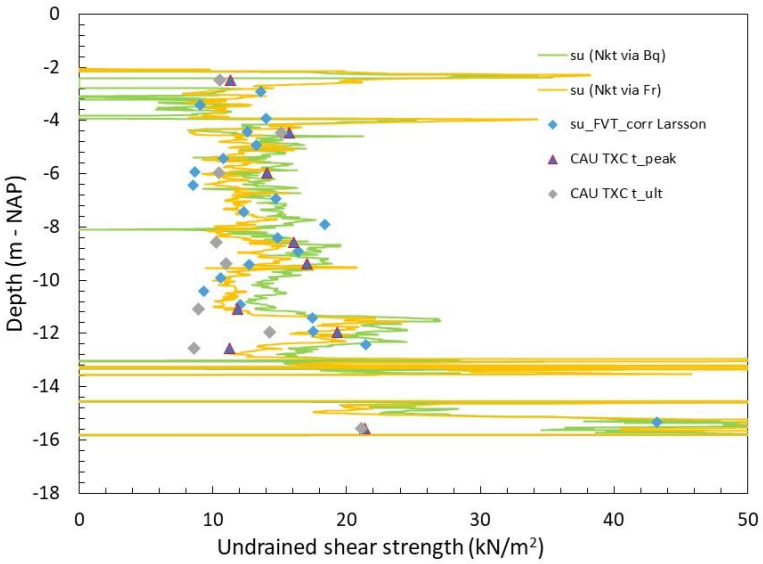


Figure 4-19 Derivation of undrained shear strength from CPT using estimated N_{kt} based on Robertson (2012) and Mayne et al. (2018) for site Kinderdijk, The Netherlands, with comparison with undrained shear strength from field vane tests (FVT) and triaxial tests (TXC).

The peak undrained shear strength from triaxial compression tests give a reasonable fit with the undrained shear strength derived with N_{kt} based on Mayne et al. (2018).

The undrained shear strength as obtained with N_{kt} based on Robertson (2012) gives lower values, more or less in line with the ultimate values of the undrained shear strength derived from triaxial compression tests. The undrained shear strength from FVT has a large scatter which overlap with all the other data.

Figure 4-20 shows the derived OCR values for the site at Kinderdijk. Various methods are applied to obtain OCR from CPTs. CRS tests are available as a reference. The Mo et al. (2023) relationship based on cavity expansion theory and critical state soil mechanics performs the best.

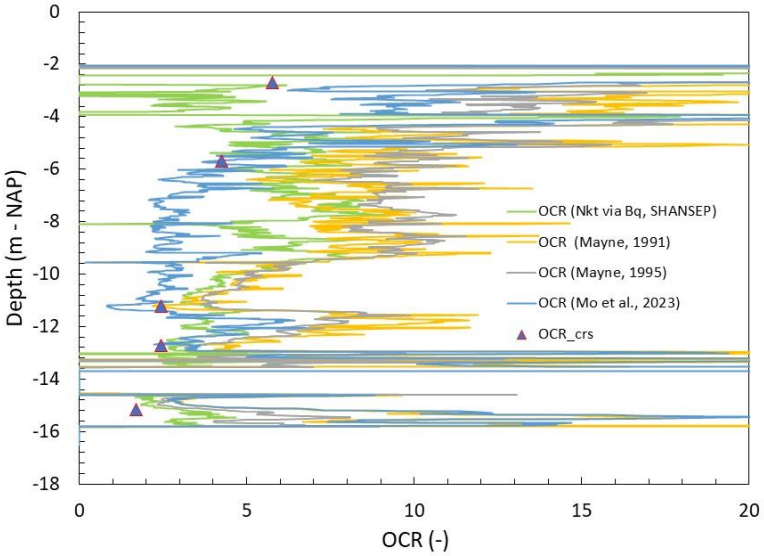


Figure 4-20 Derivation of overconsolidation ratio from CPT using different methods for site Kinderdijk, The Netherlands, with comparison with overconsolidation ratio derived from constant rate of strain (CRS) tests.

Derivation of OCR from shear wave velocity V_s via yield stress

When SCPTu are performed and the shear wave velocity V_s is available this parameter can also be used to determine the yield stress σ'_{vy} or overconsolidation ratio OCR. Mayne et al. (1998) give the relationship:

$$\sigma'_{vy} = (q_t - \sigma_{v0})^{0.702} \, (V_s/64)^{0.751}$$

Mayne (2007) gives an overall relationship for various intact geomaterials:

$$\sigma'_{vy} = 0.101(\sigma_{atm})^{0.102} \, 0.161(G_0)^{0.478} \, (\sigma'_{v0})^{0.42} \text{ with } G_0 = \rho \, V_s$$

Figure 4-21 shows the relationship from Mayne (2007).

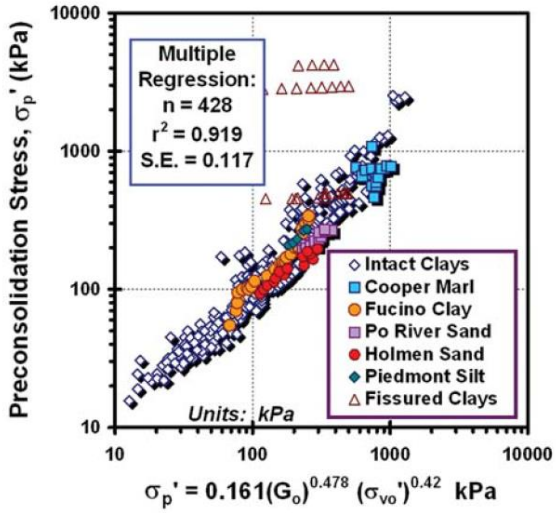


Figure 4-21 Correlation between small-strain shear modulus and yield stress (Mayne, 2007).

Derivation of OCR from shear wave velocity V_s via undrained shear strength

Several authors developed a relationship between shear wave velocity and undrained shear strength s_u (Levesque et al., 2007; Agaiby & Mayne, 2015; Agaiby, 2018; L'Heureux & Long, 2017). Using these relationships the undrained shear strength can be derived from the SCPTu. OCR can be retrieved from s_u with the SHANSEP model when the normally consolidated undrained shear strength ratio S is known. It is not reported whether these relationships are based on peak s_u values or ultimate state s_u values. Figure 4-22 shows the aforementioned relationships between shear wave velocity and undrained shear strength.

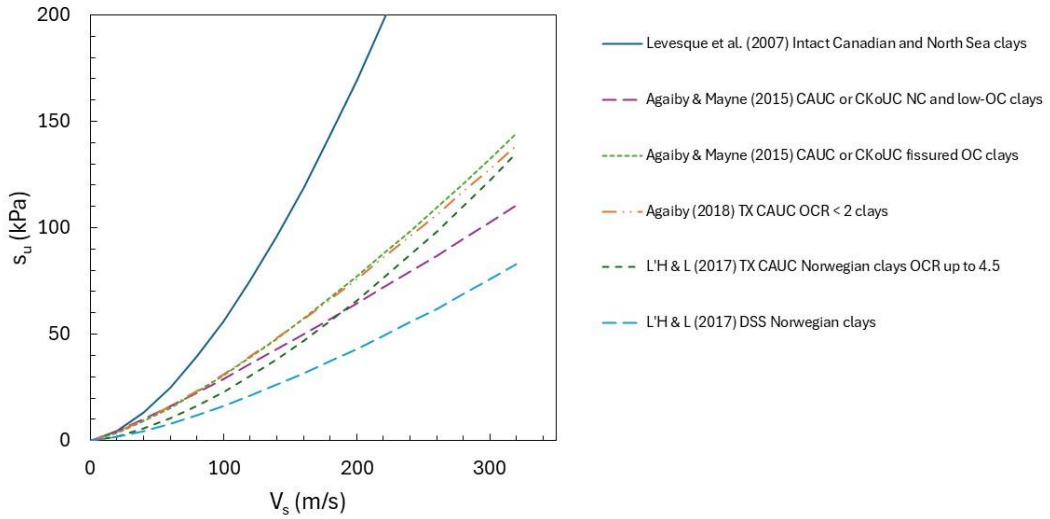


Figure 4-22 Different correlations between shear wave velocity and undrained shear strength.

Based on Figure 4-22 the L'Heureux & Long (2017) relationship for the triaxial compression mode of shearing can be seen as an average estimation:

$$s_u^{\square} = 0.021(V_s)^{1.52}$$

According to Agaiby et al. (2015) the accuracy of s_u derivation based on the cone resistance is more accurate than based on the shear wave velocity. A comparison for a dataset with 305 datapoints gives regression coefficients of 0.88 and 0.81 respectively. Agaiby et al. (2015) report a standard deviation of 0.28 for s_u based on the shear wave velocity.

Long (2022) mentions that the V_s-s_u correlations can be very powerful and useful but they should only be used for the test type and materials for which they were originally derived and they should only be applied to other materials/other test types with great caution.

Analytical model to derive OCR from normalized cone resistance Q_t and rigidity index I_R

Mayne (1991) derived an analytical model to obtain OCR from normalized cone resistance Q_t and rigidity index I_R ($= G_0/s_u$ with $G_0 = \rho\, V_s$), based on cavity expansion theory and critical state soil mechanics.

$$OCR = 2\left[\frac{\frac{2}{M}\,Q}{\frac{4}{3}[ln(I_R)+1]+\frac{\pi}{2}+1}\right]^{1/m}$$

With $M = 6\sin\varphi'/(3-\sin\varphi')$ and m the strength gain exponent.

Examples Delft and Streefkerk from The Netherlands

Some of the relationships as discussed before, including some of the relationship based on shear wave velocity V_s, are applied for two sites in The Netherlands: Delft and Streefkerk. At both sites the subsoil consists of organic clays and peat. Figure 4-23 shows the derivation of s_u from CPTs using the estimated N_{kt} based on Robertson (2012) with F_r and based on Mayne et al. (2018) with B_q for site Delft, The Netherlands. The s_u based on field vane tests (SVT) with rotation rates of 0.1 (standard rate) and 1.5 degrees per second are also presented. The figures also show s_u (peak values) based triaxial compression tests and direct simple shear tests (only for peat). Furthermore, s_u based on V_s using the correlations of L'Heureux & Long (2017) is presented.

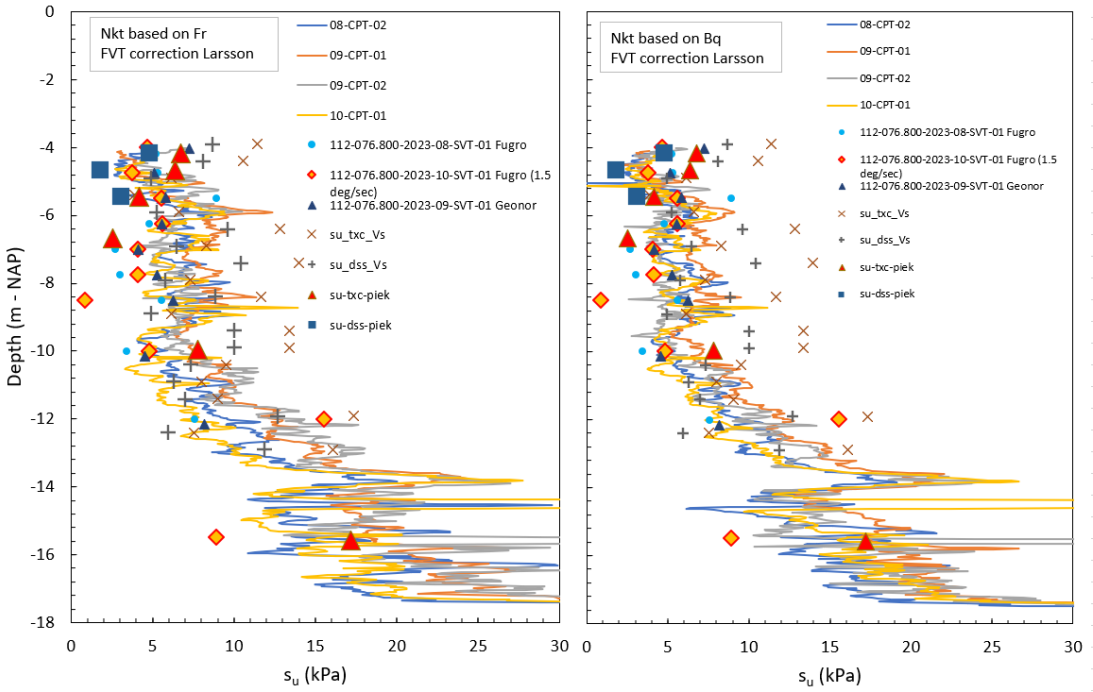


Figure 4-23 Derivation of undrained shear strength from CPT using estimated N_{kt} based on Robertson (2012) and Mayne et al. (2018) for site Delft, The Netherlands, with comparison with undrained shear strength from field vane tests (SVT) and from shear wave velocity using the correlations of L'Heureux & Long (2017).

The undrained shear strength s_u in Figure 4-23 show a reasonable comparison between s_u from CPTs, FVTs and triaxial compression tests and direct simple shear tests. There are some outliers. Much of the differences between the methods and test types is less dominant because of the spatial variability of s_u as illustrated by the variability of s_u based on the four CPTs. s_u from the CPTs with estimated N_{kt} based on Robertson (2012) with F_r is somewhat higher than s_u from CPTs using the estimated N_{kt} based on Mayne et al. (2018) with B_q. The latter seem to fit the best with the other data for this case. s_u based on the shear wave velocity V_s does not perform well.

Figure 4-24 shows the results for the site at Streefkerk. Here again s_u is derived from the CPT using the estimated N_{kt} based on Robertson (2012) with F_r and based on Mayne et al. (2018) with B_q. Data from triaxial compression tests (peak values) and direct simple shear tests (peak values; only for peat) are also plotted. s_u is also obtained using V_s with the relations of L'Heureux & Long (2017) and Agaiby & Mayne (2015).

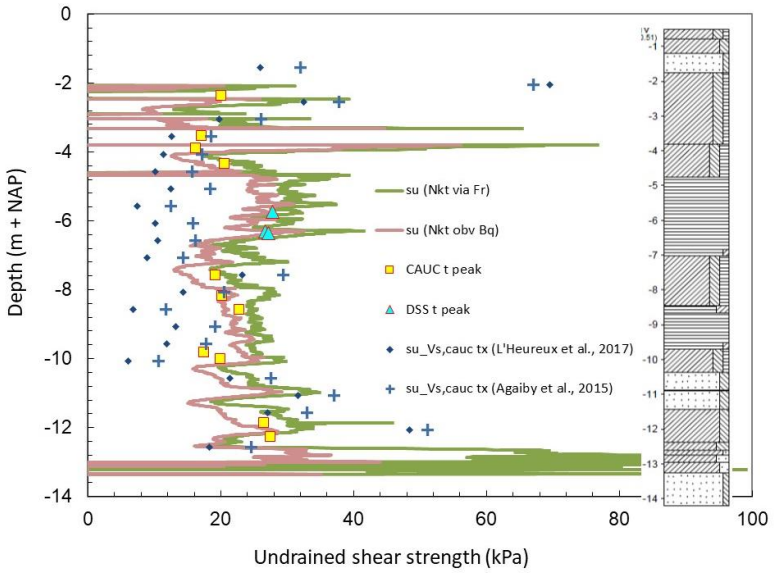


Figure 4-24 Derivation of undrained shear strength from SCPTu using estimated N_{kt} based on Robertson (2012) and Mayne et al. (2018) for site Streefkerk (SCPTu 05), The Netherlands, with comparison with undrained shear strength from triaxial compression tests (CAUC), direct simple shear tests (DSS) and from shear wave velocity using the correlations of Agaiby et al. (2015) and L'Heureux & Long (2017).

At the Streefkerk site s_u obtained from the CPT using the estimated N_{kt} based on Mayne et al. (2018) with B_q seems to fit better with the peak values of the triaxial compression tests than s_u based on the estimated N_{kt} based on Robertson (2012) with F_r. For the direct simple shear tests both N_{kt} estimations perform equally. The relations of L'Heureux & Long (2017) and Agaiby & Mayne (2015) using V_s are for this site to low. This is remarkable as the L'Heureux & Long (2017) relationship was on the high site for the Delft case.

In Figure 4-25 the derived OCR of the Streefkerk site is presented. Several methods are applied. Based on s_u from Figure 4-24 OCR is calculated using the SHANSEP model. The theoretical model of Mo et al. (2023) is also applied. Furthermore, the two models of Mayne (1998, 2007) based on V_s are applied. OCR data is also available from oedometer tests and constant rate of strain tests.

The calculated OCR using the SHANSEP model gives a good fit with the laboratory data in this case, especially the approach where the estimated N_{kt} based on Mayne et al. (2018) with B_q is used. The Mayne (1998) relationships using V_s also performs well. The Mayne (2007) relationship based on V_s gives very low values. The theoretical model from Mo et al. (2023) which performs good for the Kinderdijk site (see Figure 4-20) does not perform well for the Streefkerk site.

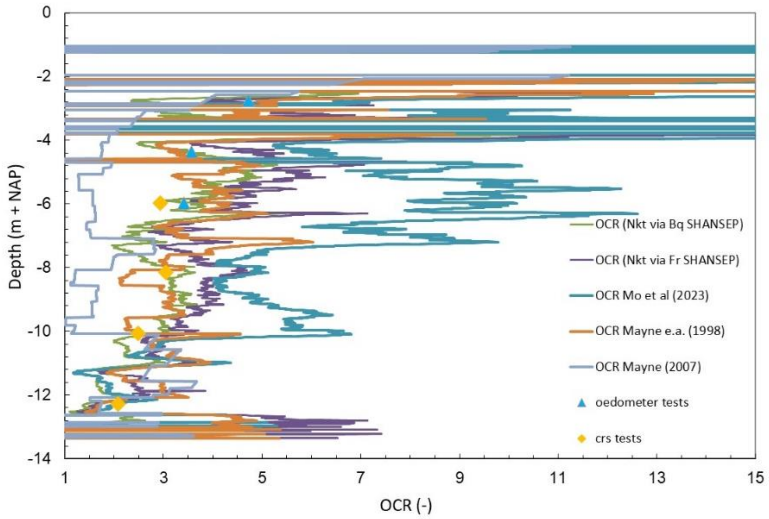


Figure 4-25 Derivation of the overconsolidation ratio (OCR) from SCPTu using different methods based on cone resistance and on shear wave velocity for site Streefkerk (SCPTu 05), The Netherlands, with comparison with overconsolidation ratio from oedometer tests and constant rate of strain tests (crs).

4.3.2 Horizontal stress

To derive the horizontal stress from CPT measurements Kulhawy & Mayne (1990) developed a relationship as presented in Figure 4-26 to derive K_0. Looking at the standard deviation as mentioned in Figure 4-26 and following Schnaid (2009) it can be said that this relationship produces no more than a rough estimate. Robertson & Cabal (2022) state that there is no reliable method to determine K_0 from CPT.

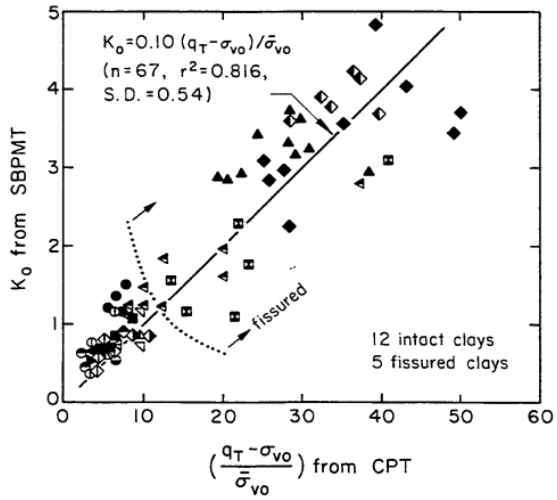


Figure 4-26 Relationship between normalized cone resistance and K_0 as derived from self boring pressure meter data (Kulhawy & Mayne, 1990).

When OCR is derived from CPT measurements, K_0 can be estimated with the Jaky equation and the Kulhawy & Mayne (1990) relationship between OCR and K_0 as discussed in Paragraph 3.5. Using this approach the accuracy of the estimated K_0 is also not high because of the uncertainties in the estimation of the friction angle, the friction angle-K_0 correlation and the derivation of OCR from s_u via N_{kt}.

4.3.3 Sensitivity

The sensitivity (S_t) of clay is defined as the ratio of the undisturbed peak undrained shear strength to the totally remoulded undrained shear strength. Sensitivity is often determined using the FVT. An estimation of S_t can be made using the CPT. Mayne (2007) and Robertson & Cabal (2022) give methods to give a first approximation of the sensitivity based on CPTs.

Mayne (2007) states that if a direct and accurate measure of in-situ sensitivity is necessary, follow-up testing with the vane shear test is prudent. Robertson & Cabal (2022) point out that for relatively sensitive clays (S_t > 10), the value of the sleeve friction can be very low with inherent difficulties in accuracy. In that case they state that the S_t derived from CPTs should be used only as a guide.

4.4 State of silt and sand

The state of sand and silt can be either defined by the relative density or the state parameter. Both parameters can be derived from CPTs. Relative density is the classical parameter to describe the state of sand. However, the accuracy of the relative density is limited as the determination of the maximum and minimum density of sand is problematic and aspects as the compressibility of the soil is not considered in a reasonable way. Furthermore, the relative density is not very well suited to define the state of sand with fines. The relative density can only be determined for drained soil behavior.

Using the state parameter as a measure to define the state of granular material gives the possibility to account for the compressibility of the soil and to assess sand with fines. The framework to derive the state parameter makes it possible to take into account site specific values of the required soil parameters and both drained and partially drained conditions.

Of course, the state parameter approach has also its complexities and limitations. For example, the accuracy of the determination of the critical state line and the fact that for natural sand the critical state line is not a straight line. Another point is that for sand and silt the critical state line and the isotropic compression line are not parallel (Wood, 2007). However, based on experiences in research on liquefaction in Canada (CANLEX-project) Robertson et al. (2000) found that the state parameter approach performed better than the relative density approach.

Several methods are developed to derive the state parameter from CPT data:

- Been & Jefferies (1987) original method based on data from calibration chamber tests on several more or less clean sands and only for drained conditions.
- Been & Jefferies (1992) screening method accounts for partially drained behavior of sands and silts during cone penetration ($B_q \neq 0$). However, Jefferies & Shuttle (2011) put forward that this method is a first approximation and might be argued as speculative.
- Plewes et al. (1992) screening method extends the work of Been et al. (1987) and estimates the state parameter for drained and partially drained soils based on B_q. Soil compressibility is estimated based on friction ratio. It gives only a fist approximation according to Jefferies & Shuttle (2011). Jefferies & Been (2016) mention this method as surprisingly accurate based on experiences across many sites. This method is frequently used for assessments of tailings stability (Monforte et al., 2023; Reid & Fourie, 2023).

- Shuttle & Jefferies (1998) complete framework for drained soil conditions. This framework includes the shear modulus and the plastic hardening modulus of the soil.
- Robertson (2009) screening method with correction of the normalized cone resistance for fines content based on classification index I_c and focus on drained soil conditions.
- Robertson (2022) with modified correction of the normalized cone resistance for fines content based on classification index I_c.
- Monforte, Arroyo & Gens (2023) approach for undrained conditions based on B_q. Based on CPTu simulations using the Particle Finite Element Method.

Jefferies & Been (2016) state that K_0 is very important to determine the state parameter accurately from CPTs. They assume that one is far more likely to estimate K_0 poorly than any of the other parameters involved. Therefore, they recommend measuring K_0 with a SBPMT.

Figure 4-27 gives an example of the determination of the state parameters from a CPT from a site at the river Spui in Oud-Beijerland, The Netherlands. Several methods are applied to derive the state parameter. The various methods give different results. Especially in the thin-layered strata between -16 m and -21 m the derived state parameter values are very different. This is because the capability of the different methods to account for partially drained behaviour. The value -0.05 is commonly applied as a safe boundary (Jefferies & Been, 2016). Based on state parameter the friction angle, undrained shear strength and cyclic resistance ratio CRR can be determined (Jefferies & Been, 2016).

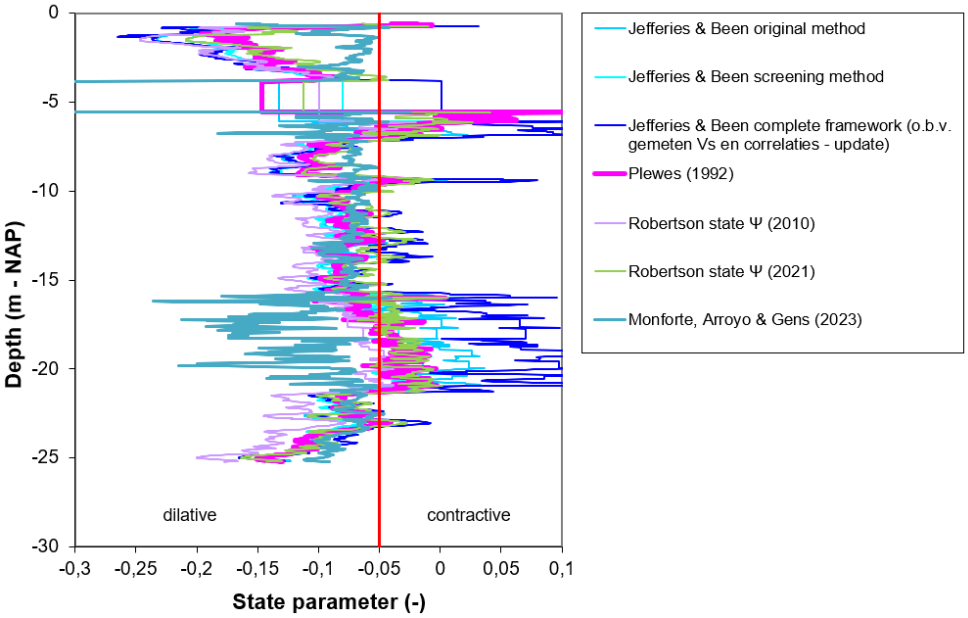


Figure 4-27 Determination of state parameter with various methods for Spui, Oud-Beijerland, The Netherlands. The methods give different outcomes especially in the thin-layered strata between -16 m and -21 m. The value -0.05 is commonly applied as a safe boundary (Jefferies & Been, 2016).

4.5 Stress state intermediate soils

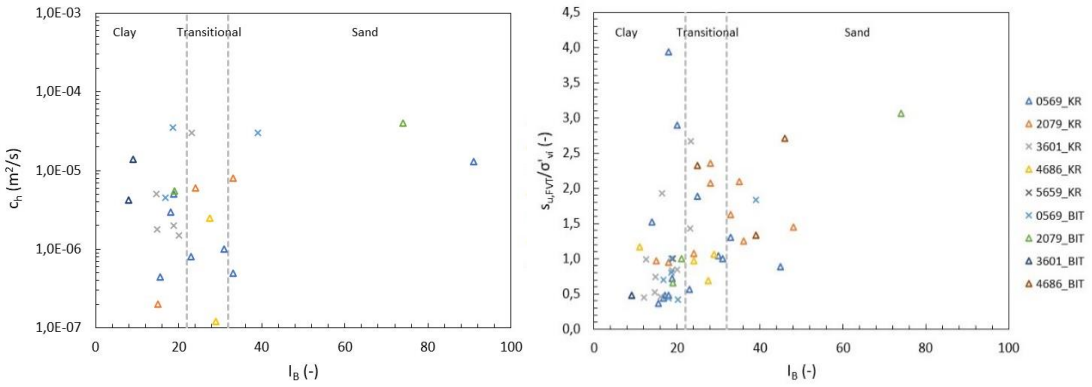
For mixed soil types such as clayey silt and sandy clay the interpretation of CPTs is not straightforward. The drainage conditions during CPT penetration may be partially drained. Methods to interpret CPTs are mostly developed for fine grained low permeable soils with undrained behaviour or for granular permeable soils with drained behaviour.

Another topic of mixed soil types is the specific behaviour in compression and during shearing. For these soils it is found that the initial void ratio continues to affect its compression behaviour and shearing behaviour to large strains, to the extent that unique normal compression lines and critical state lines cannot be found by means of conventional laboratory testing (Coop, 2015). This so called “transitional” behaviour, in which the initial soil density plays a major role, seems to result from robust soil fabrics. It is a limitation of the critical state soil mechanics framework that the compression behaviour and shearing behaviour of these materials cannot be described correctly. It is found that critical state lines can be defined for each initial void ratio (Shipton & Coop, 2015; Ponzoni et al., 2014). In triaxial compression tests mostly dilative behaviour is found, which cannot be related to OCR. Determination of OCR is not obvious as in oedometer tests continuous curved compression lines in void ratio versus stress space will be found. Therefore, the SHANSEP-model is also not capable to describe the behaviour of transitional soils. In a pragmatic way the SHANSEP-model can be used to fit the test data with the assumption of an apparent OCR or POP.

Soil classification of these transitional soils can be done with the Robertson I_c and I_B parameters. This soil classification is important to distinguish between sand-like and clay-like behaviour. When I_B is between 22 and 32 the soil behaviour is transitional.

When these intermediate soils are encountered at a site extensive in-situ testing and laboratory testing is required to get a good understanding of the soil behaviour. For the in-situ tests especially the drainage conditions need attention as these materials may behave partially drained during testing (Schnaid, 2009; DeJong et al., 2012). This behaviour may require cone penetration tests and field vane tests with faster penetration rates and faster rotation rates respectively.

Figure 4-28 shows an example of dissipation test results and field vane tests results for tidal deposits in Friesland, The Netherlands. The consolidation coefficient and the undrained shear strength ratio are plotted against I_B. The areas between the dashed lines are the areas of the transitional soils. At least for the consolidation coefficient it is clear that it makes sense to apply the I_B as a guidance for the soil behaviour. For the sandy soils the coefficient of consolidation is higher than for the transitional soils and clayey soils. For the undrained shear strength ratio, the pattern is not that clear. The undrained shear strength ratio of the transitional soils is relatively high, may be caused by the dilative behaviour of these soils. Here the drainage conditions may also play a role as the field vane tests in this project are performed using the common rotation rate of 6 degrees per minute which may be to slow in sand and transitional soils. The results of the field vane tests are only applied for the clay-like material in this project.



(a) (b)

Figure 4-28 Relationship between horizontal consolidation coefficient derived with dissipation tests and classification index IB (a). Relationship between undrained shear strength ratio derived with field vane tests and classification index IB (b). IJsselmeerdijk, Friesland, The Netherlands. The areas between the dashed lines are the areas of the transitional soils.

4.6 Partially saturated soils

When performing CPTs in the partially saturated zone, the cone resistance and sleeve friction are influenced by the capillary suction in the soil. Because the capillary suction and degree of saturation are not measured during CPT penetration the interpretation of the CPT data is difficult.

Deltares (2023) is working on a study of the shear strength of (initially) unsaturated soil in relation to the assessment and design of the slope stability of flood defences. For this research in-situ testing and monitoring at the IJsseldijk near Westervoort and the Maasdijk near Oijen in The Netherlands and laboratory testing take place.

In this research it is found that a substantial part of the soil above the normal daily phreatic surface can be permanently saturated (grey clay). On top of this permanently saturated zone lies a zone where degree of saturation and suction varies considerably (brown oxidized clay). From the shear strength measurements, it can be determined that the soil behaves undrained in the permanently saturated zone (grey clay). In the zone where degree of saturation and capillary suction fluctuates (brown clay) the soil behaviour depends on degree of saturation. In this zone the drained shear strength depends on capillary suction and degree of saturation and therefore also varies considerably. The soil behaves undrained above a certain threshold value in degree of saturation ($S_r \sim 0.85$).

To estimate the water content, degree of saturation and capillary suction to be able to interpret the CPT measurements in the partially saturated soils, the permittivity or dielectric constant is additionally measured with the CPT device. The development of the approach is ongoing work at Deltares.

Figure 4-29 shows the fluctuation of the shear strength at the site at the Maasdijk in different seasons.

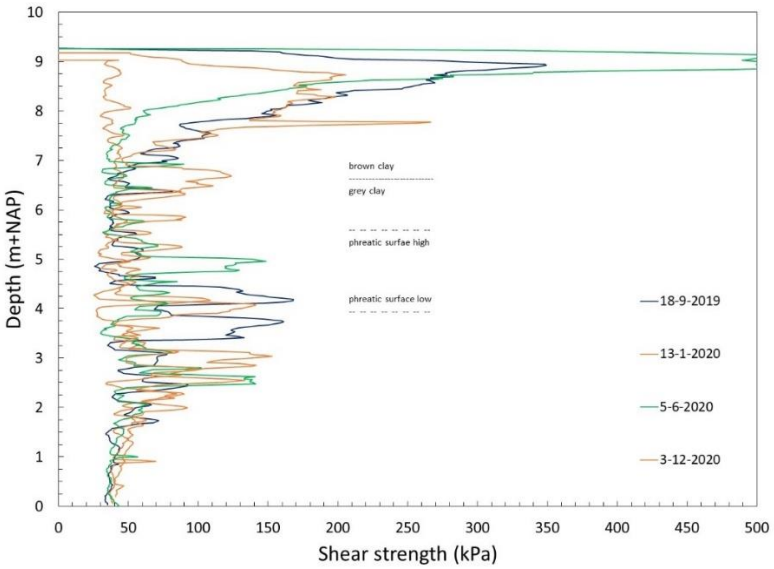


Figure 4-29 Shear strength derived from CPTs using N_{kt} = 15 at different seasons at Oijen, The Netherlands. The shear strength in the upper two meter varies throughout the seasons because of variations in capillary suction and degree of saturation.

5 Laboratory measurements to derive the stress state

Obviously, laboratory tests also play an important role to derive the stress state. For example:

- To estimate the in-situ vertical effective stress accurately determination of the bulk unit weight in the laboratory is important. This is the case as the vertical effective stress is the bases for many other estimations. Note that the pore water pressure also has to be known accurately to determine the vertical effective stress. Determination of the bulk unit weight in the lab is also relevant as the estimation of the bulk unit weight based on cone penetration tests gives not always reliable results; as discussed in Paragraph 4.2.2. So, cross checks are relevant here. Other classification tests or index tests are relevant to learn what the basic characteristics of the subsoil are, such as water content, Atterberg limits, organic content. Accurate information on these basis characteristics of the subsoil is relevant to determine the soil type and to estimate the vertical effective stress.
- To get insight in the yield stress and overconsolidation ratio incremental load tests or constant rate of strain tests can be used. Note that high quality soil samples are required to get reliable results of the yield stress and overconsolidation. Knowledge of the overconsolidation ratio can be used to estimate the earth pressure at rest coefficient $K_{0,oc}$. Here the Kulhawy & Mayne (1990) relationship can be applied, as discussed in Paragraph 3.5.
- An alternative approach to deduce the yield stress and overconsolidation ratio is to perform both overconsolidated and normally consolidated triaxial tests and/or direct simple shear tests. The difference between the overconsolidated strength and the normally consolidated strength can be attributed to the overconsolidation ratio. To get a reliable result of the in-situ undrained shear strength an overconsolidated test should be performed with applying a vertical consolidation stress equal to the best estimate of the vertical effective stress at the point where the soil sample is retrieved from the subsoil. This consolidation procedure is the so-called recompression technique (Ladd et al., 2004). Here it has also to be noted that high quality soil samples are required to get reliable results.
- The earth pressure at rest coefficient for normally consolidated conditions $K_{0,nc}$ can be measured with constant rate of strain devices, which have a steel ring with a thin diaphragm with a strain gauge all around the steel ring (K_0-CRS test). When applying thin diaphragms with strain gauges in different directions $K_{0,nc}$ can be measured in different directions, as shown by Gareau et al. (2004); see also Paragraph 3.5. This can be relevant in glacial deposits where the horizontal stress can be different in different directions.

The advantage of applying overconsolidated triaxial tests and direct simple shear tests is that they also can be used to correlate the measured undrained shear strength in the laboratory with the measured strength in the field with cone penetration tests. In this way a site specific or project specific empirical cone factor N_{kt} can be derived. Based on such a site specific or project specific correlation cone penetration tests can be used to estimate the in-situ stress state with more confidence.

6 Spatial variability

6.1 Introduction

To be able to determine the stress state accurately, a sufficiently dense site investigation is required. The thickness of the track bed and the railway embankment can vary considerably due to variations in the former surface level or due to settlement of the soft soils below the railway embankment. The stratigraphy of the subsoil can also vary considerably due to variations in the geological environment. These variations can be variations in layer thickness but can also be geological features intersecting the deposit, e.g. sandy channels within a soft clay deposit. Within a certain soil layer, the soil properties can vary considerably. All of these variations can occur on relatively short distances. When conducting a site investigation with too much distance between the boreholes and other field tests variations in stratigraphy can be easily missed.

When analyzing the results of a site investigation special attention is required for thin weak layers. Thin weak layers can determine the potential failure modes of a railway embankment. Thin weak layers can consist of for example:

- Loose sand which can liquefy.
- Soft non-ripened (low OCR) clay which may cause squeezing.
- Thin gyttja layer at the bottom of a peat layer which can cause slope instability.

These thin layers can be missed in site investigations due to the spatial variability of the subsoil in combination which the available number of boreholes and in-situ tests. Such thin layers are not always recognized in boreholes and cone penetration tests. Note that thin layers less than 0.2 m thick are difficult to recognize in CPTs because of the averaging effect of the deforming soil around the cone tip.

From The Netherlands two examples can be mentioned, where a weak layer dominated the failure mode:

- IJkdijk Booneschans, where an Allerød soil horizon occurs just below the top of the Pleistocene sand. The horizon is covered by less than 0.2 m loose sand. This soil horizon hampers vertical drainage in the sand. Peat was on top of the Pleistocene sand. Most of the lateral deformation of the initiated slope failure was found at the contact of the peat and underlying sand layers. The top of the sand layer was loose and saturated, and appeared liquified. A thin shear zone, or shear plane, involving both sand and peat, had apparently developed upon failure (Zwanenburg et al., 2012).
- Eemdijk, where the failure occurred over approximately the middle 30 m of the 60 m long test embankment and both ends of the embankment remained stable under the same loading. This clearly indicates a 3D mechanism. The slope failure is initiated by a weak layer just above the Basal peat layer. This soft clay layer with organic matter and thickness of 0.5 m is only present in the middle part of the 60 m long test embankment over a length of about 30 m at the toe of the embankment. The inclinometer shows that the deformations in the Basel peat layer are low compared to the deformations in the weak clay layer. In this case peat has not the lowest strength of the present layers (Lengkeek, 2022).

6.2 Variability in stratigraphy

The variability of the stratigraphy at relatively short distances can be substantial. At three sites in the Alblasserwaard, The Netherlands, rows of CPTs with an interval of 7 meters over a stretch of 100 m are conducted (Deltares, 2019).

Analysis of the CPT results show a continuous variation in soil stratigraphy and cone penetration resistance. Layer boundaries of soil layers can vary significant at the length scale of a potential slip plane (order of 30 – 100 m). The top of the deep sand layer varies 1 meter over a length of 100 meter and the thickness of the peat layer varies several meters over a length of 100 meter.

In accordance with common geotechnical practice in infrastructural projects, safety assessments and design of flood defences where CPT's are carried out with intervals of 100 m or more, it has to be concluded that one CPT is only a random sample of the stratigraphy and cone penetration resistance. More dense sampling is needed to capture all relevant geological features and spatial variability of layers. The significance of one CPT to describe the stratigraphy and cone penetration resistance for a stretch of 100 m or more is very low. Weak layers can be easily missed. A second CPT close to the first one or at somewhat larger distance can give a very different result. The stratigraphy may be different, and the cone penetration resistance can be substantially higher or lower. Related to the failure mechanism of slope instability where the size of a potential slip surface is in the order of 30 to 100 m, one CPT is not enough.

6.3 Variability of soil parameters within a soil layer

The intrinsic soil properties and state variables within a soil layer vary considerably. Apart from a trend, the random part of a continuous spatial variability is usually quantified explicitly by a correlation length or scale of fluctuation (for 'random field' analysis), or implicitly by the variance ratio between local and regional variability.

Previous research (De Gast, Vardon, & Hicks, 2019) (Deltares, 2019) suggests that the observed variability of CPT resistances in soil deposits cannot be described by a single (horizontal) scale of fluctuation and that the ratio of local and regional variance may not be a constant for a geological deposit. De Gast et al. report for peat a horizontal scale of fluctuation between 2 and 9 m parallel to a dyke and 4 to 5 m perpendicular to a dyke. A horizontal scale of fluctuation for peat parallel to a dyke between 3 and 33 m has been reported in the Deltares study, whereas De Gast et al. report a horizontal scale of fluctuation of 3 to 20 m parallel to a dyke and 4 to 11 m perpendicular to a dyke. For clay, the Deltares study reports a horizontal scale of fluctuation parallel to a dyke between 9 and 30 m.

The latter results are based on three locations in the same depositional environment in the Alblasserwaard in The Netherlands (see Figure 6-1). In the publication of De Gast et al. is not explicitly reported if the analyses are based on measured cone resistance q_c, cone resistance with correction for measured pore water pressure q_t or net cone resistance $q$net but it can be assumed that q_t has been used. The derivation of the horizontal scale of fluctuation in the Deltares study is based on q_c. An analysis on the normalised cone resistance Q_t has additionally been performed but resulted in much more scatter and in unrealistic short scales of fluctuation. It is assumed that the short scale of fluctuation for Q_t relative to q_c is caused by additional uncertainties from corrections for measured pore water pressure, total stress and effective stress.

An overview of different scales of fluctuation in CPT data from literature is presented by De Gast (De Gast, 2020). This overview shows scales of fluctuation for sand and stiff clays for which the horizontal scale of fluctuation mostly ranges between 0.1 and 66 m. Per material the range in the horizontal scale of fluctuation is often also large. For a silty clay a horizontal scale of fluctuation of 5 to 12 m is mentioned, which agrees with the findings of (De Gast, Vardon, & Hicks, 2018) and (Deltares, 2019).

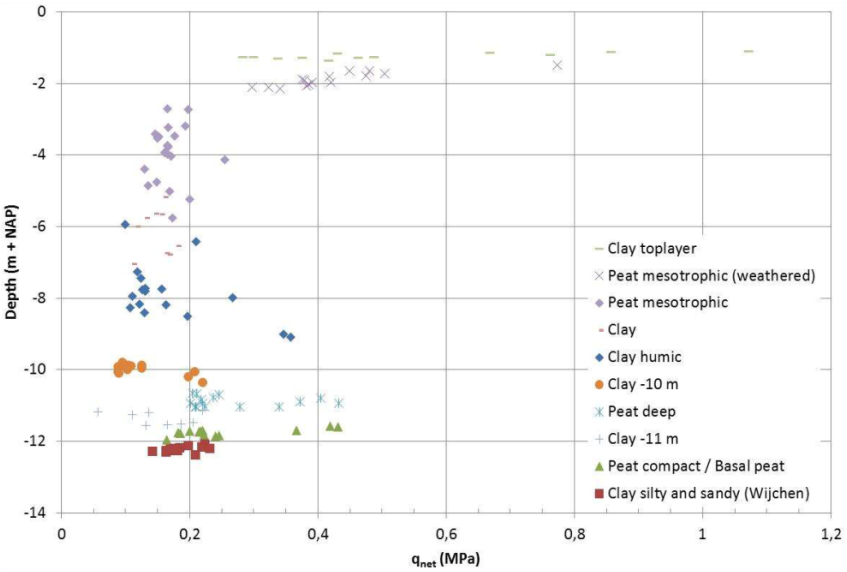


Figure 6-1 Variability of net cone resistance in 15 CPTs along a row of 100 m in the Alblasserwaard, The Netherlands (Deltares, 2019). The symbols are the mean values of the net cone resistance per CPT and per soil layer and the symbols are plotted in the middle of the soil layers.

6.4 Influence of spatial variability on stability analysis

The spatial variation of a soil variable within a deposit can consist of a trend and a random part. A trend within a deposit is possible for all soil variables, especially those that are known to depend on the local stress state and overconsolidation. For example: the undrained strength and the cone resistance.

Regarding the random part, the following two alternatives are regularly used to quantify the associated uncertainty in a stability analysis.

- In the 'random average' approach, the spatial variability of the averaged value of the random part along a slip plane is used for implicit modelling of the uncertainty in a semi-probabilistic or probabilistic analysis (Calle, 2008). The total variance of random point values σ_{tot}^2 is assumed to be a sum of the variance of the point value within the slip plane σ_{loc}^2 and the variance of the averaged value $\sigma_{loc,mean}^2$. Based on experience, most of the relevant slip planes have a width ranging from 30 to 100 meters.
- In the 'random field' approach, the random part of the spatial variability of point values is modelled explicitly in a probabilistic stability analysis. This spatial variability can be determined from measurements with "subsurface mapping" (Ching & Phoon, 2019). Mapping is based on the autocorrelation between the considered soil variable at different points, where the variables at different points become more correlated at closer distance. The autocorrelation function is parameterised by the autocorrelation length (in this case referred as scale of fluctuation, θ).

6.5 Statistical analysis with CPT data

As discussed before the significance of one CPT to describe the stratigraphy and cone penetration resistance for a stretch of 100 m or more is very low. Therefore, in the Dutch guideline for the assessment of slope stability of flood embankments (dyke) a procedure is available to derive the mean values and standard deviations of the cone resistance, undrained shear strength and overconsolidation ratio of a soil layer within a dyke section (RWS, 2019). The procedure is based on the use of the effective stress normalized cone resistance Q and the random average approach as mentioned in Paragraph 6.4**Error! R eference source not found.**.

7 Interpretations for examples of sites

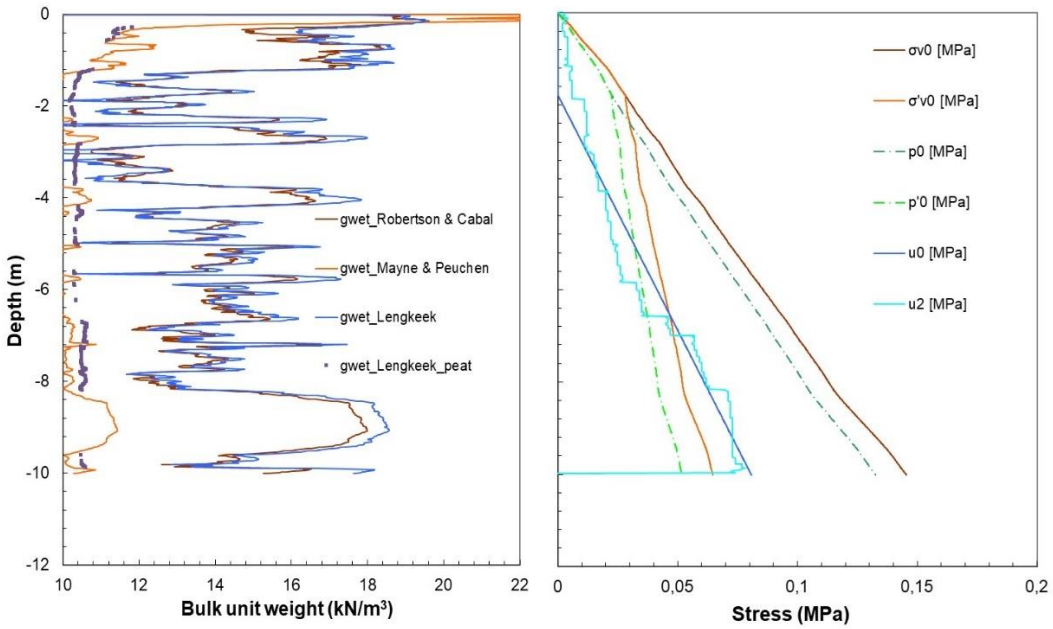
7.1 Scope

In this chapter an exploratory analysis of the provided data of the four examples of sites is conducted in order to show the outcomes for the various methods discussed in this report. This analysis is based on the findings of the literature review (see Chapter 4) and Deltares experience, focussed on the application of CPTs. For each of the examples one or two CPTs are analysed in order to derive (parts of) the stress state of the railway embankment and the subsoil. The other available geotechnical data such as boreholes and index tests are used to be able to interpret the CPTs. The results are aimed at showing the impact, possibilities and limitations of the various methods. The results are not intended to be used for formal designs or assessments.

7.2 Example 1: railway embankment on a floodplain

In this example the subsoil is formed by a river plain, which is made up of fine-grained soils and palustrine and/or peaty deposits with the presence of organic matter. Therefore, in this example the focus is on the derivation of OCR. Based on OCR also K_0 and the horizontal stress can be estimated.

For this example, CPTu7 is chosen because of the thick clayey deposits with low cone resistances. The net area ratio of the cone is 0.85. The level of the phreatic surface and hydrostatic pore water pressure are chosen as discussed in Paragraph 2.1. Four different methods are applied to derive the bulk unit weight. Figure 7-1 gives the estimated bulk unit weight and stresses for CPTu7 at example 1.



(a) (b)

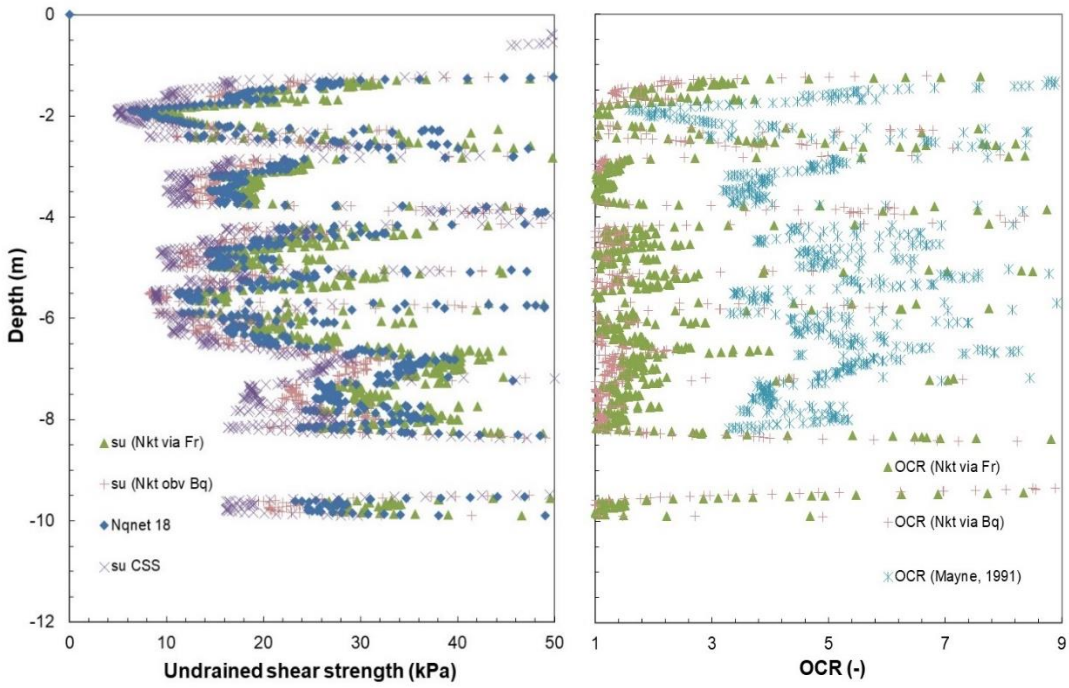
Figure 7-1 Estimated bulk unit weight (a) and stresses (b) for CPTu7 at example 1.

Figure 7-1a shows soft soil layers with relatively low bulk unit weight. The Lengkeek et al. (2022) method to estimate the bulk unit weight of peat gives much lower bulk unit weight for peat than the Robertson et al. (2022) method and the general Lengkeek et al. (2022) method. This difference between these methods has an influence on the effective stress of about 10% at level -10 m below ground level.

Figure 7-1b shows a remarkable pattern of the measured pore water pressure u_2. These measurements show a slow response and no peaks in the pore water pressure as expected in low permeable soft layers. Possibly loss of saturation of the measurement system has occurred during penetration, probably in the top layer with partially saturated soils.

The pore water pressure measurements give no clear reference of u_2 in a sandy layer (aquifer). This makes it difficult to accurately model the stationary pore water pressure gradient (u_0). These issues of the measured pore water pressure have an influence on B_q, but also on the derived effective stress and overconsolidation ratio. As the bulk unit weight also has its uncertainty as mentioned before the effective stress and overconsolidation ratio are not accurate.

Figure 7-2a gives the derived undrained shear strength s_u and overconsolidation ratio OCR for CPTu7 at example 1. s_u is derived via the empirical cone factor N_{kt} which is related to F_r according to Robertson (2012) and related to B_q following Mayne & Peuchen (2018). These methods give the peak undrained shear strength s_u for direct simple shear and triaxial compression modus respectively. Using the SHANSEP model and the theoretical relation from Mayne (1991) OCR is determined; see Figure 7-2b.



(a) (b)

Figure 7-2 Derived undrained shear strength (a) and overconsolidation ratio (OCR) (b) for CPTu7 at example 1. The undrained shear strength is derived via N_{kt} based on F_r and B_q which give peak values.

B_q is influenced by the choice of the stationary pore water pressure gradient (u_0) as discussed before. This also influences N_{kt}, s_u and OCR.

However, this uncertainty gives not much difference in N_{kt} based on F_r and B_q. The theoretical relation from Mayne (1991) seem to overestimate OCR; see also Figure 4-20. The derived OCR is very low for the whole soil profile, both for the approaches via F_r and B_q.

7.3 Example 2: railway line on a slope

The upper part of the deposits in this example consists of younger clayey silt sediments. The depositional history of the sediment is locally very complex because influences from the side valleys/slopes are also present within the silty deposits. Locally outcrops of the deep sand deposits are present at the surface. The young silty and sandy sediments are assessed as potentially liquifiable.

For this example, two CPTs are analysed:

- SCPTu13 with silty sandy material for which the state parameter is determined to assess the liquifiability.
- SCPTu17 with a thick layer of clayey material for which the overconsolidation ratio OCR is determined.

For SCPTu13 the phreatic surface is set at 1.5 m below surface level following the geotechnical report; see Paragraph 2.2. Based on the u_2 measurements it seems to be possible to set the phreatic surface level at 4 m below surface level.

The gradient of the stationary pore water pressure u_0 with depth is fitted to the measured u_2. The pore water pressure increase with depth seems to be influenced by the pore water pressure in the aquifer at the location of SCPTu 13. The pore water pressure increase with depth is much lower than hydrostatic (underdrainage condition).

The applied K_0 is set at 0.7 which is a reasonable first estimate (Jefferies & Been, 2016).

Figure 7-3 gives the estimated stresses for SCPTu13 at example 2. The response of u_2 seems to be slow and straggling.

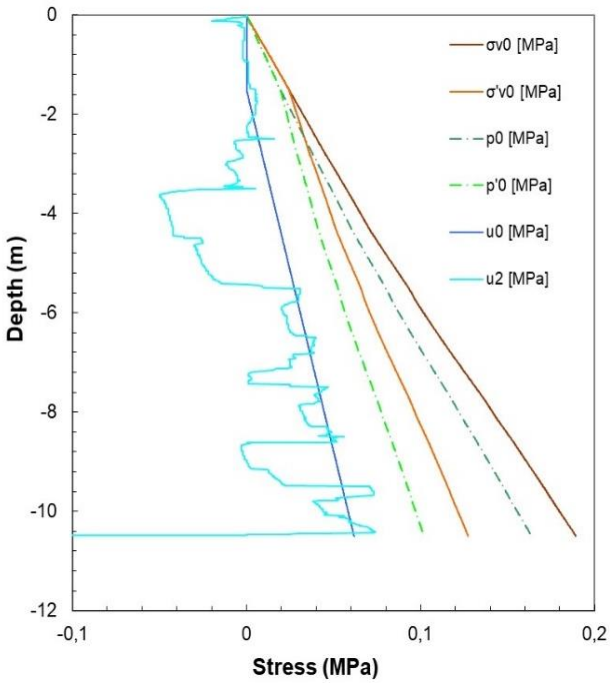


Figure 7-3 Estimated stresses for SCPTu13 at example 2.

Figure 7-4 shows the derived soil classification index I_c (a) and state parameter (b) for SCPTu13 at example 2. Figure 7-4b shows a remarkable difference in derived state parameter values for the various applied methods. At least at the upper 4 m, but also around -7 m. The material seems to be not liquifiable when the Plewes et al. (1992) and Monforte et al. (2023) methods are followed. In the upper 4 m silty material is present based on I_c and the measured pore water pressure u_2 suggest partially drained behaviour. Therefore, it is likely that the Plewes and Monforte et al. approaches, which account for partially drained behaviour, give a realistic estimation of the state parameter. Using these methods the state parameter is mostly smaller than -0.05, which is commonly applied as a safe boundary (Jefferies & Been, 2016) to conclude that the soil behaves dilative. However, some thin soil layers or soil pockets at several depth have state parameter values larger than 0.0, which indicate contractive soil behaviour and liquefaction possible.

Scaling of the derived state parameter values for the higher stress level below the embankment compared to the adjacent field next to the embankment where the CPTs are performed can only be done when the critical state line and normal compression line in void ratio – stress space are known. For sand and silt the critical state line and normal compression lines are not parallel lines in the void ratio – stress space (Jefferies & Been, 2016). Assuming that the soil below and adjacent to the embankment belong to the same normal compression line and critical state line the state parameter moves towards higher values (stronger tendency for contractive behaviour) when the effective stress below the embankment is higher.

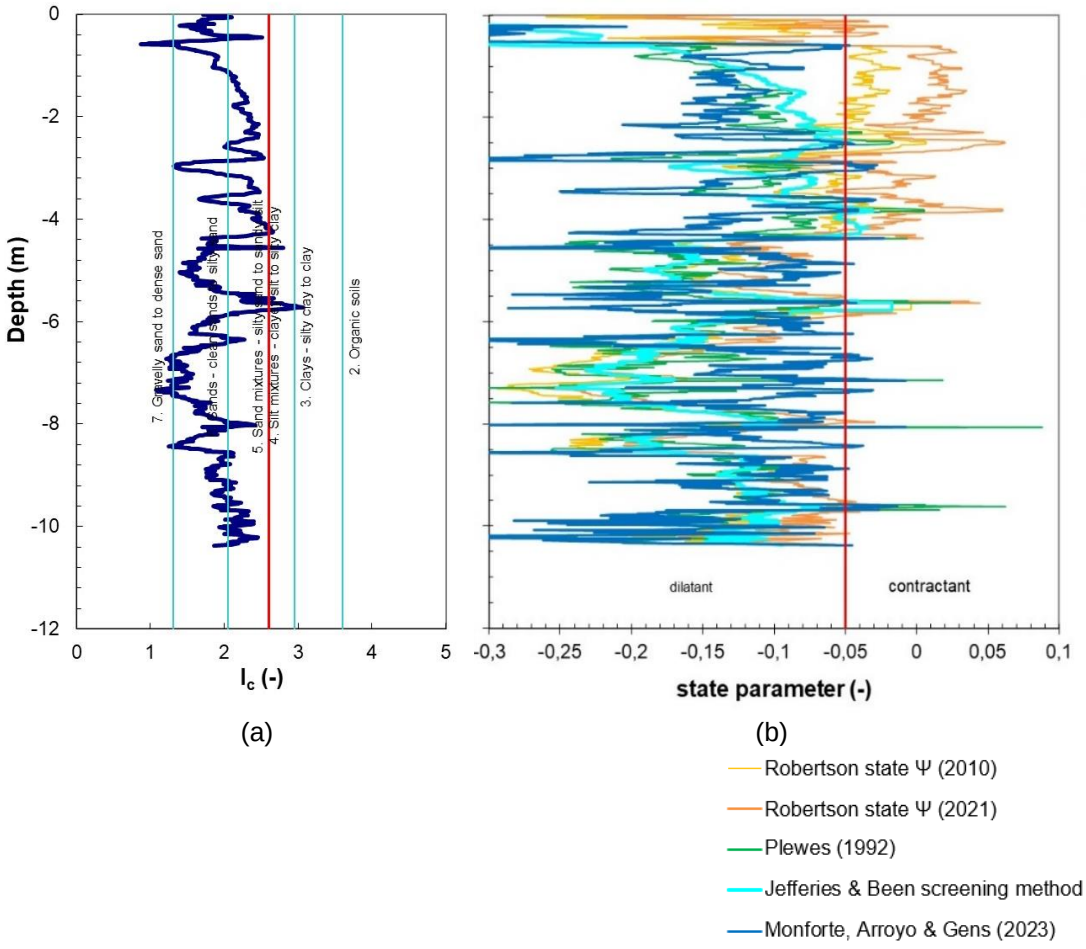


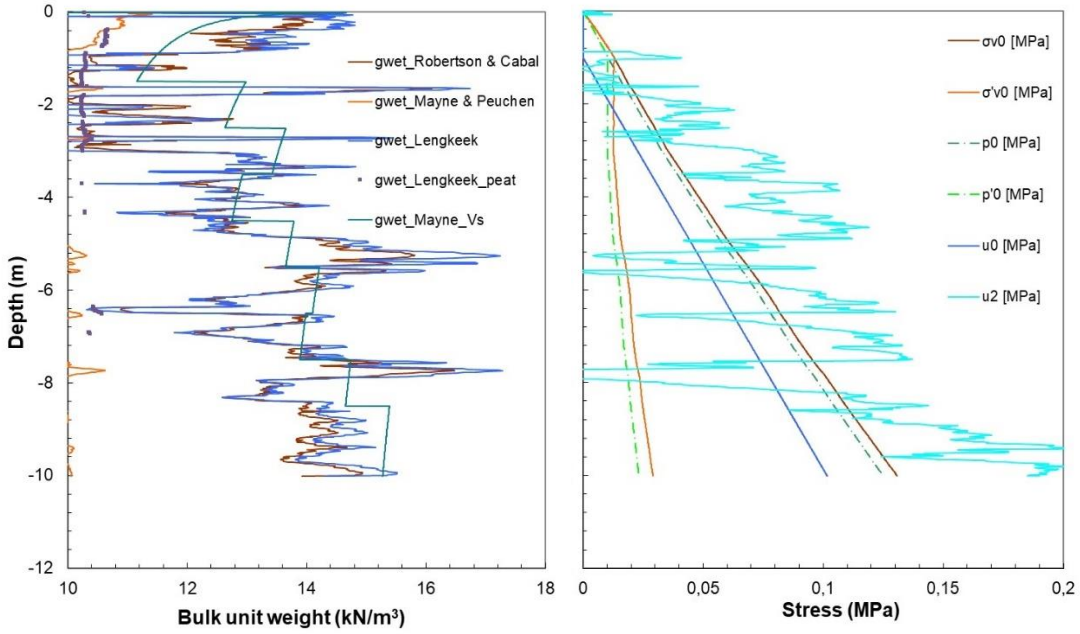
Figure 7-4 Derived soil classification index I_c (a) and state parameter (b) for SCPTu13 at example 2. The value -0.05 is commonly applied as a safe boundary (Jefferies & Been, 2016).

SCPTu17 with a thick layer of clayey material

For SCPTu17 the phreatic surface is set at 1.0 m below the surface level according to the u_2 measurements. The geotechnical report also suggests a phreatic surface close to the surface. At the location of SCPTu17 the geohydrologic situation is unclear according to the geological profile in the geotechnical report. Only an aquitard is given. The stationary pore water pressure u_0 is fitted along the measured pore water pressure u_2. However, u_0 is difficult to determine based on u_2. The fit results in a pore water pressure gradient more than hydrostatic (artesian groundwater condition). The u_2 measurements with many peaks give a reliable pattern at this site.

Figure 7-5 gives the estimated bulk unit weight (a) and stresses (b) for SCPTu17 at example 2. At this site a good agreement is found between the Robertson et al. (2022), Lengkeek et al. (2022) and Mayne (2007) approach. The last one is based on the shear wave velocity V_s.

Figure 7-6a gives the derived undrained shear strength for SCPTu17 at example 2. s_u is derived via N_{kt} based on F_r and B_q, which give peak values of s_u for the direct simple shear and triaxial compression mode respectively. s_u is also derived via the shear wave velocity V_s following L'Heureux and Long (2017). Using this method also values of s_u for the direct simple shear and triaxial compression mode can be derived. Looking at the different results these s_u values based on V_s should be large strain values (ultimate state).



(a) (b)

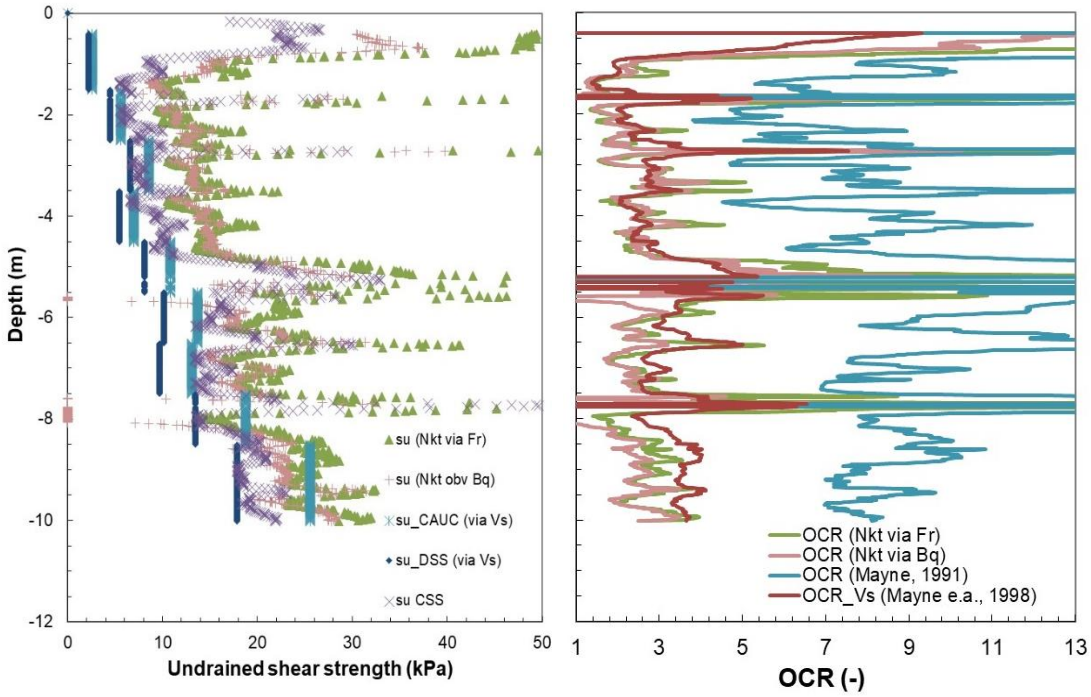
Figure 7-5 Estimated bulk unit weight (a) and stresses (b) for SCPTu17 at example 2. The stationary pore water pressure u_0 is fitted along the measured pore water pressure u_2.

Figure 7-6b shows the derived OCR. The OCR values are determined in different ways:

- Via s_u which is derived from the cone resistance using N_{kt} based on F_r and using the SHANSEP model.
- Via s_u which is derived from the cone resistance using N_{kt} based on B_q and using the SHANSEP model.
- Directly from the cone resistance using the theoretical relation from Mayne (1991).
- Directly from the cone resistance and the shear wave velocity V_s using the relation from Mayne et al. (1998).

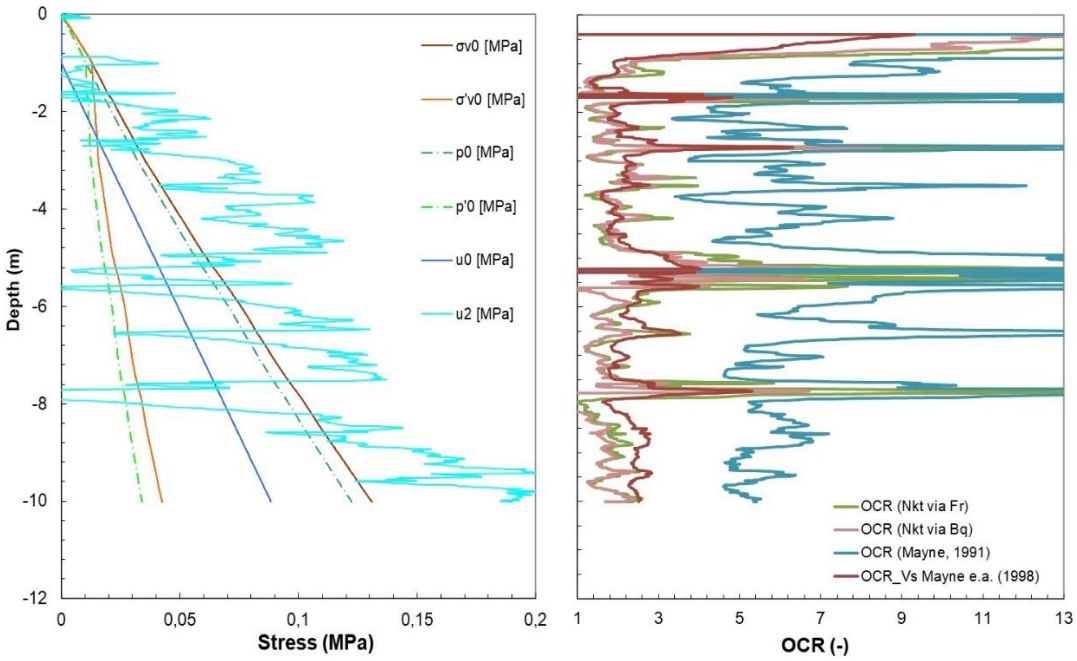
The Mayne (1991) relation again seems to overestimate OCR as this method estimates much higher values than the other methods. The Mayne et al. (1998) relation based on cone resistance and shear wave velocity V_s gives for this case a good comparison with OCR derived via N_{kt}, s_u and the SHANSEP model.

Because of the uncertainty on the stationary pore water pressure u_0 a check is done to see how sensitive the analysis is for the estimation of u_0. Figure 7-7 shows the derived stresses (a) and overconsolidation ratio (OCR) (b) for SCPTu17 at example 2, assuming hydrostatic pore water pressure u_0 instead of the assumed artesian groundwater condition in Figure 7-5. The hydrostatic pore water pressure gradient results in a higher effective stress and a lower OCR: around 2 instead of around 3 in Figure 7-6.



(a) (b)

Figure 7-6 Derived undrained shear strength (a) and overconsolidation ratio (OCR) (b) for SCPTu17 at example 2. The undrained shear strength is derived via N_{kt} based on F_r and B_q which give peak values; the others are ultimate values.



(a) (b)

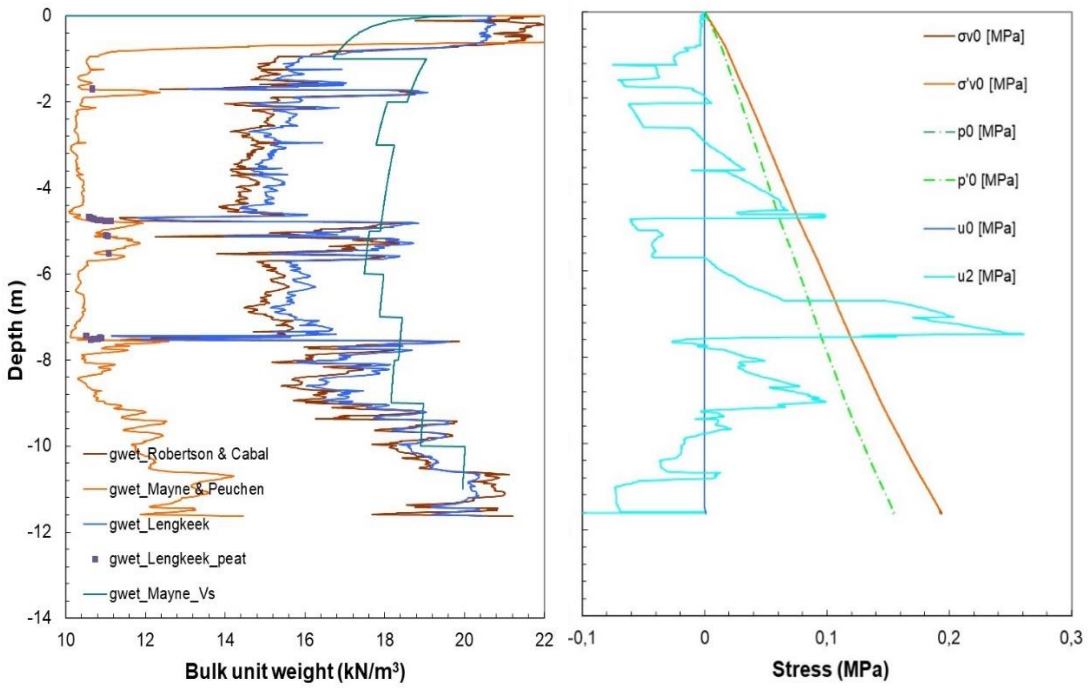
Figure 7-7 Derived stresses (a) and overconsolidation ratio (OCR) (b) for SCPTu17 at example 2, assuming hydrostatic pore water pressure u_0.

7.4 Example 3: railway line on an embankment

In this example fine-grained layers with high organic content were formed on the surface. Some artificial fillings containing foreign substances were encountered. Below this are very heterogeneously composed silty sediments, up to several meters thick, consisting of clayey to fine sandy silts with a variable proportion of organic material, interspersed with peaty material and lake chalk-containing lenses and intermediate layers. Lake chalk is sensitive soil. In this example the focus is on the derivation of OCR and sensitivity S_t. Based on OCR also K_0 can be estimated.

For this case SCPTu 4 is analysed. The starting point of this CPT is at 443.84 m above sea level. The phreatic surface is around 435 +/- 2 m above sea level (see Paragraph 2.3). So, the first 9 m of the CPT belongs to partially saturated soil. The measured pore water pressure u_2 gives irregular stammering and slow response (no rapid peaks). The pore water pressure measurement is therefore difficult to interpret. Looking at the u_2 response the phreatic surface may also be somewhat deeper. It is assumed that the soil is partially saturated (S_r above 0.8) as at some levels positive u_2 pressures are measured.

Figure 7-8 shows the estimated bulk unit weight (a) and stresses (b) for SCPTu4 at example 3. At this site again a good agreement is found between the Robertson et al. (2022) and Lengkeek et al. (2022) methods. The results from the Mayne (2007) approach which is based on the shear wave velocity V_s deviates from the other results at this site.



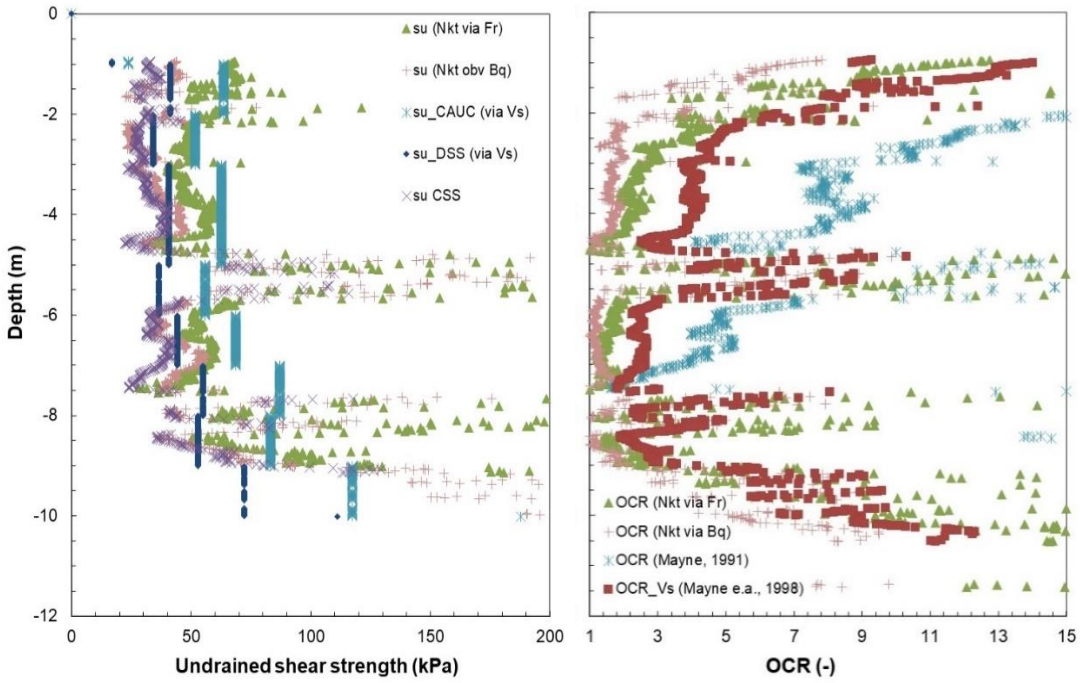
(a) (b)

Figure 7-8 Estimated bulk unit weight (a) and stresses (b) for SCPTu4 at example 3

Figure 7-9a gives the deduced undrained shear strength for SCPTu4 at example 3. s_u is derived via N_{kt} based on F_r and B_q. These methods give peak values of s_u for the direct simple shear and triaxial compression mode respectively. s_u is also derived via the shear wave velocity V_s following L'Heureux and Long (2017). Using this method also values of s_u for the direct simple shear and triaxial compression mode can be derived. In this case s_u based on V_s matches relatively well with s_u based on the cone resistance.

Figure 7-9b shows the calculated OCR values. The calculated OCR are based on the same methods as used for example 2. The Mayne (1991) relation again seems to overestimate OCR. The Mayne et al. (1998) relation which is based on the cone resistance and the shear wave velocity V_s gives for this case also higher OCR then OCR derived via N_{kt}, s_u and the SHANSEP model.

In Figure 7-10 the derived sensitivity S_t for this case is presented. The sensitive layers (lake chalk) are between -7 m and –9 m. S_t depends somewhat on the approach. When S_t is estimated based on the peak value of s_u derived from the cone resistance with N_{kt} based on B_q then S_t is somewhat lower than S_t based on the approximation of Robertson & Cabal (2022) and S_t derived from the cone resistance with N_{kt} based on F_r.



(a) (b)

Figure 7-9 Derived undrained shear strength s_u (a) and overconsolidation ratio (OCR) (b) for SCPTu4 at example 3. The undrained shear strength derived via N_{kt} based on F_r and B_q are peak values; the others are ultimate values.

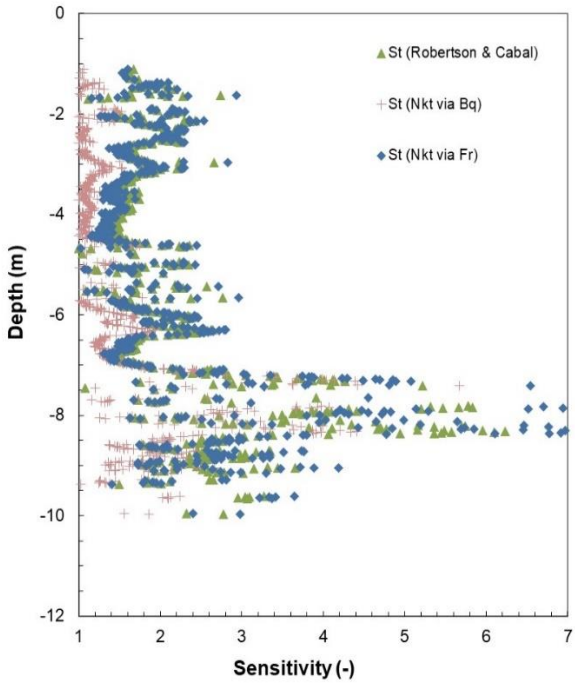


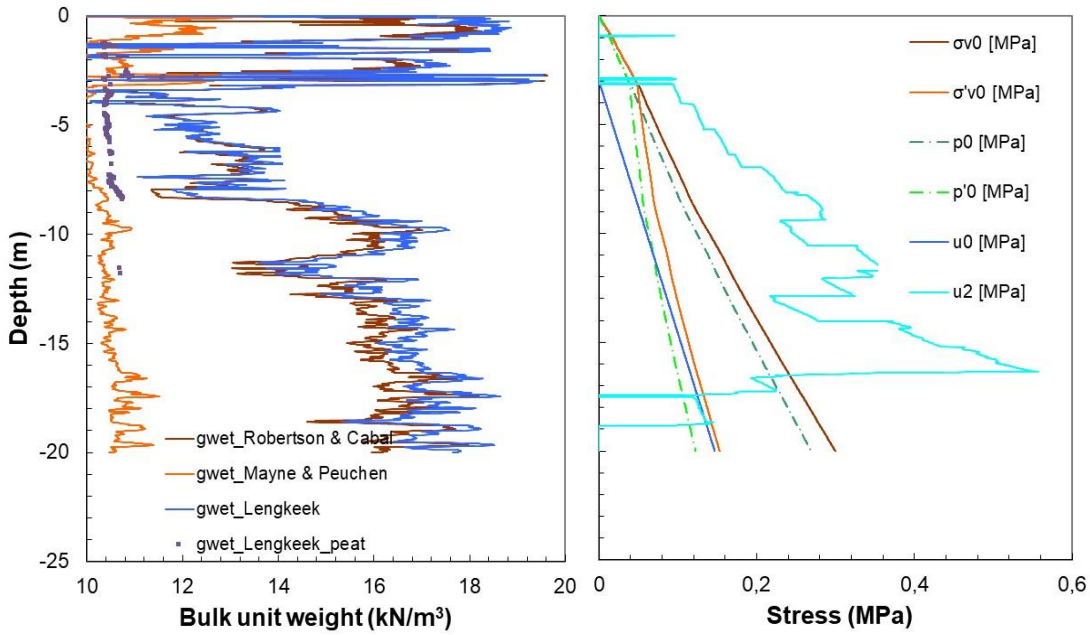
Figure 7-10 Derived sensitivity for SCPTu4 at example 3.

7.5 Example 4: railway line on soft spots with challenging soils

The peaty deposits at this site are the deepest peat bogs of Switzerland: 15 meters of peat and organic silt. A lot of settlement occurs at this site. Therefore, in this example the focus is on the derivation of OCR. Based on OCR also K_0 can be estimated.

For this site CPTu4 is analysed. The phreatic surface is assumed at -3 m based on the pattern of u_2. Also based on the pattern of u_2 it is assumed that at -18 a more permeable layer might be present. The piezometric head as measured with the piezocone at -18 m is therefore chosen to fit the pore water pressure gradient (u_0). Also at this site the response of u_2 seems to be slow and straggling.

Figure 7-11 gives the estimated bulk unit weight (a) and calculated stresses (b) for CPTu4 at example 4. For peat the Lengkeek et al. (2022) method to estimate the bulk unit weight of peat gives a much lower bulk unit weight than the Robertson et al. (2022) and the Lengkeek et al. (2022) general methods. This has an influence on the effective stress of about 6% at level -8 and about 4% at level -20 m.



(a) (b)

Figure 7-11 Estimated bulk unit weight (a) and stresses (b) for CPTu4 at example 4.

Figure 7-12a gives the calculated undrained shear strength for CPTu4 at example 4. s_u is derived from the cone resistance via N_{kt} based on F_r and B_q. These methods give peak values of s_u for the direct simple shear and triaxial compression mode respectively. Both methods give similar results.

In Figure 7-12b the calculated OCR values are presented. The calculated OCR are based on three methods:

- Via s_u which is derived from the cone resistance using N_{kt} based on F_r and using the SHANSEP model.
- Via s_u which is derived from the cone resistance using N_{kt} based on B_q and using the SHANSEP model.
- Directly from the cone resistance using the theoretical relation from Mayne (1991).

The Mayne (1991) relation seems to overestimate OCR similar to the other cases.

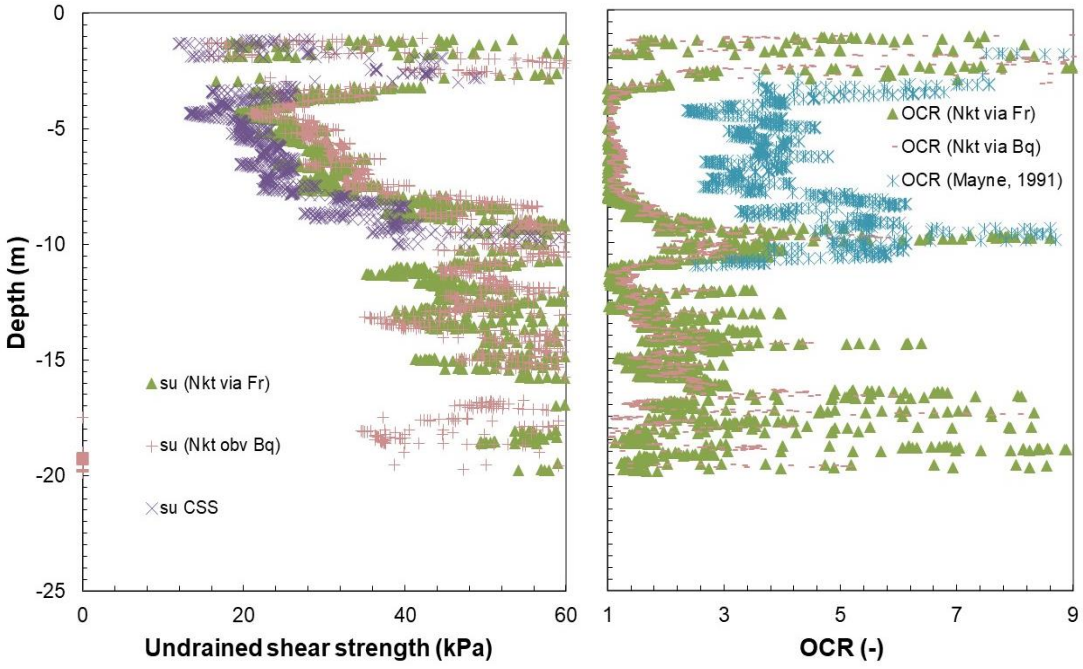


Figure 7-12 Derived undrained shear strength s_u (a) and overconsolidation ratio (OCR) (b) for CPTu4 at example 4. The undrained shear strength derived via N_{kt} based on F_r and B_q are peak values; the others are ultimate values.

8 Conclusions

This literature study gives an overview of the state-of-the-art methods and theories to determine the stress state under the track bed, with a focus on situations that are similar to the Swiss railway foundations. The stress state concerns both static and cyclic conditions. Regarding the methods to determine the in-situ stress state the focus in this study is on the CPT. However, also other in situ tests are briefly discussed. The CPT interpretation to determine the stress state is applied to four examples of railway tracks in Switzerland at different sites with different geological conditions.

Stress state at static conditions

The stress state under the track bed at static conditions depends on many factors. These are:

- Thickness and weight of the track bed and the railway embankment.
- Stratigraphy of the subsoil with different soil types having different bulk unit weights.
- Stress history of the subsoil related to the geological background (sedimentary environment) of the subsoil and also including soil forming processes (ripening, cementation) and creep (post-sedimentary processes or ageing).
- Pore water pressures in the railway embankment and the subsoil under different geohydrologic conditions, including seasonal variations.

The influence of many of these factors on the stress state is not straightforward:

- The thickness of the track bed and the railway embankment and the stratigraphy of the subsoil can vary considerably which result in variations in the effective stresses. These variations can occur on relatively short distances.
- The stress history of the subsoil and the railway embankment can also vary because there are many factors which influence the stress history, such as the geological background of the soil, building of the embankment and influence of the train loading. So, at different sites different aspects of the stress history can be dominant.
- The pore water pressures can be very different at different locations due to the geological and geohydrological conditions (artesian groundwater conditions or underdrainage conditions) or the construction of the railway embankment and also vary considerably in time due to seasonal effects. This also affects the stress state.

Careful site investigation is required to quantify the effect of all these aspects on the stress state of the track bed.

Stress state at cyclic conditions

The main findings concerning the stress state at cyclic conditions are:

- From the available literature on laboratory tests follows that significant pore-pressure build-up after a train passage seems unlikely in case of already cyclic preloaded soil (with intermediate drainage) if the cyclic shear stress amplitude is not enlarged (no change in axle loads and speeds). An increasing pore pressure build-up by subsequent train passages is also not reported in literature on the few available field measurements. This is confirmed by pore pressure measurements at the Delft – Schiedam railway line in The Netherlands. This measurement campaign shows a small increase of pore pressure due to an individual train passage with short peaks. Around 98% of the pore pressure peaks dissipate in 100 seconds or less.
- A strong attenuation of the train induced vertical displacements and excess pore water pressure with depth can be expected. This follows from dynamic 3D FEM analyses with moving loads which are performed for the Delft – Schiedam railway line.

The largest displacements (0.30 mm) are only visible in the embankment, especially in the top part of the embankment above the phreatic surface which exhibits more compression. However, at 6 m below the embankment still some train induced deformations occur. The pattern of the attenuation with depth will depend on the configuration of the track bed and the embankment and the level of the phreatic surface.

- CPTs conducted in the railway embankment of the Delft – Schiedam railway line show a strong decrease of the density of the sand in the embankment. Within about 4 m below the top of the embankment the cone resistances reduce to low values.
- Assuming quasi-static conditions (ignoring dynamic amplification of soil displacements and stresses) under the track bed is allowed as long as the train speed is below approximately 60% of the local critical train speed.
- When train speed, axle load or train frequency are going to increase one can expect an increase of the vertical displacements and excess pore pressures. These effects become stronger towards the critical train speed. How these effects attenuate with depths will depend on the configuration of the track bed and the embankment and the level of the phreatic surface.

Application of CPTs to determine the stress state for field conditions

CPTs are a powerful tool to investigate the conditions of railway embankments and the subsoil and to determine the stress state of the track bed. This concerns especially the static conditions of the stress state of soft soils. Some aspects need attention:

- The methods to estimate bulk unit weight based on CPTs are not accurate. This introduces an error in the stress state. In two of the examined examples the error in the effective stress was between 4 and 10%. This influences also OCR and K_0.
- The stationary pore water pressure in the subsoil is often difficult to deduce from CPTs. This is the case when there are no permeable soil layers within the depth of the CPT measurements. In one of the examined examples different assumptions on the pore water pressure gradient result in an uncertainty in the effective stress and therefore also an uncertainty in OCR of around a factor 1.5.
- In most CPTs which are analysed to derive the stress state for the four examples the reliability of the measured pore water pressure u_2 seems to be poor. This gives a limitation in the interpretation of the CPT concerning the soil classification.
- The methods to interpret CPT measurements are developed for certain soil types. For other soil types the interpretation methods may be less accurate. To increase the accuracy of the determination of the stress state cross checks with field vane tests or high-quality laboratory tests are essential. This is also the case for the interpretation of the seismic cone.
- It is not possible to derive an accurate estimation of the horizontal stress from CPT measurements. When the horizontal stress is important pressuremeter tests or dilatometer tests are required.
- A CPT does not give an accurate estimation of the sensitivity of soil layers, such as lake chalk. For sensitive soils additional measurements with field vane tests are required.
- To assess the state of sand and silt various interpretation methods for CPTs are available in literature. To assure that the CPT interpretation is correct, especially for partially drained soil conditions during CPT penetration, additional laboratory testing is required.
- For the interpretation of CPTs in partially saturated soils additional information on water content, degree of saturation and capillary suction is needed as the capillary suction influences the CPT measurements.

9 Recommendations

To improve the reliability of the determination of the stress state under the track bed at both static and cyclic conditions the recommendations are as follows.

Stress state at static conditions
To assess the stress state of the railway embankment and the subsoil at static conditions the following is recommended:

- Perform a site investigation including the geohydrological conditions to get a detailed insight in the construction of the railway embankment and the stratigraphy of the subsoil and in the stress state of the railway embankment and the subsoil.
- The site investigation needs to be performed sufficiently deep below the track bed. The required depth depends on the local circumstances, such as:
  - The depth until which the train load may affect the stress state.
  - The height of the railway embankment and the steepness of the side slopes of an embankment.
  - Differences in elevation of the surrounding surface level adjacent to the embankment (slopes, ditches).
  - The occurrence of soft soil layers (weak clay, peat, gyttja) or liquifiable layers (loose sand and silty sand) or collapsible layers which can cause instability of the embankment. Such layers even when they are deep below the track bed may attract a potential slip surface or deformation zone along which an embankment slope may deform or fail.
  - The occurrence of an aquifer which may have a high piezometric head due to artesian groundwater conditions (can be seasonal or after heavy rainfall).
- When the site investigation is performed next to the track, for clay and peat it can be assumed that the pre overburden pressure POP next to the track and below the track is on average the same. This follows from experiences from flood defences in The Netherlands. For sand and silt the state parameter moves towards higher values (stronger tendency for contractive behaviour) when the effective stress is higher below the embankment and assuming that the soil below and adjacent to the embankment belong to the same normal compression line and critical state line.
- Be aware of situations where the static stress state may change due to changes or variations in groundwater conditions (seasonal fluctuations, initially unsaturated soil which becomes saturated and losses strength) or due to changes in the stability of a hill slope.

Stress state at cyclic conditions
When train speed, axle load or train frequency are going to increase one can expect an increase of the vertical displacements and excess pore pressures. These effects become stronger towards the critical train speed. How these effects attenuate with depths will depend on the configuration of the track bed and the embankment and the level of the phreatic surface. When train speed, axle load or train frequency for a railway line are going to increase the following is recommended:

- Perform a site investigation including the geohydrological conditions to get a detailed insight in the construction of the railway embankment and the stratigraphy of the subsoil and in the stress state of the railway embankment and the subsoil.
- Conduct analyses to predict what the effect of the increase of train speed, axle load or train frequency on the stress state of the track bed will be.
- Design and install monitoring to get insight in the response of the stress state of the railway embankment and the subsoil to the change of the train speed, axle load or train frequency.

Application of CPTs to determine the stress state for field conditions

CPTs are very useful as a screening tool to determine the stress state under the track bed, especially for cases with soft soils. When CPTs are applied in a site investigation the following is recommended to improve the accuracy of the CPT interpretation:

- Perform cross checks as the methods to interpret CPTs have uncertainty because they are often empirical methods which are developed for certain soil types and need verification for use in other soil types:
  - Apply different interpretation methods to get insight in the bandwidth of possible outcomes of the CPT interpretation.
  - Determine the bulk unit weight in the laboratory as the bulk unit weight is the basis to derive the effective stress and other parameters from CPTs.
  - Determine the undrained shear strength on high quality soil samples in the laboratory using the recompression consolidation technique to verify the empirical cone factor N_{kt}. This will lead to accurate determination of OCR.
  - Perform laboratory tests on sand and silty sand to determine the required parameters to improve the CPT interpretation and to study the behavior of the material to verify the derived state parameter from the CPT.
- Investigate the geohydrological conditions as the stationary pore water pressure is very relevant to interpret CPT data and to derive accurate effective stresses. This can be done with piezometers. It can be considered to perform CPTs with dissipation tests which can be performed longer time until the stationary pore water pressure is reached.
- Improve the accuracy of the u_2 pore pressure measurements as these are relevant for the soil classification, estimation of the empirical cone factor N_{kt} and CPT interpretation. Perform the u_2 measurements with careful attention for filter type and fluid type and preparation of the equipment.
- When the horizontal stress or K_0 are relevant CPTs cannot give accurate results. In that case pressure meter tests or dilatometer tests should be performed. In peat it should be noted that dilatometer tests may be not accurate enough.
- When the soil is sensitive it is recommended to perform additional field vane tests.
- When the soil is partially drained (intermediate soils) faster or slower CPT penetration may be required to cause undrained or drained conditions respectively. For silty sand and sand with fines methods are available in literature to derive the state parameter for partially drained conditions during CPT penetration.
- When the soil is partially saturated determine the water content, degree of saturation and capillary suction to be able to interpret the CPT data. The capillary suction influences the shear strength and therefore also the CPT measurements.
- Take into account the spatial variability of the soil conditions. The distance between CPTs needs to reflect the geological complexity of a site.

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