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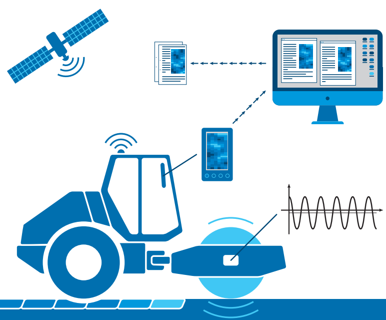

Heft 4
Wien, November 2018

D. Adam & S. Larsson (eds.)

40 Years of Roller Integrated Continuous Compaction Control (CCC)

Anniversary Symposium, November 29th, 2018, Vienna

Symposium Proceedings

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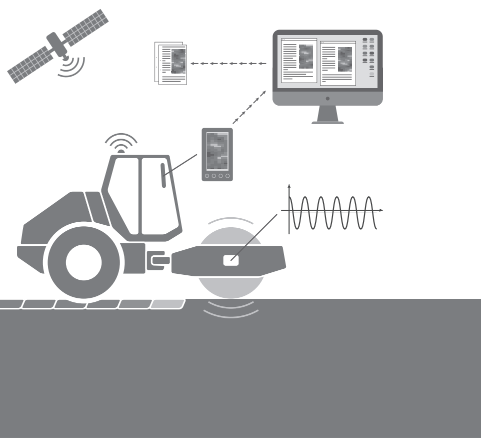

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Preface

In 1978 the first Roller Integrated Continuous Compaction Control (CCC) system was patented. Since then 40 years have passed, in which not only CCC has been established in geotechnical engineering worldwide, but has also significantly influenced the development of rollers to high tech devices.

In the scope of the Anniversary Symposium "40 Years of Roller Integrated Continuous Compaction Control (CCC)" we reviewed the past four decades together with renowned international experts and took a look into the future. The anniversary symposium offered a platform for geotechnical engineers from all over the world to exchange experience and developments in dynamic roller compaction and compaction control. In the name of the organizing committee I express my gratitude to all honorary guests, pioneers, invited lecturers and speakers for coming to Vienna to share their rich experience in the field of dynamic compaction and compaction control with all the participants.

The symposium was connected with a fine exhibition at the venue. The leading roller manufacturers Bomag, Dynapac, Hamm, Ammann and Caterpillar as well as the CDC compaction company Cofra and, moreover, the developers of compaction control devices Anix and ZORN INSTRUMENTS presented their products and prepared short techno-commercial presentations. The financial support granted by the exhibitors and consulting engineers FCP and VCE is gratefully acknowledged. The numerous non-financial co-sponsors expressed their commitment to support the developments and innovations in the field of novel compaction techniques and compaction control systems.

The International Intelligent Construction Technologies Group (IICTG) and ISSMGE TC 202 (Transportation Geotechnics) delivered short reports about their activities and their technical meetings right before and after the symposium.

The Extended Abstracts of the presentations are published in this 4th issue of the Publication Series of the Institute of Geotechnics. In 24 contributions several authors from all over the world present historic developments from Scandinavia and Central Europe, application, interpretation, classification, evaluation and implementation of CCC, and finally, new findings and novel compaction techniques.

The Institution of Civil Engineers (ICE) will publish a Themed Issue with more detailed papers covering the symposium topics.

Univ.Prof. Dipl.-Ing. Dr.techn.
Dietmar Adam

Vienna, November 2018

Content

Call for Papers
Roller integrated continuous compaction control

Proceedings of the Institution of Civil Engineers

Geotechnical Engineering THEMED ISSUE

Champions: K. Rainer Massarsch, Geo Risk and Vibration Scandinavia AB, Bromma, Sweden, and Dietmar Adam, Technische Universität Wien, Vienna, Austria

Geotechnical Engineering is pleased to announce a collaboration with the organising committee of the 2018 symposium *40 Years of Roller Integrated Continuous Compaction Control (CCC)* to produce a themed issue on roller integrated continuous compaction control in 2019.

Roller Integrated Continuous Compaction Control (CCC) is a one-day symposium organised by the Technical University, Vienna, Austria, in cooperation with the Royal Institute of Technology, Stockholm, Sweden. The symposium will be held on 29 November 2018 at the Austrian Society of Engineers and Architects (ÖIAV) in Vienna.*

As part of the symposium, proceedings containing extended abstracts will be published. Symposium authors and others who would like to publish their findings in greater detail (in addition to the four-page extended abstracts) are invited to prepare longer papers (maximum 5000 words) to be published in a themed issue in *Geotechnical Engineering*. All submitted manuscripts will be reviewed in accordance with ICE Publishing rules. Authors must cite the extended abstract in the submitted manuscript

Manuscripts are expected to cover the following topics:

- **Historic overview of roller compaction and continuous compaction control**
- **Applications and interpretation of measurements**
- **Field testing methods for compaction control (e.g. static and dynamic plate load test)**
- **Standardisation, quality assurance/control and design aspects**
- **Recent and future developments (e.g. compaction equipment, electronics, internet)**
- **Case histories of construction and engineering earthworks**

Relevant papers addressing the main theme (compaction control) will also be considered.

The deadline for abstracts is 31 December 2018. The deadline for submissions is 31 August 2019.

*Details about the symposium can be obtained from the symposium website (https://www.igb.tuwien.ac.at/en/ccc/program/) or by contacting the symposium secretary Johannes Pistrol (johannes.pistrol@tuwien.ac.at)

Invitation to authors

To submit an abstract, please visit:
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Historic Review from Scandinavia and Central Europe

Development and improvement of a roller mounted compaction meter in Sweden in the 70's and 80's

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Keywords:Compactometer; Geodynamik; Dynapac

1 COMPACTOMETER, EARLY DEVELOPMENT FROM A SWEDISH PERSPECTIVE

Geodynamik, a Stockholm based Swedish engineering company founded by Dr. Heinz Thurner and Dr. Åke Sandström started development of the first accelerometer based compaction meter for vibratory rollers in the mid-seventies. The project was financed via the Swedish Technical Development Board, Styrelsen för Teknisk Utveckling (STU). Today called VINNOVA, Verket för Innovationsutveckling. Geodynamik also developed and patented the principle of oscillating compaction, also this based on research funded by STU. Dynapac was a partner in the compaction meter project for evaluation of the practical application of a possible future product. Dr. Lars Forssblad was responsible for the evaluation with help from his team at the Dynapac research laboratory.
The possibility of using a vibrating roller as a tool for compaction measurement had been desired for many years and after three years of development, testing and evaluation, the Compactometer was presented. Based on an accelerometer mounted on the vibrating part of the drum frame, it measures, amplifies and analyzes the signals with respect to the fundamental vibration frequency and the harmonics thereof. The first version presented digital values on a display, which were calculated over a measurement period of 5 or 30 seconds. A limit could also be applied, and measurement values exceeding the limit would trigger a light that indicated that the desired soil stiffness had been reached.

Figure 1. Early Compactometer installed in a roller.

2 INITIAL DEVELOPMENT AND EVALUATION

In the summer of 1977, tests were made with the first prototype of the electronic compaction meter on a number of road and dam projects in Sweden. A wide range of materials were evaluated; rock-fill, crushed rock, gravel, sand, moraine and clay. The compaction meter results were compared with the results of static plate load tests, falling weight apparatus, water replacement density tests and levelling of surface settlement. The tests were made in the Dynapac Research Laboratory using a CA51S 16000 kg single drum roller. The results showed linear relationships between the new roller-generated compaction meter value and all four test methods when plotted in a single logarithmic graph.

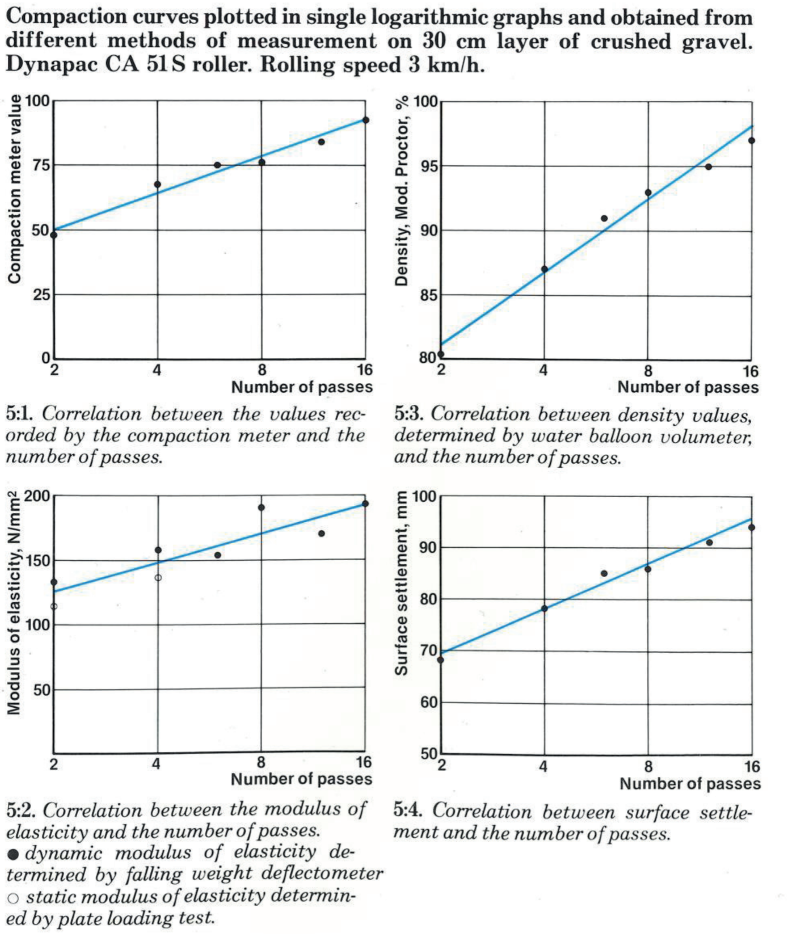

Figure 2. Compactometer correlation to other control methods

The most extensive of the initial tests was conducted during construction of the Juktan power dam in northern Sweden during August and September 1977. The tests were made in close cooperation with the State Power Board. Again, a CA51S was used for the majority of the work while a smaller 6000 kg CA15 was used for the compaction of till soils and in confined areas. In addition, here, good correlation was found between the results of the compaction meter and the traditional compaction control methods. The Swedish engineering company Jacobsson & Widmark, J&W (Eng. B. Pramborg), made additional evaluations together with Chalmers Institute of Technology (S. Hansbo) at the Riksten gravel pit in Tullinge outside Stockholm, Sweden. Silty sand, sand and gravelly sand was chosen as the materials for testing. In short, the conclusion was to recommend the use of the Compactometer for monitoring the individual layers and penetration together with pressure meter control of the fill once it had reached the final level. International tests include rock-fill compaction with a 16000 kg CA61 towed roller at the Finstertal dam in Austria and compaction of railway embankments using a CA51S on the new line from Paris to Lyon in France. The latter was made in cooperation with the French Railway administration.

In 1986 The Dynapac research department conducted studies and testing of a new type of measurement roller, a narrow steel wheel with a vibration device and a Compactometer fitted. The idea behind this was that that the narrow, rounded contact surface of the steel wheel would provide more accurate readings in comparison to those obtained from the full width measurements of the roller-mounted version. The project was later abandoned.

Figure 3. Tow-behind Compactometer

3 LEGAL ISSUES

In 1977, Dynapac signed an agreement with Geodynamik granting exclusive rights to market the Compactometer on a number of markets. This agreement included a clause stating that the funding received from STU was to be repaid based on the number of units sold. This was part of the payment for the exclusive rights to the Compactometer. In 1984, Dynapac was approached by an STU employee who previously had been consulted as a patent engineer and advisor during the development of the Compactometer. Dynapac was offered a new model of a compaction meter (principles unclear). This caused a rather agitated conversation between STU, Geodynamik and Dynapac.

4 DOCUMENTATION OF COMPACTON METER DATA

As early as 1980, a printer was added to the Compactometer. This produced lasting documentation of the compaction meter data, albeit without an absolute position.

Figure 4. Compactometer printer

Wireless transfer of Compactometer data was also possible as early as 1983. The Telemetry system, as it was called, was also developed by Geodynamik and used during evaluation trials carried out by the Technical University of Munich in 1984. Geodynamik tested their first generation documentation system, the STS-001, in 1986. This was later developed into the Compaction Documentation System, CDS and launched in 1988.

The Compaction Control Systems-Recorder and Analyzer (CCS-RA) was introduced by Dynapac in the late 80's. The CCS-RA introduced digital data storage and the option of a DOS-based PC software for more in-depth analysis and reporting of the Continuous Compaction Control (CCC) data. It also added the possibility of color print-outs.

5 SWEDISH SPECIFICATIONS FOR CCC

Research and tests made from 1985 and onwards in cooperation between Swedish stake-holders in the field of road construction resulted in a national specification that included CCC. The national Swedish Road Administration were among the earliest adapters of the CCC technology. They published a method description on how to use CCC data for localization of static plate load testing (VVMB 603:1994). This provided an incentive for contractors to invest in CCC equipment as they were allowed to reduce the amount of test locations from a maximum of eight, randomly placed tests per 4500 or 6000 square meters (depending on material) down to only two. The tests should be made in the points identified by CCC system as being the ones with the lowest stiffness. The publication also covers the fundamentals of accelerometer based compaction meters, what factors affect the results as well as a method for calibration of the Compactometer with relation to the static plate load test. The method was introduced in the national standards called VÄG94 (Road 94) and has been part of the national specifications ever since.

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Schwingungstechnische Grundlagen der FDVK

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Schlagwörter: FDVK; Bodenmodellierung; Schwingungssystem

1 ZUSAMMENFASSUNG

Seit nunmehr 4 Jahrzehnten werden durch Messungen des Schwingungszustandes am Verdichtungswerkzeug bereits während des Verdichtungsvorganges Messgrößen zur Beurteilung des Verdichtungszustandes ermittelt. Dabei handelt es sich häufig um indirekte Messgrößen, die durch eine Kalibrierung auf die klassischen Messgrößen zurückgeführt werden können. In diesem Beitrag werden vier unterschiedliche Bodenmodelle vorgestellt und untersucht. Zunächst wird nachgewiesen, dass sie geeignet sind, das reale dynamische Verhalten des Verdichtungswerkzeuges zu beschreiben. Ferner werden neun unterschiedliche Kriterien untersucht, um ein tieferes Verständnis im Zusammenhang mit der flächendeckenden Ermittlung des Verdichtungszustandes zu vermitteln. Zusammenfassend kann man feststellen, dass es viele Möglichkeiten gibt, die Steifigkeit des Untergrundes bei Anregung mit einer Vibrationswalze zu ermitteln. Für eine bestimmte Walzenkonfiguration lässt sich stets eine Korrelation zwischen Auswertekriterium und Steifigkeit des Untergrundes erzeugen. Den ausführlichen Beitrag zum Beispiel findet man unter (Kröber 2018b).

2 ANSÄTZE FÜR DAS SCHWINGUNGSSYSTEM WALZE UND BODEN

Im Folgenden sollen vier Modelle für das System Walze/Boden erarbeitet und vorgestellt werden. Für Modellmodell A wird für den Boden der standardmäßige Linearansatz verwendet.

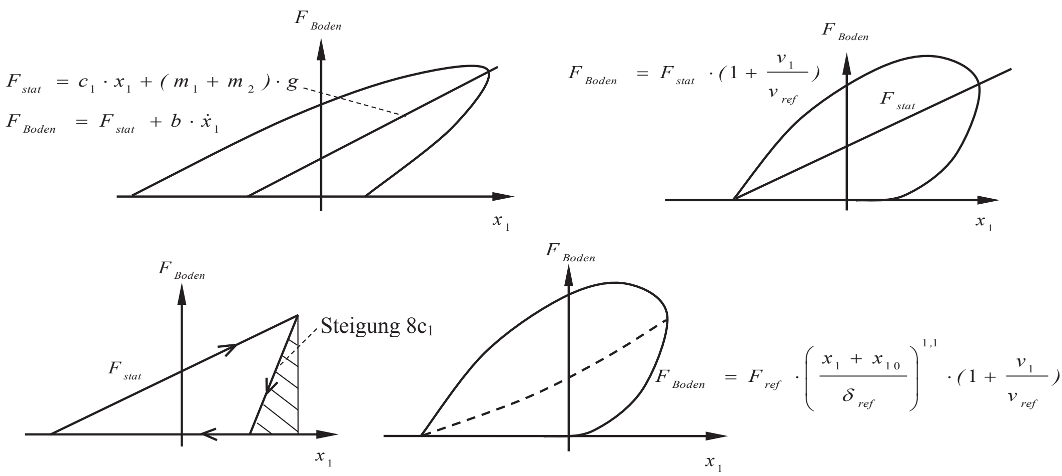

Abbildung 1. Kennlinien für Modelle A (oben links), B (oben rechts), C (unten links), D (unten rechts)

Für Bodenmodell B ist der statische Ansatz gleich. Die Beschreibung der Dämpfung erfolgt proportional der statischen Kraft in Abhängigkeit der Geschwindigkeit. Bodenmodell C erfolgt ohne Einfluss der Geschwindigkeit. Dafür geht die Bewegungsrichtung ein. In Bodenmodell D wird gegenüber Bodenmodell B die lineare Steifigkeit durch eine progressive Potenzfunktion ersetzt.

3 GRUNDLAGEN DER FDVK, VARIATION DER STEIFIGKEIT C_1

Bei der flächendeckenden Verdichtungskontrolle ist als Standardfall die Unwuchtmasse konstant. Die Vibrationsfrequenz ändert sich auch nur in einem geringen Maße. Die Vibrationsfrequenz nimmt wegen des steigenden Leistungsbedarfs und den volumetrischen Wirkungsgraden der Ölhydraulik etwas ab. Übrigens ist die Abnahme der Vibrationsfrequenz auch ein Maß für die Steifigkeit des Bodens. Was hier jedoch die entscheidende Rolle spielt, ist die Veränderung der Steifigkeit c_1.
In der nächsten Abbildung ist die sich einstellende Schwingamplitude (Schwingwegdifferenz peak-to-peak dividiert durch 2) in Abhängigkeit von c_1 dargestellt. Die Darstellung erfolgt hier exemplarisch für Modell 4.

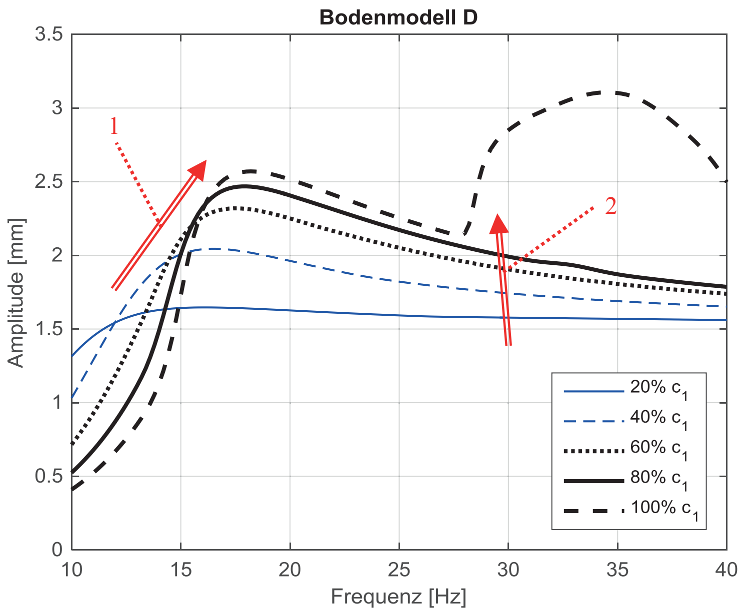

Abbildung 2. Amplitude über Frequenz, Parameter c_1

In Abb. 2 ist erkennbar, dass mit wachsender Steifigkeit die Eigenfrequenz Walze/Boden ansteigt ("Pfeil 1").

"Pfeil 2": Bedingt durch den charakteristischen Verlauf der Resonanzkurve steigt die Schwingamplitude im Betriebszustand mit wachsender Steifigkeit an. Die sich einstellende Schwingamplitude wäre also bereits das erste Kriterium, um einen Aufschluss für die Steifigkeit der Unterlage zu erhalten. Das spürt auch der Fahrer auf der Walze. Aufgrund der gefühlten Schwingintensität konnte er bereits eine Aussage über die Steifigkeit persönlich empfinden und einschätzen, bevor objektive Kriterien zur dynamischen Verdichtungskontrolle vor 40 Jahren entwickelt wurden.

4 KRITERIEN ZUR BEURTEILUNG DES VERDICHTUNGSZUSTANDES

Mit der Simulationsrechnung werden im Folgenden exemplarisch 9 Kriterien zur Ermittlung des Verdichtungszustandes aus den dynamischen Signalverläufen entwickelt und erläutert. In dieser Kurzfassung ist die Abhängigkeit für Bodenmodell D in Abb. 1 (unten rechts) dargestellt.

Kurzbeschreibung der neun untersuchten Kriterien:

- Oberschwingung der Ordnungszahl 2, in Graphik mit "2" bezeichnet
- Grundschwingung, Ordnungszahl 1, in Graphik "1"
- Subharmonsiche Schwingung, Ordnungszahl 0,5, in Graphik "0.5"
- Maximale Beschleunigung, in Graphik "b_{max}"
- Differenz Maxima Beschleunigung, in Graphik "d b_{max}"
- Differenz Steigung Kompression und Expansion, in Graphik "steig"
- Maximale Bodenkontaktkraft, in Graphik "F_{Bod}"
- Steigung Kompressionskennlinie, in Graphik "c_{Bod}"
- Geodätische Schwinghöhe Rahmen, in Graphik "geod"

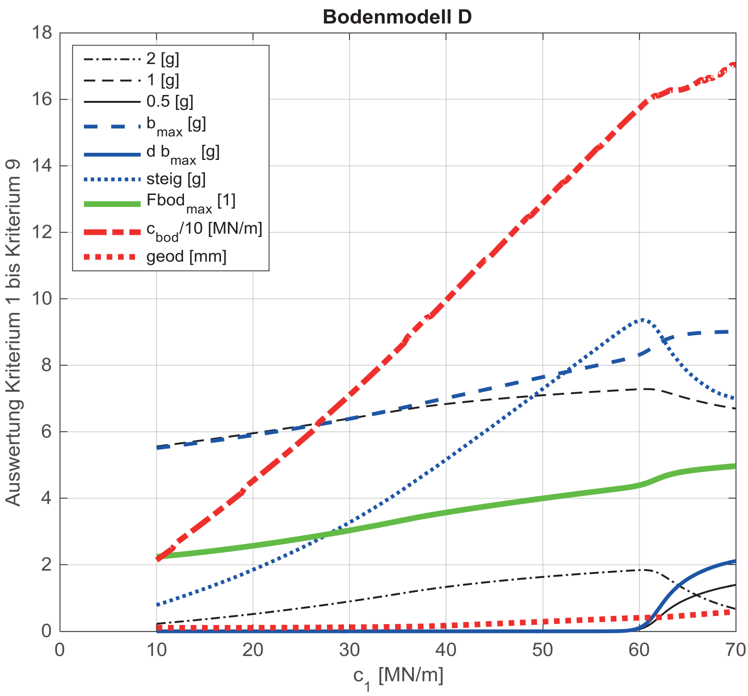

Abbildung 3. Abhängigkeit aller Kriterien für Bodenmodell D.

Zusammenfassend kann man feststellen, dass es viele Möglichkeiten gibt, die Steifigkeit des Untergrundes bei Anregung mit einer Vibrationswalze zu ermitteln. Für eine bestimmte Walzenkonfiguration lässt sich stets eine Korrelation zwischen Auswertekriterium und Steifigkeit des Untergrundes erzeugen.

5 AUSBLICK

Wenn eine mathematische Beschreibung des Untergrundes vorliegt (beispielhaft Modell A), dann kann man die Beschleunigung während des Verdichtens messen und dann im Hintergrund das Modell rechnen lassen.

Bodenmodell A:

$$F_{Boden} = c_1 \cdot x_1 + b_1 \cdot \dot{x}_1 + (m_1 + m_2) \cdot g \qquad (1)$$

wobei stets

$$F_{Boden} \geq 0 \qquad (2)$$

Hier sind im Wesentlichen c_1 und b_1 des Bodens als unbekannt anzusehen. Nun variiert man in der "Hintergrundrechnung" c_1 und b_1 so lange, bis der errechnete Beschleunigungsverlauf mit dem simulierten Beschleunigungsverlauf im quadratischen Mittel möglichst gut übereinstimmt. Dies wird permanent durchgeführt. Auf diese Weise erhält man als Abfallprodukt b_1, aber auch das gesuchte c_1. Vielleicht wird dies auch schon in der Praxis angewandt.

Im "Google Play Store" gibt es vom Verfasser einige APPs (Kröber 2018a). Sie sind einfach zu finden, indem man den Namen "Kröber" eingibt. In dem hier aufgezeigten Zusammenhang ist besonders "frequency acoustics" (Frequenzanalyse aufgrund Mikrofonaufnahme) und "vibration analysis" (Auswertung Beschleunigungssensoren) zu nennen. Hier könnte man den Verdichtungszustand mit einer APP auf dem Smartphone messen. Für den Anwender wäre das eine preisgünstige Alternative. Eine solche APP wäre sicher für wenige Euro verfügbar.

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Automatische Verdichtungskontrolle: eine Anwendung der nichtlinearen Schwingungstheorie

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Keywords: Compaction Control; Nonlinear Vibrations; Optimization of Compaction Process

1 EINFÜHRUNG

Die nichtlineare Dynamik einseitig gebundener Systeme stellt die Grundlage der Mess- und Regelungstechnik dynamischer Bodenverdichtungsgeräte, insbesondere von Strassenwalzen und Walzenzügen dar. Diese Technologie lässt sich gezielt auf eine stark nichtlineare Dynamik erweitern, um den Energietransfer zwischen Maschine und Untergrund zu maximieren. Parallel zur Energieeffizienz nimmt der Aufwand für ein stabiles Maschinenverhalten zu. Alternativ zur Optimierung der einzelnen Verdichtungsgeräte kann der Strassenbauprozess über den Einsatz aller Maschinen auf einer Baustelle optimiert werden. Diesen Aspekten ist der nachstehende Artikel gewidmet.

2 NICHTLINEARE DYNAMIK VON VERDICHTUNGSGERÄTEN, KLASSISCHE TECHNIK

2.1 Klassische Mess- und Regelungstechnik bei Vibrationsverdichtungsmaschinen

Bei der klassischen Verdichtungskontrolle und der darauf aufbauenden Regelungstechnik für Verdichtungsmaschinen, vgl. Abb. 1, spielt primär die Vertikaldynamik die zentrale Rolle in der dynamischen Analyse des Bewegungsverhaltens der Maschine.

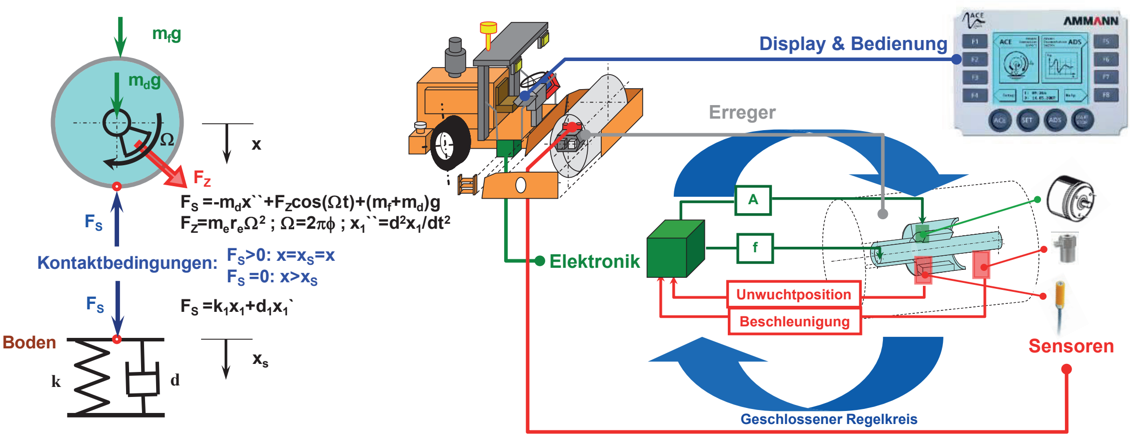

Abbildung 1. Modell einer klassischen Vibrationswalze samt zugehörigem Mess- und Regelsystem.

Klassische Verdichtungsgeräte zeichnen sich dadurch aus, dass ihr Chassis in Ruhe bleibt, d. h. nicht am Verdichtungsprozess teilnimmt, vgl. Hierzu die Modellierung (Anderegg, Kaufmann 2004). Dementsprechend kann die Lösung der Bewegungsgleichung (1) mit $x_f \approx 0$ angenähert werden. Auf diesem Weg wird das rund 2 bis 3-fache Maschineneigengewicht dynamisch mit der Periode der Anregung in den Untergrund eingeleitet. Damit diese Dynamik stabil erfolgt, werden die maximal auftretenden Kontaktkräfte pro Schwingungsperiode durch Regeln von Amplitude (Exzentrizität r_u) und Frequenz f des Erregers automatisch auf einen Maximalwert geregelt. Bei Asphaltmaschinen ist das typischerweise das doppelte Eigengewicht, bei Erdverdichtungsgeräten das rund 2.5 bis 3-fache der statischen Achslast.
Parallel zum geregelten Verdichtungsprozess wird die Steifigkeit k_B des Bodens gemessen und als Verdichtungskontrollwert verwendet. Diese Technologie ist mittlerweile rund 20 Jahre alt.

3 METHODEN DER LEISTUNGSOPTIMIERUNG, AUTOPARAMETRISCHE RESONANZ

Ein eine interessante Weiterentwicklung dieser Technologie stellt die Nutzung des nichtlinearen Bewegungsverhaltens von Bodenverdichtern dar. Dabei geht es primär darum, mit grösseren Unwuchtmomenten $m_u r_u$ eine nichtlineare Dynamik zu erzeugen, bei welcher das Chassis einen aktiven Anteil an der resultierenden Dynamik hat und die maximalen Kräfte F_S, die auf den Boden einwirken, otimiert werden, d. h. $x_f \neq 0$ und $F_{S,max}/(m_f+m_d)g$ ist maximal. Abb. 2 zeigt den Aufbau eines entsprechenden Walzenzugs, die Gleichungen (1) beschreiben die zugehörigen Differentialgleichungen.

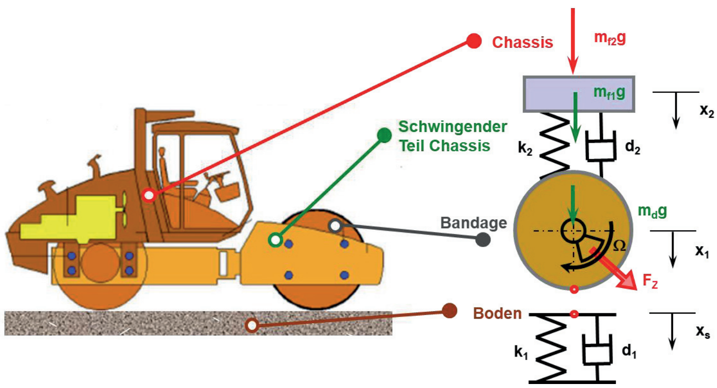

Abbildung 2. Analytisches Modell eines Verdichtungsgerätes mit unwuchterregter Bandage und schwingendem Chassisanteil (ein Teil schwingend, ein Teil statisch/stationär)

3.1 Stark nichtlineare Dynamik

Die stark nichtlineare Dynamik weist einen grossen Bereich analytischer Lösungen auf, welche im Allgemeinen instabil sind und für die Verdichtung nicht genutzt werden können. Die wesentliche Ausnahme ist die Wirkung der vibrierenden Bandage als selbsterregte (auto)parametrische Resonanz bezüglich der schwingenden Masse des Chassis.

$$\begin{aligned} m_d\ddot{x}_d &= -F_S + m_d g + m_u r_u\ \omega^2 cos(\omega t) - k_2(x_2 - x_1) - d_2(\dot{x}_2 - \dot{x}_1) \\ m_{f1}\ddot{x}_f &= (m_{f1} + m_{f2})g + k_2(x_2 - x_1) + d_2(\dot{x}_2 - \dot{x}_1) \\ F_S &= k_B x_S + d_B \dot{x}_S\ Falls\ F_S > 0;\ F_S = 0\ sonst \end{aligned} \tag{1}$$

Parameter: m_{f1}: Masse Rahmen mitschwingend; m_{f2}: Masse Rahmen statische Auflast; m_d: Masse Bandage (schwingendes Element, beinhaltet den Unwuchterreger);
x_f: Schwingweg Chassis; x_d: Schwingweg Bandage; ω: Erregerfrequenz [rad/s]
r_u Exzentrizität Unwucht; x_f: Schwingweg Chassis; k_i: Steifigkeit I; d_i: Dämpfung i

Für diesen Fall ist in Abb. 3 der zugehörige Schwingungszustand des Systems dargestellt. Man erkennt, dass die Bandage nur jede zweite Unwuchtumdrehung Bodenkontakt hat, dementsprechend ist die dabei auftretende, maximale Bodenkraft $F_{S,max}$ wesentlich höher als bei der herkömmlichen Verdichtungstechnik.

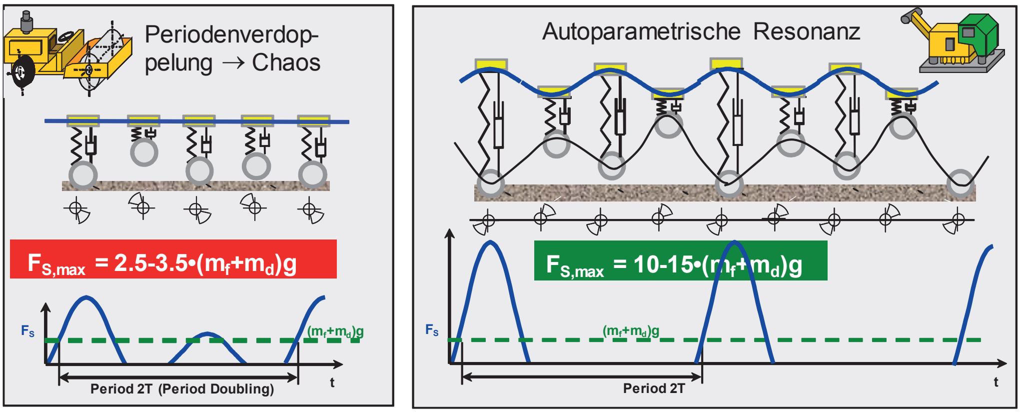

Abbildung 3. Tilgereffekt einer schwingenden Chassismasse, eine Leistungssteigerung von rund 400% ist realisierbar. Gut erkennbar die statische Wirkung des Chassis heutiger Maschinen und die Dynamik bei der Ausnutzung der autoparametrischen Resonanz

Es kann praktisch nachgewiesen werden, dass die autoparametrische Resonanz die maximale Verdichtungskraft um bis zu 400% gegenüber herkömmlichen Methoden vergrössert. Für eine 20 to Maschine mit teilweise schwingungsfähigem Chassis wurde dieser Effekt nachgewiesen, vgl. Abb. 4. Die Verdichtungsschläge erfolgen mit 12.5 Hz, während dem Unwuchterreger und Chassis mit 25 Hz, also der doppelten Frequenz vibrieren. In Abb. 3 erkennt man die Wirkung dieser Dynamik: die zweifrequente Schwingung maximiert die Amplitude in Bodenrichtung und minimiert diese in der Flugphase. Anders ausgedrückt erhält man eine asymmetrische Maschinendynamik, deren Maxima in den Boden gerichtet sind und damit die transferierte Verdichtungsenergie maximieren.

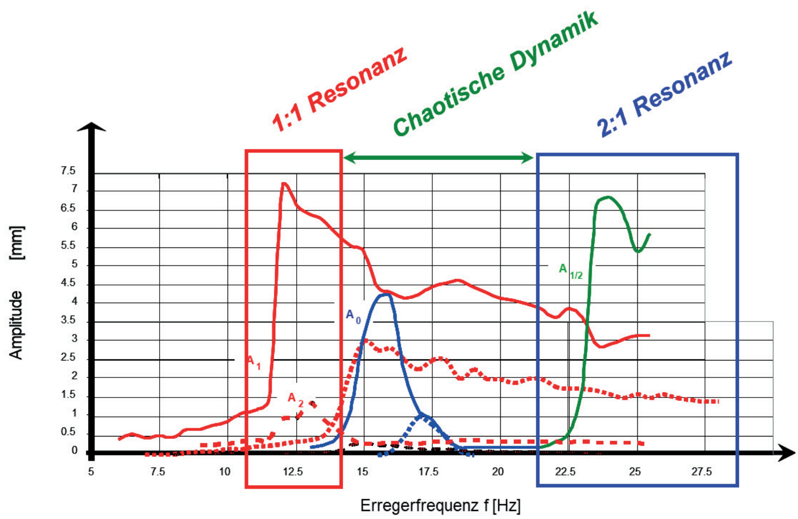

Abbildung 4. Gemessene Schwingungen eines Walzenzugs mit teilweise mitschwingendem Chassis, welches als Tilger wirkt. Die Messungen wurden auf sandig-kiesigem Boden in Spanien durchgeführt A_i bezeichnet die Amplitude des Schwingungsanteils mit der Frequenz i•f, wobei f die Erregerfrequenz bedeutet.

Die praktischen Messungen haben auch gezeigt, dass seine solche Dynamik die Maschinenmasse maximal ausnutzt, im Gegenzug aber eine schnelle und präzise Regelung der Maschinendynamik erfordert. Da durch die nichtlineare Dynamik sehr hohe Leistungen im Maschinen-Boden-System auftreten, ist die Vermeidung chaotischer Betriebszustände augenblicklich durch Anpassen der Erregerfrequenz zu unterbinden (Anderegg und Recher 2010). Je höher die resultierende Nichtlinearität der Dynamik, umso wichtiger wird die Regelung der Frequenz! Theoretische Grundlagen zur Regelung stark nichtlinearer und chaotischer Schwingungssysteme finden sich in (Kapitaniak 1996). Diese damaligen theoretischen Ansätze lassen sich vortrefflich auf die Anwendungstechnik der dynamischen Bodenverdichtung applizieren.

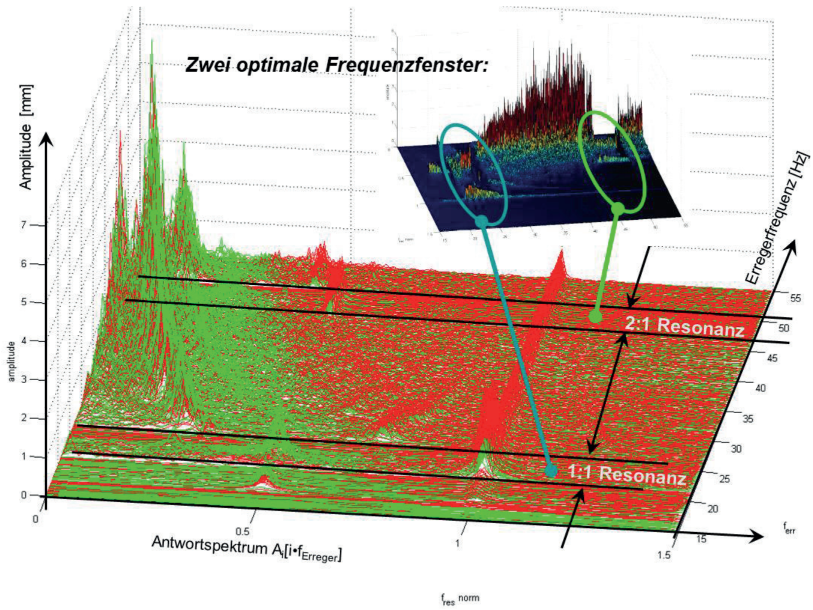

Abbildung5. Übersicht über das Frequenzspektrum eines Walzenzugs in Abhängigkeit der Erregerfrequenz der Maschine. Gut zu erkennen ist das Schwingungssignal mit der Anregungsfrequenz $f_{Erreger}$, sowie die 1:1- und die 2:1-Resonanzzone. Diese beiden Frequenzbereiche sind stabil, alle andern entsprechen chaotischem Verhalten.

Die Abb. 5 zeigt eine Systemsimulation der Maschinen-Boden-Interaktion eines Zweimassenschwingers. Die Dynamik ist stark nichtlinear, bzw. chaotisch, lediglich die angegebene 1:1- und 2:1-Resonanzzonen eignen sich für den praktischen Betrieb. Eine Regelung der Maschinenparameter ist dabei immer erforderlich.

3.2 Eine Klassifikation der Verdichtungsgerätetechnik

Aufgrund der gemachten Erfahrungen mit stark nichtlinearen Schwingungssystemen wurde eine stark vereinfachte Klassifikation von Verdichtungsgeräten definiert. Dabei wird der Quotient aus Zentrifugalkraft und Eigengewicht als Mass der zu erwartenden Nichtlinearität genutzt. Je höher diese dimensionslose Grösse (vgl. Gleichung (2)), umso starker die zu erwartende nichtlineare Wirkungsweise.

$$\text{"Leistungsdichte"} := \frac{m_u r_u\ \omega^2}{(m_f + m_d)g} \qquad (2)$$

Die Abb. 6 zeigt auf, dass seine implizite Leistungsgrenze bei heutigen Verdichtungsmaschinen existiert, die durch eine autoparametrisch wirkende Maschinentechnik hin zu grösseren Leistungen

durchbrochen werden kann. Diese Innovation, diese Leistungssteigerung, erfordert aber in jedem Fall die Verwendung einer sehr leistungsfähigen Regelungstechnik, insbesondere der Erregerfrequenz, um die Schwingungsstabilität bei veränderlichem Untergrund zu gewährleisten.
Diese Verdichtungstechnik kann wie ihre Vorgänger durch die kontinuierliche Verdichtungskontrolle FDVK, z. B. mittels Steifigkeitsmessung, ergänzt werden.

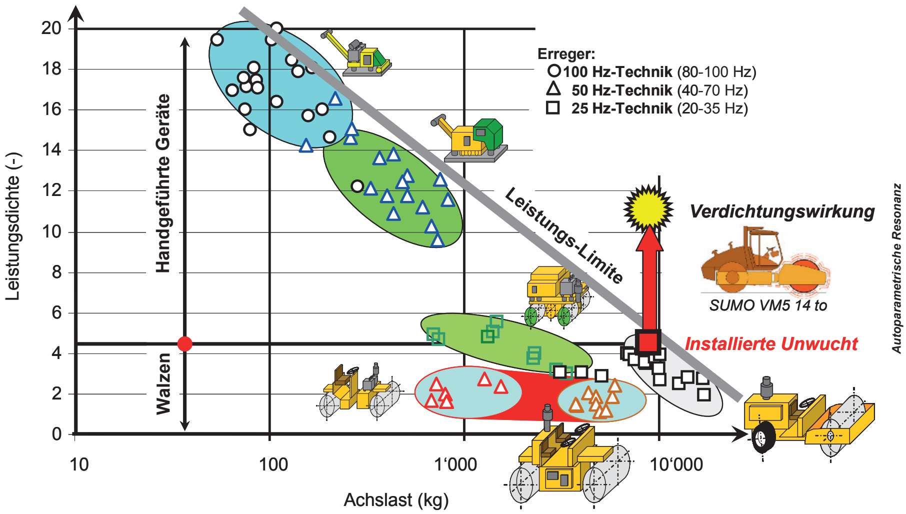

Abbildung 6. Klassifikation der Verdichtungsgeräte, dargestellt in halblogarithmischem Massstab. Die "Leistungsdichte" bezeichnet den Quotienten von Zentrifugalkraft und schwingender Masse (in kN).

4 KÜNFTIGE ENTWICKLUNGEN IN DER VERDICHTUNGTECHNOLOGIE

Die Verdichtungskontrolle und die mit ihr einhergehende Mess- und Regelungstechnik des Verdichtungsprozesses optimiert die einzelne Maschine. Die moderne Entwicklung der mobilen Automationstechnik und der damit eng verknüpfte Einsatz des GPS-Positionierungstechnologie erlaubt die Optimierung ganzer Fahrzeugflotten und verschiebt die Innovation in der Verdichtung vom einzelnen Arbeitsgerät hin zum optimalen Einsatz ganzer Fahrzeugflotten, gewissermassen eine Entwicklung hin zur "Baustelle 4.0"

4.1 Die digitale Baustelle

Abb. 7 zeigt den systematischen Aufbau einer vernetzten Verdichtungsgeräteflottentechnologie, die auf Basis einzelner geregelter Verdichtungsmaschine, welche den Verdichtungszustand in situ messen, ein Gesamtbild einer Baustelle generieren.
Die drahtlose Kommunikation zwischen den Maschinen ermöglicht die hierarchische Vernetzung untereinander und basiert damit auf dem klassischen Pyramidenmodell, wie es in der industriellen Automation verwendet wird. Diese Technologie lässt sich kombinieren mit einer digitalen Baustellenplanung, welche es erlaubt, den Arbeitsfortschritt eines Verkehrswegs in situ zu überwachen und allfällige Anweisungen an die Maschinenführer direkt online zu übertragen. Sehr aktuell ist die Entwicklung von Optimierungsalgorithmen für die optimale Bahnplanung des Walzeneinsatzes auf einer Baustelle (Haliti 2018).
Abb. 7 zeigt auf, dass die aktuellen Entwicklungstendenzen in der mobilen Automation den Prinzipien der klassischen industriellen Automation folgen. Jede Maschine erhält vor ihrem Einsatz den zu bearbeitenden Arbeitsraum und die zu erreichende Verdichtung (mittels FDVK überprüft). Die Verdichtungsdaten der Maschinen werden an die Leitzentrale übertragen und zu einem Gesamtbild des Baufortschritts zusammengefügt. Der Bauführer kann in der Folge notwendige Anpassungen und

Optimierungen direkt digital an die einzelnen Maschinen übertragen. Andererseits sind der Baufortschritt und die termingerechte, überprüfbare Verdichtungsqualität jederzeit abrufbar, bei Prozessende kann das Resultat an den Auftraggeber übertragen und die Verrechnung ausgelöst werden.

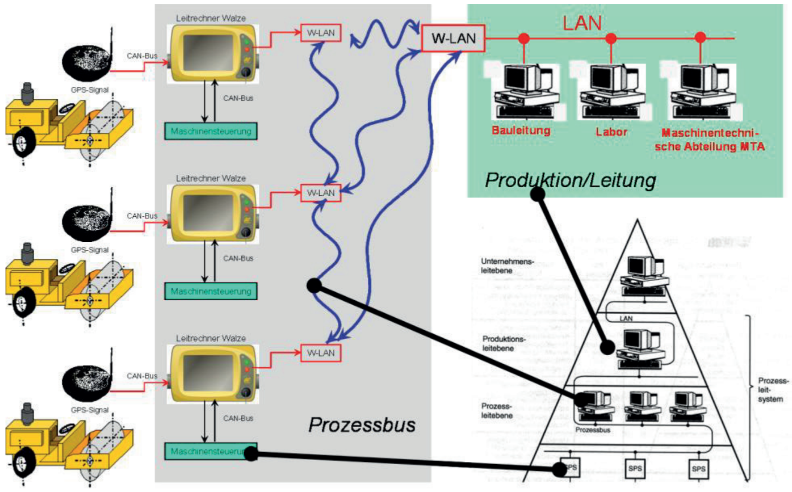

Abbildung 7. Zukunft der Verdichtungstechnologie – Maschinen warden vernetzt, ein modernes Bauunternehmen wird zu einer digitalen Fabrik – Strassenbau 4.0

5 SCHLUSSFOLGERUNGEN

Zusammenfassend kann gesagt werden, dass in zwei Richtungen die Leistungsfähigkeit von Verdichtungsgeräten vorangetrieben werden kann. Zum einen kann die Verdichtungswirkung einer einzelnen Maschine maximiert werden durch die Nutzung einer stark nichtlinearen Dynamik. Dies bedingt parallel eine sehr leistungsfähige Mess- und Regelungstechnik, welche die nichtlineare und chaotische Wirkung begrenzt in der Praxis. Zum andern eröffnet die Kommunikationstechnik zwischen den Fahrzeugen und der Einsatz des GPS und der digitalen Bauplanung die Möglichkeit, den Verdichtungsprozess mehrerer Maschinen in situ zu überwachen und optimieren. Letztere Technologie eröffnet zudem die Möglichkeit, den Verdichtungsprozess vollständig zu überwachen und zu dokumentieren. Dies dürfte mit der voran schreitenden Digitalisierung und den laufend steigenden Anforderungen an die Bauqualität eine ebenfalls wichtige Funktionalität darstellen.

VERDANKUNGEN

Die Untersuchungen zur Nutzung der nichtlinearen Dynamik bei Vibrationsverdichtern wurden zusammen mit dem Industriepartner, der Ammann Group in Langenthal mit finanzieller Unterstützung der Kommission für Technologie und Innovation (KTI, Bern) durchgeführt.

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Estimating stiffness in layered and spatially heterogeneous ground

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Keywords: Continuous Compaction Control; Layered Ground; Elastic Modulus

1 INTRODUCTION

Vibratory roller-based measurement of soil properties during and after compaction, an approach termed Continuous Compaction Control (CCC) or Intelligent Compaction (IC), has gained considerable momentum in the United States. One limitation of current CCC technology is that the estimated soil stiffness provided by CCC on vibratory smooth drum rollers (12-15 ton) is a composite measure of ground stiffness to a depth of 1.0-1.2 m (Rinehart & Mooney 2009). This is considerably greater than a 15-30 cm thick layer or lift of subgrade, subbase or base material typically used in the US. While a composite measure of stiffness is informative, it does not provide a measure of layer elastic modulus/stiffness that is prescribed in design and performance-based construction specifications. A methodology was developed to extract layer elastic modulus/stiffness from composite soil stiffness and GPS-based position provided by currently available vibratory IC rollers (Fig. 1). The research to develop this methodology was carried out over a ten year period culminating in a series of publications listed at the end of this abstract, in addition to many conference proceedings papers not listed due to space constraints.

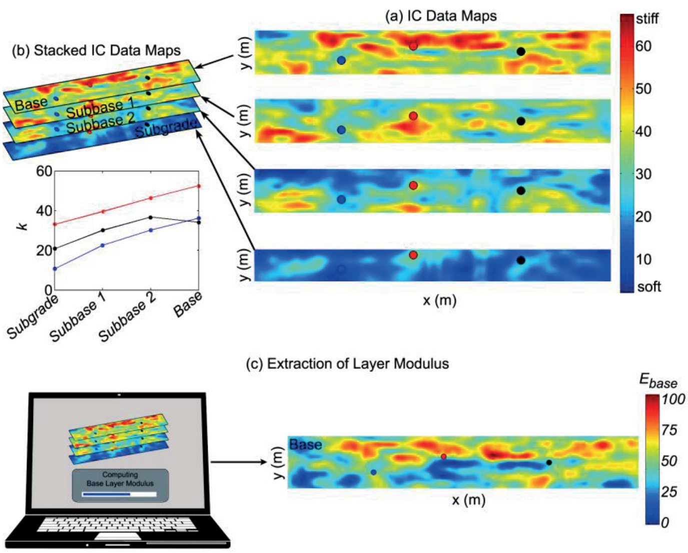

Figure 1. Measurement depths of IC roller and common field tests.

2 INVERSE ANALYSIS OR BACKCALCULATION

The developed methodology combines forward modeling and inverse analysis. Forward modeling involves the estimation or prediction of roller-measured composite stiffness values k for ranges of layer elastic moduli and layer thickness combinations expected in practice. Forward modeling can be physically-based, e.g., lumped-parameter, finite element, or boundary element modeling; it can be statistically-based; or it can be a combination of both. In the lumped parameter approach, the soil is modeled as a discrete mass-spring-dashpot element. This approach has been successful in 'lumping' all of the ground into a single object and in characterizing dynamic drum interaction. A number of excellent publications on these topics by Messrs, Anderegg, Adam, Kröber, Selig, and others, are cited within the references listed at the end of this abstract. However, this approach is incapable of modeling individual earthwork layers and their contribution to IC roller response. Further, this approach cannot model the variable width contact problem between the drum and soil surface that is critical to correctly capturing layered soil response. In this investigation, continuum-based finite element (FE) analysis and boundary element (BE) analysis were employed to physically model individual layers and the nonlinear curved drum interaction with the ground surface (Fig. 4a).

Inverse analysis, also called backcalculation, works in reverse and provides an estimate of individual layer elastic modulus E_i using sets of IC data maps, i.e., k and layer thickness from stacked data sets (as shown in Fig. 1). The back-calculation of layer elastic modulus E_i for an n-layer system progression that follows a bottom-up construction process, beginning at the bottom layer n. For pavement earthwork, layer n is the subgrade soil and is assumed to be at least 1-2 m thick and therefore behaves as a semi-infinite half-space. In this case, we can create a forward model (f_n) which relates the modulus of the bottom layer (E_n) to the roller measured stiffness (k_n) on the bottom layer.

3 DATA ANALYSIS AND RESULTS

3.1 Field Data

One set of data used in the process to calibrate/validate the forward model is shown here. Further data is presented in a number of the references, including Mooney & Facas (2012). The data was collected with an instrumented Sakai SV 510D smooth drum roller. On top of the granular subgrade, a 20 cm layer of stone base material was first placed and compacted. Then, a 30 cm layer of stone base material (for a total of 50 cm overlying layer) was placed and compacted. The contact force – drum displacement (F_c - z_d) response from a low amplitude pass on each layer at the same spatial coordinates is shown in Fig. 2. Independent density and stiffness testing revealed that each stone base layer reached similar levels of compaction. As shown, the 20 cm and 50 cm thick layers of stiffer base course have a significant effect on the F_c - z_d response. Here, the F_c - z_d response reflects the composite two-layer system. F_c increases significantly with base course thickness, z_d increases slightly, and the composite stiffness k increases appreciably.

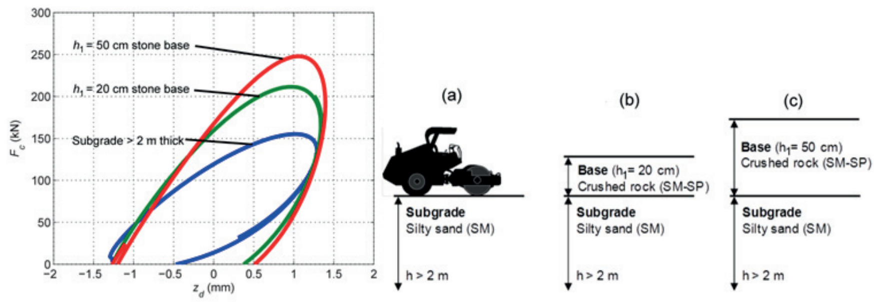

Figure 2. Field measured F_c - z_d loops from IC roller on vertically homogeneous subgrade soil

3.2 Two layer Inversion

Here we illustrate the back-calculation of layer modulus for a two-layer system (Fig. 3). Three key parameters dictate the roller response for a two layer system: the top layer elastic modulus (E_1), the thick bottom layer elastic modulus (E_2), and the thickness of the top layer (h_1), which is determined from GPS measurements. E_2 is presumed known because it was determined from inversion of IC results from layer 2 on down.

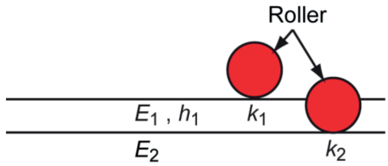

Figure 3. Two-layer system used for inverse analysis

The required nature of the back-calculation approach was determined through investigation of the roller-layered soil response. The forward model-predicted k was simulated over a wide range of E_1, E_2 and h_1 consistent with what is observed in practice. The k values vs. E_1 and E_2 are plotted for h_1 =15, 30 and 50 cm in Fig. 4. The results are physically intuitive, e.g., k increases with increasing E_1 and E_2. All of the contours exhibit monotonically decreasing k from left to right as E_2 decreases. Fig. 4 conveys the uniqueness of k in that there is only one E_1 - E_2 combination that produces a measured k for a given h_1. Additionally, as h_1 increases, a lower E_2 is needed to maintain the same k_1 for fixed E_1 (for $E_1 > E_2$). This implies that as h_1 increases, the top layer has a larger influence on k_1. The gradients or partial derivatives of the Fig. 4 contours reflect the relative influence of each parameter on k. These are shown and described in Mooney & Facas (2012).

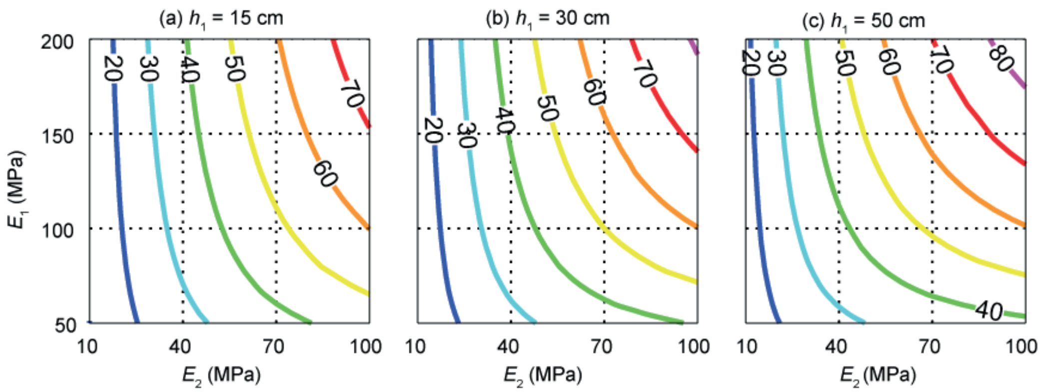

Figure 4. Simulated k values (MN/m) for (a) h_1 = 15 cm (b) h_1 =30 cm and (c) h_1 = 50 cm

Fig. 5 presents the model-determined k vs. E_1 for various values of E_2 and h_1. The relationships are monotonically increasing functions, implying that the inversion should be unique, i.e., for each k there exists exactly one value of E_1 for given E_2 and h_1. Thus, there are no local minima or maxima when performing the inversion. Based on these findings, simpler minimization algorithms such as a root finding algorithm can be used. More complex minimization techniques such as genetic algorithms are not required.

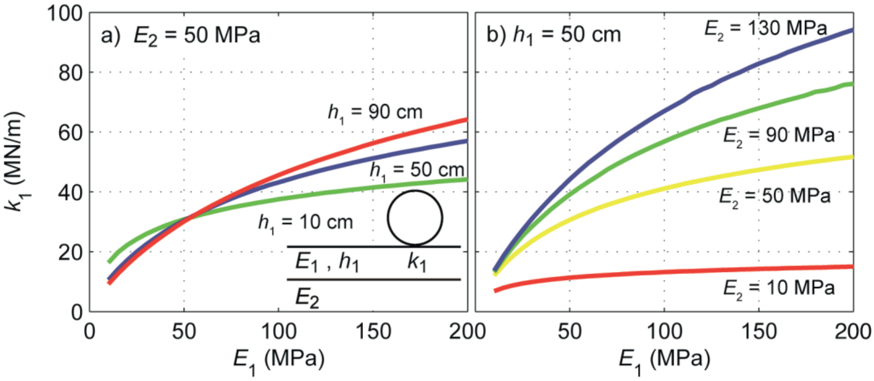

Figure 5. Model-simulated k vs E_1 for a two-layer system for: (a) variable h_1, (b) variable E_2

4 CONCLUSIONS

A methodology was developed to extract layer elastic modulus/stiffness from currently available CCC / IC data, i.e., composite soil stiffness and GPS coordinates from vibratory IC rollers. Forward model results matched relatively well with available experimental data. Inverse analysis was pursued with a traditional gradient approach that proved successful but time-intensive. The developed methodology can be implemented in practice through software algorithms. These algorithms can be integrated directly into the IC equipment on-board an IC roller, within office software that analyzes IC data, or both. The methodology generated is generic and can be applied to any currently-available proprietary measures of ground stiffness from vibratory rollers. The methodology developed from this investigation will allow all transportation stakeholders (owners, consultants, contractors) to directly evaluate the elastic modulus of individual lifts or layers using vibratory IC roller data.

ACKNOWLEDGEMENTS

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Anniversary Symposium – 40 Years of Roller Integrated Continuous Compaction Control (CCC)
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Derivation of the permissible spatial variation of CCC data from the requirements for driving comfort

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Keywords: CCC data; driving comfort; soil; spatial variation; cmv; dynamic traffic loads

1 INTRODUCTION

Spatial variations of soil properties occur even within nominally homogenous soil layers. In order to perform a reliable geotechnical design, it is necessary to consider the local variance of the soil properties. A variation of the soil stiffness leads to spatially variable settlements which imply increasing dynamic forces on passing vehicles (Mitschke 1984). Higher dynamic forces result in additional settlements and simultaneously reduce driving comfort. This process continues steadily until a repair of the pavement is required. Thus, the evenness of the pavement, soil stiffness and driving comfort are related directly. The aim of this paper is to provide a numerical model which allows modeling the long-term evenness of pavements due to passing vehicles taking into account a spatial variation of the subsoil stiffness. From this numerical model conclusions on the driving comfort requirements are drawn regarding the permissible spatial variation of Continuous Compaction Control (CCC) data.

2 ROAD UNEVENNESS

Road unevenness can be differentiated into geometric imperfections and settlements due to the deformation of the subsoil. Due to the unevenness of the pavement, dynamic loads on passing vehicles occur depending on the velocity of the vehicle and the amplitudes of spectral road unevenness (Grabe 1992). Measurement data of road unevenness reveal that the amplitudes of spectral road unevenness are proportional to the inverse of the angular frequencies Ω of wave length. Hence, this characterizes an unsteady statistical process called random walk (Papoulis 1965). The unevenness *u* can be characterized by one of the factors such as the spectral unevenness Φ_h (Ω=1). With increasing number of passing vehicles *N* the spectral road unevenness is shifted to higher amplitudes. Hence, the spectral unevenness density is a function of the number of passes, see Eq. (1)

$$\Phi_h\,(\Omega=1) = f(N). \tag{1}$$

Additionally, this function depends on the spatial variation of the initial soil stiffness and the type of dynamic loading. Hence, knowing this function allows the derivation of the permissible spatial variation of the soil stiffness after compaction which can be measured, for instance by the compaction meter value *cmv* (Grabe 1992).

3 SUBSOIL

From CCC data the spatial variation of the subsoil stiffness can be derived (Grabe 1993). Comparing the local variation of soil stiffness measured in terms of compaction meter value and the road unevenness in terms of spectral density, it can be concluded that both graphs reveal the same characteristic. This leads to the assumption that the spatial variation of soil stiffness is transferred to the road unevenness with increasing number of passing vehicles (Grabe 1994).

4 DRIVING COMFORT

The driving comfort is significantly influenced by the road unevenness, driving velocity and the vehicle dynamics. Humans do not asses the influence of a given vibration solely by its intensity, for instance acceleration amplitude. They perceive vibrations of different frequency but of same intensity, differently. Thus, the evaluation of driving comfort is based on a subjective frequency-dependent perception. Humans are most sensitive in the range of frequency $f = 4 - 8$ Hz (Mitschke 1984).

5 SIMULATION MODEL

In order to derive the permissible spatial variation of CCC data with respect to driving comfort requirements a numerical model is presented. First, the unevenness of a young pavement and the spatial variation of the soil stiffness after compaction are modelled numerically. Second, the effect of dynamic forces from passing vehicles on the long-term evenness of the pavement is simulated. The soil behavior is described using a simple constitutive law assuming oedometric conditions.

5.1 Modeling of Pavement

The unevenness of the pavement in the initial state u_0 is modelled by adding uncorrelated random independent identically distributed samples $a_i \sim N(O, \sigma^2)$, i. e., a_i is sampled from a Gaussian distribution with zero mean and variance σ^2

$$u_{0(i)} = a_i. \tag{2}$$

The unevenness of the young pavement can be seen in Fig. 1, top left. The considered road section has a length of 1000 m. The corresponding frequency spectrum is a straight horizontal line.

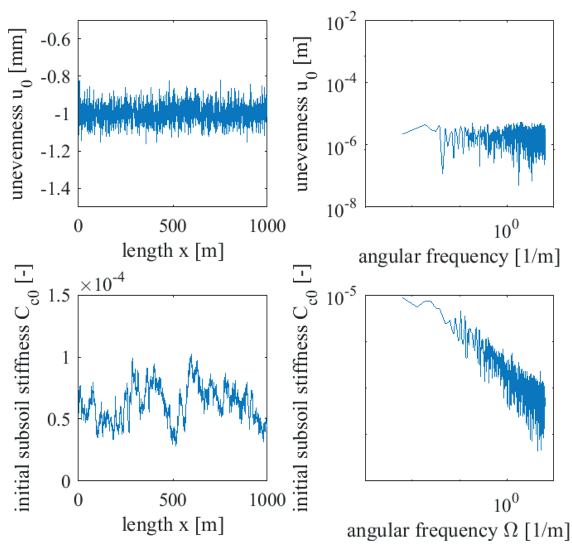

Figure 1. Unevenness u_0 and subsoil stiffness C_{c0} in initial state (left) and as a function of angular frequency (right).

5.2 Modeling the Subsoil
The soil is modelled using a simple constitutive law for oedometric conditions:

$$u_{res} = u_0 - C_{c0} \cdot B \cdot \ln(1+N) \cdot h \tag{3}$$

wherein N is the number of load cycles and B is a factor taking into account the effect of increasing dynamic loads with developing long-term unevenness

$$B = P_{dyn}/P_{dyn0}/100. \tag{4}$$

It is assumed that the settlements are influenced by dynamic loads in the upper part of soil layer up to height of $h=1$ m. The initial stiffness of the subsoil C_{c0} is modelled similar to the initial unevenness of the young pavement by adding correlated independent identically distributed random samples a_i:

$$C_{c0(i)} = \phi \cdot C_{c0(i-1)} + a_i, \tag{5}$$

wherein $\phi = 0.99$. The spatial variation of the subsoil stiffness coefficient and the corresponding frequency spectrum are presented in Fig 1, bottom left and right. The frequency spectrum of the subsoil stiffness shows the $1/\Omega$ characteristic, described in Grabe and Mahutka 2005.

5.3 Vehicle Model
The vehicle model consists of a two mass spring-dashpot-system, see Fig. 2. The mass of the truck is $m_1=3.5$ t and of the wheel $m_2=0.8$ t. The spring stiffness is $k_1=430$ kN/m and $k_2=2000$ kN/m. The damping coefficients are $c_1=25.5$ kNs/m and $c_2=6.4$ kNs/m (Mitschke 1984).
The dynamic load P_{dyn} is determined by solving the differential equation for the spring-dashpot-system

$$\boldsymbol{P}_{dyn} = \boldsymbol{M} \cdot \boldsymbol{a} + \boldsymbol{C} \cdot \boldsymbol{v} + \boldsymbol{K} \cdot \boldsymbol{u}, \tag{6}$$

wherein **M**, **C** and **K** are the mass, damping and stiffness matrices and ***a***, ***v*** and ***u*** are the acceleration, velocity and displacement of the vehicle. Knowing the dynamic force acting on the pavement the resulting settlements due to loading and unloading can be determined as described in section 5.2.

6 RESULTS

The long-term unevenness as well as the spatial variation of the initial soil stiffness can be seen in Fig. 3(a). With increasing number of passing vehicles the evenness of the pavement approaches the stiffness distribution of the subsoil. The associated distribution of the spectral unevenness density is shown in Fig. 3(b). The obtained curves are mostly below the range of four to eight hertz. Hence, the comfort criterion defined by Mitschke 1984 is met. With increasing number of passing vehicles the lines are shifted to higher amplitudes. As the unevenness of the pavement rises continously, compare Fig. 3(c), it can be expected that with further passing vehicles and ongoing unevenness development

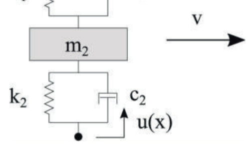

Figure 2. Vehicle model consisting of a two mass spring-dashpot-system.

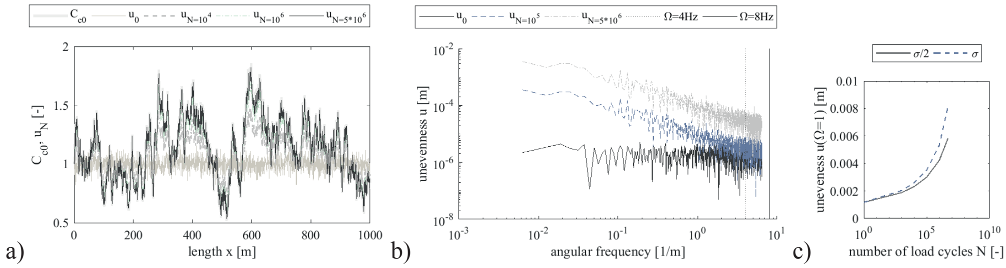

Figure 3. a) Longterm-unevenness approaching the initial soil stiffness (scaled). b) Unevenness *u* as a function of angular frequency for different load cycles. c) Unevenness *u* as a function of load cycles *N* (Ω=1) varying the initial compaction of the soil foundation in terms of the variance.

the lines of the spectral unevenness density will be shifted into the frequency range in between of four to eight hertz and therefore driving comfort quality will be reduced significantly. In light of the spatial variation of CCC data, the effect of a systematic compaction of the subsoil on the long-term unevenness was investigated, by creating a subsoil stiffness distribution with halved variance $\sigma^2/2$.The result can be seen in Fig. 3c). Compaction at variance of the subsoil foundation lead to a slower development of unevenness and therefore reduces dynamic forces on passing vehicles. This results in an enhanced driving comfort quality and also extends the serviceability of the pavement.

7 CONCLUSION

Unevenness of the pavement leads to an increase of dynamic forces on passing vehicles and consequently to a loss of driving comfort. Therefore, a numerical model has been developed for modeling the long-term evenness of pavements considering spatial variations of the subsoil stiffness. The numerical model is based on a simple constitutive law and takes into account the influence of dynamic traffic loads due to passing vehicles. The results indicated that the long term quality of the pavement depends essentially on the spatial variation of the soil stiffness. Hence, initial compaction of the soil foundation leads to a slower development of unevenness and thereby enhances the long-term driving comfort quality. Hence, a need exists to derive compaction criteria in order to meet the requirements of driving comfort reliably. The presented model needs to be verified against in-situ measurement data. Additionally, the model will be extended to use an advanced constitutive model capable of modelling soil behavior under cyclic loading.

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Anniversary Symposium – 40 Years of Roller Integrated Continuous Compaction Control (CCC)
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How can the use of CCC influence our understanding of soil behaviour during compaction?

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Keywords: CCC; marginal materials; quality control; soil compaction

1 INTRODUCTION

Slovenia, a member of EU since 2004, has fairly small surface – 20,273 km^2 and 2 million inhabitants. Due to the morphological and geological diversity, the construction of motorways and railways require many deep cuts, high embankments and numerous structures such as viaducts, tunnels, etc.

Internal analysis performed in the third biggest construction company in Slovenia, CMC Celje, in the early 1990ies showed that 30% of premature failures of asphalt pavements origin from the improper behaviour of unbound base and subgrade, although the QA – QC tests, performed during construction, confirmed the required quality of material and compaction. The reasons for the premature pavement failures that originated from the unbound base and subgrade could be divided in some typical groups:

– Softening of fill material in embankments, constructed from flysch type rock masses, clay bearing soft rocks and highly over-consolidated clays due to permanent changes in water content. The migration of fines was observed from the subgrade towards to the unbound upper layers due to the permanent increase of water content
– Loss of stiffness of natural gravel in low embankments and unbound subbase under the traffic loads due to the abrasion of metamorphic rock silicate particles and the increase of mica fines (phenomena explained well in Brandl, 2001)
– Heterogeneity of stiffness in cases when the excavated rock from karst limestone and dolomites, mixed with terra rossa, was used as fill material.

To reduce the shortcomings in front of the huge National Motorways Construction Program, several advanced steps were proposed to improve the quality assurance programme and new test methods for quality control were introduced for the first time, like:

- the use of CCC (Continuous Compaction Control),
- the use of repeated triaxial load test to study the gravel behaviour under the dynamic loading
- the use of Soil Water Retention Curve to improve the understanding of softening in marginal materials, like highly over-consolidated clays and flysch type rock masses.

In the period 1994 – 2018, different types of rollers with various systems of CCC were used in Slovenia at large earthwork jobsites. However, the method has not been accepted yet as a part of a regular quality control method for earthworks, which consists of reference Proctor density and minimum required relative compaction (D_{pr}) calculated from nuclear gauge dry density, and static and dynamic plate load test. Even more, we are facing quite loud opposition against the use of CCC from different parties included in Quality assurance programme. The paradox is that the use of CCC was strongly supported by the designers and investors (clients), where a huge mass of marginal materials was available for earthworks and the use of good quality fill material from borrow pits would bring huge additional costs as well as construction delays. Three such case histories in the past 20 years are presented. Some specifics that do not go well with the common description of CCC calibration results and expectations are emphasized.

2 CASE HYSTORIES

In all case histories vibratory roller Bomag BW 216 D was used and the calibration was performed by comparing CCC dynamic measuring values Evib with light falling weight device dynamic modulus E_{vd} and/or dry density (ρ_d) measured by nuclear gauge. Multiple point measurements were made for the same measuring point.

2.1 Low motorway embankments at Dolga vas

The embankments were constructed from round clean gap graded Mura river gravel excavated in gravel pits below ground water table. The gravel was too wet for compaction and drying was performed by spreading it at the construction site. Previous experiences with Mura gravel showed problematic compaction due to variations in water content, as well as its poor performance as subbase.

Four different calibration attempts with same material are presented in Fig. 1, left. A shift to higher E_{vd} values is visible through time indicating the changes in material gradation, water content and saturation. The 3rd test field was also compacted after 24 hours of rest. There was an increase of E_{vd}, but no increase of Evib was detected. On the same test field, the height of the embankment was decreasing with track number and on 4th track the results of Evib were heavily influenced by soft sediments deposited near river Lendava. E_{vd} did not detect any soft soil below the embankment.

The test fields confirmed problematic compaction of Mura gravels and non-homogeneities of compacted layers. No Evib-E_{vd} calibration was possible. It seems that CCC confirmed previous finding that low embankments on soft ground do not exhibit proper stiffness, although E_{vd} (or E_{v2}) measurements were good enough. The impossibility of calibration additionally confirms that round Mura gravel is not suitable for subbase due to inadequate homogeneous stiffness of compacted layer. Partial crushing and modification by cement were proposed for the subbase on real construction sites.

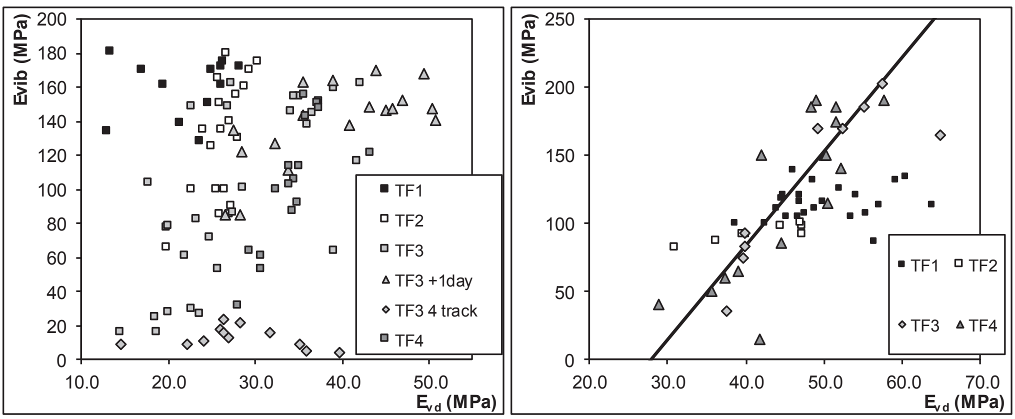

Figure 1. Results from test fields at Dolga vas (left) and PSP Avče (right).

2.2 PSP Avče

For the construction of Pumped-storage plant (PSP) Avče app. 0.5 MIO m^3 of fill material was needed for the accumulation. The only available fill material at the construction site was flysch material consisting of marlstones, claystones and calcarinite. The material had non-homogenous gradation with maximum grain size of 100 mm and occasional oversize boulders.

Two regimes of compaction were used for calibration. Large amplitude of vibration (1.9 mm) on the 1st and 2nd test fields, and small amplitude (1 mm) on the 3rd and 4th test fields (Fig. 1. right). It was difficult to select points for E_{vd} measurements due to open structure of the compacted layer. Scatter in the results of E_{vd} measurements was large. At the first two test fields, where compaction with large amplitude was used, the overall range of Evib was smaller, if compared with test fields, where compaction using small amplitudes was used. On the 1st and 2nd test fields calibration was not possible in these cases, whereas on the 3rd and 4th test fields the calibration was possible.

CCC was recognised as relevant quality control for the first time in Slovenia. During quality control, the CCC was found to be a good indicator of weak clayey zones.

2.3 HPP Brežice

A temporary dike was constructed to protect deep excavation for power house foundation against flooding. The fill material for the dikes was a mixture of gravel and sandy silt/silty sand mixed in mass ratio 4:1. The compacted mixture had permeability lower than $1\ 10^{-6}$ m/s and no sealing layer was installed. The designer of the HPP Brežice decided to use CCC in order to improve the homogeneity of the dikes.

At the presented test field, the first layer was prepared by loading gravel from the left and then silty sand from the right on a truck, which was then unloaded at the location and then spread longitudinally by bulldozer. For the second layer, the previous procedure was followed by ploughing until homogenous mixture was obtained.

The 1st layer exhibited clear relationship between Evib-E_{vd} or Evib-ρ_d (Fig. 2, left) as expected and the test filed seemed to be successful. Upon a closer inspection of the compacted layer, material segregation was observed as a result of spreading method. Based on nuclear gauge measurements, siltier material, which was compacted at water contents above optimal water content, had after compaction almost full saturation. It seems that excess pore pressure prevented further compaction, reduced layer stiffness (Evib < 15 MPa), while pumping of fines to the surface impeded the measurements of E_{vd}, as thin layer of "liquid" silt formed on the layer surface. The laboratory measurements performed on samples taken at different locations of the 1st layer, confirmed that Evib was directly influenced by fines content and/or water content (Fig. 2, right). At fines content between 14% and 20% or at water content between 8% and 11%, the behaviour of material changes, because there is a sudden decrease of soil stiffness. It seems that at higher fines content the stiffness is controlled by fines. The 2nd layer was more homogenous due to ploughing before compaction. The water content and the fines content were similar to those in the 1st layer, when there is a jump in stiffness. High scatter in E_{vd} and low values of both E_{vd} and Evib made the calibration impossible.

3 CONCLUSIONS

The use of CCC in the presented case histories shows:

1. In most of the marginal materials used as fill materials, the CCC calibration was not possible.
2. There could be a large scatter in point measurements around the same measuring point.
3. CCC excellently detected the influence of fines and water content to layer stiffness of compacted silty gravel. Although the material is recognised as silty gravel (GM, fines content 7-17%), it behaves as silt during compaction.
4. The 5% increase of average Evib of track could be used as a valid criterion for gravels used as a fill material. In silty gravel, this criterion was not always valid.
5. At low values of Evib, the criterion of 5% increase could not be met.
6. CCC excellently detected weak areas in embankment compacted from coarse grained crushed flysch bedrock. By using CCC, the total settlements of the 23 m high embankment were less than 2 cm or less than 0.1% of embankment height.

7. In karst region, where coarse mixture of dolomite with clayey matrix was used, CCC was the only tool that definitely showed improvement of properties due to compaction.

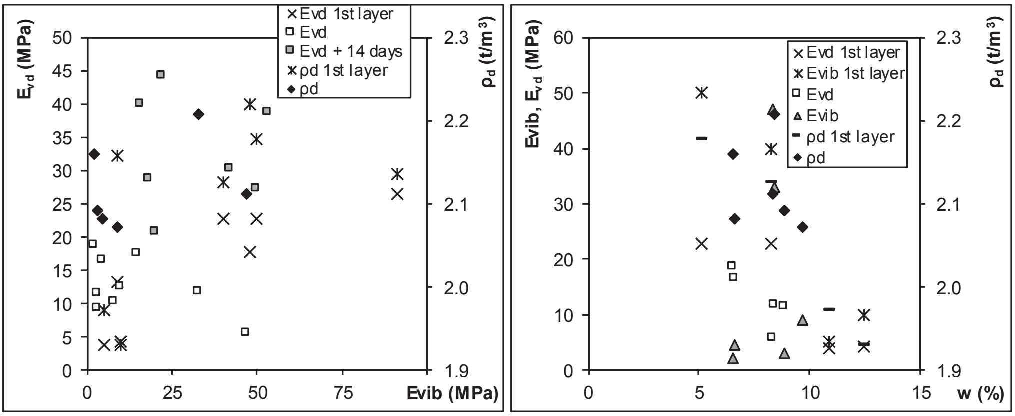

Figure 2. Field test results at test filed for HPP Brežice temporary dikes.

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Anniversary Symposium – 40 Years of Roller Integrated Continuous Compaction Control (CCC)
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Extended 2nd Proctor Lecture: Compaction improvements from an industry 4.0 perspective

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Keywords: CCC; Intelligent Compaction; Compaction Improvement

1 INTRODUCTION

Compaction is one important action applied on geomaterials (soils, rock, mixtures of soil and rocks, stabilised soils and unbound and bounded granular materials) by different technologies to achieve design target values assuring the desired performance of earth structures and structural layers in pavement and railways structures (Gomes Correia 2018). Fundamentals to explain phenomena of compaction curve supported by microscopic observations are presented allowing a conceptual framework for a systematic evaluation of microstructural effects on measurable macroscopic engineering variables, such has elastic stiffness, strength, compressibility, yielding behaviour and permeability (Alonso et al. 2013). This can also support the recent practical approach developed by Tatsuoka and Gomes Correia (2018) for soil compaction control. These authors show quantitatively that the macroscopic engineering properties (mechanical and hydraulic) are all controlled by dry density ρ_d and "degree of saturation S_r at the end of compaction". As a standard method, they proposed to control the values of water content w and ρ_d in such that S_r becomes $S_{r,opt}$ (S_r when $\rho_{d,max}$ is obtained for a given compaction energy level (CEL)) while ρ_d becomes large enough to ensure soil engineering properties required in design fully taking advantage of available CEL provided by the compactors. As a consequence, an efficient and effective soil stiffness control during and after compaction related with design performance can only be assured if combining with the above method which means, combined with the control of degree of saturation. This corroborates the geotechnical engineering knowledge that compaction parameters are key index properties influencing performance based properties and consequently geostructures performance. These above aspects, as well as, an overview of laboratory and field compaction technologies used for earth structures and structural layers in pavement and railways structures were presented by the author during the 2nd Proctor Lecture and described in (Gomes Correia 2018). A special emphasis was addressed in their developments during the years and mainly addressing the link between laboratory and field developments in the framework of an integrated chain of engineering design. Additionally the use of soft computing in laboratory and field tests was introduced demonstrating the power of this tool to deal with big data (laboratory, field, monitoring) allowing the retrieve of existing data, predictive and discovery capacities, as well as, optimisation in helping decision making (Gomes Correia et al. 2013). An application was shortly presented for an optimization of earthworks (Parente et al. 2015).

2 CONCLUSIONS

This Lecture summarises some of the above aspects and conclude with the introduction of the concept of Industry 4.0, showing the potential of its application as a future framework for intelligent optimization of earthworks (Parente et al. 2018) and in particularly for intelligent compaction optimization operations. Indeed, this concept preconizes agile and autonomous decisions as a way to tackle volatile and unpredictable environments, such as the latter operations (see Fig. 1).

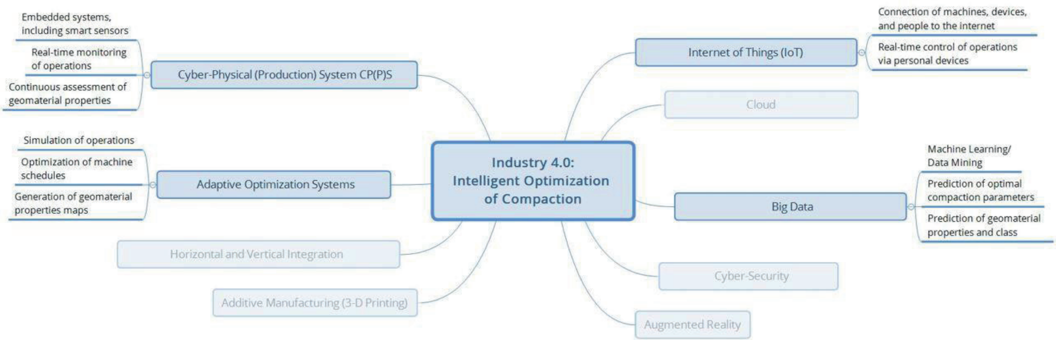

Figure 1. Optimization of the compaction process from an Industry 4.0 perspective

In order to achieve this, one of its main focuses is to leverage on technologies such as smart sensors to monitor the compaction process in real-time, thus generating a virtual counterpart of physical compactors in a simulation model, as well as to continuously asses the geomaterial properties (e.g. continuous compaction control). On the one hand, this comprises the basis for real-time scheduling and optimization of compactor tasks, while simultaneously processing the data associated with geomaterial feedback. On the other hand, the latter data can additionally be the target of machine learning and data mining algorithms, allowing not only for the prediction of optimal compaction parameters, but also for the classification of geomaterials according to their properties, which displays the potential for further optimizing and improving the quality of the compaction process. Finally, the entire process is supported by the connection of compactors, devices, and people to the internet, in line with the Internet of Things concept, which provides transparency, improves communication, and provides real-time control of operations from any worldwide location with internet access by resorting to any personal communication device (i.e. smart phones).

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Application | Interpretation | Classification Evaluation | Implementation of CCC

Anix GmbH: Precision electronic test equipment for road and railway construction: Static Plate load test, Planograf and Benkelman beam

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Key words: Anix; Plate load test; Planograf; Benkelman beam

1 ANIX GMBH

The company Anix GmbH is focused on the development and production of modern testing equipment for road construction and civil engineering. In the design of our test equipment, we pay attention to user-friendliness and reliability. Our team is personally there for your clients. Friendliness, speed and high quality of service are very important to us.

2 THE STATIC PLATE LOAD TESTER AX01a

Our static plate load test device AX01a (Figure 1) provides a simple and easy way to determine the bearing capacity and deformability of soils and layers without binders, e.g. anti-frost layers.

2.1 Measuring principle

The soil to be tested is gradually loaded under a load plate. The mean normal stresses and the deflections of the load plate are measured and then shown in a diagram as load-settlement lines. Our electronic box of the AX01a evaluates automatically the deformation modulus E_{v1} and E_{v2} according to DIN 18134:2001-09 – as key characteristics for the bearing capacity – and the ratio E_{v2}/E_{v1} – as key

Figure 1. Set up of the static plate load test device AX01a

Figure 2. Electronic box AX01a

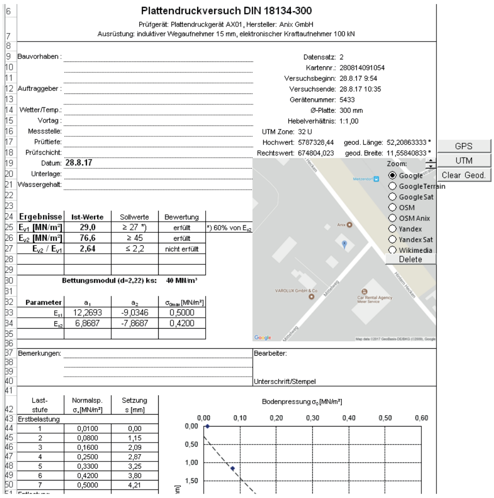

Figure 3. Section of the Excel sheet

characteristics for the compression. In addition to the DIN-standard, many other European Standards are supported (e.g. for Austria, Switzerland, Italy, Slovenia).

2.2 Water proof design

Our plate load test device AX01a is featured by its compact and robust design. The electronic box (Figure 2) is water proof. Thanks to the additional external buttons and after closing of the transparent cover, it can also be used under harsh environmental conditions, e.g. during rain. It has a built-in mini printer for printing of small test records on the construction site. The internal rechargeable battery can be recharged within 2 hours and allows continuous operation of up to 48 hours. The test data can be saved on a SD card and are available for further processing under Microsoft Excel®. In case the electronic box is equipped with the optionally available GPS module, a map section is also displayed on the Excel test report (Figure **3**).

3 THE PLANOGRAF AX02

The Planograf is used to capture the longitudinal profile of road surfaces (Figure 4). It is standardized in the test specification "TP Eben – Berührende Messungen" of the FGSV (“Technical paper – touching measurements” of the German Research Society for Road and Traffic).

3.1 Testing principle

The Planograf consists of a 4.5 m long rigid frame with 10 wheels, a support wheel and a centrally located measuring wheel. The measuring wheel detects the deviation of the road surface profile from the 4 m virtual zero line formed by the 10 wheels. If a limit value (of typically 4 mm) is exceeded, a signal sounds and the highest deviation of the profile of the corresponding lane section is included in the deduction calculation.

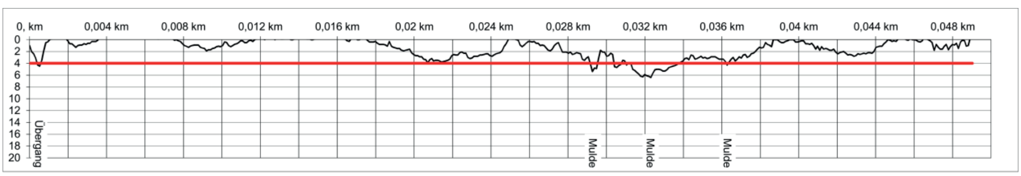

Figure 4. Planograf AX02: longitudinal profile, with comments, red limit line at 4 mm

3.2 Can be separated into three parts

The special feature of our Planograf AX02 (Figure 5) is: it can be disassembled into three parts for easy transportation. The three parts – when put together – require very little space (Figure 6) and fit into a minibus, space wagon or kombi car.

The key features of the electronic box correspond to those of the AX01a. The longitudinal profile is saved on the SD card. During the measurement, it is possible to comment critical points of the longitudinal profile (e.g. „through“, „dirt“, „special road installation“). These comments are user-configurable. After the measurement, a short report can be printed out via the built-in mini-printer. For the creation of test records and the further evaluation of the measurements Microsoft Excel® is used, as with the plate load test device.

Figure 5. The Planograf AX02

Figure 6. Planograf, disassembled put together

4 THE BENKELMAN-BEAM AX06

In response to frequent customer demand Anix GmbH has re-launched the Benkelman beam this year.

4.1 Newly developed

The Benkelman beam AX06 (Figure 7) is a new development in which special attention has been paid to the following features:

- truss grid beam construction made of stainless steel
- is delivered calibrated
- 1:2 measuring bridge with precision bearings
- easy to assemble, simply hang in the front part of the beam and screw it tight
- pack size of less than 2 meters
- improved mounts for dial gauge, the height adjustment of the feet is clamp-able
- according to the requirements of "FGSV Working Paper No. 33, Bearing capacity of Roads, Part B1: Benkelman Beam, Device Description, Measurement Execution, Part C1: Evaluation and Evaluation of Dredging Measurements"

4.2 Applications

The Benkelman beam is used to determine bearing capacity of asphalt and paving slabs:

- Identification of visually unrecognizable vulnerabilities
- Formation of homogeneous sections of comparable bearing capacity
- Determination of the carrying behavior over longer periods of time and after extreme hydrological events
- Determination of structural causes of damage and assessment of the structural condition
- Selection of suitable measures in the context of road maintenance.

It is an inexpensive alternative to the FWD (Falling Weight Deflectometer).

4.3 Measuring principle

A loading vehicle with a twin-tired rear axle and a wheel load of 5 tons is located on the test track. The measuring tip of the Benkelman beam is placed in the middle between the twin tires. The dial gauge is set to zero. The loading vehicle moves away and the deflection of the road surface is read off the dial gauge. The modulus of elasticity is determined from the wheel load and the deflection.

4.4 „Two-gauges“ option

The Benkelman beam is available in two versions: classic with a dial indicator (Figure 7), and as a "two-gauge" version. The "two gauge" variant is measuring one additional point of the deformation bowl (in about 50 cm distance to the first point). This allows conclusions about the deformation modules of the base layer in addition to the modulus of the asphalt layer. Two inductive displacement sensors and the proven Anix electronics box are used for the measurement. Measurements can be captured together with GPS coordinates (optional), saved to the SD card and printed with the built-in mini printer.

Figure 7. The classical Benkelman beam AX06 with a dial gauge

FURTHER INFORMATION

Further information on the products presented can be obtained directly from the manufacturer:
Anix GmbH
Hintern Hecken 1
D-39179 Barleben
Germany
Internet: https://www.anix.biz, Email: info@anix.biz
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Flächendeckende Dynamische Verdichtungskontrolle – FDVK Anwendungen und Interpretation von Messergebnissen

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Schlagwörter: Kalibrierung; Schwachstellenanalyse; Prozessdokumentation; Erdbau; Asphaltbau; BCM0

1 EINLEITUNG

Vor 40 Jahren wurden in Deutschland und Schweden die ersten walzenintegrierten Messgeräte entwickelt und auf dynamisch arbeitenden Erdbauwalzen mit dem Ziel eingesetzt, den Maschineneinsatz zu optimieren sowie den Verdichtungsfortschritt und den Verdichtungszustand qualitativ zu beurteilen. Die Entwicklung wurde durch grundlegende Untersuchungen zum dynamischen Verhalten von Vibrationswalzen und durch die parallel laufenden Innovationsschübe der elektronischen Messtechnik und Datenverarbeitung ermöglicht. Es folgten die Visualisierung der Messdaten auf Dokumentationssystemen und die Anbindung der walzenintegrierten Messtechnik an GNSS Positionierungssystemen, um die Walzenposition während des Verdichtungs- und Messvorgangs zu erfassen. Zurzeit konzentriert sich die Weiterentwicklung auf die mobile Datenübertragung zwischen Walze und Büro.

Der Einsatz der Systeme auf Großbaustellen des Straßen-, Eisenbahn- und Flughafenbaus führte Anfang der 90 er Jahre zum Verfahren der Flächendeckenden Dynamischen Verdichtungskontrolle (FDVK), mit der die Verdichtung bzw. Tragfähigkeit von Böden während des Arbeitsprozesses einer Vibrationswalze flächendeckend beurteilt werden kann. Während mit konventionellen Methoden nur stichprobenartige Überprüfungen im Nachhinein möglich sind, die sich auf sehr kleine Prüfvolumen beziehen, kann bei Anwendung der FDVK die gesamte bearbeitete Fläche bereits während des Verdichtungsprozesses überprüft, die Messwerte dem Walzenfahrer visualisiert und für Bauleitung und Qualitätssicherung dokumentiert werden. Schwachstellen können sofort erkannt und entsprechende Maßnahmen getroffen sowie Aussagen über die Gleichmäßigkeit der geprüften Fläche angestellt werden.

In Deutschland, Österreich, Schweiz und Schweden wurden Regelwerke für die Anwendung der flächendeckenden dynamischen Verdichtungskontrolle bereits in den 90er Jahren erarbeitet und seitdem im Straßen-, Eisenbahn-, Flughafenbau, sowie im Erdbau angewendet. Die Verbreitung der Systeme hat in den letzten Jahren stark zugenommen. Sie werden auf Baustellen unterschiedlichster Größenordnung eingesetzt. Inzwischen werden Mess- und Dokumentationssysteme von fast allen Walzenherstellern oder deren Systemausrüster angeboten. Im Dezember 2016 wurde im Zusammenhang mit der Europäischen Normung der Erdarbeiten ein erstes europäisches Dokument zur FDVK: CEN/TS 17006 – Earthworks - Continuous Compaction Control (CCC) veröffentlicht, welches bereits zur Anwendung der FDVK auf Großprojekten in Großbritannien, Frankreich, Italien und der Türkei geführt hat.

Seit ca. 10 Jahren wird die FDVK Mess- und Dokumentationstechnik auch im Asphaltbau eingesetzt und erprobt. Eine bauvertragliche Nutzung, wie sie im Erdbau schon länger praktiziert wird, muss für die Asphaltanwendung noch entwickelt werden.

2 MESS-UND DOKUMENTATIONSSYSTEM

Das von BOMAG entwickelte FDVK System ermittelt aus dem Beschleunigungsverhalten der Bandage und der Kraftweganalyse den dynamischen Steifigkeitsmodul " E_{VIB} " als Kenngröße. E_{VIB} reagiert auf Dichte- und Tragfähigkeitsunterschiede bzw. Änderungen und zeigt bei grobkörnigen und gemischtkörnigen Böden gute Korrelierbarkeit mit der Dichte.

Das Mess- und Dokumentationssystem besteht aus dem Messystem Terrameter mit zwei Beschleunigungssensoren, einer Rechnereinheit, Wegsensor, Bedien-und Anzeigeeinheit und Drucker zur Dokumentation der Messwerte als Linienschrieb unmittelbar auf der Baustelle, sowie dem Dokumentationssystem mit Bildschirm und GPS zur flächigen Visualisierung der bereits erfassten Messwerte und der durchgeführten Überfahrten (Kloubert and Wallrath 2012).

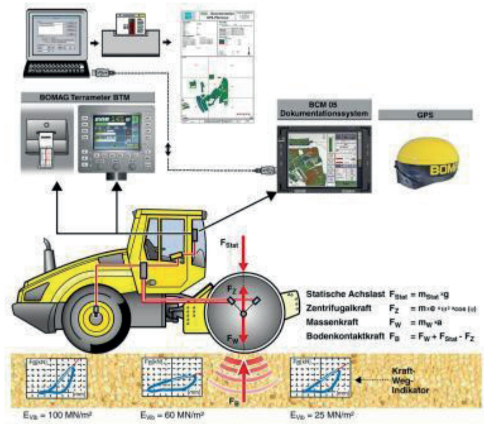


Abbildung 1. Messprinzip und Ausrüstung einer FDVK Walze.

3 ANWENDUNGSMÖGLICHKEITEN DER FDVK BEI ERDARBEITEN

Seit Einführung der ZTVE – StB 94/97 können in Deutschland die walzenintegrierten Mess- und Dokumentationssysteme im Rahmen der Eigenüberwachung und Fremdüberwachung für Erdarbeiten im Straßenbau eingesetzt werden. Im Vordergrund steht hierbei die Anwendung der FDVK als Vollprüfung auf Grundlage einer Kalibrierung der dynamischen Messwerte der Walze an die bauvertraglich definierten Prüfmerkmale Verdichtungsgrad und Verformungsmodul. Die Verfahrensweise ist als Prüfmethode M2 in der ZTVE – StB 94/97 und in den technischen Prüfvorschriften für Boden und Feld im Straßenbau (TP BF – StB) Methode FDVK – Teil E 2 verankert.

Darüber hinaus bietet die FDVK eine Reihe von weiteren Anwendungsmöglichkeiten, die keine Kalibrierung erfordern (M FDVK E 2014). Eine mit dem BCM 05 durchgeführte flächendeckende Schwachstellensuche durch ein Proof Rolling ist auf allen Bodenarten einsetzbar. Schwachstellen mit niedrigen E_{VIB} Werten werden dabei erkannt und dokumentiert. Die Verdichtungsprüfung mit Einzelversuchen kann gezielt an diesen Schwachstellen durchgeführt werden. In der Kombination von dynamischem Messwert der Walze und Einzelprüfung kann die Fläche dann insgesamt beurteilt werden.

Eine weitere wichtige Anwendung ist die Dokumentation der Verdichtung auf Grundlage einer Probeverdichtung und der damit verbundenen Festlegung einer Arbeitsanweisung. Hierzu werden mit dem BCM 05 System und einer GPS Anbindung die E_{VIB} Werte und die Position der Walze kontinuierlich dokumentiert und damit auch die Verdichtungsübergänge kontrolliert. Die mit den

Anwendungsmöglichkeiten der FDVK verbundenen Vorteile im Hinblick auf Steigerung der Verdichtungsleistung und Verbesserung der Verdichtungsqualität lassen sich wie folgt zusammenfassen:

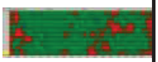
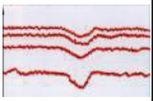
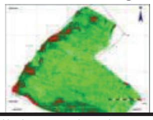
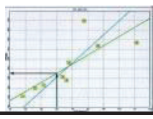
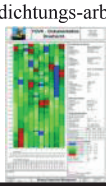
ANWENDUNG	Empfohlene Walzen-ausstattung	Beispiele	Vorteile	Eignung
Schwachstellenanalyse  	Kleine Baumaß-nahmen: Terrameter BTM Größere Baumaß-nahmen: zusätzlich BCM 05	Gründungsflächen, Planum und Damm-schüttungen	Erkennung von Bereichen mit hohen und niedrigen Mess-werten (Schwach-stellenanalyse / Proof Rolling) ⇒ gezielter Einsatz von herk. Prüf-verfahren	Alle Böden
Nachweis der maximal möglichen Verdichtung  	Kleine Baumaß-nahmen: Terrameter BTM Größere Baumaß-nahmen: zusätzlich BCM 05	Dammschüttungen Frostschutz-schichten Ungebundene Tragschichten	Vergleich der Meßwerte zwischen einzelnen Über-gängen Optimierung des Geräteeinsatzes	Alle Böden
Festlegung einer Arbeitsanweisung  	BTM und BCM 05 und GPS		Dokumentation der Übergänge und der Schwachstellen	Alle Böden
Nachweis der Verdichtung durch Kalibrierungen  	Kleine Baumaß-nahmen: Terrameter BTM Größere Baumaß-nahmen: zusätzlich BCM 05	Planum Frostschutz-schichten Ungebundene Tragschichten Dammschüttungen	Flächendeckende Verdichtungs-prüfung	Steinschüttungen grobkörnige Böden Mischböden Mineralgemische
Steuerung der Verdichtungs-arbeiten  	Kleine Baumaß-nahmen: Terrameter BTM Größere Baumaß-nahmen: zusätzlich BCM 05	Dammschüttungen Frostschutz-schichten Ungebundene Tragschichten	Wirtschaftlichkeit Optimierung des Geräteeinsatzes	Alle Böden

Abbildung 2. Anwendungsmöglichkeiten der FDVK.

4 FLÄCHENDECKENDE VERDICHTUNGSKONTROLLE BEIM ASPHALTBAU

Während im Erdbau die flächendeckende Erfassung des Verdichtungsfortschritts viele Jahre praktiziert wird, werden Asphaltwalzen erst seit einigen Jahren mit GPS gestützten Mess- und Dokumentationssystemen ausgestattet, um die erreichte Verdichtung während des Einbauprozesses flächendeckend zu kontrollieren. Weil der Verdichtungsprozess von Asphalt möglichst in einem engen Temperaturfenster erfolgen muss, sind die Asphaltwalzen auch mit Temperatursensoren zur Erfassung der Asphalttemperatur ausgestattet. Eine Korrelierbarkeit des auf Steifigkeit basierten FDVK Messwertes mit der Dichte ist aufgrund des Einflusses der Asphalttemperatur auf die Steifigkeit und der gegenüber Erdbauanwendungen relativ dünnen Einbaulagen schwieriger. Dennoch werden bei guten Vorrausetzungen Verdichtungszunahmen erkannt und visualisiert sowie Schwachstellen lokalisiert. Gute Voraussetzungen sind gegeben, wenn die Asphaltunterlage eine gleichmäßige Steifigkeit aufweist und die Verdichtung bzw. Verdichtungsmessung innerhalb einer gewissen

Temperaturspanne, in der Regel bei Heißasphalt zwischen 100 – 150 ° C erfolgt. Mit zunehmender Verdichtung nehmen dann auch die Messwerte zu, sodass sich im Idealfall eine Kalibrierung des Walzenmesswertes auf die Dichte entwickeln lässt und ein Anforderungsband definiert werden kann. Große Vorteile liefert die FDVK auf Asphaltbaustellen, wenn die Walzen, wie in Abb. 3 gezeigt, z.B. durch ein WLAN – Netz verbunden sind. Damit werden der Verdichtungsprozess und das praktizierte Walzschema für alle Beteiligten transparent. Die zu Einbaubeginn festgelegte Anzahl der Überfahrten werden verlässlich und flächendeckend visualisiert und dokumentiert. Dies führt zu einer effizienteren, gleichmäßigeren und qualitativ hochwertigen Verdichtung und Prozessdokumentation (Kloubert and Wallrath 2010).

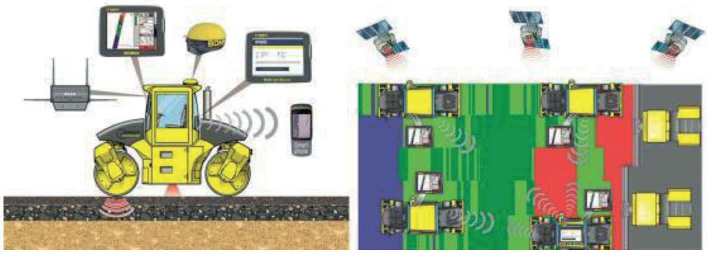


Abbildung 3. Vernetzung von mehreren Walzen zur Prozessdokumentation im Asphaltbau.

5 ZUSAMMENFASSUNG

Seit 40 Jahren werden Messsysteme als Zusatzausrüstung für Vibrationswalzen angeboten, die eine flächendeckende Messung eines dynamischen Kennwertes auf einer verdichteten Schicht zulassen. Mit Hilfe von Anzeige- und Positionierungssystemen werden dem Walzenfahrer bereits während der Verdichtungsarbeit Kenntnisse über den jeweils herrschenden Verdichtungs- und Tragfähigkeitszustandes, den Verdichtungsfortschritt und den durchgeführten Überfahrten der überwalzten Fläche visualisiert. Daraus hat sich in Deutschland, Österreich und Schweden für den Erdbau bereits sehr früh das Prüfverfahren der flächendeckenden dynamischen Verdichtungskontrolle (FDVK) mit unterschiedlichen Anwendungsmöglichkeiten entwickelt.

Zwischenzeitlich wurde durch die Nutzung von Mikroprozessorsteuerungen und die Einführung von automatischen Dokumentations- und GPS gestützten Positionierungssystemen, ein Verdichtungsmanagement ermöglicht, das die Positionsdaten der Walze mit den Verdichtungsdaten verknüpft und sowohl die Verdichtungsqualität und ihre Gleichmäßigkeit, als auch die Walzenübergänge kontrolliert und dokumentiert. Die damit verbundenen Vorteile haben zu einer weiteren Verbreitung und Anwendung der flächendeckenden Verdichtungskontrolle auch beim Einbau von Asphalt geführt. Die im Asphaltbau notwendige Vernetzung der Walzen ermöglicht eine prozessgesteuerte Verdichtungstechnik mit Dokumentation.

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Anniversary Symposium – 40 Years of Roller Integrated Continuous Compaction Control (CCC)
D. Adam & S. Larsson (eds.)
November 29th, 2018, Vienna, Austria

Intelligent Compaction Measurement Values – A Systematic Classification

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Keywords: Intelligent compaction; Accelerometer-based measurements; Systematic classification

1 INTRODUCTION

Intelligent compaction (IC) is a vibratory roller-based technology to improve compaction quality control by extending the Continuous Compaction Control (CCC) technologies with global positioning system (GPS)-based mapping. IC vibratory rollers are equipped with a high precision GPS, infrared temperature sensors (for asphalt compaction application), an accelerometer-based measurement system, and an onboard color-coded display. Intelligent Compaction Measurement Value (ICMV) is a generic term, coined by the US FHWA, for a calculated value based on accelerometer measurements on IC vibratory roller drums. ICMV are in different forms of metrics from various manufacturers, with various levels of correlation to the mechanical properties (e.g., moduli) and physical properties (e.g., densities) of the pavement layer materials after compaction. The purpose of this document is to provide a systematic classification for ICMV and to provide a road map towards truly intelligent compaction monitoring, control, and acceptance. This is a summary of the report by Chang (2017).

2 BASIC MECHANISM OF ICMV

2.1 Basic Common Mechanism of ICMV

The basic common mechanism for calculating all types of ICMV is to measure the vertical acceleration of the vibrating drum and compute ICMV using various mathematical models and methods. The concept is simple and ingenious; measuring the properties of compacted materials during compaction allows real time compaction monitoring and control. The roller drum exerts compaction force on the compacted materials and the compacted materials react the force back to the roller drum. The harder the compacted materials, the larger the reactive force. The reacted force is captured by the accelerometer in terms of acceleration. The control system will then process the acceleration signals and compute ICMV.

2.2 Challenges for Computing ICMV

There are numerous factors affecting ICMV computation. From the roller side, vibration types, eccentric force, vibration amplitude and frequency, and roller speed are major factors. From the compacted materials, soils types and moisture contents, asphalt mixture proportioning, asphalt mix temperatures, and underlying support condition are major factors (Xu 2016).

There are many challenges to produce ICMV including the followings:

(a) Vibration force is not equal to compaction force: The vibration force from the eccentric weight in the drum is not equivalent to the effective compaction force that asserts on compacted materials. The compaction force fluctuates as the vibration frequencies increase. On the other hand, the vibration force increases monotonically with increase of frequencies. This is a frequent mistake for researchers who start modelling the drum and material interaction.

(b) Actual strain measurement is different from theoretical computation: The actual strains and displacement under a drum is constant. However, the computed theoretical strains and displacement are variable across a drum width based on Lundberg's and Hertz's theories to solve the contact problem of cylindrical drum of finite length on compacted materials. The error is due to the incorrect assumption of theoretical contact area of the models to mimic the field drum contact condition. Therefore, the theoretical computation needs corrections to match actual field measurements.
(c) When decoupled, the computed modulus is erroneous: When a drum and compacted materials lose contact or are decoupled, the computed modulus are often too low or non-obtainable. The actual modulus should be high, based on the reactive force, in comparison with the computed values. Therefore, the technique of using the impact model and reactive force can overcome the difficulties of computing ICMV during double-jump movements of the roller drums.

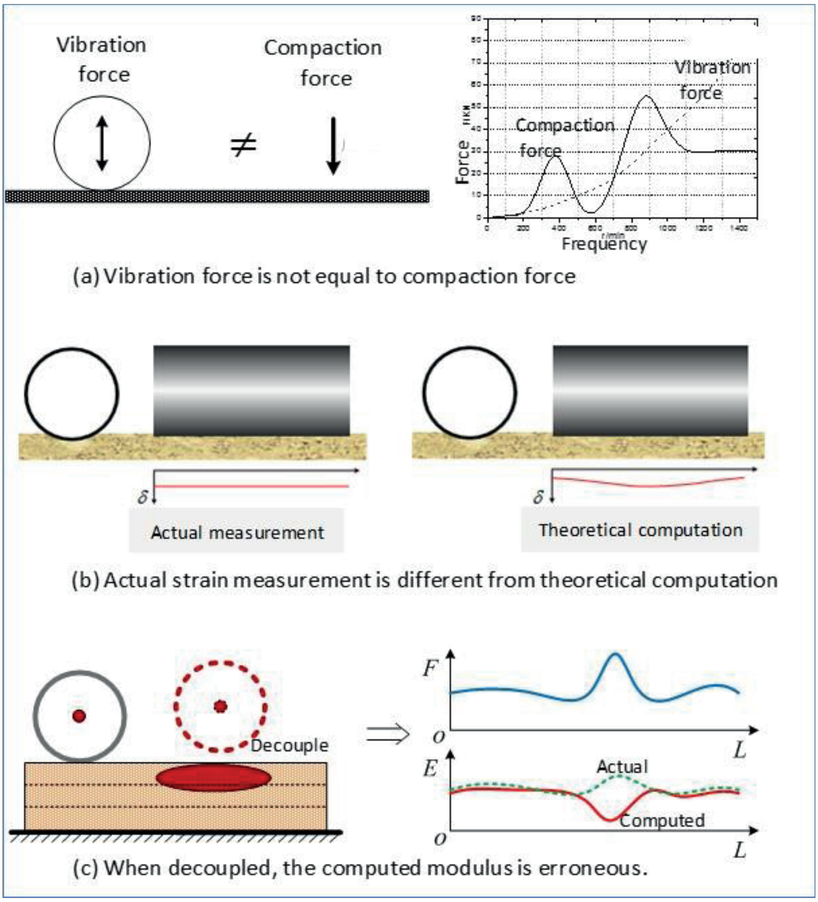

Figure 1. Main challenges to compute ICMV.

3 MATHMATICAL MODELS FOR SOLVING ICMV

To systematically classify ICMV, the mathematical models for solving ICMV are summarized in five categories based on the types of dynamic, static, empirical, and mechanistic solutions. These models include those that have been developed in the past 40 years and the ones that are still under development. Detailed description of ICMV mathematical models and solutions can be found in US FHWA Technical Brief: ICMV – A Road Map.

Table 1. Five Mathematical Models for Computing ICMV.

Model	Description	Mechanistic/ Empirical	Dynamic/ Static
A	Empirical Reactive Models	Empirical	NA
B	Continuum Roller and Half-Space Layered System	Mechanistic	Dynamic/Static
C	Discrete Drum and Spring-Dashpot Coupled System	Mechanistic	Dynamic
D	Dynamic Impact Model for Decoupled Drum and Layer System	Mechanistic	Dynamic
E	Artificial Intelligence Method	Mechanistic	Dynamic data

4 SYSTEMATIC CLASSIFICATION OF ICMV

There are currently solutions of ICMV based on the above mathematical models. There is also next generation ICMV under development. The following describe the "Levels of ICMV" for the systematic classification of ICMV. The classification is based on four criteria: (1) applicability to a variety of pavement materials, (2) correlation with material's mechanical (modulus) and physical properties (density), (3) validity during decoupling (when a drum loses contact with compacted materials), and (4) applicability to obtain layer-specific mechanical and physical properties of compacted materials. This lays out the road map for past, present, and future ICMV development.

4.1 Five Levels of ICMV Solutions

4.1.1 Level 1 ICMV – Empirical Solutions based on Frequency Responses
The Level 1 ICMVs are empirical solutions based on frequency responses represented by frequency ratios. It is computed based on the Model A1: Reactive Models with frequency responses or Model A2: Oscillation Frequency Reactive Model.

4.1.2 Level 2 – Empirical Energy and Rolling Resistance Solutions
The Level 2 ICMVs are empirical energy and rolling resistance solutions. It is computed based on Model A3: Reactive Models with rolling resistance principle. The computation requires machine specific parameters and measurements such as machine movement angle and energy loss coefficients.

4.1.3 Level 3 – Simplified Static Mechanistic Solutions
The Level 3 ICMVs are simplified static mechanistic solutions. It is a static solution based on Model B-Continuum Roller and Half-Space Layered System or based on Model C- Discrete Drum and Spring-Dashpot Coupled System. Due to its static solution and simplified method, the solution is invalid when double jumps (or loss of contacts) occur.

4.1.4 Level 4 – Dynamic Mechanistic Solutions
The Level 4 ICMVs are dynamic mechanistic solutions. They are dynamic solutions based on Model D- Dynamic Impact Model for Decoupled Drum and Layer System. The reactive force ICMV (F-ICMV) can be computed based on roller vibration acceleration, velocity, and displacement. It also requires dynamic correction factors and phase lags to account for actual field condition. The modulus ICMV (E-ICMV) can be computed either based on the reactive force or based on the time duration of impact.

4.1.5 Dynamic Mechanistic and Artificial Intelligence Solutions
The Level 5 ICMVs are a combination of dynamic mechanistic solutions and artificial intelligence solutions. Layer-Specific Modulus ICMV (E_n-ICMV) and Density ICMV (ρ_n-ICMV) are based on a combination of Model D- Dynamic Impact Model for Decoupled Drum and Layer System Model E-Artificial Intelligence Method. Though still under development, it is envisioned that both models can be

merged, and field measurement methods can include layer-by-layer mapping to measure E_n-ICMV and ρ_n-ICMV. A true real-time auto-feedback system can then be deployed to optimize compaction without human intervention. Then, an autonomous compaction machine can be realized.

4.2 Summary of All Levels of ICMV

The above five levels of ICMVs are summarized below:

Table 2. Five Levels of ICMV Solutions.

Level	Model	Measurement Values	Correlation	Decouple	Layer Specific
1	A1, A2 Empirical	Harmonic ratio	Weak or Poor	No	No
2	A3 Energy	Energy index	Weak or Poor	NA[4]	No
3	B + C Discrete vibration, continuous static	Stiffness, Coefficient of resistance force, Modulus	Good	No	Difficult
4	D Hybrid	Coefficient of resistance force, Modulus	Good	Yes	Yes
5	D + E Continuous dynamic	Density, Modulus	Excellent	Yes	Yes

5 CONCLUSIONS AND FUTURE DEVELOPMENT

This document summarizes the mathematical models of ICMV and a systematic classification for ICMV solutions. To advance the ICMV levels, the International Intelligent Construction Technologies Group (IICTG) and the International Intelligent Construction Research Institute (ICTRI) were founded in 2016 and 2018, respectively, to facilitate the international cooperation on ICMV research. Further IICTG/ICTRI research will involve the US National Road Research Alliance (NRRA) to conduct field evaluation of Level 3-4 ICMV in 2018-2019. Future development will also include Continuous Compaction Reference Device (CCRD) for future ICMV certification programs. The ultimate goals of the above development are to realize the modulus-based compaction acceptance and true intelligent compaction feedback controls.

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Evaluation of weak spots in geotechnics in terms of size, distribution and relevance

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Keywords*: CDCC weak spots; Geostatistical analysis; Nearest neighbour; Fuzzy clustering*

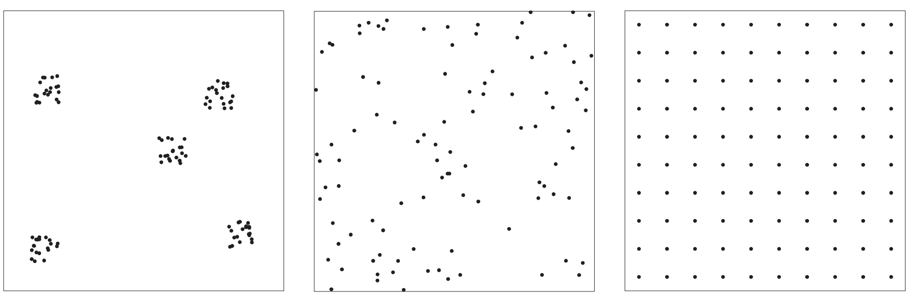
1 INTRODUCTION

In a statistical sense, the *CDCC* in earthworks allows the user to check an inspection lot completely. Thus, a characteristic compaction value derived from the measured data is available for each point. Due to the statistical assessment according to the 10% quantile in earthworks the compaction requirement does not have to be fulfilled at every point. A shortfall of the required value is tolerated with a maximum of 10% of the measured values. Beside this limitation in the number of weak spots they also have to be evenly distributed over the entire area. Otherwise, areas in which weak spots concentrate may lead to stability problems or to uneven settlement in connection with areas without weak spots. So far, the assessment of this areal distribution is done visually by the parties involved with different interpretations being the rule. In contrast, the presented methods should show possibilities for an objective assessment.

2 GEOSTATISTICAL METHODS

2.1 Principles

The distribution of weak spots in an inspection lot corresponds to a spatial point distribution. In order to assess the uniformity of this distribution, its probability is tested by assuming a random distribution process. Hence the null hypothesis of the statistical test corresponds to a random point distribution. This type of distribution is known as *complete spatial randomness (CSR)* and can be represented as a realization of the homogeneous *Poisson* point process. Depending on the statistical test, it is possible to deduce the type of deviation to a random distribution. As shown in Fig. 1, the deviation could either be a concentration (clustering, (a)) or a regular distribution (dispersion, (c)) of the weak spots.



(a) (b) (c)

Figure 1. Types of weak spot distribution: (a) clustered, (b) random (null hypothesis), (c) regular.

2.2 Data Preparation

2.2.1 Triangulation and interpolation of the measuring points

For the geostatistical analysis of a weak spot distribution the inspection lot size needs to be known. This size is defined below as the area enclosed by the *CDCC* measurement points. Therefore, a triangulation of all measured points is carried out. As a result the gray-shaded area in Fig. 2(b) is obtained from the measured values in Fig. 2 (a).

The *CDCC* rollers take a measurement at approximately every 0.2 - 0.5 meters. With a roller drum width of approximately 2 meters the measured values in the driving direction are much closer together than between the individual measuring tracks. To compensate for this uneven data acquisition, an interpolation of the *CDCC* measurements at equidistantly distributed reference points in the inspection lot is performed before the actual geostatistical analysis of the weak spots can be carried out. The interpolated values in Fig. 2 (c) are obtained by using the *natural neighbour interpolation* method (Sibson et al., 1981).

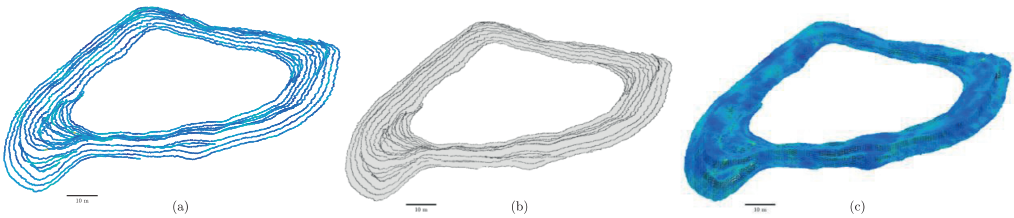


Figure 2. (a) Colour-coded CDCC values along several measuring tracks; (b) triangulation of the measurement points in order to define the grey shaded inspection lot; (c) interpolation of the measurements from (a) at equidistantly distributed reference points

2.2.2 Filtering out weak spots

The weak spots are filtered out by determining all *CDCC* measured values which fall below a certain required value. The corresponding weak spots for the inspection lot in Fig. 2 (c) are marked in red in Fig. 3. A further subdivision of the weak spots by the degree of their lower deviation of the required value is not part of this work.

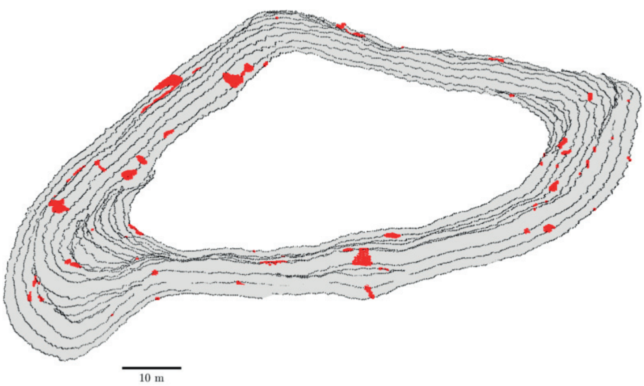


Figure 3. Red marked weak spots within the inspection lot.

2.3 Examples of applied methods

2.3.1 Nearest neighbour

In this approach the distance between a weak spot and its nearest neighbor weak spot is measured. In the simplest case the average nearest neighbor distance (*NN* distance) is determined and used as an indication of the characteristic of the weak spot distribution. The assumption shown in Fig. 4 is based on the fact that a concentrated weak spot distribution has a small average *NN* distance whereas a regular distribution of the weak spots tends to have a large mean *NN* distance. Consequently, a random distribution lies between these two extremes.

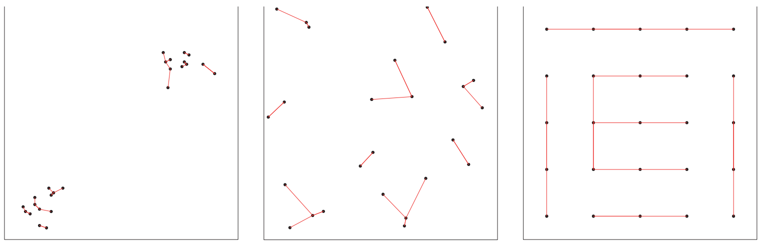
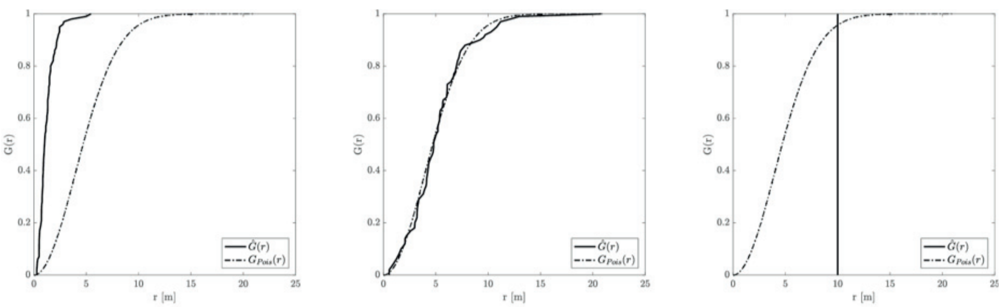


Figure 4. *NN* distances for a (a) clustered, (b) random and (c) regular weak spot distribution.

However, this simple procedure fails when weak spot distributions contain both clusters and isolated outliers because in this case the small distances in the clustered areas compensate for the disproportionately large distances to the outliers. Therefore, the cumulative distribution of all measured *NN* distances *r* should always be considered (see Fig. 5).



(a) (b) (c)

Figure 5. Cumulative Distribution (cf. Equation 1) of all measured NN distances in Fig. 4 (a) to (c).

The subframes show the empirically determined cumulative distribution function $\hat{G}(r)$ as well as the theoretical curve for a random distribution. According to Baddeley et al. (2015) this theoretical function is defined as follows:

$$G_{Pois}(r) = 1 - e^{-\lambda\pi r^2} \tag{1}$$

The distribution of the *NN* distances for a clustered weak spot distribution plotted in Fig. 5 (a) lies above the theoretical curve of a random distribution since the proportion of short distances between the neighboring weak spots is relatively high. As expected the distance function (b) for the randomly distributed weak spots follows the theoretical curve for the homogeneous *Poisson* process whereas the regular distribution (c) deviates in direction of greater distances and even lies on a straight line in this extreme example of a perfectly regular lattice.

2.3.2 Fuzzy clustering

Fuzzy C-means clustering (FCM) according to Bezdek et al. (1984) distinguishes itself from other geostatistical methods by the fact that possible clusters can be classified more precisely in terms of size, position and shape. For this purpose, each weak spot is assigned to a cluster. The assignment can be more or less accurate and is evaluated by its membership grade. The algorithm initially sets up a random number of clusters and randomly distributes their centers in the inspection lot. Afterwards the number and position of the clusters are adjusted iteratively by optimizing the membership grades. The result of the *FCM* analysis for the weak spot distribution in Fig. 3 is shown in Fig. 6.

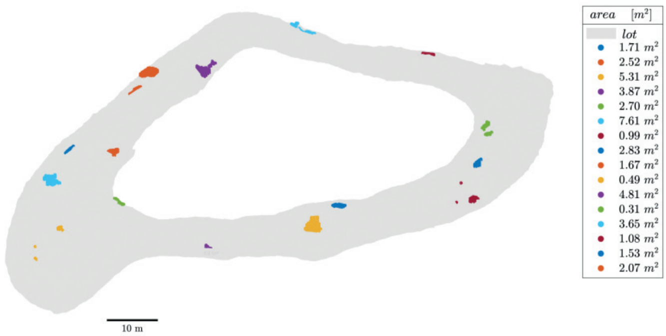


Figure 6. Identified weak spot clusters within the inspection lot in Fig. 3 based on the FCM method.

3 CONCLUSIONS

There are numerous geostatistical methods for assessing the uniformity of weak spot distributions which can help to provide an objective assessment. In addition, these methods can be used within certain limits to express the spatial distribution characteristics by a simple numerical value, which plays an important role in the formal acceptance protocol of an inspection lot. However due to the uneven data acquisition, *CDCC* measurement data cannot be geostatistically examined without prior interpolation of the measured values at equidistantly distributed reference points. Besides simple distance based methods, such as the nearest neighbour distance analysis, fuzzy clustering in particular promises very good application possibilities since it is possible to define clusters more precisely.

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Anniversary Symposium – 40 Years of Roller Integrated Continuous Compaction Control (CCC)
D. Adam & S. Larsson (eds.)
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Institutional Challenges and Opportunities in Implementation of Intelligent Compaction for Quality Management of Earthwork

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Keywords: Quality Control; Earthwork; Field Testing; Intelligent Compaction; Continuous Compaction Control

1 INTRODUCTION

With the emphasis on mechanistic-empirical pavement design procedures in the last decade, significant research effort has been devoted to understanding and implementing modulus-based performance management of compacted geomaterials (e.g., Von Quintus et al. 2009; Nazarian et al. 2014). Performance management signifies the following two phenomena: (1) modulus of each layer is close to the value specified by the designer, and (2) compacted layer is uniform in terms of modulus.

In reaction to the first item (modulus verification), Nazarian et al. (2014) systematically enumerated the technical and institutional complications related to incorporating modulus-based spot testing devices (e.g., Light-Weight Deflectometer or LWD) in the mechanistic-empirical pavement design procedures and provided practical solutions to them. These complications include (1) relating pavement design parameters to construction quality control parameters, (2) incorporating the impact of moisture content on modulus, and (3) developing field-calibrated numerical models to be used in proper evaluation and acceptance of compacted geomaterials.

In reaction to the second item (uniformity), many US highway agencies require the traditional proof rolling as a rudimentary way of evaluating the compacted geomaterials. Proof rolling with the intelligent compaction (IC) technique can be an effective technology for identifying the less stiff spots systematically. IC measurement values (ICMVs), which are obtained in different manners, are typically used for that purpose. If implemented properly, IC can cover 100% of compacted materials (a major shortcoming of the spot testing with less than 1% coverage). Despite significant efforts, there are still gaps that prevent IC from being used widely for eventual performance management of geomaterials. The major technical gap often cited includes a lack of robust and practical methodology to determine lift-specific target ICMV. The main institutional issues include (1) confusion among contractors and owner agencies on when the contractor process control ends (termed continuous compaction control, CCC) and the owner agency quality acceptance process (termed IC proof rolling) begins, (2) the absence of a rational means of relating one ICMV to another reported by different roller vendors, (3) lack of a rational way to interpret the data, and (4) lack of strong correlation between ICMV and spot test results.

In this presentation we intend to share our efforts toward assisting highway agencies to identify their technical and institutional issues that impeded the estimation of layer mechanical properties with IC proof-mapping, to identify the associated causes, to propose a solution through supplementary field and laboratory investigations, and to demonstrate that the proposed solution can be used by highway agencies to obtain ICMVs that are related to layer-specific mechanical properties of compacted embankment, subgrade and base materials.

2 INSTITUTIONAL CHALLENGES AND OPPORTUNITIES OF IMPLEMENTING CCC

Fig. 1 illustrates an ideal process implementing a quality management protocol. Successful implementation of IC proof mapping to achieve compaction uniformity and improved life of pavement structures is associated with multiple criteria. These criteria categorized as installation (including IC hardware and data acquisition software); field operation (including data collection, global positioning system (GPS) setup and roller operating parameters such as vibration frequency, vibration amplitude and roller speed); interpretation of ICMV (including outlier detection, color coding criteria to visualize ICMVs on the map, and quality management); and inconsistencies among different data collection algorithms and reporting formats provided by various IC manufacturers.

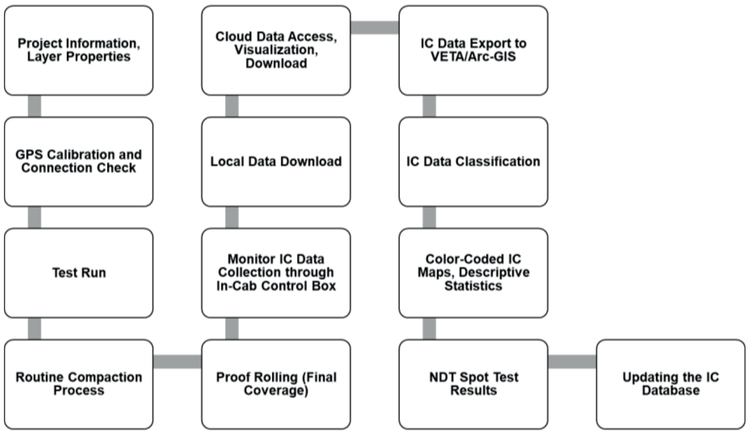

Figure 1. An Ideal Flowchart for Application of CCC in Quality Management

Although a variety of roller manufacturers provide the data collection and management systems to implement CCC and IC proof mapping, the non-uniformity in data collection methods as well as inconsistency among data reduction algorithms have generated several complications for highway agencies to implement the IC proof mapping in their quality management process. Fig. 2 shows the components of a research-grade IC data collection system that provides a straightforward and implementable data for quality management. That system has been successfully implemented in a number of construction projects to obtain different ICMVs recommended by different manufacturers from one set of vibration data, in order to establish basic uniformity among different makes and models of rollers. (Nazarian et al. 2014, Nazarian et al. 2015, Mazari et al. 2016).

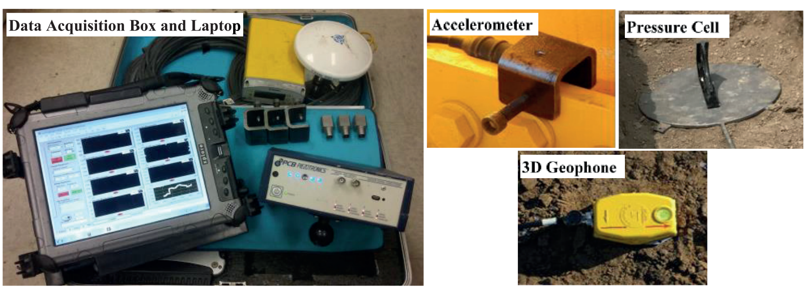

Figure 2. Components of CCC Data Validation System

A common practice in implementation of IC proof mapping for quality management is to pair it with an established spot testing device (e.g., Light Weight Deflectometer, LWD) (Nazarian et al. 2014; Schwartz et al. 2017). A typical process for such paired approach is shown in Fig. 3. The less-stiff areas are identified from color-coded ICMV maps and then evaluated with LWD. The estimation of field moisture content is beneficial in adjusting the field target values.

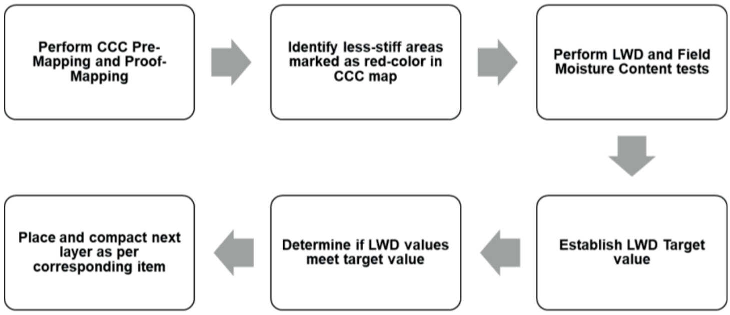

Figure 3. CCC Quality Management Process paired with Modulus-Based Spot Testing

Another complication is involved with current data visualization approach where a color-coded map of collected ICMV is presented on a fine-grained network of geospatial locations based on the frequency of GPS and roller vibration data collection. Although these maps are "sharp" for visualization, they might not preserve the "fidelity" needed for a robust quality management. Presenting the information in a number of field blocks could be more implementable in terms of quality management purposes.

Fig. 4 illustrates an example of color-coded blocks of CCC data for a typical test section. The test section is divided into several blocks with a representative ICMV for each block. The color-coding criteria are not based on the standard deviation of the measurements but set based on the coefficient of variation (COV) that a typical construction company can achieve with reasonable effort (in Texas 25% based on a database of the CCC projects for four years). In addition, a color-coded map of COV for each block is provided to highlight the blocks with less uniform ICMVs. Fig. 4 also represents spot tests (in this case LWD) results in the same format. The representative ICMV and LWD graphs seem to provide similar information about less stiff areas that might require re-working.

Color Code

- (red) < 75% of Average Measurement
- (yellow) 75% of Average $\leq x <$ Average
- (green) $\geq$ Average Measurement

Descriptive Statistics

NDT	Mean	Standard Deviation
CMV	4.1	1.8
LWD Modulus (MPa)	43	17

- (red) > 35%
- (yellow) 25% < COV $\leq$ 35%
- (green) $\leq$ 25%

Chainage (m)	(a) ICMV D	C	B	A	(b) Coefficient of Variation of ICMV (%) D	C	B	A	(c) LWD Modulus (MPa) D	C	B	A
0	8.2	3.9	4.6	4.6	32	41	44	26	59	58	26	62
7.5	4.2	4.2	3.6	4.8	42	43	10	22	41	86	41	59
15.0	3.9	3.9	2.5	4.3	29	40	10	11	42	58	34	38
22.5	3.8	2.1	2.3	6.0	26	39	37	11	52	54	31	50
30.0	4.6	3.3	5.1	8.0	23	21	13	18	70	56	47	61
37.5	3.9	2.6	6.5	7.6	20	34	17	15	20	37	41	56
45.0	1.9	1.1	4.8	4.7	22	59	22	27	11	29	30	46
52.5	2.0	1.8	3.1	6.5	33	39	26	20	61	25	25	38
60.0	2.6	1.6	3.4	5.6	35	22	24	22	9	25	17	46

Figure 4. Spatial Variation of (a) CCC Data Validation System, (b) Coefficient of Variation of CCC Data, and (c) LWD Modulus

To further investigate the effectiveness of CCC approach in representing the mechanical properties of compacted pavement layers in the field, a nonlinear vibrating and moving finite element model has been developed that simulates the roller compaction energy to provide the response of layers based on their properties. This model, which is calibrated with field data in one and two-layer geomaterials, is being used to develop an algorithm to estimate the stiffness parameters of compacted geomaterials. Further information about that software and the process explained above can be found in Tirado et al. (2018).

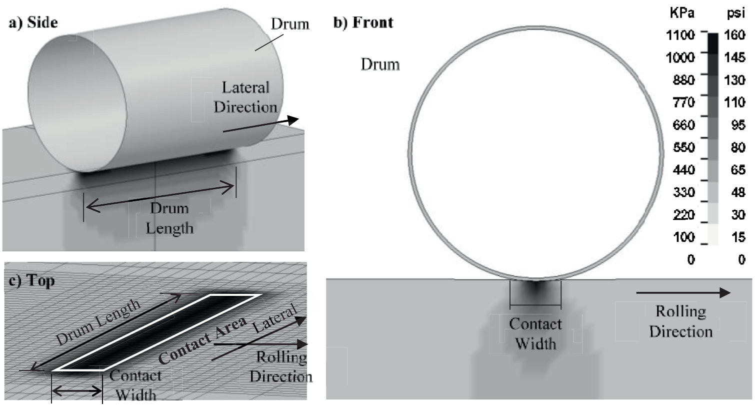

Figure 5. Finite Element Modelling of Roller-Soil Interaction

CONCLUSIONS

The results of an extensive study associated with the acceleration of the implementation of IC proof mapping for compaction quality control are summarized here. The proposed methodology is rigorous but practical that can be readily implemented.

ACKNOWLEDGEMENTS

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Intelligent Compaction Implementation in US – Issues and Solutions

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Keywords: Intelligent Compaction; Implementation; Issues and Solutions

1 INTRODUCTION

Intelligent compaction (IC) and Continuous Compaction Control (CCC) technologies has been existed for 40 years. However, IC/CCC (IC) has not been widespread used around the world. The United States (US) started the IC implementation in early 2000's, then accelerated it during the IC Transportation Pooled Fund Study from 2007 to 2012. Since then, the US IC industry is arguably the largest in the world and grows exponentially. Through these years of IC implementation in the US, numerous issues and obstacles have hindered the progress of IC implementation at various stages. However, the US responded with solutions through coordinated efforts among the US Federal Highway Administration (FHWA), US States Departments of Transportation (DOTs), consultants, and IC manufacturers and solution providers. This document is to summary the above issues and solutions.

2 BRIEF HISTROY OF IC IN US

The timelines and milestones for IC efforts in the US are shown as below.

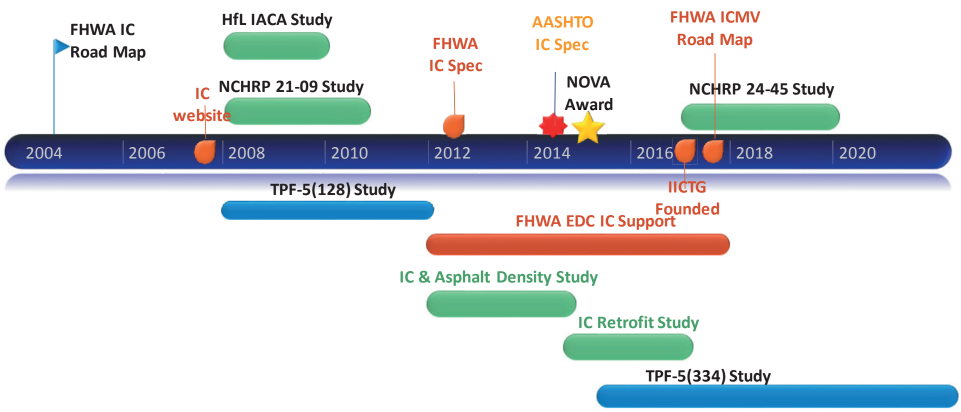

Figure 1. Timeline and Milestones of IC Implementation in US.

The followings highlight the US IC implementation:

- 2004: FHWA produced a "Road Map" for IC implementation after a SCAN tour in Europe. This document has sparked interests in IC implementation at several, but limited, DOTs.

- 2007-2010: NCHRP 21-09 was a soils IC research project to provide guidance in field calibration and specification development.
- 2007-2009: The FHWA Highway for Life (HfL) IC study produced a system for asphalt IC application, called IACA (Intelligent Asphalt Compaction Analyzer). Later, IACA was commercialized by Volvo as a commercial product.
- 2007-2010: TPF-5(128) was a demonstration pooled fund study project to accelerate implementation of IC for soils, subbase, and asphalt materials. The momentum of US national IC implementation starts to build up during this period.
- 2007-current: The IC website (www.IntelligentCompaction.com) was launched to provide one-stop shop for IC and it has been constantly updated.
- 2012-2014: The FHWA IC & Asphalt Density Study is to investigate the relationship between Intelligent Compaction Measurement Value (ICMV) and asphalt in-place density. This study produces an IC-based model for asphalt in-place density prediction.
- 2012-2014: The FHWA IC retrofit study was conducted to evaluate IC retrofit systems and provide guidelines for IC retrofit instrumentation and associated field calibration.
- 2012: The first FHWA IC specification for soils and asphalt was published.
- 2013-2017: The FHWA Everyday Counts (EDC) IC support project provided DOT and industry technical support for implementing IC with training workshops, equipment demonstration, and phone/email/online support. This effort accelerates further the US national IC implementation.
- 2014: The first AASHTO IC specification for soils and asphalt was published.
- 2016-2019: The NCHRP 24-45 IC research aims to produce methods to measure layer-specific mechanical properties (such as modulus and stiffness) with ICMV.
- 2016: International Intelligent Construction Technologies (IICTG) was founded to take IC to the next levels and to integrate IC with other Intelligent Construction Technologies (ICT).
- 2015-2021: The TPF-5(334) pooled fund study "Intelligent Construction Data Management (ICDM) - Veta" provided guidance and continuous funding to enhance the AASHTO Standard Veta software tool to meet DOTs and industry's needs for standardization and data management.
- 2017: The FHWA Tech Brief on ICMV Road Map was published to provide guidance in ICMV classification and future development.

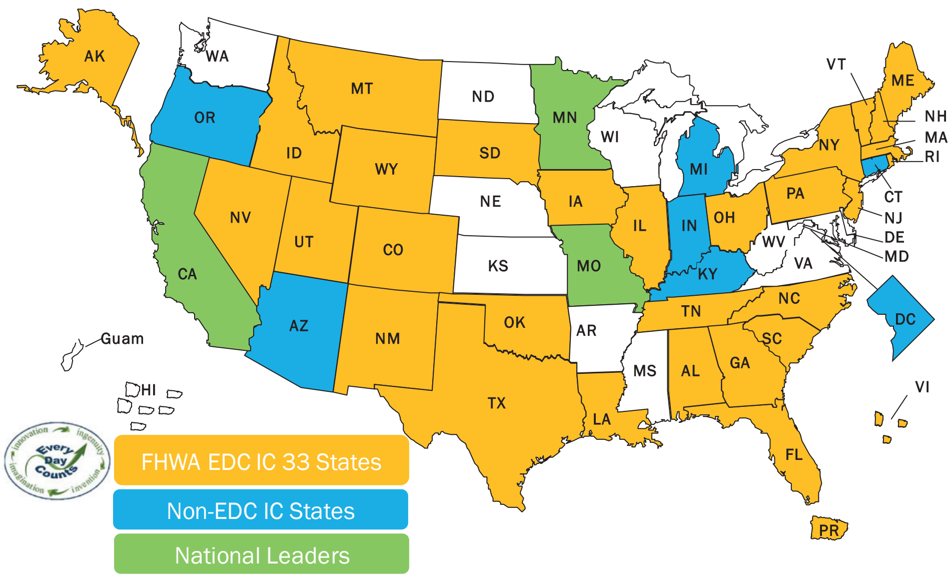

Figure 2. Distribution of US DOTs for IC Implementation.

3 ISSUES AND SOLUTIONS IN US IC IMPLEMENTATION

3.1 Specifications

Issues:

- Lack of IC specifications
- Inconsistent IC specifications across the US
- Lack of IC data exchange/storage standards

Solutions

- National Guidelines/Standards: The US FHWA IC standards were published in 2012 and the AASHTO standards were published in 2014. The AASHTO ICT data standard is anticipated to be published in 2019.
- Local/State specifications: There are more than 26 DOTs that have developed their IC specifications as of 2018.
- Agency-Industry Committees: Many DOTs have formed agency-industry committees or task groups to develop and refine IC specifications.
- Agency initial investment: Many DOTs were willing to pay for the IC cost for initial projects.

3.2 Equipment Supplies and Tech Support

Issues:

- Lack of IC equipment supplies
- Lack of equipment dealers/tech supports

Solutions

- Tackle Chick-&-Egg problem: Supply-and-demand drives the IC market as a constant battle.
- Vendors ramp up equipment supplies: IC vendors and dealers started to stock IC equipment as the market demands started to increase.
- Vendors ramp up national tech support and local dealership training.

3.3 Agency-Institutional

Issues:

- Lack of buy-in of upper management and districts
- Lack of personnel and resources

Solutions

- Benefit-Oriented Executive summary: There are lots of efforts to provide such presentation to executive levels to get their buy-in, whom are in-turn influential in driving the IC implementation at agencies.
- Seek (initial) consulting support: Consulting support is effective to bridge the gap of lack-of-resource at agencies.
- Assign dedicated personnel: Agencies have assigned dedicated personnel and resource to facilitate the IC implementation.
- Create new dedicated positions: Agencies have created new positions and resource to manage the IC implementation.
- Train agency personnel: Agencies started to train their personnel to manage IC projects.

3.4 Training

Issues:

- Lack of training for dealers/tech support
- Lack of training for industry
- Lack of training for agency

Solutions

- Vendors have ramped up training for dealers (and industry).
- Dealers have ramped up training for industry (and agency).
- Consultant training bridges the gap of training for the agencies and industry.

3.5 Field Validation

Issues:

- Lack of practical field validation for GNSS/GPS
- Lack of practical field validation for temperature sensors

Solutions

- Practical procedures for field validation for GNSS/GPS were developed in FHWA and AASHTO IC specifications.
- Practical procedures to validate temperature sensors are expected to begin during the 2018 construction season.

3.6 Certification

Issues:

- Lack of certification for IC Quality Control (QC) managers and technicians
- Lack of certification for equipment

Solutions

- Certification (qualification) for IC QC managers and technicians have begun in CA, MN, etc.
- There are no IC equipment certification programs yet.

3.7 ICMV Levels

Issues:

- Lack of Level 3-4 ICMVs (ref: FHWA Tech Brief on ICMV Road Map)
- Lack of Level 5 ICMVs

Solutions

- It is expected that a National Road Research Alliance (NRRA) study will evaluate the Level 3-4 ICMVs and produce a plan for future IC certification.
- There are not yet Level 5 ICMVs available commercially, but they are under research and development to take IC to the next levels.

4 KEYS TO SUCCESS

The keys to success of IC implementation in US can be summarized as follows:

- Passion: IC champions within agencies or companies are key driving forces.
- Patience: It takes time for IC implementation. Well-thoughts plans are the keys.
- Communication: Forming Expert Task Groups among agency, IC suppliers, earthwork/paving contractors, and consultants are crucial.
- Strategy: Start with Low-Hanging-Fruits to minimize risk.

5 CONCLUSIONS

This document summarizes the issues and solutions of IC implementation in the US. The purpose of this document is to share the lessons-learned during the US implementation since 2004 with issues encountered and solutions responded. Though not intended to be duplicated outside of US, these experiences would be valuable to those countries who are in the process or plan for IC implementation in the future.

REFERENCES

All references for this document can be found in the IC website (www.IntelligentCompaction.com)

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Progress towards implementation of CCC in the UK

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Keywords: Compaction; Specification; Method; Performance

1 INTRODUCTION

The UK Specification for Highways Works (Anon. 2016a; 2016b) currently makes no specific provision for the use of Continuous Compaction Control (CCC) in the construction of earthworks for roads and highways. Such techniques can be used by the constructor but the provisions of the Specification take primacy leaving little incentive to do so.

The work presented here selectively summarises Winter (2017) and is intended to independently demonstrate the proven benefits of CCC in the context of the UK Specification. The background science and technology of earthworks compaction were set out in the context of the UK Specification and of the potential use of CCC. Historic and currently available CCC specifications were reviewed and examples of the use of CCC were provided from real construction projects by Winter (2017).

It was concluded that there was no major technical obstacle to the introduction of CCC to road and highway earthworks in the UK. The work presented by Winter (2017) was intended to help guide this process and to provide an introduction to infrastructure owners and constructors to the potential benefits of CCC while highlighting the limitations of CCC and the potential pitfalls of an uncontrolled introduction.

2 POTENTIAL APPLICATIONS

Three means of using CCC within the construction of UK earthworks, with examples of the use of each being presented by Winter (2017), were identified, as follows:

a) GPS and mapping (QC): As a means of monitoring and recording the coverage of compaction plant including the number of passes applied at a given location. This approach uses the built-in GPS facilities of the plant and supports the effective application of QC procedures to the existing method specification.
b) Soft spots (QA): As a means of identifying 'soft spots' that require further compaction and of monitoring the relative increase in compaction as successive passes are applied. This is particularly helpful in ensuring that over-stressing does not occur resulting in low density and high air voids (Parsons 1992).
c) Performance: As a means of determining the stiffness or relative density of a compacted material subject to performance specification (Carder et al. 2009).

The UK Specification for Highway Works (Anon. 2016a) and the associated Notes for Guidance on the Specification for Highways Works (Anon. 2016b) acknowledge that the prevention of high air void contents within compacted fill materials is the priority if long term weakening and settlement is to be avoided. While the best measure of an acceptable state of compaction is generally accepted to be the dry density and air voids, their field measurement is difficult. For this and other reasons compaction specifications are often written in terms of the method to be used to attain a given compaction state as determined by trials (Parsons 1992).

2.1 The UK Specification for Earthworks

The UK specification for earthworks compaction (Anon. 2016a; 2016b), is primarily based on method specification, in Series 600 and NG600. The method (plant type, plant mass, layer thickness, number of passes) is generally defined such that it will produce an air voids content of 10% or less within a defined moisture content range for a given soil type. Within 600mm of the pavement structure additional compactive effort is required so as to achieve an air voids content of 5% or less. Some selected fills (e.g. fill to structures and pulverised fuel ash) are required to achieve either a relative compaction value (usually relative to a British Standard maximum dry density) and/or a specific maximum percentage of air voids (i.e. the specifications for these materials and/or classes of fill are end product); end product was first introduced to the UK specification in 1986.

These method compaction requirements were based on a programme of research of more than four decades duration and are summarised by Parsons (1992). Underlying the specification was the principle that the constructor should have as free a choice of construction plant as possible and that once the type of plant is selected the specification then sets-out the maximum layer depth and the minimum number of passes required for the particular fill type to be used. The compactive effort thus specified was designed to achieve, in well-graded soils, an average state of compaction of 10% air voids at the lower end of the likely natural moisture content of each soil type. Thus for soils dry of the design condition an air content higher than 10% would be produced, whilst with increasing moisture content beyond the optimum moisture content/maximum dry density condition, dry density would increase at an approximately constant air voids content. Overstressing and shear failure of the soil may occur when the minimum air void condition is reached if excess pore water pressures are generated beneath the compactor and this may lead to remoulding of the surface and rutting beneath the wheels or rollers of the compactor. The principles of the method specification are illustrated in Fig. 1.

Notwithstanding this, controls on the moisture acceptability of fills are applied. This is necessary in order to prevent compaction of the fill in a state too dry of optimum, in which case high stiffness may be achieved at low density and high air voids potentially leading to excessive settlement as the fill subsequently takes on water (e.g. Matheson and Winter 1997; Winter 2004).

The use of CCC does, however, reduce the need for earthworks inspections and reduces the risks associated with such site-based activities.

2.2 Documentation of the Compaction Method

The opportunity to use CCC plant to monitor compliance with a method specification represents the path of least resistance to its use in the UK. A plan of the area to be compacted can be produced showing the number of passes applied and therefore highlighting where additional compaction is required to meet the requirements of the specification.

This information can also be presented to the operator of the compaction plant in real time in order that the process can be adjusted in order to ensure full compliance during the actual compaction operations. Clearly CCC can be used to document the compaction method for any soil to which the compaction plant is suited, whether operated as a vibratory roller or as a dead weight machine.

2.3 Identification of Soft Spots

Continuous Compaction Control can be readily employed as a means of identifying 'soft spots' that require further compaction, and of monitoring the relative increase in compaction as successive passes are applied. This approach can also be used to determine weak areas of an existing subgrade that may require treatment prior to the construction of an embankment, for example.

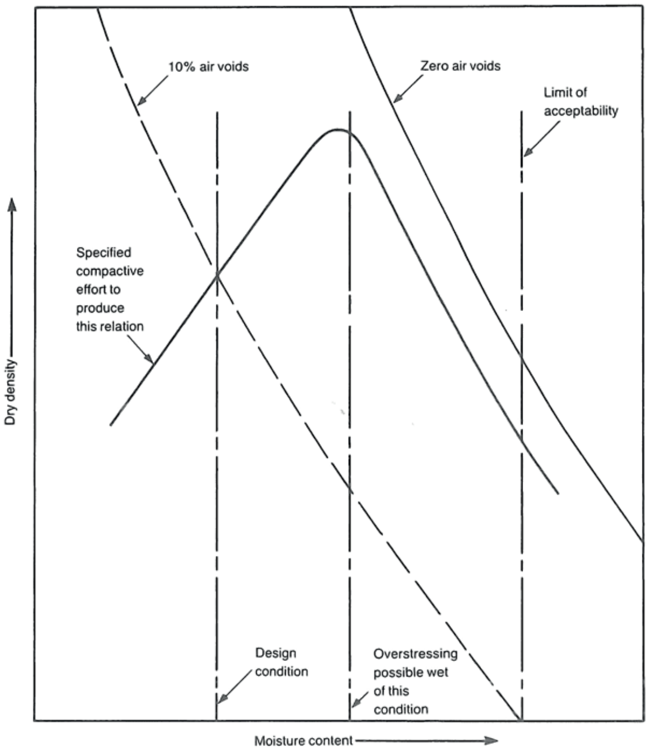

Figure 1. Principle of the method specification for the compaction of well-graded soils (from Parsons 1992).

In general, CCC can be used to identify soft spots or weak areas for any soil type, although if stiffness measurements are used for other than assessing simply which areas are soft/weak then comments regarding the lack of applicability of the resulting values to some soil types prevail (see Section 2.4).

This approach can also be an aid to the prevention of over-stressing thus avoiding a reduction of the stiffness with successive passes as the compaction energy is applied to an already densely compacted soil causing dilation. Such effects can be especially prevalent on sands with low coefficients of uniformity (i.e. those that are of a single-sized nature) (Parsons 1992).

2.4 Performance Specification

The primary use of CCC is as a means of determining the stiffness or relative density of a compacted material subject to a performance specification (Carder et al. 2009) and it seems fair to state that it was this objective that first drove the development of such equipment in the late-1970s and early-1980s. It is important to note that this approach is generally applicable only to earthworks materials for which the proportion passing the 63μm sieve is less than 15%.

Calibration of the stiffness and density is required and this confirms the necessity of the continued application of acceptability testing in order to ensure that the material is in a suitable condition for compaction (Section 2.1). This ensures that high measured stiffness is not achieved in association with low density/high air voids – this is of course essential at both the trial/calibration stage and the implementation stage when the works compaction is undertaken. Thus the pitfalls associated with refocussing the specification from dry density/air voids to a measure of stiffness and/or strength are effectively avoided. This requirement is implicit in, for example, the current German Specification (Anon. 2007) in which limitations on the allowable moisture content are placed.

3 CONCLUSIONS

It is clear that the use of CCC in earthworks compaction offers significant benefits. These include quality control, using GPS, to determine the coverage and number of passes. This can assist the operator to monitor and correct progress in real-time and to reduce the constructor's risk of non-compliance while giving the client an auditable trail, potentially within a BIM system, to demonstrate compliance with a method specification. CCC also can be used for the identification of soft spots as part of a quality assurance programme, again providing confidence for all parties that a reliable earthwork product has been delivered. This can also be an aid to the prevention of over-stressing, when a reduction in stiffness will be apparent with successive passes. Finally, the opportunity to introduce a performance specification to control earthworks construction in a manner more closely to related to in-service performance should be an attractive proposition for all parties. Of course, none of these approaches to the use of CCC removes the need to control the acceptability of the soil.

Notwithstanding the fact that there is relatively little encouragement to use CCC in the UK Specification for Highway Works there is no obstacle to its use. The benefits are considerable and the existence of a specification designed to work with the UK Specification (Carder et al. 2009 and updated by Winter 2017) and the EU Technical Specification (Anon. 2016c) together significantly reduce the number and height of the barriers to the use of CCC.

At the time of writing consultations are ongoing with major UK Overseeing Organisations and constructors to elicit their support and involvement in a trial (or trials) to demonstrate the use of CCC in earthwork construction and related moves are being made by other major infrastructure owners.

ACKNOWLEDGEMENTS

The funding of Bomag (Great Britain) Ltd for this work is gratefully acknowledged.

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Einsatz der FDVK beim Bau des dritten Flughafens von Instanbul

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Schlagwörter: FDVK; Verdichtungskontrolle; Erdbau; Flughafen

1 EINLEITUNG

Zurzeit entsteht 40 km nordwestlich von Istanbul an der Küste des Schwarzen Meeres einer der größten Flughäfen der Welt. Der neue dritte Flughafen von Istanbul soll im Endausbau auf einer Fläche von rd. 78 Millionen Quadratmetern über sechs Start- und Landebahnen verfügen und eine Kapazität für rd. 200 Millionen Passagiere pro Jahr bieten. In der ersten Ausbaustufe wird der Flughafen im Oktober 2018 mit einer Kapazität von jährlich bis zu rd. 90 Millionen Passagieren und 2 Start- und Landebahnen nach nur dreieinhalb Jahren Bauzeit in Betrieb gehen. Die Projektrealisierung und Finanzierung erfolgt als PPP Projekt durch ein Konsortium der türkischen Unternehmen Cengiz, Kolin, Limak, Kalyon und Mapa, die den Flughafen auch über 25 Jahre betreiben und anschließend an den türkischen Staat übergeben werden.

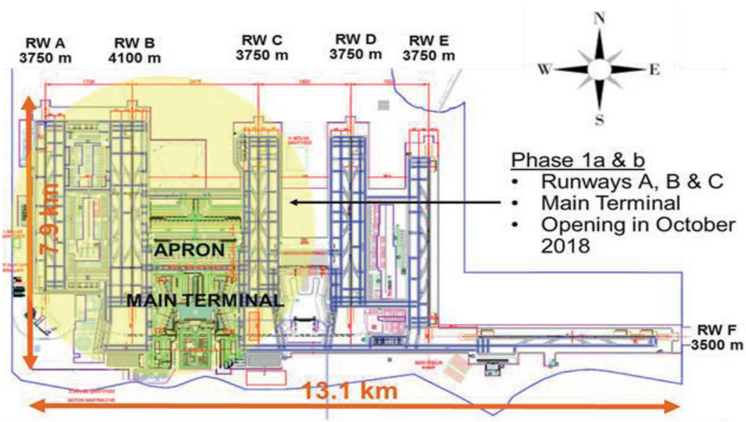

Abbildung 1. Neuer dritter Flughafen von Istanbul.

Der neue Flughafen befindet sich im Gebiet eines ehemaligen Braunkohletagebaues, welcher bis in die 50er Jahre des letzten Jahrhunderts in Betrieb war. Das ursprünglich hügelige Areal mit Geländehöhen zwischen rd. Null bis rd. 120 m über dem Meeresspiegel wurde im Verlauf der Bauarbeiten zu einer ebenen Fläche auf einer Geländehöhe bei rd. 80 m über dem Meeresspiegel umgewandelt. Die Höhe der Aufschüttungen betrug bis zu rd. 70 m. In den Einschnittbereichen wurde das Gelände um bis zu rd. 40 m abgetragen.

In der ersten Ausbaustufe wurden mehr als 400 Mio. m³ Erde bewegt und ca. 350 Mio. m³ kontrolliert lagenweise wieder eingebaut. Hierfür standen ca. 2200 Muldentransporter, 255 Bagger, 116 Raupen, 65 Grader und 158 Verdichtungswalzen zur Verfügung. Bei den im Erdbau eingesetzten Verdichtungswalzen handelte es sich mehrheitlich um schwere 26t Walzenzüge, ausgestattet mit Glattmantel, Schaffuß oder Polygonalbandage. Vierzig 26t Walzenzüge mit Glattmantelbandage verfügten zusätzlich über die gerätetechnische Ausstattung zur Durchführung der „Flächendeckenden Dynamischen Verdichtungskontrolle“ (FDVK). Als Schüttmaterial, welches im Zuge der Aushubarbeiten in den Abtragsbereichen gewonnen wurde, standen im Wesentlichen leicht- bis ausgeprägt plastische Tone der Danişmen Formation zur Verfügung. Die tägliche Einbauleistung betrug, bei einer Schüttlagenstärke von rd. 30 cm, bis zu rd. 750.000 m³.

2 PRÜFKONZEPT IM ERDBAU

Tägliche Einbauleistungen von bis zu rd. 750.000 m³ bzw. von bis zu rd. 2,5 Mio m² verdichteter Fläche erforderten eine Anpassung der Prüfmethodik im Hinblick auf die baubegleitende Überwachung der Qualität der eingebauten Lagen durch die Bestimmung des erreichten Verdichtungsgrades. Die Zielgröße betrug $D_{pr} \geq 95\%$ bezogen auf die modifizierte Proctordichte. Die Anwendung der im Erdbau üblichen konventionellen Verfahren zur Bestimmung der Dichte (z.B. Sandersatz-, Ausstechzylinderverfahren) verdichteter Lagen erforderte, unter Berücksichtigung der bereits optimierten Prüfanforderung von mindestens einer Kontrollprüfung je 4000 m² Fläche (im Normalfall eine Kontrollprüfung je 1000 m²) die Durchführung von mehreren hundert Einzelversuchen pro Tag. Dieser Umfang stellte nicht nur besondere Anforderungen an die Menge und Arbeitsleistung des hierfür vorzuhaltenden Personals als auch an die Logistik zur Verarbeitung und Interpretation der Ergebnisse der Kontrollprüfungen.

Vor diesem Hintergrund wurde das walzengestützte Verfahren der Flächendeckenden Dynamischen Verdichtungskontrolle (FDVK), entsprechend den Anforderungen der CEN/TS 17006 (Technical Specification, Earthworks – Continuous Compaction Control (CCC)), als Hauptverfahren zur Überwachung der Qualität verdichteter Lagen eingesetzt.

Abbildung 2. Angewendetes Prüfkonzept zur Überwachung der Qualität verdichteter Lagen.

Mit dem Einsatz der FDVK konnten Tagesleistungen von ca. 90.000 bis 110.000 m² je Walze erreicht werden. Eine drahtlose Übertragung der gemessenen Daten unmittelbar nach der Fertigstellung jeder Prüffläche, an die mit der Auswertung und Interpretation der Messdaten betrauten Einheiten der Eigenüberwachung, ermöglichte eine weitere Steigerung der Effektivität des Systems zur Qualitätsüberwachung.

Im Fall der Ablehnung einer Prüffläche infolge einer quantitativen Unterschreitung des Zielwertes T_M (Acceptance Value) um mehr als 10% erfolgte eine Überprüfung der identifizierten Flächen durch zusätzliche konventionelle Verdichtungskontrollprüfungen. In Abhängigkeit der Ergebnisse dieser Versuche wurde die Prüffläche für die Überbauung freigegeben oder es wurden zusätzliche Maßnahmen zur Wiederherstellung des qualitätskonformen Zustandes angeordnet (z.B. Nachverdichtung, Austausch nasser Bereiche gegen geeignetes Schüttmaterial einschließlich Verdichtung).

Der durchgeführte Arbeitsablauf zur Überwachung der Qualität verdichteter Lagen ist in Abb. 3 dargestellt.

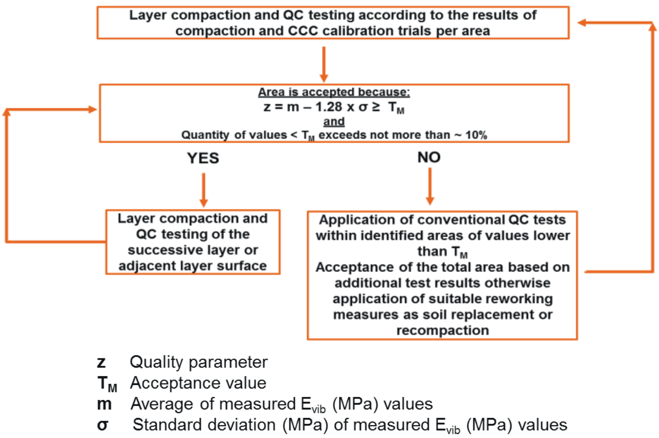

Abbildung 3. Arbeitsablauf zur Überwachung der Qualität verdichteter Lagen bei Einsatz der FDVK als Hauptprüfmethode.

Das eingesetzte Prüfkonzept ermöglichte eine Reduzierung des Umfanges an konventionellen Kontrollprüfungen um bis zu 80% bei einer gleichzeitigen Erhöhung der Qualität und der Aussagefähigkeit der durchgeführten Prüfungen.

3 PRAKTISCHE UMSETZUNG DER VERDICHTUNGS- UND PRÜFARBEITEN

Zu Beginn der Erdarbeiten wurden anhand von mehreren Versuchsfeldern Untersuchungen zur Festlegung des geeigneten Arbeitsverfahrens für die Verdichtung sowie Prüfung mittels FDVK durchgeführt. Hierbei stellte sich heraus, dass eine im Kreis fahrende Kolonne aus 5 bis 6 Walzenzügen die besten Ergebnisse hinsichtlich Arbeitseffektivität und erfolgreicher, qualitätskonformer Verdichtung, erbrachte.

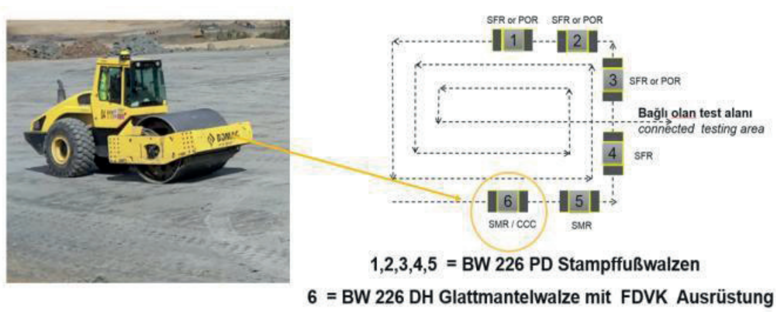

Abbildung 4. Walzschema für 6 im Kreis fahrende Walzenzüge.

Eine statistische Auswertung der gewonnenen Messdaten im Hinblick auf die Walzen- bzw. Verdichtungsgeschwindigkeit in Abhängigkeit der Messergebnisse, der sogenannten z-Werte (Quality Parameter), die zu einer Annahme bzw. Ablehnung einer Prüffläche führen, zeigt den Einfluss der Geschwindigkeit auf den Verdichtungserfolg und die Qualität der gemessenen z-Werte.

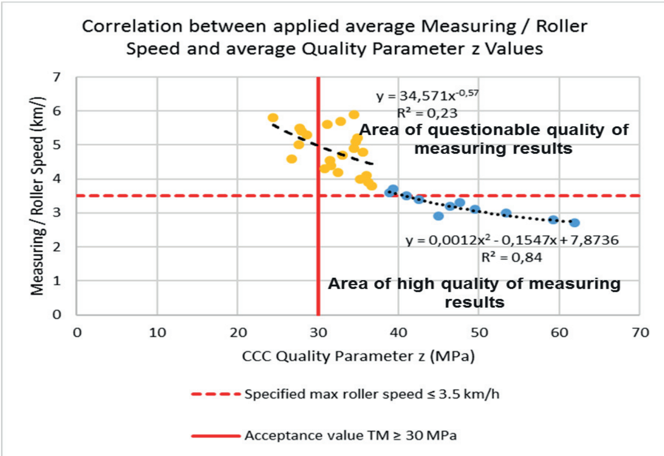

Abbildung 5. Zusammenhang zwischen Walzen- bzw. Verdichtungsgeschwindigkeit und Verdichtungserfolg.

Die in Abb. 5 dargestellten Untersuchungsergebnisse zeigen, dass bis zu einer maximalen Walzengeschwindigkeit von rd. 3,5 bis 4,0 km/h von einer hohen Qualität der FDVK Ergebnisse sowie des Verdichtungserfolges infolge einer geringen Streuung der Einzelwerte ausgegangen werden kann. Bei einer Erhöhung der Verdichtungsgeschwindigkeit beginnen die Werte stark zu streuen, sodass die Qualität und der Verdichtungserfolg entsprechend abnehmen.

Demzufolge führt das in Abb. 4 dargestellte Walzschema für im Kreis fahrende Walzenzüge zu besseren Ergebnissen als im Parallelbetrieb. Dieses Verfahren ermöglicht eine einfache, aber effiziente Kontrolle der Geschwindigkeit aller im Bereich der Prüffläche operierenden Walzen. Dies ist im konventionellen Parallelbetrieb nicht möglich.

4 ZUSAMMENFASSUNG

Bei den Erdbauarbeiten des neuen dritten Flughafens von Istanbul stellten die sehr hohen Einbauleistungen von täglich bis zu 2,5 Mio. m² die Qualitätssicherung der einzubauenden Schüttlagen vor eine besondere Herausforderung. Mit Hilfe der walzengestützten, flächendeckenden, dynamischen Verdichtungskontrolle und ergänzenden konventionellen Einzelprüfungen konnte für die einzubauenden leicht bis mittelplastischen Tone ein leistungsfähiges, arbeitsintegriertes Prüfkonzept praktiziert werden, das eine Reduzierung des konventionellen Prüfumfanges um bis zu 80 % ermöglichte.

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Influence of vibratory compaction on slope stability ongoing research in Norway

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Keywords: Vibration influence zone; Slope stability; Analysis procedure

1 INTRODUCTION AND BACKGROUND

A landslide-triggered tsunami caused extensive material damage at Statland, Norway in 2014. The investigation of technical cause concluded the slide was likely triggered by vibratory compaction (NVE, 2014). During a follow-up study (NVE, 2016), a basis on how to account for vibrations during vibratory compaction near slopes with vibration sensitive material has been established and is described here.

Vibrations from construction activities rarely induce landslides, but there have been a few earlier cases where compaction work was a contributing factor or trigger. Bernander (2011) describes a landslide south of Uddevalla, Sweden in 1990. It was most likely triggered by a heavy vibratory roller which caused the failure of a berm designed to provide additional stability to an embankment for the E6 highway. The Åsele-slide in 1983, also in Sweden, (Ekström and Olofsson 1985) caused the failure of a road embankment that was partially submerged due to filling up of hydro-electric reservoir. The slide was triggered by a tractor-drawn 3.3 ton vibratory roller. The 1987 Lake Ackerman slide caused damage to Highway 94 in Michigan, USA (Hryciw et al. 1990). Six 22-ton trucks generating vibrations for a seismic refraction study triggered the slide. The road embankment consisted of hydraulic fill of loosely deposited sand. Hryciw et al. estimated the shear strains induced in the fill of up to 0.06%, which is enough to generate pore pressure build-up and cause failure in fine silty sand (Dobry, 1985).

2 VIBRATIONS DUE TO VIBRO COMPACTION

Ground vibrations from vibratory rollers transmit large loads to the soil which can cause build-up of pore pressure and reduce soil strength in vibration susceptible soils such as loose silt and sand, and sensitive clays. This should be considered when carrying out construction work near slopes with such soils. The strength reduction is dependent on soil state, load amplitude and number of cycles.

Vibratory roller compaction is performed by passing over the same area up to 8 times (NS 3458) which means a soil element is exposed to a large number of vibration cycles. The number of load cycles a soil element is subjected to depends on the speed of the roller, the vibration frequency and the depth. Vibratory rollers have vibration frequencies between 20-40 Hz and both the load amplitude and vibration frequency varies with the type of soil and the thickness of the compacted layer and the speed is usually between 0.5 m/s to 1.5 m/s. NVE (2014) estimated soil elements were subjected to several hundred cycles. A shallow soil element is in general subjected larger amplitudes than a deep soil element. Even though a deeper soil element is subjected to smaller vibration amplitude it is influenced by vibratory equipment over a wider area.

To estimate the effect of compaction induced vibrations on a slope with vibration sensitive material one can use empirical equations (see e.g. Caltrans 2013) to estimate vibration amplitudes. Such equations give vibration amplitude on the ground surface while the slope failure is likely to be induced at some depth beneath the vibratory equipment. Field measurements of vibration amplitudes in vibration susceptible soft/loose material are not available in the literature, therefore we have in first phase resorted to numerical modelling which should be followed up with field measurements for validation.

2.1 Numerical modelling of compaction vibrations

The effect of vibratory roller compaction on soil degradation has been evaluated with a finite element model. Based on a 2D axis-symmetric FE-model with Comsol Multiphysics, frequency domain analysis of vibratory compaction has been performed. The details of the modelling are given in (Johansson, 2017, and NVE, 2016).

The effect of vibrations varies with parameters such as soil properties and nonlinearity, bedrock depth and geometry, and presence of thin soft layers. To develop a vibration measurement procedure, we suggest, based on a threshold shear strain of 0.025%, a 10 m wide and 5 m deep influence zone around the vibratory equipment outside of which the soil strength is not reduced. The influence zone in shown in **Figure 1**. A tentative vibration limit is set to 10 mm/s near slopes with vibration susceptible materials.

In addition to the load amplitude, the loading frequency of the compaction equipment can have a large impact on the vibration amplitudes at depth, due to resonance effect in the soil deposits. Further studies of the effect of vibration frequency on the induced shear strains are needed, especially since there is ongoing development to make use of resonance effects to increase compaction efficiency (Wersäll et. al 2018).

The experimental results of Wersäll et. al 2018 have been used to validate a novel numerical procedure which accounts for soil nonlinearity with help of an equivalent linear model to compute the non-linear dynamic response of the drum and soil interaction. The equivalent linear model will be used further to investigate the effect of compaction vibrations on slope stability.

3 TENTATIVE PROCEDURE FOR EVALUATING SLOPE STABILITY ACCOUNTING FOR VIBRATIONS

Here follows a proposal for a three step procedure for dealing with a potentially vibration susceptible slope. A flow chart for the procedure is given in (NVE 2016). The first step is to evaluate the geology and if the soil type is vibration susceptible. The second step is to perform a slope stability evaluation accounting for the effect of construction vibrations. If the stability does not fulfil the requirements, the third step is to suggest remedial measures such as vibration measurements and/or increasing the stability before construction work starts.

3.1 Evaluate soil type and vibration susceptibility

Step 1: First evaluate the geology and soil types. A detailed geological and geotechnical investigation is performed to evaluate if the slope consists of vibration susceptible soil types.
Step 1a: Determine the type of geological deposit at the site and nearby areas. Information in (NVE 2016) can be used to determine the probability for vibration induced strength degradation based on terrain and geological maps. Also determine if the area is within or near an existing landslide hazard zone.
Step: 1b. Determine if the soils are vibration susceptible. Different figures for evaluating vibration susceptibility of clayey and sandy soils are compiled in NVE (2016) based on a literature review. Also

evaluate if the soil deposit is layered. Layered soil deposits are more vibration susceptible and are often found in fjord and fluvial deposits, near shore areas and deltas.
If the site under consideration is close to a hazard zone or if the soil is considered vibration susceptible based on the evaluation in step 1 above, a pseudo static stability analysis is recommended as described below.

3.2 Slope Stability evaluation

If the slope contains vibration susceptible soil types a slope stability analysis to account for the effect of construction vibrations is recommended. The method suggested here is based on the use of a two-dimensional limit equilibrium analysis considering the effect of vibrations from compaction equipment on the local slope stability. In Norway there exists also a concept of area-stability, in which the possible retrogression has to be accounted for in the slope stability evaluation, which is not described here. Point 1-7 below are under the assumption that the static slope stability does not fulfil the requirement by the design standards. If the static slope contains vibration susceptible material but static stability fulfils the requirements in the design standard it is recommended for documentation purposes to only perform limited vibration monitoring, as indicated in Figure 1.

1. Identify critical profiles, based on terrain and rock geometry, and distance to vibration source etc.
2. A 10 m wide and 5 m deep zone is assumed to subjected to strength degradation due to vibrations, as shown in Figure 1.
3. Determine residual strength for the zone subjected to strength degradation with help of figures and equations compiled in NVE (2016). The strength will depend on material type and number of vibration/loading cycles. The number of cycles is on the order of 100 to 1000, and depends on the size of the equipment and number of roller passes. There is figure in NVE (2016) for correcting strengths for number of cycles. If unknown use 1000 cycles for determining a correction factor. It is also recommended to compare the selected residual strength with empirical equations from the literature (Olson and Stark 2002).
4. Selection of drained and undrained strengths not subjected to strength degradation is performed according to geotechnical practice.
5. Stability during and before construction activities should be compared to understand the effect of vibratory compaction on slope stability.
6. Compare results with requirement in design standard for failure mechanisms and consequence classes.
7. If stability requirements are not fulfilled more detailed analysis or remedial measures are necessary to increase the stability.

3.3 Vibration measurement

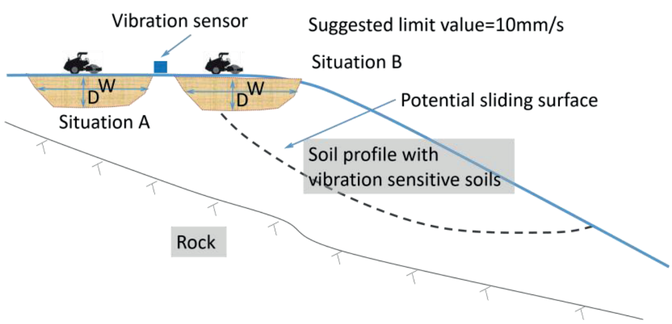

Figure 1. Zone subjected to strength degradation due to vibratory compaction. Vibration sensor location is tentative and has to be adapted to the specific project. The idea is to measure vibrations as close as possible to vibratory equipment and the potential sliding surface.

4 CONCLUSIONS

The case histories show vibratory compaction could cause failures of slopes with vibrations susceptible soils. Often thin layers of silt and sand are involved. Thus, it is important to investigate the existence of such layers.

Resonance effects due to soil deposit geometry or resonance in the interaction between soil and vibratory equipment can increase the loads on the slope. The effect of loading frequency needs further analysis.

A simple procedure for vibration susceptibility and slope stability analysis accounting for compaction vibrations has been proposed. The method is conservative and will likely result in the slope not fulfilling the safety requirements in the standard and thus slope improvements are needed before performing compaction work.

The sizes of the influence zone and the tentative vibration limit of 10 mm/s are based mainly on numerical analysis and further field measurements of compaction vibrations are needed for validation. Such measurements are being planned in the Remedy (2018) project.

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MDP and CMV	✓			
Temperature			✓	✓
GNSS Mapping	✓	✓	✓	✓
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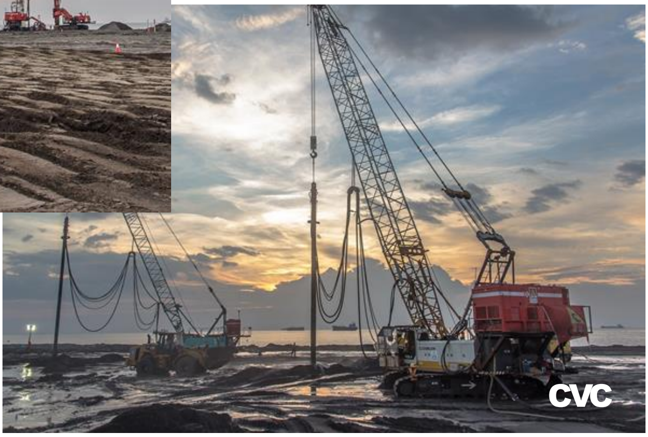

CONSOLIDATION – COMPACTION – ELEMENTS - BARRIERS

Research & Development
Novel Compaction Techniques

Roller compaction of rock-fill with automatic frequency control

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Keywords: Compaction; Frequency; Roller; Resonance

1 INTRODUCTION

Conventional vibratory soil compaction rollers are operated under a drum vibration frequency that is fixed or manually adjustable within a narrow range. The compaction frequency is generally far above the resonant frequency of the coupled roller-soil system and has traditionally been focused on obtaining a high centrifugal force while avoiding double-jump of the drum. However, recent full-scale tests have shown that resonance in the roller-soil system can be utilized to obtain a more efficient compaction process at a significantly lower frequency and at the same time reducing energy consumption and machine wear (Wersäll et al. 2017, Wersäll et al. 2018). The concept of using resonance to amplify vibrations in the soil to obtain better compaction was proposed already in the early age of vibratory compaction (Bernhard 1952, Converse 1953) but was not applied in practice due to technical limitations at the time. The first application of resonant compaction was in deep compaction using vibratory probes (Massarsch 1991). Based on results from the recent full-scale tests, a prototype roller has been developed by Dynapac that automatically and continuously adjusts the compaction frequency based on integrated roller measurements in order to obtain resonance in the roller-soil system. The concept is denoted automatic frequency control (AFC) and results in a significantly lower compaction frequency. This paper describes results from the first field study using AFC to compact a trial embankment consisting of rock-fill.

2 METHODS AND MATERIALS

The tests were carried out adjacent to the new freight railway line Kardonbanan in Norrköping, Sweden, which is currently under construction. The prototype roller is based on a Dynapac CA6500D smooth-drum soil compaction roller with static linear load 65 kg/cm and total weight 20 900 kg (Fig. 1). It can be operated both conventionally with a fixed frequency (28 Hz for high amplitude and 30 Hz for low amplitude) and with AFC. The AFC frequency ranges for the prototype roller are 17-28 Hz for high amplitude and 17-33 Hz for low amplitude. The nominal amplitudes are 2.1 mm for the high setting and 0.8 mm for the low setting.
The trial embankment (Fig. 1) was 1 m thick and consisted of crushed rock 0-150 mm in accordance with Swedish regulations for sub-ballast material and was placed directly on a compacted subgrade of gravelly silty till. The high fines content of the subgrade (around 35 %) in combination with heavy rains preceding the test resulted in a low stiffness which was quantified by static plate load tests (PLT) on the subgrade. The average reload modulus, E_{v2}, of the subgrade was equal to 65 MPa. The embankment was compacted in only one layer, i.e. at the full height. However, to facilitate installation of monitoring equipment, it was placed in two layers by a bulldozer. Heavy construction traffic caused some initial compaction in the surface of the bottom half of the layer, as well as at the top.

Figure 1. Compaction tests (left) and trial embankment (right).

The embankment was 10 m wide (excluding slope) and 28 m long (excluding ramp) and was not compacted over its entire surface. In order to study the relative effects of AFC, two parallel paths were compacted, spaced 2 m apart from edge to edge and with widths equalling the width of the drum, 2.13 m (Fig. 1). The instrumented portions of the compaction paths were 20 m long, centred along the length of the embankment surface. One path was compacted with AFC and the other conventionally using a fixed frequency. The total number of passes was 32 for each path and every pass was compacted with high amplitude.

The settlement of the surface was measured in 63 points per path after 0, 2, 4, 8, 12, 16, 20, 24 and 32 passes, resulting in a total number of 1134 settlement measurements. Static plate load tests were conducted in accordance with the German standard DIN 18134 (Deutsches Institut für Normung 2012) in 8 points per path after 8 and 32 passes, resulting in a total of 32 tests. Falling weight deflectometer (FWD) measurements were conducted in 4 locations along each path after 32 passes, with the loads 30 kN and 50 kN, i.e. a total of 16 tests.

Settlement within the embankment was measured by inductance coil sensors (Epsilon Measuring Unit, EMU), installed in two sections per compaction path. The coils consist of copper wire, wrapped numerous times around a circular disc, and placed in vertical alignment in the embankment material. Inducing an electric current in one coil gives rise to a magnetic field, which is detected by adjacent coils and can be correlated to the distance between them. Thus, the EMU system reveals the depth-dependent settlement in the embankment. Each measurement section consisted of six coils, resulting in five measurement distances from the subgrade level up to 30-50 mm below the embankment surface. Strain readings were performed after 0, 2, 4, 8, 12, 16, 20, 24 and 32 passes.

3 RESULTS

Fig. 2 shows the vibration frequency of the drum, which is constant at 28 Hz for conventional fixed-frequency compaction, and varying with AFC. Since AFC automatically adjusts the frequency to approach resonance, it becomes significantly lower. The adjusted frequency is somewhat affected by the driving direction (forward: odd passes, backward: even passes) since the interpreted soil stiffness depends on the rotation of the eccentric mass in relation to the rotation of the drum. Fig. 2 also shows the resulting displacement amplitude. The large increase in amplitude for AFC, which is the result of resonance, is remarkable considering the significantly reduced centrifugal force at the lower frequency (centrifugal force is proportional to the square of frequency). The average settlement of the surface is shown in Fig. 3. The total settlement is slightly larger at fixed frequency but these results do not reveal the relation between the compression of the embankment and the subgrade.

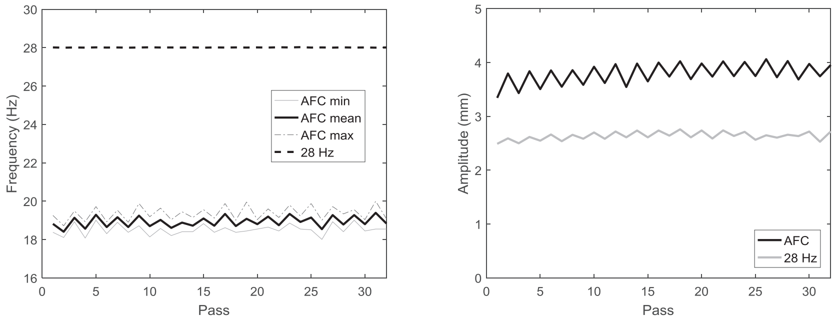

Figure 2. Compaction frequency (left) and displacement amplitude (right).

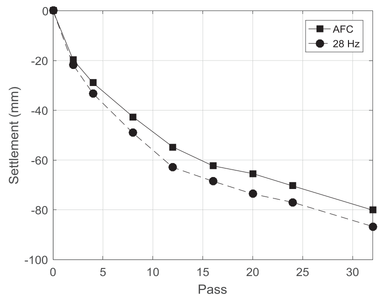

Figure 3. Settlement of the embankment surface.

Fig. 4 shows results from the EMU measurements. The presented compressions are the averages between the two measurement sections in each compaction path. The total compression of the embankment layer is greater with AFC than with fixed frequency. In the compression profile after 32 passes, it is observed that fixed frequency has a slightly higher depth effect while AFC produces a significantly better compaction close to the surface. This is due to the dynamic interaction between the drum and soil, where the drum, at resonance, takes advantage of the oscillation of the soil whereas the high-frequency oscillation cause the drum to move out of phase with soil, resulting in compaction energy being converted into friction. The reduced compression close to the bottom of the layer is due to initial compaction, resulting from the placement procedure, as discussed above. Better compaction at the surface, as obtained by AFC, can eliminate the need for subsequent static or low-amplitude passes. The greater depth effect at high frequency is likely due to the shorter distance between impacts and is causing significant settlement of the quite soft subgrade, which is the reason for the higher settlement at the surface in Fig. 3.

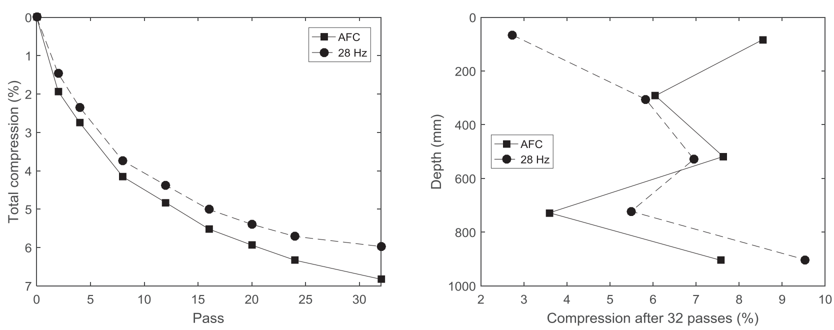

Figure 4. Total compression (left) and compression profile after 32 passes (right) from EMU sensors.

Table 1 shows the average measured E_{v2} from PLT after 8 and 32 passes and the average surface modulus from FWD after 32 passes. The stiffness on the surface after 8 passes is very similar for the two compaction methods. After 32 passes, however, there is a significantly higher stiffness in all measurements on the part of the embankment compacted using AFC, which is a result of the improved compaction effect at the top of the layer. This shows that the end result is better with AFC. Furthermore, double-jump, which has a negative effect on compaction and machine wear, occurs for fixed frequency after approximately 18 passes while it cannot be observed in the 32 passes of AFC.

Table 1. Average stiffness values from PLT (E_{v2}) and FWD (surface modulus)

	PLT (8 passes)	PLT (32 passes)	FWD 30 kN (32 passes)	FWD 50 kN (32 passes)
AFC	108 MPa	157 MPa	197 MPa	201 MPa
28 Hz	105 MPa	135 MPa	168 MPa	170 MPa

4 CONCLUSIONS

Resonant compaction with automatic frequency control was tested under field conditions. By utilizing resonance in the coupled roller-soil system, dynamic strains in the subgrade are amplified, while reducing conversion of compaction energy into friction at the surface and thereby optimizing the dynamic roller-soil interaction. Previous tests have indicated that compaction can be improved while significantly reducing energy consumption and machine wear when compacting at a lower frequency. This study shows that resonance can be obtained by automatic parameter adjustment and that AFC improves the end result in relation to stiffness acceptance criteria. The main improvement is at the top of the compacted layer, resulting in a considerable increase in surface stiffness, which can eliminate the need for subsequent passes. It is thus feasible to reduce the total number of passes using AFC and in turn the operating time. The trial embankment was compacted in single roller paths and in a single layer on relatively soft subsoil. It is recommended that the difference between conventional compaction and AFC is studied during construction of a full multiple-layer embankment with compaction of the entire surface.

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Innovative Ideen bei Messungen am Walze-Boden-Interaktionssystem bei der dynamischen Verdichtung

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Schlagwörter: FDVK; Flächendeckende Dynamische Verdichtungskontrolle; Messung

1 ALLGEMEINES

Die Flächendeckende Dynamische Verdichtungskontrolle (FDVK) ist eine Methode der arbeitsintegrierten Dokumentation und Überwachung des Verdichtungserfolges bei der Bodenverdichtung durch dynamisch angeregte Walzen. Das Bewegungsverhalten der Walze wird gemessen und Rückschlüsse auf die „Bodensteifigkeit“ werden aus diesen Signalen gezogen. Stand bei den frühen FDVK-Messungen noch der empirische Zugang zwischen dem Bewegungsverhalten der Walze und der Bodensteifigkeit im Vordergrund, so konnten in den vergangenen Jahrzehnten durch die systematische Analyse des Walze-Boden-Interaktionssystems das Kontaktproblem Walze-Boden, die Betriebszustände, in denen dynamische Walzen arbeiten, und die Kraft-Verformungsrelationen erforscht und FDVK-Messgrößen auf physikalischer Grundlage entwickelt werden. Aufgrund dieser Erkenntnisse wurden auch dynamische Walzen mit stufenlos variablen Prozessparametern (Frequenz, Vertikalamplitude, Fahrgeschwindigkeit etc.) entwickelt, welche in der Lage, sind diese zu regeln und automatisch optimal auf den zu verdichtenden Boden einzustellen. Die Anwendung der FDVK ist mittlerweile zur Routine auf Baustellen geworden, sie ist aus dem modernen Erdbau nicht mehr wegzudenken und ist in vielen Ländern durch Vorschriften geregelt, welche den Stand der Technik festschreiben und praktikabel anwendbar machen.
All dies wäre nicht möglich gewesen, wenn nicht systematisch Messungen und Experimente durchgeführt worden wären. Einerseits ist die Messung an der Walze (Bandagenbeschleunigung an der Lagerschale, Frequenz, Lage der Unwucht, Fahrgeschwindigkeit und -richtung, Exzentrizität, Position der Walze) ein integraler Bestandteil jeder FDVK-Methode. Anderseits waren und sind für die Grundlagenforschung, die angewandte Forschung, die bodenmechanische Interpretation sowie die Entwicklung und Erprobung verbesserter FDVK-Messmethoden Messungen und Experimente erforderlich, die weit über die Messungen der FDVK Standardprozedur hinausgehen.

2 STANDARDMESSUNG DER FDVK AN DER WALZE

2.1 Bandagenbeschleunigung an der Lagerschale

Die Messung der Walzenbewegung ist die Grundlage jeder FDVK-Messung. Da die Bandage während der Verdichtungsfahrt rotiert, kann die Beschleunigung (horizontal in Fahrtrichtung und vertikal) nur an der Lagerschale gemessen werden. Bei den meisten Walzen für den Erdbau kann die Bandagenbeschleunigung nur an der Seite des Antriebsmotors für die Unwucht messtechnisch erfasst werden, da auf der Seite des Fahrmotors die dynamische Entkoppelung mittels Gummipuffern bereits an den rotierenden Teilen erfolgt.

2.2 Lage der Unwucht

Die Lage der Unwucht ist wichtig für die Bestimmung des Phasenwinkels zwischen der Zentrifugalkraft der Unwucht und der Bandagenbewegung. Unter Einbeziehung der Massenträgheit der Bandage kann die Bodenkontaktkraft bestimmt werden. Die Relation zwischen Bodenkontaktkraft und Bandagenverschiebung bildet in Form eines Arbeitsdiagrammes die Grundlage für die meisten modernen FDVK-Messwerte.

2.3 Position der Walze

Jeder der kontinuierlich ermittelten FDVK-Messwerte muss in der Dokumentation einer Position im Feld zugeordnet werden. Durch die Messung der Raddrehung (bzw. Walzendrehung) kann eine Längenmessung vorgenommen werden; sie dient auch der Messung und Regelung der Fahrgeschwindigkeit. Für die Dokumentation ist jedoch die Zuordnung zu den einzelnen Fahrspuren manuell erforderlich. Wird die Position der Walze über GPS (Global Positioning Sysem) dokumentiert, so können FDVK-Systeme die Messwerte automatisch auf den gefahrenen Spuren im Grundriss in globalen Koordinaten darstellen. Diese Systeme sind einfacher zu bedienen und haben eine größere Sicherheit gegen Manipulation, funktionieren jedoch nicht immer zufriedenstellend.

3 MESSUNGEN AN DER WALZE ZUR GRUNDLAGENFORSCHUNG

3.1 Winkelmessung zur Bestimmung der Lage der Unwucht

Die Bestimmung der Lage der Unwucht erfolgt standardmäßig mittels Näherungssensor bzw. über die Auswertung der Horizontalschwingung in Fahrtrichtung. Für die Standardmessung ist diese Genauigkeit in der Regel ausreichend, setzt jedoch eine gleichförmige Rotation der Unwuchtwelle voraus. Diese muss gerade im Sprungbetrieb von Vibrationswalzen nicht zwingend gegeben sein. Für die Forschung, die Kalibrierung der Standardmethoden und die Fehlerabschätzung werden Sensoren zur kontinuierlichen Winkelmessung an der Unwuchtwelle eingesetzt.

3.2 Bestimmung der Lage der Bandage

Es gibt FDVK-Systeme, die bereits auf geringe Imperfektionen und Unausgewogenheiten der Bandage äußerst empfindlich reagieren (besonders bei Oszillationswalzen), weil bei der Rollbewegung der Bandage die FDVK-Messungen durch ein periodisches Signal überlagert werden. Zur Erforschung und Kompensation dieser störenden Einflüsse ist die Kenntnis der Lage der Bandage erforderlich. Bei konstanter Fahrgeschwindigkeit genügt ein Sensor, der bei jeder Umdrehung ein Signal abgibt; dazwischen kann linear interpoliert werden. Es werden aber auch Sensoren zur kontinuierlichen Winkelmessung bei der Bandagenrotation eingesetzt.

3.3 Bestimmung der Aufstandsbreite der Bandage

Die dynamische Walzen-Boden-Interaktion wird maßgeblich von der Kontaktbedingung zwischen Walze und Boden bestimmt. Durch die gekrümmte Bandagengeometrie hat die Kontaktfläche eine variable Größe. Bei weichen Bodenbedingungen und hoher Vertikalbelastung durch die Walze kommt es zu einer großen Setzungsmulde, die Bandage dringt definiert in den Boden ein und die Aufstandsfläche wird größer. Vor allem im Auflastbetrieb (z.B. Oszillation oder horizontal gestellte Vario-Walze) lässt sich die Aufstandsbreite messtechnisch gut bestimmen. Es werden in den Viertelpunkten des Bandagenumfanges Sensoren (Wegsensor oder Pfeife) installiert, welche den Kontakt mit dem Boden registrieren. Mit Hilfe eines vertikal genau gegenüberliegenden Referenzpunktes kann nicht nur die gesamte Bandagenaufstandsbreite sondern auch deren Asymmetrie (längerer Teil auf der Seite der Bugwelle) bestimmt werden.

3.4 Messungen auf der rotierenden Bandage

Wenn die Kenntnis der Lagerbeschleunigungen nicht ausreichend ist, können an der rotierenden Bandage, beispielsweise an den Viertelpunkten des Umfangs, die Beschleunigungen gemessen werden. Die Beschleunigungen werden entweder in einem synchronisierten, an der dynamisch angeregten Bandage angebrachten Messsystem erfasst und gespeichert oder mittels eines Telemetriesystems von der rotierenden Bandage auf den Walzenrahmen übertragen und vom Messsystem, welches mit der Walze mitfährt, aufgenommen. Mit diesen Daten kann z.B. der Rotationsmittelpunkt von Oszillationswalzen ermittelt werden.

3.5 Messung der Rahmenbeschleunigung

Die dynamisch angeregte Bandage ist über elastische Elemente (Gummipuffer) mit dem Rahmen der Walze verbunden. Diese Elemente sorgen für eine dynamische Entkoppelung, sodass der Rahmen weitgehend vibrationsfrei bleiben und die Bandage mit einer annähernd statischen Kraft belasten sollte, um ihr dynamisches Verhalten möglichst wenig zu beeinflussen. Mittels Beschleunigungsmessungen am Rahmen kann dessen dynamischer Einfluss auf die Bandage bestimmt werden.

3.6 Position der Walze

Die Bestimmung der genauen Position der Walze auf einer Spur eines Testfeldes kann mittels Laser-Distanzmessung (kontinuierlich) zu einem Fixpunkt oder mittels Lichtschranke (z.B. Spuranfang und -ende) realisiert werden.

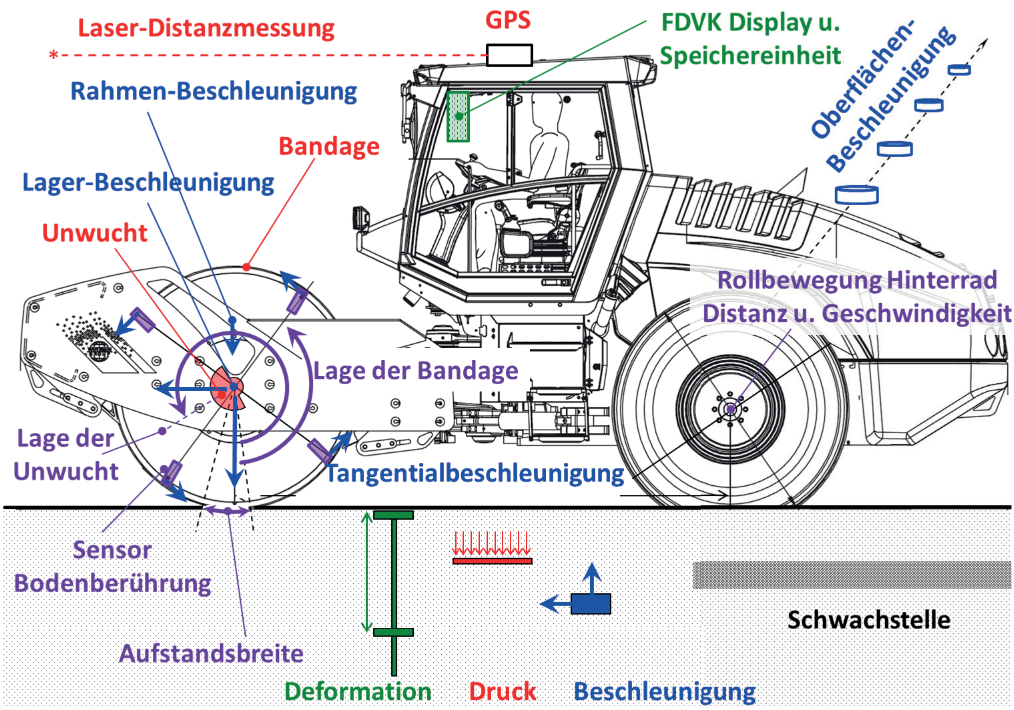

Abbildung 1. Mögliche messtechnische Instrumentierung des Walze-Boden-Interaktionssystems.

4 MESSUNGEN IM BODEN ZUR GRUNDLAGENFORSCHUNG

4.1 Schwachstellenversuch

Messungen zu Forschungszwecken finden in der Regel auf definierten Testfeldern statt. Um eine Variation der Bodensteifigkeit herbeizuführen, können künstliche Schwachstellen überschüttet oder Schichten keilförmig geschüttet werden. Die Bestimmung der Verdichtungstiefe und der Messtiefe ist auf diese Weise möglich.

4.2 Druck im Boden

Mittels Drucksensoren sind die dynamischen Bodenspannungen messbar. Auf den Kontakt zwischen Korngerüst und Druckpolster sowie die Steifigkeitsverhältnisse ist Bedacht zu nehmen.

4.3 Relativverschiebung im Boden

Die Bodenverdichtung kann über die Messung der vertikalen Relativverschiebungen, das „Dünnerwerden der Schichten“, dynamisch verfolgt werden.

4.4 Beschleunigung im Boden

Bei der Messung der Beschleunigungen im Boden sollen sich die in unterschiedlichen Tiefen angebrachten Beschleunigungssensoren so gut wie möglich in die Kornstruktur des Bodens integrieren (Abmessung, Dichte), um so wenig wie möglich zu stören.

4.5 Beschleunigung an der Oberfläche

Zur Beurteilung der Erschütterungswirkung unterschiedlich angeregter dynamischer Walzen auf die Umgebung kann die Erschütterungsausbreitung an der Oberfläche, durch Messung mit einer Reihe an Sensoren, quer zur Fahrtrichtung aufgestellt, bestimmt werden.

4.6 Vergleichsversuche im Boden bzw. an der Oberfläche des Testfeldes

Zwischen bzw. nach den Messfahrten können weitere Messungen und Versuche im Boden bzw. an dessen Oberfläche vorgenommen werden. Die Setzungsmessung mittels Nivelliergerät, die Messung der Wellenausbreitungsgeschwindigkeit im Untergrund, Vergleichsversuche zur Bestimmung der Dichte (Ersatzverfahren, Isotopensonde) und der „Tragfähigkeit“ (statischer und dynamischer Lastplattenversuch) dienen zur Korrelation mit den FDVK-Messwerten und zur Beurteilung der Verdichtungswirkung der Walze. Die Bestimmung der Bodenkennwerte des verdichteten Materials im Erdbau-Labor ist obligatorisch.

5 SYNCHRONISATION DER MESSUNGEN AN DER WALZE UND IM BODEN

Ziel jeder Messung ist es, einen einzigen Datensatz zu generieren, der die Daten sämtlicher autarker Messsysteme, an der rotierenden Bandage, auf der fahrenden Walze, bei den im Boden installierten Sensoren und bei den Messungen im Fernfeld etc., exakt synchronisiert zusammenführt. Dies kann mittels Funksignalen über CB-Funk, mittels eigener Funkmodule oder mittels Synchronisation über Satelliten-Zeitsignal realisiert werden.

6 ZUSAMMENFASSUNG UND AUSBLICK

Die FDVK birgt bereits systemimmanent die Messtechnik in sich. Für eine erfolgreiche Weiterentwicklung und den Erkenntnisgewinn durch Grundlagenforschung und angewandte Forschung werden experimentelle Untersuchungen und neue Ideen zur messtechnischen Erfassung der auftretenden Phänomene auch in Zukunft unerlässlich sein.

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Anniversary Symposium – 40 Years of Roller Integrated Continuous Compaction Control (CCC)
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Measuring soil compaction on dynamic compaction technologies - Field tests and laboratory investigations using the PIV method

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Keywords: PIV-Method; 1g model test; field investigation; Dynamic Compaction

1 INTRODUCTION

Contrary to modern rollers, dynamic compaction technologies have a lack of process accompanying compaction monitoring systems, like the continuous compaction control CCC. Nowadays only empirical criteria of compaction, for example a specific amount of impacts are used practically. A comprehensive study focused on the theoretical development of a criterion of compaction for dynamic compaction technologies is missing. Several experimental studies (Poran et al. 1992), (Hajialilue-Bonab and Rezai 2009) investigated the densification mechanisms of soil subjected to impact loads in scaled laboratory tests. Furthermore field studies are presented which investigate possible metrics of soil densification, measured on the compactor (Kopf and Paulmichl 2005), (Adam et al. 2011). Other field studies are focused on the validation of numerical models (Kirstein et al. 2016). We are interested in the measurable interaction of the compaction technology and the soil and its interdependencies. Therefore we carried out experimental studies with the PIV/DIC - Method (Particle Image Velocimetry / Digital Image Correlation) on small scaled models in the field of gravity, so called 1g model tests. Within this article we present the reliance of the depth of the crater and the maximum acceleration of the falling weight. The results of these experiments show similarity with the theoretical densification model called common compaction curve (Sawicki and Swidzinski 1989). The laboratory results regarding the behaviour of the falling weight are in good compliance with recent field investigations done in real scale. These results increase the understanding of the mechanism of densification done by dynamic compaction technologies and are another stride towards a compaction monitoring system for transient dynamic soil compactors.

2 METHOD

2.1 Laboratory investigation using the PIV-Method

The experimental setup used for the laboratory investigation is presented in Figure 1(a) schematically. The soil specimen made of dry sand has a dimension of 880 x 620 x 400 mm (length x height x width) and is prepared by air raining. With this method of sample preparation the initial density of the specimen was $\varrho_0 = 1.62$ g/cm³. The used raining method is presented more into detail in (Trudeep and Dasaka 2012), (Knut et al. 2017). A 20 mm thick sheet of PMMA on one side of the container allows an insight into the soil during compaction. The loads are applied by a guided, half-round tamper falling close to the PMMA. The mass of the falling weight was 4.8 kg. The falling height was 1.22 m. The behaviour of the soil during the impact is captured by a high-speed camera recording at 1600 fps. The images are processed with the PIV/DIC-method using the commercial software ISTRA4D. The PIV/DIC-Method calculates the field of displacement by a cross-correlation algorithm which compares discrete regions of the picture pair-by-pair. A uniaxial acceleration transducer gather the behaviour of the falling weight during the impact. Because the behaviour of the soil and the compactor is captured, we are able to investigate its interdependency. The metrics of the laboratory investigation presented in this article are the depth of the crater and the acceleration measured on the falling weight. The depth of the crater with respect to the soil surface is the lowest point of the falling weight before the falling weight is lifted for the next drop. According to the work of (Poran et al. 1992) the depth of the crater is related to the diameter of the falling weight to minimize scaling effects. The acceleration is measured with a sampling rate of 10 kHz and low-pass filtered with a cut off frequency of 100 Hz. The used metric regarding the behaviour of the falling weight is the maximum of acceleration of the filtered signal.

2.2 Field investigation

To compare our laboratory investigations with the results of the field test we needed granular, dry soil. The thickness of the test site should be 4 m to ensure that compaction is measured in the granular, dry soil only. To satisfy these requirements the test site was built in layers especially for this investigation. Approximately 2.500 m³ well graded gravel were used for this investigation. During the earthwork we monitored the density and the water content in different layers to ensure the quality of the investigation. The final test site and the used crawler crane are shown in Figure 1(b). The falling weight was made of concrete with a mass of 9800 kg. We observed 24 impacts with a constant falling height of 2 m. Two acceleration transductors each measuring in three axis have been attached on the falling weight. Similar to the laboratory investigations we measured the depth of the crater and the maximum of the acceleration in vertical direction. The depth of the crater was measured with a high precision levelling instrument after the first, third, sixth, twelfth and twenty-fourth impact. The signal of the acceleration transductor, measuring at 10 kHz, was low-pass filtered with a cut off frequency of 100 Hz. The used metric regarding the behaviour of the falling weight is the maximum of acceleration during the impact of the falling weight shown in Figure 1(c-d).

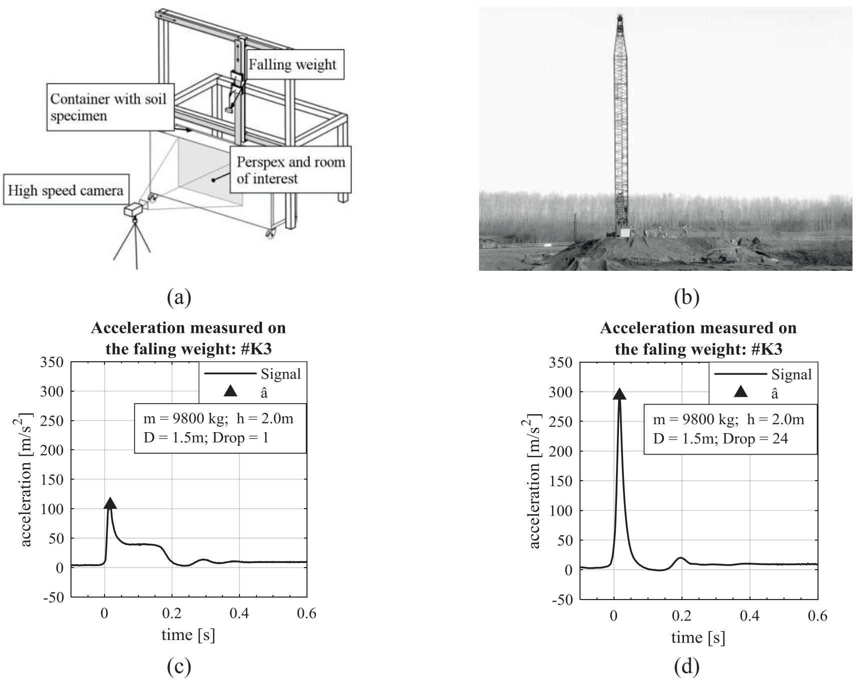

Figure 1. The experimental setup for the laboratory investigation with the PIV/DIC Method (a). A picture of the test site with the crawler crane just before the investigation took place (b). The acceleration measured on the falling weight in vertical direction after the first (c) and the 24th impact (d).

3 RESULTS

The result of the laboratory and the field investigation are presented in Figure 2. The number of impact N are considered in logarithmical scale. On the left side of the figure the maximum value of acceleration â over 24 drops is presented for the laboratory investigation and the filed test (Figure 2(a), 2(c)). The depth of the crater is presented in Figure 2(b) and Fig. 2(d). As mentioned, the depth of the crater measured in the scaled laboratory investigation is related to the diameter of the falling weight. All results can be fitted with the same equation which is based on the common compaction curve adapted in Eq. (1) and presented in (Sawicki and Swidzinski 1989).

$$y = C_1 \ln(1 + C_2 N) \tag{1}$$

The prediction bounds for the laboratory investigation of the measurement of the maximum acceleration and the depth of the crater are ±3.2% and ±1.7% respectively. The prediction bounds for the field investigation, especially for the measurement of the maximum acceleration, are clearly higher. The fitted curve of the depth of the crater has prediction bounds of ±6.0%. The prediction bounds for the measurement of the maximum acceleration are ±25%. The behaviour of the acceleration in time domain (Fig. 1(c)) for the laboratory and the field investigations both agree with measurements presented in (Adam et al. 2016), (Kopf and Paulmichl 2005) or (Kirstein et al. 2016).

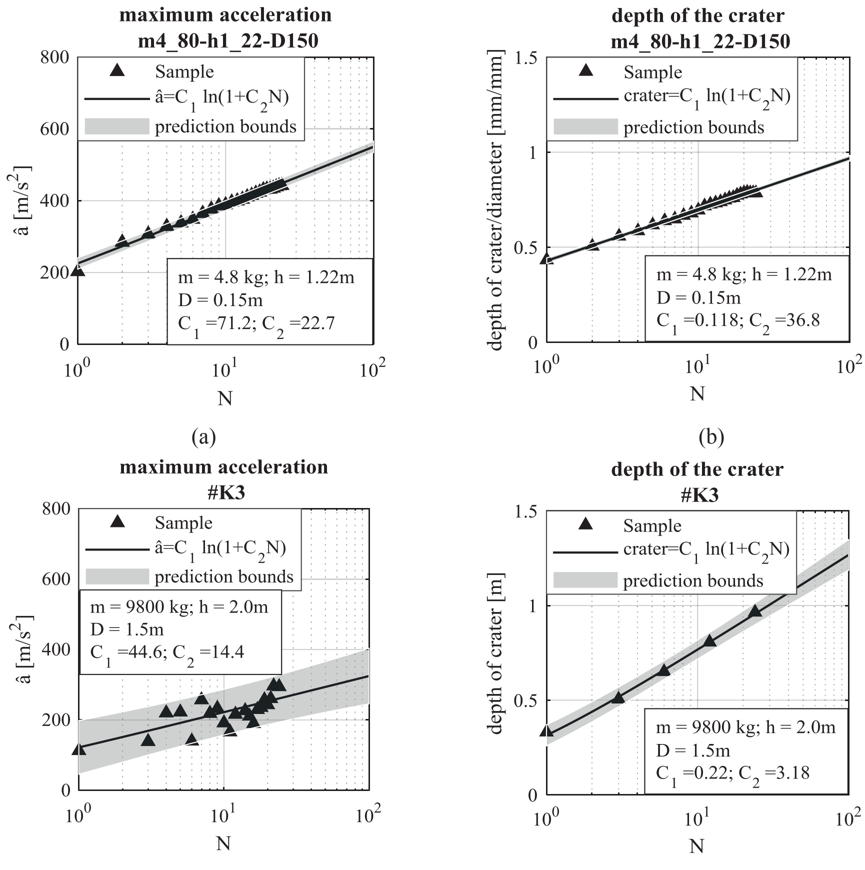

Figure 2. The results of the laboratory investigation (a-b) and the field investigation (c-d)

4 CONCLUSION

The results are showing a linear relationship between the depth of the crater and the maximum value of acceleration in laboratory and field conditions (see Figure 3(a), 3(b)) because both indicators can be fitted with a function based on the common compaction curve. The accuracy of this relation is low, especially for the field condition, which is induced by the scatter of the peak value of the acceleration. The scatter is certainly caused by slightly hitting the border of the crater and shows the sensitivity of this parameter. As mentioned in (Adam et al. 2016) the decay of the acceleration could be a more robust parameter. The investigation of the dependency of the acceleration, the depth of the crater and the measured depth of densification is in process recently.

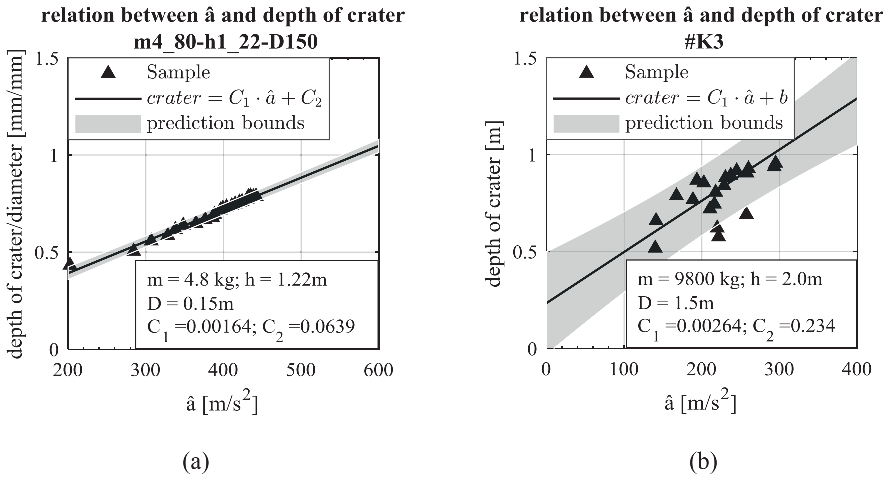

Figure 3. The maximum value of acceleration is in linear relationship with the depth of the crater in laboratory (a) and field condition (b).

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Numerische Simulation des Verdichtungsvorganges in Böden mit dynamisch angeregten Walzen

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Schlagwörter: FEM; Simulation; Hypoplastizität

1 EINLEITUNG

Der Vorteil von FEM-Simulationen im Vergleich zu semianalytischen Berechnungsmodellen liegt darin, dass neben der Ermittlung des dynamischen Verhaltens des Verdichtungswerkzeuges auch die dynamischen Vorgänge im Boden beschrieben werden können. Unter Berücksichtigung des elastischen Halbraumes für den Untergrund können bei Verwendung von geeigneten elastischen und plastischen Materialgesetzen für Böden Aussagen zu Verdichtungseffekten und Auswirkungen auf Maschine und Untergrund gemacht werden. Aufgrund der Modellierungsart können unterschiedliche Steifigkeiten wie z.B. Unstetigkeiten oder geschichtete Aufbauten im Untergrund abgebildet und deren Auswirkungen auf den Verdichtungsvorgang untersucht werden. Bei einem semi-analytischen Modell ist die Umsetzung kaum oder gar nicht realisierbar.
Das in diesem Beitrag vorgestellte FEM-Berechnungsmodell wurde entwickelt, um, im Gegensatz zu bisherigen numerischen Untersuchungen, alle Aspekte der Bodenverdichtung mit dynamischen Walzen abzudecken. Dabei sollen das dynamische Verhalten der Bandage und auch die Reaktionen im Untergrund nachgebildet sowie der Aspekt der FDVK durch Ermittlung der dynamischen Verdichtungskennwerte berücksichtigt werden.

2 SIMULATIONEN MIT ELASTISCHEM MATERIALANSATZ

Die Bilder in Abb. 1 zeigen das zur Nachrechnung der Bodenverdichtung mit Vibrationswalzen entwickelte FEM-Gesamtmodell mit dem Bodenausschnitt (links) sowie der Diskretisierung der Bandage mitsamt der an ihr aufgebrachten Randbedingungen zur Nachbildung der Masseverteilung, der Fahrbewegung sowie der wirkenden Kräfte (rechts).

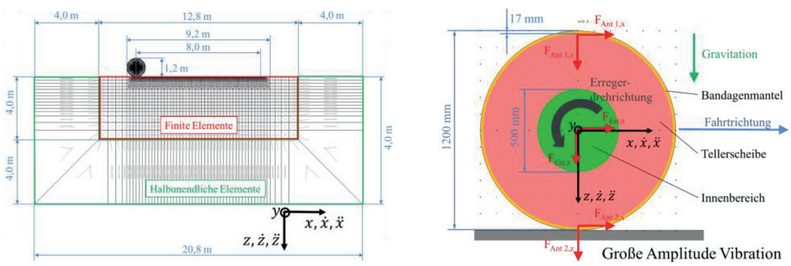

Abbildung 1. Allgemeiner Aufbau des Berechnungsmodells.

Die Behandlung eines solchen Problems setzt voraus, dass das verwendete FE-Programm in der Lage ist, ein sog. Kontaktproblem zu lösen, d.h., dass in diesem Falle die mechanischen Effekte wie

Stoßeffekte, Grenzflächendeformation, Haftung, Reibung oder auch wieder die Trennung der beiden voneinander unabhängigen Kontaktkörper (Bandage und Boden) ermittelt werden müssen. Das in diesem Kapitel gewählte Berechnungsmodell lässt keine Veränderungen von Dichte und Steifigkeit des Untergrundes zu, da das verwendete elastische Materialmodell diese Effekte nicht beschreiben kann. Durch die Wahl unterschiedlicher Untergrundsteifigkeiten können jedoch die Zustände, die beim Verdichtungsfortschritt auftreten, nachgebildet und simuliert werden.
Mittels der Variation des Untergrund-E-Moduls können unterschiedliche Verdichtungszustände mit Hilfe des FEM-Modells simuliert werden. Bei Betrachtung der Bandagenreaktionen während einer Überfahrt können deutliche Unterschiede zwischen weichem und hartem Untergrund festgestellt werden. Die Diagramme in Abb. 2 zeigen Bandagenbeschleunigungskurven aus Verdichtungsüberfahrten mit einem 9t-Walzenzug, links aus Messungen (Kröber, 1988) und rechts aus entsprechenden FEM-Nachrechnungen. Bei der ersten Überfahrt (A.) befindet sich die Bandage im Zustand des Kontaktes. Mit zunehmender Anzahl an Überfahrten nehmen die Beschleunigungen in negativer Richtung zu, so dass die Bandage über den Zustand des Abhebens ((B.) und (C.)) in das Springen übergeht ((D.) bis (F.)). Die Diagramme aus der Simulation zeigen dabei eine sehr gute Übereinstimmung mit den Messergebnissen, sowohl hinsichtlich der Größen der Beschleunigungswerte als auch bezüglich der dynamischen Zustände der Bandage.

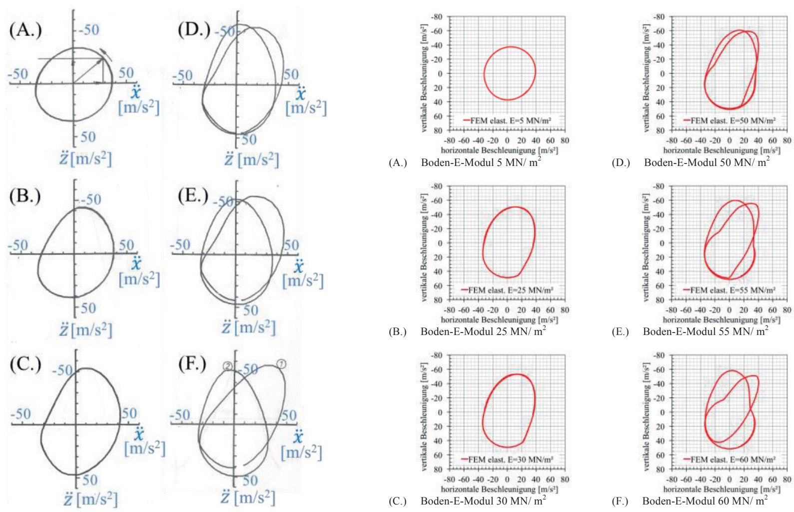

Abbildung 2. Bandagenbeschleunigungen – Vergleich von Messergebnissen (Kröber, 1988) (links) mit Berechnungsergebnissen (rechts).

Sämtliche aktuellen FDVK-Messwerte für Vibrationswalzen basieren auf der Auswertung des Beschleunigungssignals in vertikaler Richtung. In (Erdmann und Kopf 2005) wurden bereits Ergebnisse aus Parameterstudien zur Abhängigkeit der jeweiligen Verdichtungskennwerte von der Erregerkraftgröße und der Bodensteifigkeit vorgestellt. In (Rinehart 2008) sind Versuche mit einem 12t-Walzenzug beschrieben, bei denen ein tragfähiges Material, ein weitgestufter, gebrochener Kies der Bodengruppe GW, auf einem tonigen Sand mit geringer Tragfähigkeit schichtweise aufgebracht und verdichtet wurde. Die Diagramme in Abb. 3 zeigen die Verläufe der Mittelwerte des k_B-Wertes aus der jeweils letzten Überfahrt je eingebauter Schicht in Abhängigkeit von der Größe der Erregerkraft und der Schichtdicke ((A.) aus Messungen (Rinehart 2008) und (B.) aus entsprechenden Simulationen). Auch hier zeigen sich gute Übereinstimmungen zwischen Mess- und Simulationsergebnissen. Die Messtiefe bei den praktischen Versuchen liegt zwischen 0,9 m und 1,2 m und bei den Nachrechnungen bei 1,2 m. Im Berechnungsmodell können scharfe Grenzen zwischen dem steiferen und dem weicheren Material definiert werden, weshalb die Kurven für die verschiedenen Erregerkräfte schlüssig und ohne Überschneidungen verlaufen. Im praktischen Versuch entsteht hingegen ein Übergangsbereich in der untersten Schicht, in dem das tragfähigere Material aufgrund des fehlenden Widerlagers nicht vollständig verdichtet werden kann.

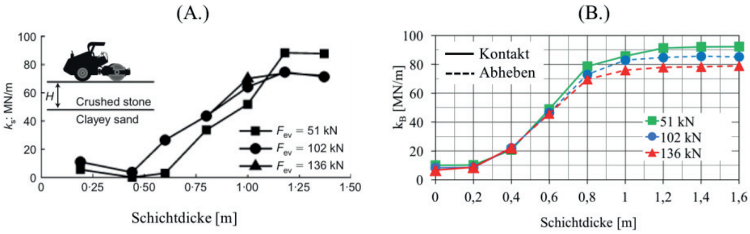

Abbildung 3. Verläufe des k_B-Wertes in Abhängigkeit von der Schichtdicke und der Erregerkraft – Vergleich von (A.) Messergebnissen (Rinehart 2008) und (B.) Berechnungsergebnissen.

3 SIMULATIONEN MIT HYPOPLASTISCHEM MATERIALANSATZ

Die Hypoplastizität ist ein Materialgesetz für granulare Materialien, das an der Universität Karlsruhe in den 1970er Jahren entwickelt wurde (Kolymbas 1978). Seitdem ist das Materialmodell stetig verbessert und optimiert worden zur besseren Nachbildung des realen Materialverhaltens von granularen Medien wie z.B. Böden (Herle 1997). Die weiterentwickelte Version der Hypoplastizität mit intergranularer Dehnung (Herle und Niemunis 1997) ermöglicht die Berechnung von dynamischen, zyklischen Vorgängen, wie zum Beispiel der Verdichtung mit Vibrationswalzen, da das elastische Verformungsverhalten an den Korngrenzen berücksichtigt wird. Das hypoplastische Stoffgesetz ist ein Ratengesetz. Es beschreibt die Änderung der Spannung für eine Änderung der Dehnung. Das nichtlineare Verhalten des hypoplastischen Stoffgesetzes wird durch die der Spannungsabhängigkeit der Steifigkeit gesteuert.
In Abb. 4 sind Ergebnisse aus Berechnungen von Verdichtungsfahrten auf einer Kiesschicht mit einer Vibrationsbandage einer 9-to HAMM-Tandemwalze vom Typ HD90+VO aufgeführt. Die Diagramme zeigen Kurven der Mittelwerte der unterschiedlichen FDVK-Werte ((A.) E_{vib} und k_B und (B.) CMV/RMV und Omega) mit unterschiedlichen Anfangsporenzahlen.

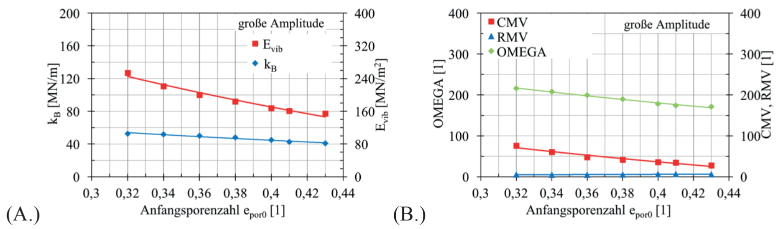

Abbildung 4. Verläufe der verschiedenen FDVK-Messwerte in Abhängigkeit von der Anfangsporenzahl.

Alle Kurven weisen nachvollziehbare und schlüssige Verläufe auf, in denen die FDVK-Messwerte umso kleinere Werte annehmen, je größer die Porenzahl gewählt wird. Eine hohe Porenzahl repräsentiert dabei ein weicheres und weniger verdichtetes Schichtmaterial.
Mit Hilfe einer Verdichtungsbahn mit stetig zunehmender Schichtdicke eines zu verdichtenden Kiesmaterials auf endverdichtetem Untergrund können Untersuchungen zur Tiefenwirkung durchgeführt werden. Abb. 5 zeigt eine Skizze zum Modellaufbau. Neben entsprechenden Ein- und Auslaufbereichen wird die Schichtdicke auf einer Bahnlänge 680 cm von 20 cm auf 220 cm erhöht.
Die Berechnungsergebnisse (Abb. 6) zeigen, dass die Verdichtungswirktiefe nicht mit den Messtiefen der FDVK-Messwerte übereinstimmt. Während die Verdichtungsüberfahrt zu Porenzahl-reduzierungen bis in einer Tiefe von 220 cm führt, liegen die Messtiefen je nach FDVK-Messwert zwischen ca. 40 cm und 95 cm.

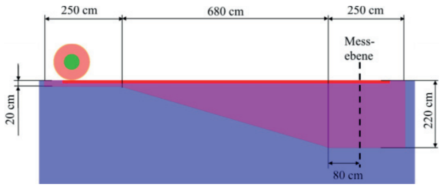

Abbildung 5. Aufbau einer Verdichtungsbahn mit ansteigender Schichtdicke

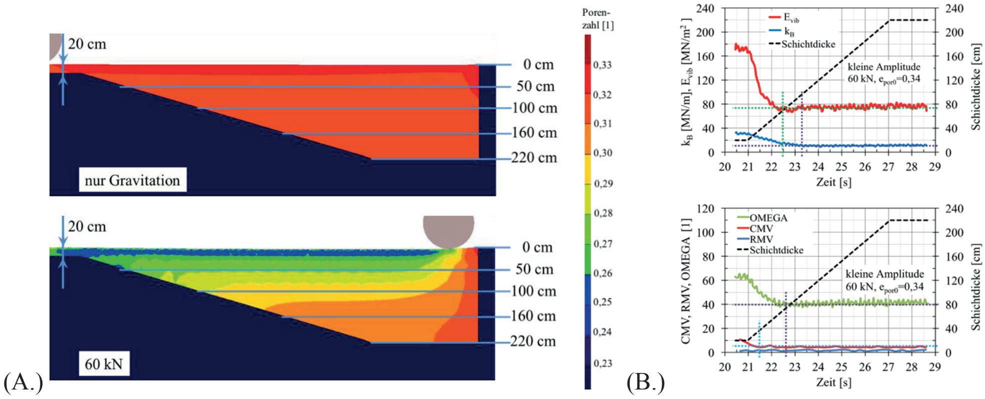

Abbildung 6. (A.) Porenzahlverläufe und (B.) Verläufe der verschiedenen FDVK-Werte über der Schichtdicke bei der Überfahrt mit der Vibrationsbandage einer HAMM HD+90 VO mit kleiner Amplitude.

4 ZUSAMMENFASSUNG

Die vorgestellten FEM-Modelle mit elastischem und hypoplastischem Materialverhalten für den Untergrund beschreiben das Bewegungsverhalten des Verdichtungswerkzeuges sowie die dynamischen Vorgänge im Untergrund sehr gut. Insbesondere durch die Implementierung der Hypoplastizität als Materialgesetz für den Boden können die Vorgänge im Untergrund, wie Setzungen und plastischen Verformungen, realitätsnah beschrieben werden. Die Modelle eignen sich hervorragend zur Simulation unterschiedlicher Verdichtungsaufgaben und auch zur Untersuchung der maschinenintegrierten Verdichtungsmesssysteme. Dieses neu entwickelte Modell kann als Werkzeug zur Optimierung des Verdichtungsprozesses selbst und/oder zur Verbesserung der Maschinentechnik im Zusammenspiel mit experimentellen Untersuchungen genutzt werden. Zudem kann es als Hilfsmittel zur Weiterentwicklung der walzenintegrierten Verdichtungsmessung dienen.

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CCC with Oscillating Rollers – Fundamentals and Application in Experimental Field Tests

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Keywords: CCC; Oscillatory Rollers; Intelligent Compaction; Field Tests

1 INTRODUCTION

1.1 Dynamic roller compaction

Dynamic roller compaction has become the commonly used method for near-surface compaction, because dynamic rollers are much more efficient compared to static rollers. However, the continuously improved compaction techniques in earthworks and geotechnical engineering also require the use of adequate test equipment to assess the achieved compaction success. The sole use of conventional spot like compaction testing methods, especially at large construction sites, is inefficient and does not represent the state of the art anymore. Continuous Compaction Control (CCC) is a suitable method to overcome the drawbacks of spot like compaction testing methods.

1.2 Oscillating rollers

Two types of excitation are mainly used for dynamic roller compaction, the vibratory drum and the oscillatory drum.
The eccentric masses of a vibratory drum are shafted concentrically to the drum axis, resulting in a significantly higher vertical loading, but also increased ambient vibration.
The torsional motion of an oscillatory drum is caused by two opposed, rotating eccentric masses, which shafts are mounted eccentrically but point symmetric to the drum axis (see Fig. 1). Soil is loaded horizontally by the drum motion and vertically by the dead load of the drum and the roller.

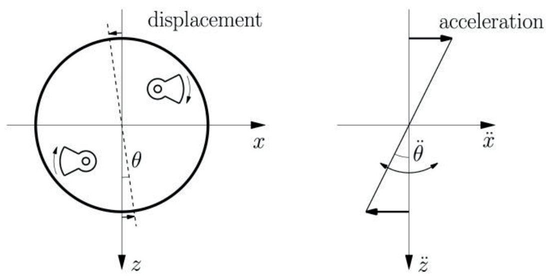

Figure 1. Excitation of an oscillatory drum.

While CCC systems have become the state of the art in compaction control for vibratory rollers during the last decades, the lack of a CCC system for oscillating rollers has been a major disadvantage for these rollers.

2 DEVELOPMENT OF A CCC VALUE FOR OSCILLATING ROLLERS

A comprehensive research project on oscillatory rollers was launched by the German roller manufacturer HAMM AG in 2011 in cooperation with the Institute of Geotechnics at TU Wien. The aim of the project has been the development of a better understanding for the motion behaviour of an oscillatory drum and its impact on the compacted soil as well as the development of a CCC system for oscillatory rollers. Within this project, large-scale in situ tests were performed with a tandem roller in a gravel pit near Vienna Airport. Details on the test setup and measurements can be found in (Pistrol 2016).

2.1 Motion behaviour of oscillating drums

The interaction of oscillatory drum and soil does not only cause accelerations in the soil, but also has an influence on the motion behaviour of the drum itself. The interaction results in a distortion of the signal and a formation of a secondary vibration with double frequency in the vertical accelerations measured in the bearing of the drum (see Fig. 2).

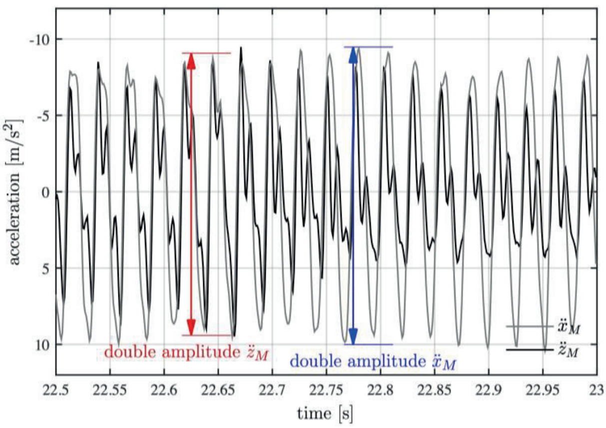

Figure 2. Horizontal and vertical accelerations in the bearing of the oscillatory drum for the eleventh pass on lane 2 of the test field (Pistrol 2016).

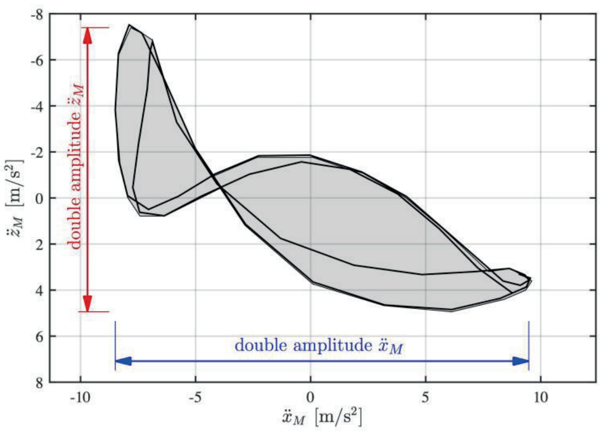

Figure 3. Horizontal and vertical accelerations in the bearing of the oscillatory drum for two periods of excitation during the eleventh pass on lane 2 of the test field (Pistrol 2016).

In Fig. 3 the same accelerations in the bearing of the drum are plotted for two consecutive periods of excitation as in Fig. 2. However, the accelerations are not plotted in the time-domain, but in a diagram with horizontal accelerations on the abscissa and vertical accelerations on the ordinate. The chronologically connected pairs of accelerations ($\ddot{x}_M \mid \ddot{z}_M$) form a reproducible shape, similar to a recumbent eight in this type of representation. The superposition of oscillatory motion and roller travel causes an asymmetry of the settlement trough and subsequently an asymmetry of the recumbent eight-shape of accelerations. A comparison of test runs at different states of compaction showed an expansion of the recumbent eight-shape with increasing soil stiffness.

2.2 Definition of the CCC value for oscillating rollers

The centrifugal forces of the two eccentric masses of an oscillatory drum cancel each other out. Therefore, the horizontal and vertical soil forces are the dynamic impact, which dominates the motion behaviour of the drum. The accelerations in the bearing of an oscillatory drum are proportional to the soil forces as shown in the results of the experimental investigations and can therefore be used for an assessment of the force path.

The formation of the recumbent eight-shape of drum accelerations changes depending on the soil stiffness. Various characteristics of the described shape can be used for assessing the soil stiffness. These include the extent in horizontal and vertical direction, gradient and curvature of various regressions, or the area circumscribed by the eight-shape. The described area showed the greatest significance of all characteristics, tested on measurement data. However, the calculation of this area cannot be done easily, especially when real measurement data shall be processed. The shape changes continuously and if one period of excitation is considered the last measurement point of the shape does not necessarily equal the first measurement point. Therefore, an algorithm has been developed to approximate the area of the eight-shape with sufficient accuracy (Pistrol 2016).

The calculated area is defined as novel CCC value for oscillatory rollers and adopts the theoretical unit of m^2/s^4. A factorisation of the presented CCC value with process parameters or machine parameters, e.g. drum diameter, static line load, vibrating mass or unbalance torque, can be used to obtain more practical CCC readings.

3 CCC IN EXPERIMENTAL FIELD TESTS

3.1 The CCC algorithm tested on real measurement data

The CCC value for oscillating rollers was evaluated in experimental field tests discussed in (Pistrol 2016). For the calculation of the CCC values a time frame of 1,024 sampling points was considered, which equals approximately one CCC value for each second.

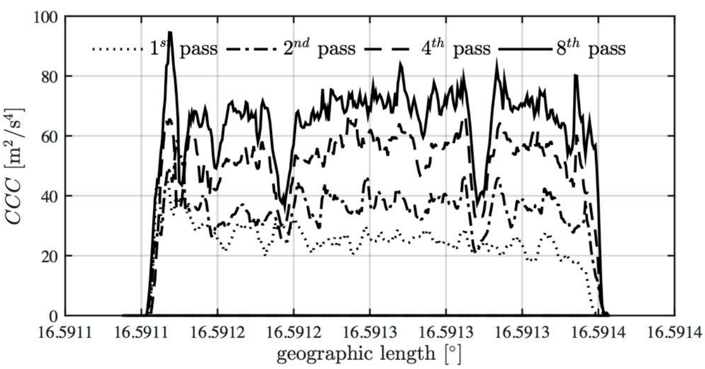

Figure 4. Progress of the measured CCC values of the passes 1, 2, 4 and 8 on lane 2 of layer 2 of the test field (Pistrol 2016).

The progress of the measured CCC values is shown in Fig. 4 for the passes 1, 2, 4 and 8 on layer 2 of lane 2 of the test field. When the CCC curves for various passes are compared to each other, an increase of the level of CCC values can be observed. Fig. 4 shows a significant increase within the first four passes on lane 2 and another smaller increase during the passes 5 to 8. This is in accordance with the common experience, that the first passes of a roller on less compacted soil gain the largest increase in soil stiffness. When the soil gets closer to its state of maximum compaction, the increase in soil stiffness becomes asymptotically smaller with each pass of the roller.
Two artificial weak spots under lane 2 (mattresses buried in 15 cm and 55 cm below ground level) cannot be located in the measurement curve of the first pass on the less compacted soil. However, their location becomes clearer with every pass of the roller. Weak spot 2 was buried in a depth of only 15 cm below ground level and shows a linear elastic behaviour. The soil above this weak spot can hardly be compacted and the CCC values of the eighth pass are only slightly larger than the CCC values after the first pass. Although weak spot 1 was buried in a depth of 55 cm below ground level of lane 2, it is still clearly visible in the CCC curves in Fig 4.
The presented CCC value for oscillating rollers is properly reflecting the increase in soil stiffness with an increasing number of roller passes and capable of detecting poorly compacted areas. Further results and a comparison to results of dynamic plate load tests can be found in (Pistrol 2016).

3.2 Validation of the CCC system for oscillating rollers

The company HAMM AG used the results of the research project and the algorithm for calculating the CCC value for oscillating rollers to produce a CCC system for oscillating rollers. Two single-drum rollers and a tandem roller were equipped with the novel CCC system to test the systems functionality in another round of experimental field tests. The tests proofed that the calculation of the CCC value for oscillating rollers can also be done online in real time and under site conditions. The CCC systems of all tested rollers showed a reproducibility of their results (see Fig. 5). However, the tests also showed the dependence of the CCC value on machine parameters like the weight of the roller compared to the excitation and the ratio between mass of the drum and mass of the roller, which results in different sensitivities of the CCC system for oscillating rollers.

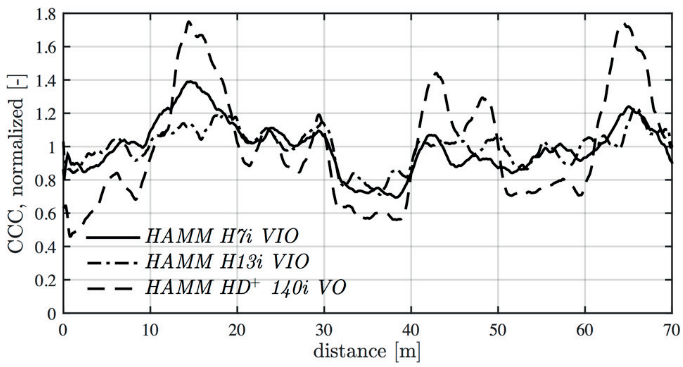

Figure 5. Validation of the CCC system exemplary for three oscillating rollers (Pistrol 2016).

ACKNOWLEDGEMENTS

The financial support granted by the German roller manufacturer HAMM AG made this research possible and is gratefully acknowledged.

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Anniversary Symposium – 40 Years of Roller Integrated Continuous Compaction Control (CCC)
D. Adam & S. Larsson (eds.)
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Analytical modelling of the motion of an oscillating roller during soil compaction assuming pure rolling contact

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Keywords: Analytical model; Continuous Compaction Control; Oscillating roller; Rolling without slipping

1 INTRODUCTION

Continuous Compaction Control (CCC) (Adam 1996) has become the standard technology for controlling the soil compaction induced by *vibratory* rollers. This control technique is based on the dynamic response of the interacting drum-soil system recorded during the roller passage, and thus, allows an instant continuous assessment of the degree of compaction. Recently, also for *oscillating* rollers an efficient roller integrated compaction measurement method has been developed (Pistrol 2016). Pistrol has shown that the vertical acceleration component of the drum centre plotted against the corresponding horizontal acceleration component generates a figure in the shape of a “recumbent eight”. According to this study, the area enclosed in the “recumbent eight” is a characteristic quantity of the compaction degree of the subsoil. Since this CCC technique has been found primarily in an empirical manner, both analytical and numerical studies are required for its theoretical verification. To fill partially this gap, in the present contribution a semi-analytical parametric study of the motion of an oscillating drum during soil compaction is conducted. The obtained results allow to detect, to understand and to explain better the dynamic response of the interacting oscillating drum-soil system, and ultimately to validate this novel compaction measurement method for oscillating rollers.

2 ANALYTICAL MODELLING

The present study is based on the simplified analytical model of the interacting oscillating roller – subsoil system shown in Fig. 1 (Pistrol 2016). This model allows to studying the motion of the oscillating drum in its settlement trough. The drum is represented as a rigid circular plate of radius r, mass m and mass moment of inertia I. It is assumed that the settlement trough has a circular shape with constant radius R, and thus it is rigid as well. At the bottom of the settlement trough the effect of the subsoil is captured through two Kelvin-Voigt bodies, one in vertical (subscript “V”) and another in horizontal (subscript “H”) direction. As such, the settlement trough exhibits a translational motion in both horizontal and vertical direction. Rotation of the settlement trough is not permitted. The so-called cone model of Wolf (Wolf 1994) is used to derive stiffness coefficients k_H and k_V of the springs, damping coefficients c_H and c_V of the dashpot dampers, and the trapped soil mass Δm, considering the measured contact length between drum and soil (Pistrol 2016). Between drum and circular guiding track continuous rolling contact is assumed, i.e. rolling without slipping. Hence, the arc length between support point A and contact point P_{cont} is equal to the length between contact point P_{cont} and point P_D on the drum, see Fig. 1. The relationship between the absolute rotation angle φ of the drum and the centre angle δ can be expressed as follows:

$$\varphi = \frac{R-r}{r}\delta = \frac{l}{r}\delta \tag{1}$$

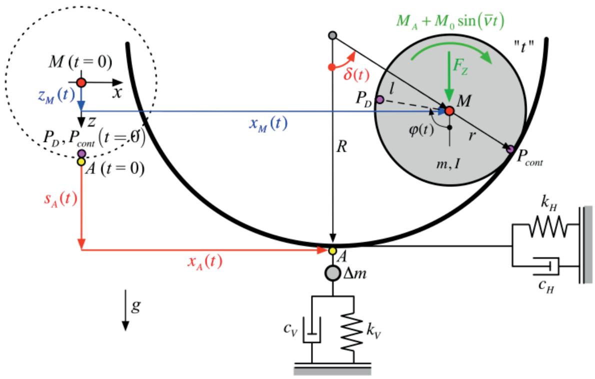

Figure 1. Mechanical model of the oscillating drum in its settlement trough (deformed position).

The motion of the three-degree-of-freedom model is described by centre angle δ, and settlement s_A and horizontal displacement x_A of support point A. Application of Lagrange's equations (Ziegler 1995), conservation of momentum and angular momentum yields the following coupled equations of motion of this model:

$$m\ddot{x}_A + c_H\dot{x}_A + k_H x_A + ml\cos\delta\ddot{\delta} - ml\sin\delta\dot{\delta}^2 = 0 \tag{2}$$

$$(m+\Delta m)\ddot{s}_A + c_V\dot{s}_A + k_V s_A - ml\sin\delta\ddot{\delta} - ml\cos\delta\dot{\delta}^2 = (m+\Delta m)g + F_z \tag{3}$$

$$\left(1+\frac{I}{mr^2}\right)ml\ddot{\delta} + m\cos\delta\ddot{x}_A - m\sin\delta\ddot{s}_A + (mg+F_z)\sin\delta = \frac{M_A}{r} + \frac{M_0}{r}\sin(\bar{\nu}t) \tag{4}$$

Variable l in Eq. (1) is equal to $R-r$, g in Eqs (3) and (4) denotes the acceleration of gravity, F_z in Eqs (3) and (4) represents the axle load of the drum (static load in vertical direction with point of application M), M_A in Eq. (4) denotes a constant driving torque, and M_0 and $\bar{\nu}$ in Eq. (4) represent amplitude and angular frequency of the oscillating moment (harmonic excitation), respectively. The second order nonlinear ordinary differential equations (ODEs), Eqs (2) to (4), are solved numerically by the MATLAB function *ode45* (Mathworks 2017) for δ, s_A and x_A. Then, the governing response quantities for the investigated CCC technique, i.e. the horizontal ($\ddot{x}_M$) and vertical ($\ddot{z}_M$) acceleration components of the drum centre (point M in Fig. 1), are calculated using the following relations:

$$\ddot{x}_M = l\left(\cos\delta\ddot{\delta} - \sin\delta\dot{\delta}^2\right) + \ddot{x}_A\,, \quad \ddot{z}_M = -l\left(\sin\delta\ddot{\delta} - \cos\delta\dot{\delta}^2\right) + \ddot{s}_A \tag{4}$$

3 RESULTS

For the subsequent studies the properties of a HAMM HD$^+$ 90 VO tandem roller with the following parameters are utilized: radius of the drum $r = 0.6$ m, mass of the drum $m = 1{,}851$ kg, mass moment of inertia of the drum $I = 411.78$ kgm^2, static axle load $F_z = 27{,}066$ N, amplitude of the oscillating moment $M_0 = 54{,}947$ Nm, excitation frequency of 39 Hz (HAMM AG 2017). It is assumed that all considered subsoil conditions exhibit a Poison's ratio ν of 0.3 and a density ρ of 1,900 kg/m^3. The shear modulus G of the subsoil is varied between 5 and 70 MN/m^2. Based on experience (Kopf 1999), the damping coefficients c_H and c_V resulting from the cone model according to Wolf are multiplied by 2. In the following, selected results derived from the acceleration components of the drum centre (point M in Fig. 1) are presented and discussed. In particular, Fig. 2 shows for one subsoil and two different radii R of the settlement trough the vertical steady state acceleration component $\ddot{z}_M$ in the frequency domain. In Fig. 3, $\ddot{z}_M$ is plotted against horizontal acceleration component $\ddot{x}_M$ for various subsoil conditions, yielding figures in shape of a “recumbent eight”. Finally, in Fig. 4 the area enclosed in the “recumbent eight” is shown as a function of the soil stiffness.

- Simulations with varying radius R of the settlement trough reveal that the function $\ddot{z}_M$ over $\ddot{x}_M$ ($\ddot{z}_M - \ddot{x}_M$ plot) only exhibits an intersection if R is slightly larger than the drum radius, i.e. r (= 0.60) < $R \leq 0.61$ m. In this range, vertical acceleration $\ddot{z}_M$ is dominated by a frequency that is two times the excitation frequency, as shown in Fig. 2, left subplot. In contrast, for $R = 0.7$ m the dominant frequencies of $\ddot{z}_M$ are close to the excitation frequency (Fig. 2, right subplot).
- The horizontal drum centre acceleration $\ddot{x}_M$ is dominated by the excitation frequency for both radii of the settlement trough, i.e. $R = 0.606$ m and $R = 0.7$ m.
- Thus, the shape of the $\ddot{z}_M - \ddot{x}_M$ plot is similar to a “recumbent eight” (Fig. 3) only if radius R of the circular settlement trough is slightly larger than drum radius r (here $r < R \leq 0.61$ m).
- Initially, the horizontal peak acceleration $\ddot{x}_M$ increases with increasing soil stiffness, its maximum is attained at about $G = 40$ MN/m^2, and it decreases subsequently with further increasing soil stiffness (Fig. 3).
- Initially, the vertical peak acceleration $\ddot{z}_M$ increases with increasing soil stiffness, its maximum is attained at $G = 22.5$ MN/m^2, and it decreases subsequently with further increasing soil stiffness (Fig. 3). Thus, the maximum of $\ddot{z}_M$ is related to a lower shear modulus compared to the maximum of $\ddot{x}_M$.
- The shape of the $\ddot{z}_M - \ddot{x}_M$ plot represents a so-called Lissajous (or Bowditch) curve (Klotter 1981). Depending on the phase shift between the vertical and horizontal accelerations it is either similar to the lemniscate of Gerono (i.e. a “recumbent eight”) (Lawrence 1972) or to a general besace.

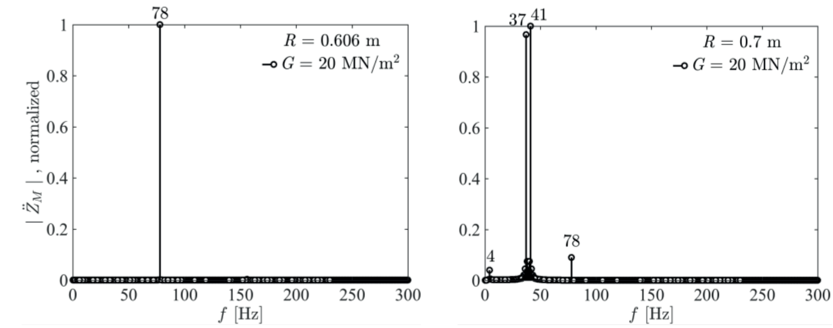

Figure 2. Vertical accelerations of the drum centre in the frequency domain for $R = 0.606$ m (left subplot) and $R = 0.70$ m (right subplot).

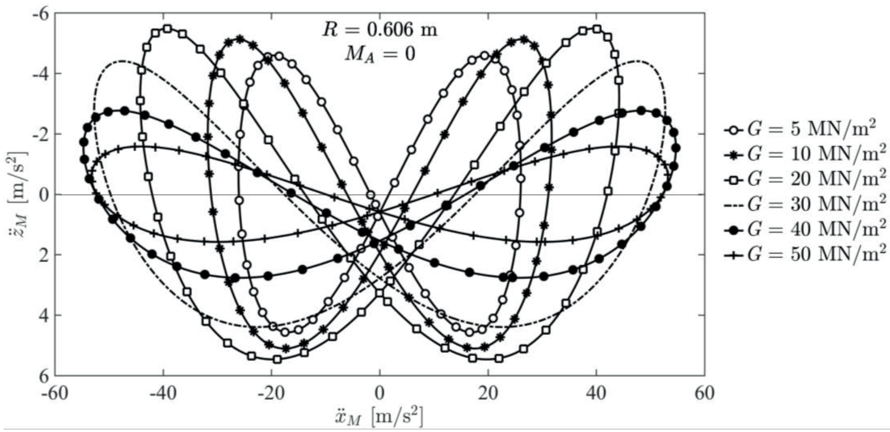

Figure 3. Vertical acceleration plotted against horizontal acceleration of the drum centre.

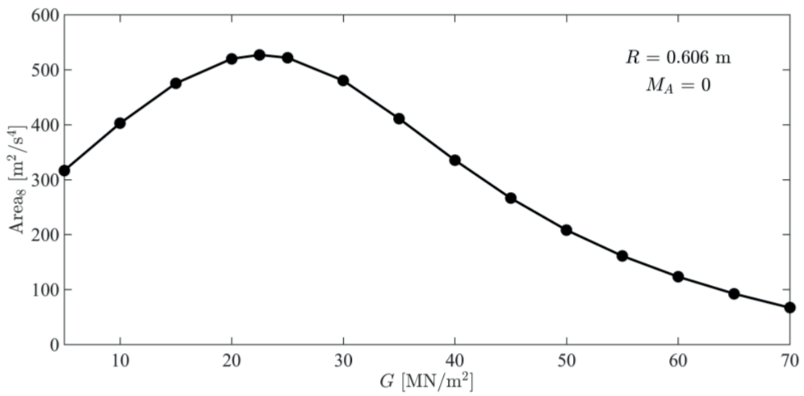

Figure 4. Area enclosed by the "recumbent eight" as a function of soil stiffness G.

- With increasing soil stiffness (in the range G = 10-30 MN/m^2), the "recumbent eight" degenerates to a general besace similar to a parabolic segment. When further increasing the soil stiffness in the range G = 40-70 MN/m^2 the curve resembles the lemniscate of Bernoulli (Lawrence 1972).
- The shape of the $\ddot{z}_M - \ddot{x}_M$ plot is symmetric with respect to the vertical axis if no driving torque M_A is applied ($M_A = 0$).
- Initially, the area enclosed by the "recumbent eight" increases with growing soil stiffness and it attains its maximum at G = 20-25 MN/m^2. Subsequently the area decreases with further increasing soil stiffness (Fig. 4).

4 CONCLUSIONS

Semi-analytic simulations of the motion of a selected oscillating roller have demonstrated that the simplified mechanical model reproduces the findings from in situ tests if the radius of the circular settlement trough is slightly larger than the drum radius. The vertical steady state acceleration of the drum centre ($\ddot{z}_M$) is dominated by a frequency that is two times the excitation frequency, whereas the dominant frequency of the corresponding horizontal acceleration ($\ddot{x}_M$) is equal the excitation frequency. It has been confirmed that function $\ddot{z}_M$ over $\ddot{x}_M$ is in the shape of a "recumbent eight", and the area enclosed by this figure is related to the soil stiffness.

ACKNOWLEDGEMENTS

The financial support granted by the manufacturer of compaction equipment HAMM AG makes the ongoing research on oscillating rollers possible and is gratefully acknowledged.

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Anniversary Symposium – 40 Years of Roller Integrated Continuous Compaction Control (CCC)
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Vibratory plate resonance compaction

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Keywords: Compaction; Gravel; Plate; Resonance, Sand; ibrator

1 INTRODUCTION

Deep soil compaction methods are generally effective below a depth of approximately 3 m, where the effective confining stress is sufficient to achieve soil densification. On the other hand, surface vibratory rollers have a limited depth effect and are effective down to maximum 1 m, due to the limited drum-soil contact area, as well as restrictions with respect to drum size and oscillating force. Vibratory plates can compact soil layers efficiently within the depth range of 1 to 4 m. In this presentation, the application of the resonance plate compaction method will be presented and its practical application illustrated by an example. In addition to the enhanced efficiency of compacting granular soils at the resonance frequency of the vibrator-plate-soil system, detailed monitoring of the compaction process makes it possible to determine in the field that the anticipated compaction effect is achieved.
Testing of vibratory plate resonance compaction dates back to investigations in the early 1950's. Converse (1952) used a dragged vibratory plate to compact loose sand in California. During field trials, a vibrator with a mass of approximately 4.5 tons and variable frequency (8 - 24 Hz) was used. The movement amplitude of the plate in the vertical and horizontal direction was measured, Fig. 1. One objective was to study resonance effects of the vibrator-plate soil system.

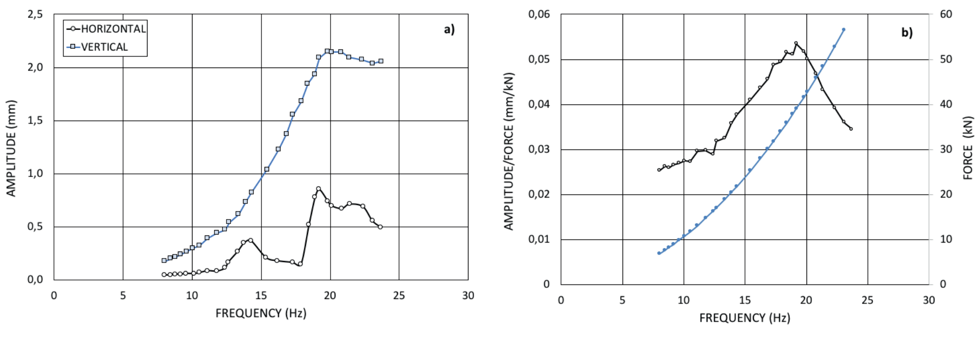

Figure 1. Effect of vibrator frequency on vertical displacement, data from Converse (1952). Influence of frequency on peak-to-peak amplitude. a) vibration amplitude as function of frequency, b) normalized vibration amplitude and centrifugal force.

Based on initial field trials, the following conclusion was drawn: "*The results of these tests indicated that very excellent densities were obtained to depths of at least one and one-half times the width of the surface plate.*" The sharpness of the resonance peak is somewhat masked by the fact that displacement varies with dynamic force, dynamic force varies with frequency and soil non-linearity reduces the amplitude at resonance. The resonance frequency becomes clearly visible when the displacement amplitude is divided by the centrifugal force, which increases with frequency (Fig. 1b). The effectiveness of the system was verified by field density tests before and after compaction. It was concluded that by the resonance compaction method: "The density in the field was actually approximately 100% of the value obtainable in the laboratory at the same moisture." It was concluded that at the system resonance frequency (vibrator-plate-soil) displacements are at a maximum, resulting in optimal compaction effect.

Nelissen et al. (1983) described the application of vibratory plate compaction under water. The main objective of the system was to achieve maximum compaction when operating at large water depth, using a perforated compaction plate. The operating frequency of the vibrator was adjusted to achieve optimal transfer of the dynamic force to the ground surface, avoiding "disconnection of the plate" which resulted in uncontrollable movement.

2 RESONANCE COMPACTION

Resonance deep compaction using vertically oscillating probes was introduced by Massarsch (1991). The objective of the system was to insert the compaction probe at a high frequency and thereafter, to adjust the vibration frequency to the system frequency of the vibrator-probe-soil system. The resonance frequency is usually significantly lower than the frequency used for sheet pile installation. Another advantage of the resonance compaction system is that all phases of the compaction process are monitored and can be optimized.

2.1 Dynamic tuning of the system

An important aspect of the vibratory plate compaction system is the optimal tuning of the static mas-vibrator/plate-soil system, assuring smooth performance and avoiding unnecessary wear of the compaction equipment. The optimization of the system is carried out using theoretical models, as shown in Fig. 2.

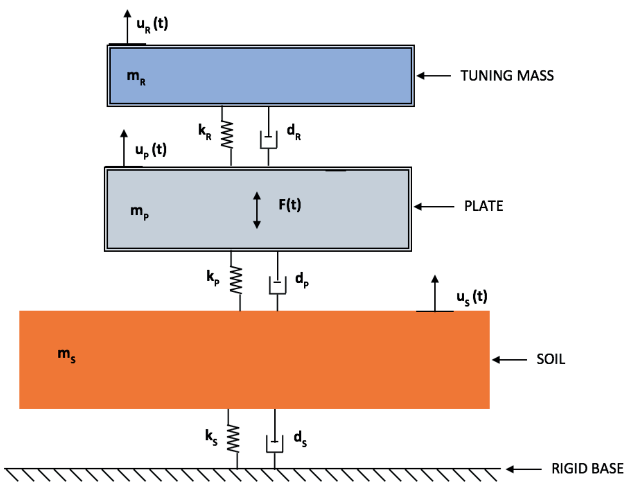

Figure 2. Theoretical model for optimization of vibratory plate compaction system.

The dynamic properties of the soil are important when designing the optimal compaction equipment. However, by continuously adjusting the excitation frequency of the vibratory plate and at the same time measuring the displacement amplitude, resonance amplification can be achieved without prior knowledge of the resonance frequency ("resonance compaction"). The advantages of resonance compaction are increased amplitude and contact force, in spite of lower centrifugal force and less energy consumption. This facilitates compaction at a lower frequency which causes the plate to move in phase with the soil, while higher frequencies cause the plate and the soil moving out of phase, resulting in a significant portion of the compaction energy being transformed to friction. Fig. 3 shows results from model tests using a compaction plate, 84 mm in diameter, to compact sand at various frequencies (Wersäll et al., 2015). The settlement below the plate, closely related to soil compaction, shows a maximum around the resonance frequency, approximately 40 Hz. At higher frequencies, the compaction effect again increases due to a high centrifugal force. Under field conditions, however, high frequencies cause double-jump and chaotic movement of the plate which is not suitable for compaction. The most effective compaction frequency is thus at resonance.

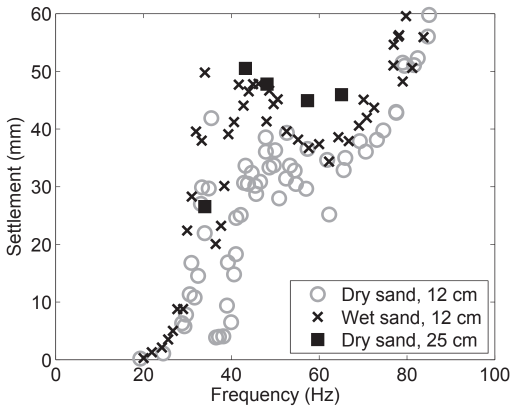

Figure 3. Frequency-dependent settlement/compaction in model tests, from Wersäll et al. (2015).

3 APPLICATION OF RESONANCE PLATE COMPACTION

The following gives an example of the practical application of the resonance compaction plate system on a natural sand deposit. The site conditions were investigated using cone penetration tests (CPT). The subsoil down to 8 m depth consisted of very dense sand with occasional layers of silt. The average cone stress down to 4 m depth varied between 10 and 30 MPa. The compaction unit used for the trials consisted of a vibrator Dieseko PVE 38M, provided with static weights, attached to a compaction plate of size 2.6 m x 2.6 m. The technical specifications of the compaction unit are given in Table 1.

Table 1. Specifications of Compaction Equipment PVE 38M with compaction plate.

Eccentric moment	38	kgm
Max. frequency	1 700	rpm
Max. frequency	28	Hz
Centrifugal force	1 200	kN
Total mass	45 570	kg
Total dynamic mass	12 600	kg
Static mass	25 800	kg
Maximum pressure	35	MPa
Nominal displacement amplitude	6	mm

A monitoring and process control system (MPCS) was used to optimize and document the compaction process. The following parameters were measured: a) acceleration of vibrator; b) depth of plate; c) hydraulic pressure and d) time. From these measurements, the following parameters were derived: 1) displacement amplitude; 2) frequency; 3) plate penetration 4) plate penetration rate. During compaction, the frequency was adjusted to close to the resonance frequency of the system. Fig. 4 shows the compaction plate and measured parameters during one compaction test. Since the soil below the plate was initially very dense, the resulting compaction is moderate, but the monitoring illustrate how frequency can be adjusted to obtain resonance by observing the displacement amplitude. At the highest frequency, the dynamic displacement amplitude decreases, indicating that the resonance frequency was exceeded. Towards end of compaction, the frequency is lowered and the amplitude increases again as the frequency pass through resonance.

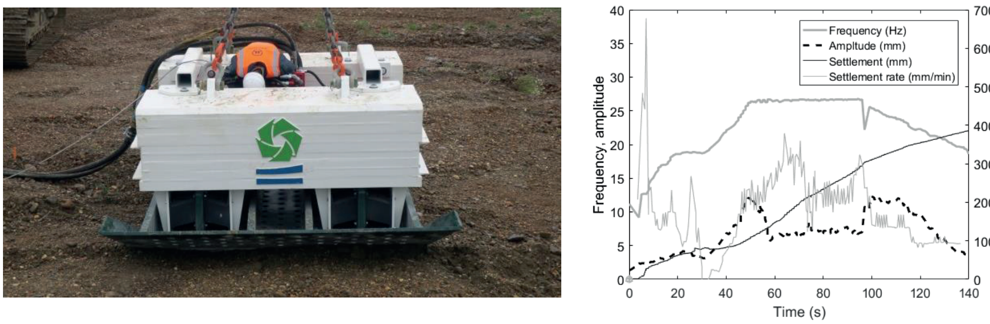

Figure 4. The vibrator and results from one compaction test.

4 CONCLUSIONS

The resonance plate compaction system has evolved during more than 50 years. However, resonance compaction has been made practically applicable only recently, thanks to the availability of powerful vibrators and development of advanced monitoring and control systems. By continuously monitoring and adjusting dynamic parameters, resonance amplification can be utilized throughout the compaction process. This generally implies a lowering of the compaction frequency, which decreases energy consumption and wear of equipment and at the same time increases the compaction effect. The efficiency of resonance compaction has previously been shown in model tests and its practical use was, for the first time, demonstrated in this study by full scale tests using a powerful vibratory compaction plate.

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Raumfüllende Verdichtungskontrolle: Bodenarterkennung mit künstlicher Intelligenz am Anbauverdichter

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Schlagwörter: Verdichtungskontrolle; Künstliche Intelligenz; Anbauverdichter

1 EINLEITUNG

Die hier vorgestellte Raumfüllende Verdichtungskontrolle (RFVK) soll bei der Verdichtungsprüfung die Tiefenwirkung des Anbauverdichters, die Bodenart und den Wassergehalt mitberücksichtigen. Damit soll eine Aussage zur erreichten Verdichtung im Raum (nicht nur zur Bodensteifigkeit in der Fläche wie bei der FDVK) ermöglicht werden, und zwar auch bei nassen oder bindigen Böden und bei variabler Auflast auf den Anbauverdichter und ohne die Notwendigkeit einer Kalibrierung. Die Voraussetzung dafür ist die Kenntnis der Bodenart und des Wassergehaltes. Beides soll mit Methoden des maschinellen Lernens aus Mustern der Interaktion der Verdichterplatte mit dem Boden ermittelt werden. Das Projekt MUSKETIER wurde im Rahmen des BMBF-Programms „IKT 2020“ als Kooperationsprojekt der MTS AG, der Universität Tübingen und der TU Freiberg gefördert.

2 ANBAUVERDICHTER

Anbauverdichter sind dynamische Verdichtungsgeräte, die an einen Bagger angekoppelt und von deren Unwuchterreger von der Baggerhydraulik angetrieben werden. Typische Einsatzgebiete sind Böschungen, Grabenverfüllungen und Bauwerkshinterfüllungen. Die Fliehkraft der Unwucht versetzt die Platte im frei schwingenden Zustand in eine sinusförmige Bewegung. Bei Bodenkontakt ändert sich die Sinusform in Abhängigkeit von Boden- und Geräteeigenschaften. Bagger-Anbauverdichter wechseln aufgrund der Stützlast des Baggers normalerweise nicht in den Sprungbetrieb und erzielen wegen der breiten Auflagefläche und der hohen Antriebsleistung eine gute Tiefenwirkung.

Abbildung 1. Anbauverdichter auf der Kanalgrabenhauptverfüllung mit Anzeigedisplay

Eine Besonderheit ist die variable Auflast. Um die Bodenkontaktkraft zu bestimmen, ist die Messung der Auflast notwendig. Dem Geräteführer wird auf einem Display angezeigt, ob er zu viel oder zu wenig andrückt, ob die Betriebsfrequenz passt und wann der Verdichtungsprozess abgeschlossen ist.

3 PROBLEMATIK DER VERDICHTUNGSKONTROLLE AM ANBAUVERDICHTER

Walzenzüge protokollieren im Rahmen der Flächendeckenden Verdichtungskontrolle die Bodensteifigkeit. Grundlage sind Beschleunigungsaufnehmer an der Bandage und eine Auswertung entweder hinsichtlich des harmonischen Inhalts der Beschleunigungen oder hinsichtlich eines Last-Setzungsverhaltens, berechnet aus zeitintegriertem Beschleunigungssignal (Einsenkung der Bandage), Unwuchtvorlaufwinkel (zeitlicher Verlauf der Fliehkraft) und dem bekannten Gerätegewicht. Beide Verfahren setzen ein konstantes, im letzten Fall auch ein bekanntes, Gerätegewicht voraus. Anbauverdichter werden angedrückt. Die statische Auflast ist damit variabel. Eine Auflastmessung für Anbauverdichter ist erst seit 2017 auf dem Markt.

Die Werte der FDVK sind von der Bodensteifigkeit und vom Betriebsverhalten des Gerätes abhängig (Adam und Kopf 2004). Um aus dem Messwert auf die Bodensteifigkeit zu schließen, muss der Betriebszustand (Kontakt, Abheben, Springen oder Taumeln) erkannt werden können. Für Anbauverdichter sind komplexe und chaotische Bewegungsmuster bekannt (Nohlen und Berquin 2015). Der Wechsel der verschiedenen translatorischen und rotatorischen Modi erfolgt spontan, z.B. durch eine geringfügig in der Richtung veränderte Krafteinleitung durch den Bagger. Damit sind die gemessene Beschleunigungs- oder Wegamplituden stark von der Position des Sensors auf der Platte abhängig. Die Bestimmung der Bodenkontaktkraft ist trotzdem möglich und liefert gute Werte, wie anhand eines Vergleichs mit den im Boden gemessenen Kraftemmisionsspektren gezeigt werden konnte (Jahnke et. al. 2018). Auch das Ende der Verdichtung, d.h. der Übergang von plastischer Verformung zu rein elastischer Verformung, kann bestimmt werden. Das Verdichtungsende wird dem Geräteführer angezeigt, aber auch bei der Auswahl der Merkmale für die weiter unten beschriebene Bodenarterkennung verwendet.

Eine FDVK mit Bestimmung der Bodensteifigkeit (Tragfähigkeit) ist am Anbauverdichter nicht praxisgerecht. Anbauverdichter werden kleinräumig wechselnd auf verschiedensten Verfüllmaterialien eingesetzt, mit Vorteilen auf bindigen Böden. Für solche Verfüllungen gibt es Vorgaben zum Verdichtungsgrad (i.d.R. 97% Proctordichte), nicht aber zur Tragfähigkeit. Gerade die bindigen Böden entziehen sich einer Kalibrierung. Um auf diese verzichten zu können, muss die Bodenart, und der relative Wassergehalt bekannt sein, und es müssen passende Stoffgesetze vorliegen, um aus der bekannten Einwirkung des Anbauverdichters auf die tatsächlich erreichte Verdichtung schließen zu können. Anbauverdichter werden punktuell umgesetzt und arbeiten nicht lagenweise. Daher sollen die Aussagen zur erreichten Verdichtung räumlich, nicht nur flächenhaft erfolgen.

4 BODENARTERKENNUNG MITTELS KÜNSTLICHER INTELLIGENZ (KI)

4.1 Voraussetzungen

Die Mustererkennung ist nicht deterministisch, d.h. es werden keine Annahmen zu Ursache und Wirkung getroffen, sondern eine große Anzahl „Trainingsdaten“ werden mit der bekannten Bodenart solange verknüpft, bis das System grundlegende Muster selbst zuordnen kann. Falsche Fährten müssen deshalb vorab eliminiert werden. Beispielsweise könnte es zu einer Fehleinstufung kommen, wenn die Trainingsdaten für Lehm mit einem gut eingestellten Bagger, die für Kies jedoch mit instabiler Hydraulik gewonnen werden würden. Die Voraussetzungen für die Mustererkennung müssen zunächst deterministisch geschaffen werden (Nohlen und Berquin, 2015).

4.2 Trainingsdaten und Signalverarbeitung, Merkmalsextraktion

Den Trainingsdaten kommt eine elementare Bedeutung zu. Weil die Bewegungsmuster typspezifisch sind, muss jeder Gerätetyp extra angelernt werden. Masseverschiebungen, wie sie bei Modelländerungen vorkommen, können das Ergebnis stören. Deshalb wurde für die Aufnahme der Trainingsdaten immer derselbe Verdichter (Typ MTS V8 WA mit Auflastsensor) verwendet. Berücksichtigt man, dass für jede Bodenart und jeden Wassergehalt eine Vielzahl von Datensätzen zu erheben sind, und dass für jede Messung auch ein Bagger benötigt wird, ist klar, dass die Erhebung der Trainingsdaten der Flaschenhals bei der Nutzung künstlicher Intelligenz für die Verdichtungskontrolle ist. Die Signalverarbeitung entspricht in etwa den üblichen Verfahren der FDVK (Analyse im Zeit- und im Frequenzbereich). Die Merkmale für die Bodenarterkennung werden vorwiegend am Ende der Verdichtung extrahiert, wenn rein elastisches Materialverhalten vorliegt. Die Merkmalsextraktion selbst verlangt Verständnis für die Abläufe im Boden und für die physikalischen Gegebenheiten des gesamten Verdichtungsprozesses. Die Merkmalsextraktion erfolgt deterministisch aufgrund von menschlichem Sachverstand und ist der fachliche Kern der Entwicklungsarbeit.

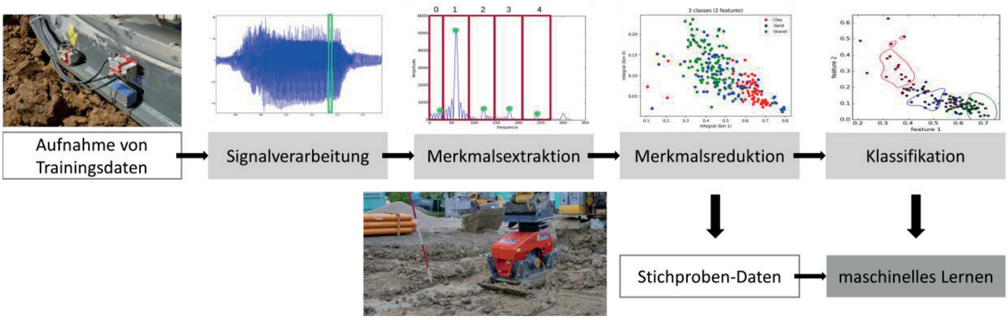

Abbildung 2. Schematischer Ablauf des maschinellen Lernens

4.3 Merkmalsreduktion und Klassifikation der Trainingsdaten

Die auf Wertetupel oder Vektoren reduzierten Merkmale ergeben einen mehrdimensionalen Datensatz. Weil für die Bodenarterkennung die Methode des „supervised learning“ angewandt wird, wird jedem Datensatz auch die nach Sichtprüfung oder im Labor bestimmte Bodenart mitgegeben. Die traditionell bodenmechanische Klassifikation wird dann der Klassifikation durch verschiedene Methoden des maschinellen Lernens gegenübergestellt. Verglichen wurden folgende Verfahren: k-Means, k-NN, Random Forests, Gaussian Process/Bayes, Support Vector Machines (SVM) sowie Neuronale Netze (feed forward, recurrent). Zunächst wurde mit Bayes‘schen Wahrscheinlichkeitsdichtefunktionen (Gaussian Process) gearbeitet, weil zu erwarten war, dass dieser Ansatz den realen Gegebenheiten gemischter Bodenarten und der kontinuierlichen Änderung des mechanischen Verhaltens aufgrund von Wassergehaltsänderungen am ehesten entspricht. Am Ende haben sich dann allerdings die Neuronalen Netze als am besten geeignet herausgestellt.

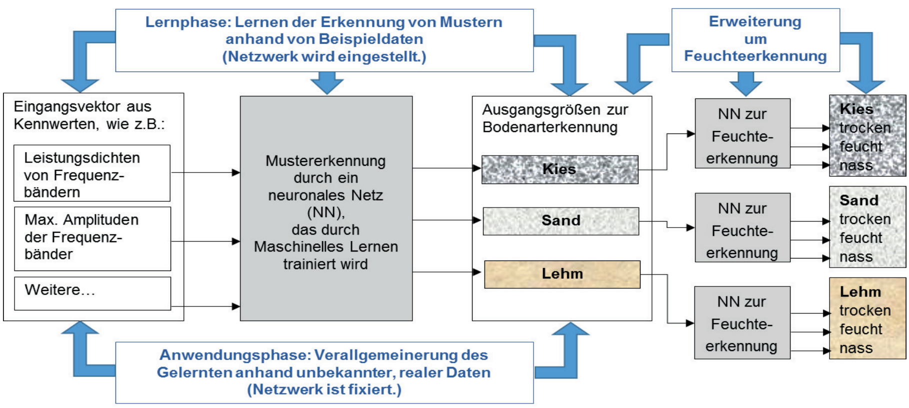

Abbildung 3. Ablauf des maschinellen Lernens am Beispiel hintereinandergeschalteter Neuronaler Netze

Was das Neuronale Netz mit den Daten tatsächlich tut, ist dem Nutzer komplett verborgen („black box“) – für Ingenieure eine ungewohnte Situation, mit der Befürchtung, dass die Klassifikation letzten Endes doch wieder nur auf die Bodensteifigkeit zurückzuführen sei. Ein überraschendes Ergebnis zeigte, dass das nicht der Fall ist. Für sehr nassen Lehm lagen uns zunächst keine Trainingsdaten vor. Weil KI kein Bewusstsein für die eigenen Grenzen kennt - eine Klasse „weiss ich nicht“ gibt es nicht - wurden die später erhobenen Daten für sehr nassen Lehm jeweils als Kies klassifiziert. Die Analyse der Daten deutet an, dass die Sättigung des undurchlässigen Materials zu einer bestimmten, von der sehr geringen Steifigkeit unabhängigen, Änderung des harmonischen Inhalts der Beschleunigungs-signale geführt haben könnte. Andere unbekannte, gemischte Böden, so z.B. gebrochener und etwas aufgeweichter Mergel, werden physikalisch sinnvoll wechselnd dem Kies und dem Lehm zugeordnet.
Das Neuronale Netz erwies sich außerdem als robust gegenüber einer schlecht eingestellten Baggerhydraulik und daraus resultierenden Abweichungen von der gewünschten Betriebsfrequenz.

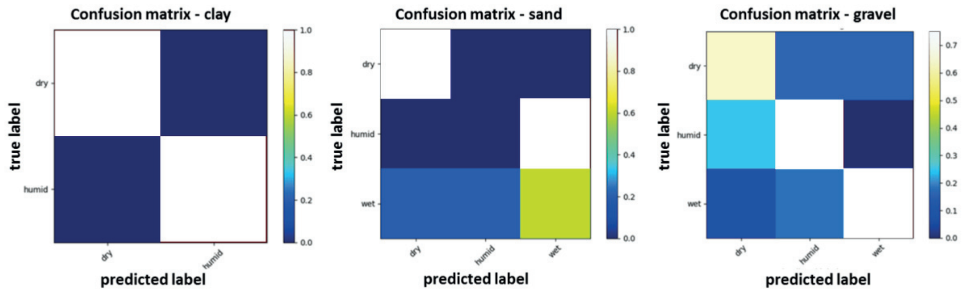

Abbildung 4. Drei wichtige Bodenarten werden mit dem relativen Wassergehalt zufriedenstellend erkannt.

5 ZUSAMMENFASSUNG UND AUSBLICK

Drei bautechnisch relevante Hauptbodenarten (Kies, Sand, Lehm) in jeweils drei Wassergehaltsbereichen (zu trocken, optimal, nass) können detektiert werden. Die grundsätzliche Eignung der gewählten Methode (hintereinandergeschaltete Neuronale Netze) ist bestätigt, so dass die Weiterentwicklung bis zu einer anwendungsreifen Praxislösung vorgesehen ist. Auf der Bauma 2019 soll ein erster Demonstrator gezeigt werden.

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Task Force 1. Promoting of use of non-traditional (such as recycled, large-sized or by-product) materials in road embankments and structural layers.

Task Force 2. Stabilization and reinforcement of geomaterials and its implications in pavement and rail track design.

Task Force 3. Intelligent construction in earthworks, development of guidelines, codes and specifications for effective use of IC technologies in earthworks.

Task Force 4. Rail track substructures, including transition zones, and transportation geodynamics.

Task force 5. Harbor geotechnics.

Task Force 6. Subsurface sensing for transportation infrastructure condition diagnostics among others.

Task Force 7. Climatic effects on geomaterial behavior related to mechanics of unsaturated transportation foundations.

Task Force 8. Organization of TC202-hosted Conferences, Workshops, and Webinars.

- 8th International Symposium on Environmental Vibration (ISEV) and Transportation Geodynamics and 2nd Young Transportation Geotechnics Engineers (YTGE) Meeting in Changsha, Hunan, China, Oct. 26-28, 2018.
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- 4th International Conf. on Transportation Geotechnics (ICTG) including 3rd Proctor lecture & 3rd YTGE Meeting, Chicago, USA, Aug. 30-Sep. 2, 2020.

2nd Proctor Lecture by Prof. A.G. Correia

Task Force 9. Transportation Geotechnics; Elsevier journal of TC202. Editors-in-Chief: António Gomes Correia; Erol Tutumluer, and Yunmin Chen. First issue was published in March 2014 of this quarterly journal, which aims to publish high quality, theoretical and applied papers on all aspects of geotechnics for roads, highways, railways, airfields and waterways. http://www.journals.elsevier.com/transportation-geotechnics/

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