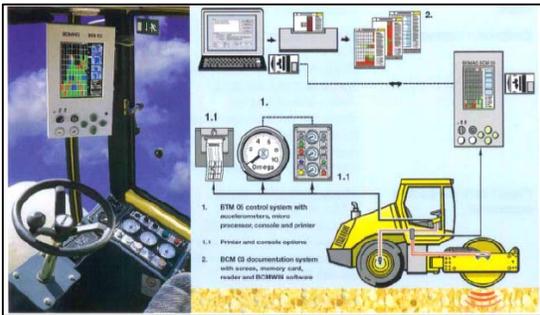
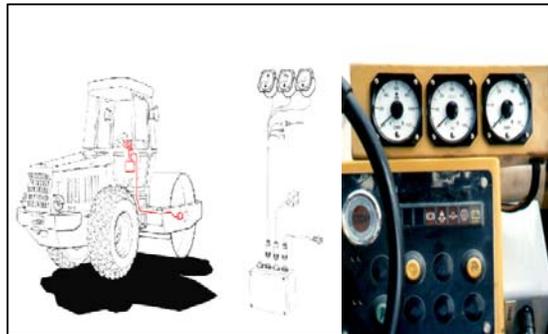


INTELLIGENT COMPACTION: OVERVIEW AND RESEARCH NEEDS

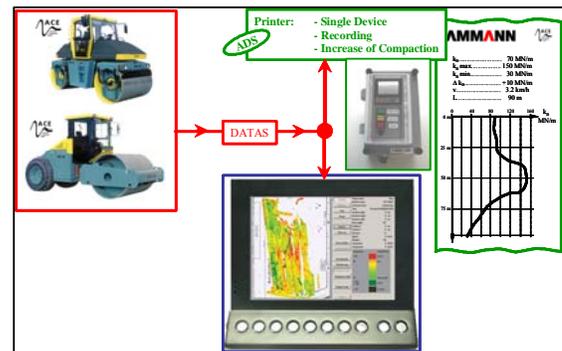
Jean-Louis Briaud (briaud@tamu.edu)
and Jeongbok Seo



From BOMAG



From GEODYNAMIK



From AMMANN

December, 2003
Texas A&M University

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Dr. Briaud can be contacted at (briaud@tamu.edu) and Dr. Seo at (jeongbok69@hotmail.com). The authors wish to thank the following people for their contributions to this report. Many of the figures and many of the references were contributed by the following individuals. Their willingness to let the authors use their work in this report is very much appreciated.

Name	Company	Email	Country
Roland Anderegg	AMMANN	r.anderegg@ammann-group.ch	Switzerland
Carl Petterson	Geodynamic	Info@geodynamik.com	Sweden
Chris Connolly	BOMAG	chris.connolly@compactionamerica.spx.com	U.S.A.
Chris Dumas	FHWA	chris.dumas@fhwa.dot.gov	U.S.A.
Hans Kloubert	BOMAG	kloubert@bomag.com	Germany
Alain Quibel	CETE	Alain.quibel@equipement.gouv.fr	France

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NOMENCLATURE AND DEFINITIONS

1. ACE: AMMANN Compaction Expert

2. ADS: Asphalt Documentation Control

3. BTM-E

BTM-E is a modified version of the BOMAG Terrameter, a computerized display of many important compaction parameters including the dynamic modulus E_{vib} in MN/m^2 (Figure 6.2)

4. BVC: BOMAG VARIOCONTROL

5. CAC: Continuous Asphalt Compaction

6. CCC: Continuous Compaction Control

7. CDS: Compaction Documentation Control

8. CMV (Compaction Meter Value, Adam and Kopf, 2000 - Geodynamik)

This value can be defined by assuming that the force amplitude (F) of the blows is proportional to the first harmonic of the acceleration and that the displacement (s) during the blow can be approximated by the amplitude of the double integral of the fundamental acceleration component (Sandstorm, 1985). From this assumption, the CMV is defined as

$$CMV \sim \frac{\hat{a}(2\omega_o)}{\hat{a}(\omega_o)} \quad \text{where } \hat{a} : \text{Amplitude, } \omega_o : \text{exciting frequency}$$

9. Double Jump

Double jump is an unstable vibration of the roller drum that occurs when the excitation is too strong for the layer that is compacted. The drum hits the ground hard, rebounds and then makes a full cycle in the air before it hits the ground again. The result is that the bounces on the surfaces are harder than normal, but occur with half the normal frequency. The spacing between the individual blows along the compacted strip is twice the normal spacing. Working in double-jump is undesirable and is likely to degrade the compaction performance, loosen already compacted areas, and shorten the life of the roller.

10. E_v (BOMAG, 1993)

In the plate load test, the calculations of the moduli for the second and first loading cycle should be based on smooth load-settlement curves. These curves can be expressed by using equation below:

$$s = a_0 + a_1 \cdot \sigma_0 + a_2 \cdot \sigma_0^2$$

where s is the settlement at the center of the loading plate, σ_0 is the average nominal stress under the plate, in MPa, a_0, a_1, a_2 are factors calculated from the plate loading test results. The modulus can be calculated from the gradient of the secant through the points $0.3 \cdot \sigma_{1\max}$ and $0.7 \cdot \sigma_{1\max}$, using the following equation:

$$E_v = 1.5 \cdot r \cdot \frac{1}{a_1 + a_2 \cdot \sigma_{1\max}}$$

where r is the radius of the loading plate, in mm, $\sigma_{1\max}$ is the maximum average normal stress of the first loading cycle, in MPa. The subscript 1 shall be used to denote the first loading cycle, and the subscript 2 to denote the second loading cycle.

11. ICM: Intelligent Compaction Machine

12. OMEGA (Adam and Kopf, 2000)

The acceleration of the drum \ddot{Z}_1 is measured in two orthogonal directions and the drum velocity \dot{Z}_1 is determined by integrating the acceleration components. Taking into consideration the mass of drum m_D , the rotating excentric mass m_E , the static force \vec{F}_{stat} and the excentric force \vec{F}_E , the energy W_{eff} is calculated as follows:

$$OMEGA \sim W_{eff} = \oint \left[-(m_D + m_E) \ddot{Z}_1 + \vec{F}_{stat} + \vec{F}_E \right] \times \dot{Z}_1 dt$$

The integration is performed for two cycles of excitation.

13. Oscillometer (Adam and Kopf, 2000)

The Oscillometer values (OMV) results from the horizontal acceleration signal of the oscillating drum \ddot{x} . Because of the partially slipping behavior of the drum, the response is different from a sinusoidal signal. Investigations have revealed that the gradient of the response signal when the acceleration is zero corresponds

with the soil stiffness. Thus, the OMV is calculated from the first derivative of the acceleration signal at $\ddot{x} = 0$ for each cycle with period t_{per} :

$$OMV = \left| \frac{d \ddot{x}}{dt} t_{per} \right| \text{ at } \ddot{x} = 0$$

14. PSI: Pavement Quality Indicator

15. RMV (Resonant Meter Value, Adam and Kopf, 2000))

$$RMV \sim \frac{\hat{a}(0.5\omega_o)}{\hat{a}(\omega_o)} \quad \text{where } \hat{a} : \text{Amplitude} \quad \omega_o : \text{exciting frequency}$$

EXECUTIVE SUMMARY

1. What is intelligent compaction?

Intelligent Compaction is achieved by a smooth drum vibratory roller with a measurement/control system. This measurement system uses the information collected to adapt the equipment performance continuously, to optimize compaction and meet required conditions. This system controls the different compaction parameters for the roller such as: drum vibration, amplitude, frequency and working roller speed (impact distance). The output parameter is a soil modulus which is calculated continuously on the basis of the monitored drum acceleration.

2. Advantages and drawbacks of intelligent compaction.

Intelligent Compaction gives an instantaneous and complete evaluation of the zone being compacted. It helps to remediate weak spots and avoids over-compaction. It reduces the number of roller passes, the number of conventional proof tests, and provides a soil modulus at all locations where the roller has traveled. It gives a more uniformly compacted layer. The drawback is that the equipment is more expensive than ordinary rollers. Intelligent Compaction evolved in Europe starting in the late seventies.

3. Contract types accelerated the development of intelligent compaction.

Today, Intelligent Compaction (IC) is more popular in European countries than in the U.S.A. One of the reasons is the difference in the type of contracts which exists in these countries for construction projects. In Europe, best-value awards are widely used in all types of procurements and design-build is the contracting method of choice for many types of projects. Long warranty periods are often in effect. Specifications requiring certain soil moduli values are in use (45 MPa for low volume roads, 120 MPa for freeways).

4. Theory is well established.

The theory to obtain the modulus from the measurement of the drum acceleration is clear and well established. It relies on the equations of equilibrium and the solution of a

drum on an elastic half space. The modulus calculated from the combined use of the measurements and the theory corresponds to stress levels as low as 100 kPa (first pass of a light roller) to 5000 kPa (last pass of a heavy roller on a well compacted and well graded soil), strain levels in the range of 1 to 5%, times of loading between 10 and 50 milliseconds, and a very low number of cycles because the vibrations are such that the soil at a certain location is “hammered” once per pass. An airplane has tire pressures approximately equal to the pressure generated by a medium roller (2000 kPa), a heavy truck has tire pressures approximately equal to the pressure generated by a light roller (600 kPa) while a car has much smaller tire pressures (200 kPa).

5. Equipment is readily available.

BOMAG, AMMANN, and Geodynamik are equipment manufacturers which provide fully equipped rollers or attachments to place of existing rollers. These pieces of equipment have the ability to optimize compaction according to the intelligent compaction principles and give various displays of the parameters recorded. A number of case histories in the USA and in Europe have been documented.

6. Research needs exist.

There are a number of research needs which have to be addressed before intelligent compaction can reach its full potential in the USA.

- a) Demonstrate that intelligent compaction leads to better compaction than conventional compaction. Costs differences between the two techniques should be documented.
- b) Understand the interdependence between the modulus and the water content, as well as the shape of the modulus versus water content curve.
- c) Develop a simple laboratory test to obtain ahead of time the target modulus from a modulus versus water content curve; this target modulus must be verified in the field using the same test.
- d) Study the depth of compaction that can be achieved by various rollers for various soils.
- e) Study existing specifications and draft standard specifications for the USA.

7. Proposed approach.

An approach is proposed which satisfies the ideas developed above. It consists of running the usual Proctor test in the laboratory but adding the Briaud Compaction Device test (BCD lasts 2 seconds) on top of the soil in the mold to get the modulus. This gives the target values. In the field, the instrumented roller performs intelligent compaction and is checked at chosen intervals with the BCD. The advantage of the BCD is that the same test can be run in the lab and in the field in very little time.

1. INTRODUCTION

1.1. General

For road and embankment construction, a new compaction method called intelligent compaction has been used in some European countries. This new method aims at providing a higher quality road or embankment by implementing a reliable quality and interactive assurance system from the very start of construction. This report describes intelligent compaction as well as associated research needs.

Traditionally, soil and rock fill materials are compacted with static or vibrating rollers (Figure 1.1). Compaction of a certain area is carried out by parallel strips (edge to edge or with some overlapping) covering each strip with a fixed number of passes. Most rollers are vibrating rollers; their vibration frequency and amplitude is kept constant and the operator chooses the roller speed. A certain number of passes and a constant roller speed, vibration frequency and amplitude do not necessarily lead to a homogeneous compaction result on a layer due to variation in material properties, water content of the layer being compacted, and stiffness of the underlying layer. A constant number of passes and constant roller parameters will often leave a certain part of the area insufficiently compacted, another part over-compacted and the rest sufficiently compacted.

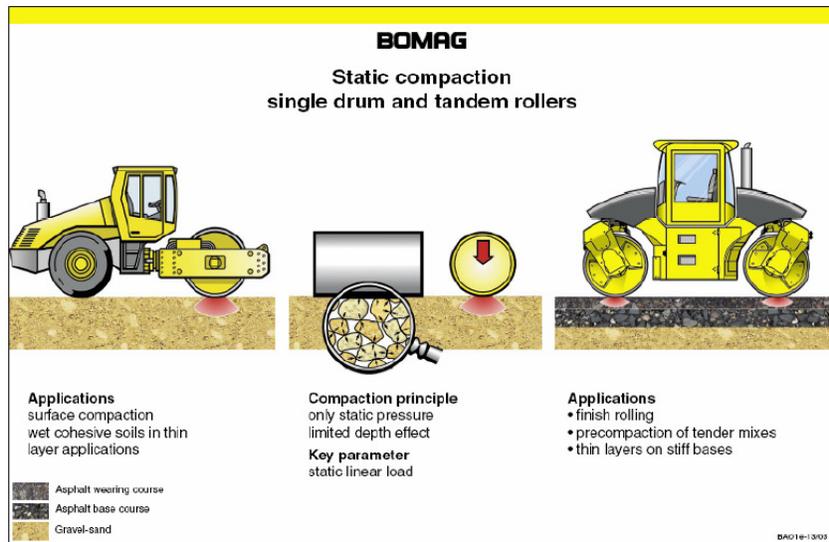


Figure 1.1 Rollers (Single Drum and Tandem Drum, from BOMAG Brochure)

1.2. Conventional Spot Testing Methods

The most common quality control test method in road construction consists of carrying out spot tests, used systematically or statistically. These tests include for example the static plate load test and the falling weight deflectometer test (modulus), the nuclear gauge, the water balloon and the sand replacement test (density, Figure 1.2). In asphalt compaction, drilled cores, radiometric sounds (modulus) and Pavement Quality Indicator (PQI) tests are used. These methods are standardized and their results may be compared with each other, as long as their performance meets the standard. It should be noted that the depth range tested by each method is different both in soil and in asphalt compaction. For example, the depth range of modulus value obtained with the static plate load test is estimated to be 1.5 times the plate diameter; therefore results obtained with a 0.30 m and a 0.60 m diameter plate may not be compared directly since the modulus is always an average value within the zone of influence. Also the measured modulus values may differ significantly within a short distance due to heterogeneity. Density values measured from replacement methods (water balloon, sand, and Bentonite) are valid only for the depth where the volume was taken. Radiometric sound tests, used to check soil and asphalts compaction, have different depth range depending on the equipment and the source, with deeper zones having less influence on the average value obtained. In addition, the water content or bitumen content as well as chemical differences will affect the average density value.



Figure 1.2 Equipment for the Sand Replacement Test
(<http://www.intec.com.my/product/soilequip/sandpouring.html>)

Most spot test methods are time consuming and one has to wait hours or days until the results are available. Sometimes, delay or stop of construction work occurs, because the tests have to be carried out without any disturbing vibrations around the test spot.

1.3. Relative Compaction Testing Methods

This kind of system measures and provides relative values, i.e. the system compares index values for two successive passes with a compaction equipment and shows the difference between the values for these two passes. It does not give the absolute percent compaction, stiffness or density achieved. These systems are available as an attachment for any compaction roller and are typically called “Compactometer” or Compaction Meters. Geodynamik Corp in Sweden is a company which sells such equipment.

Methods called “Continuous Compaction Control, CCC,” are part of the relative compaction testing methods. They are included in national specifications in Austria, Germany, Finland, and Sweden. These methods are based on roller integrated compaction meters (Figure 1.3) that continuously measure the acceleration of the roller drum and calculate a compaction meter value from the acceleration signal.

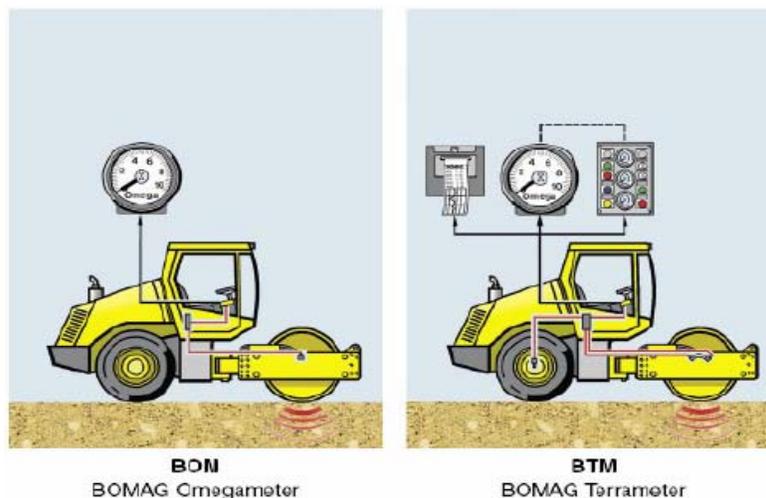
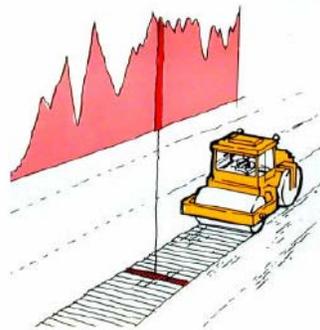


Figure 1.3 Omegameter and Terrameter (from BOMAG Brochure)

The drum of the vibrating roller subjects the soil to repeated cycles of vibration. Analogous to a dynamic plate load test, the blows or cycles of a cylindrical drum can be considered to be a load test of the soil. The value recorded by the compaction meter is presented to the roller operator instantaneously and continuously, enabling him to evaluate where compaction work is finished, where additional passes are required and what sections cannot be sufficiently compacted with the present roller. Examples of such compaction indices are the compaction meter values (CMV) for the Compactometer from GEODYNAMIK, the Omega value for the Terrameter, from BOMAG and the OMV for the Oscillometer from BOMAG also. These compaction indices are dimensionless, relative values requiring the roller parameters (drum diameter, linear load, frequency, amplitude, speed etc.).

The general overview of the compacted area based on the dimensionless compaction meter values gives the roller operator sufficient information to avoid over and under compaction (Figure 1.4). When the roller operator has decided that the compaction work has been accomplished, he can print out a Continuous Compaction Control protocol or CCC-protocol at the site. The protocol is his quality assurance documentation which can also be used to locate spot tests for calibration.



**Figure 1.4 Compactometer Value in Continuous Compaction Control, CCC
(Thurner & Sandstrom, 2000)**

1.4. Absolute Compaction Testing Methods

By measuring density and/or stiffness continuously, these methods provide absolute values of the achieved compaction for the operator of the compaction equipment. In the past, absolute compaction results could only be provided by

“independent” measuring equipment, i.e. equipment that was not attached to the compaction machine. Today certain manufacturer have equipment giving modulus values on the fly through the use of absolute measuring systems which can be attached to compaction equipment. These systems give the operator and the contractor a proof that proper compaction has been reached. Figure 1.5 shows an example of such system from AMMANN.

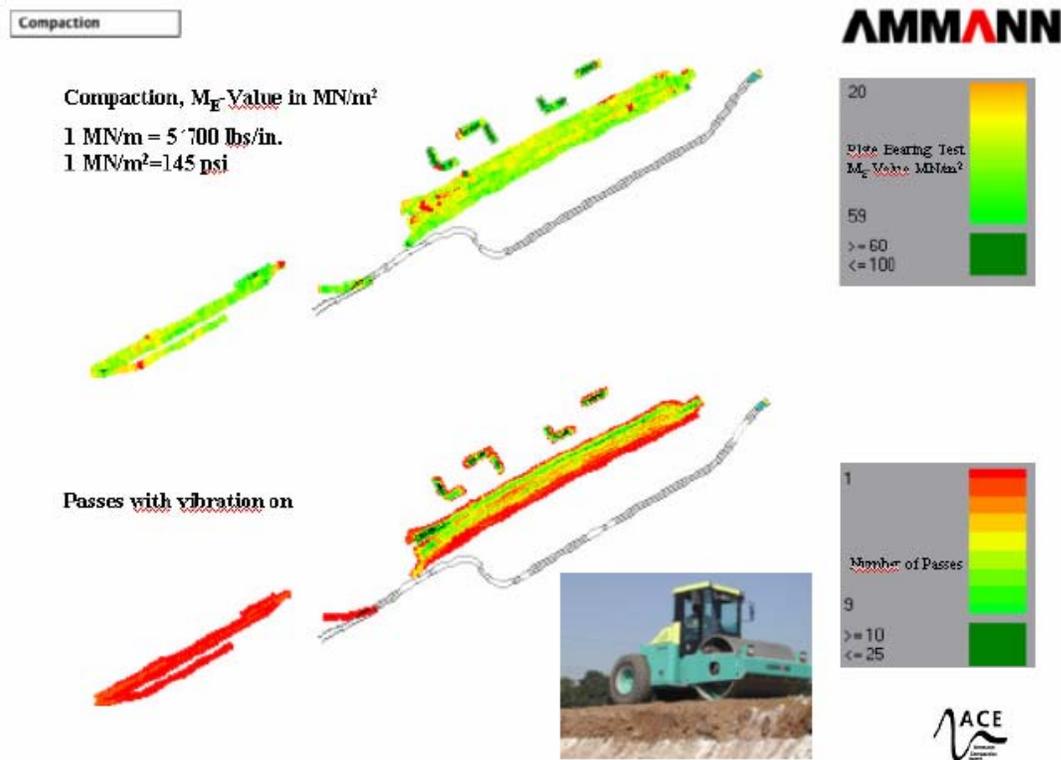


Figure 1.5 Absolute Compaction Method using ME (from AMMANN Brochure)

1.5. Intelligent Compaction Method

This “state of the art” technology is the combination of the absolute measurement technology with a regulation system that uses the measured information to adapt the equipment performance continuously to the required conditions. This system (Figure 1.6) controls the different compaction parameters of the roller: amplitude, frequency and working speed (impact distance). The modulus measurements are made by the instrumented roller itself. The system is preconditionned with the range of acceptable

modulus values (target value) and automatically adjusts the roller settings to achieve the target modulus if the readings are not within tolerance.

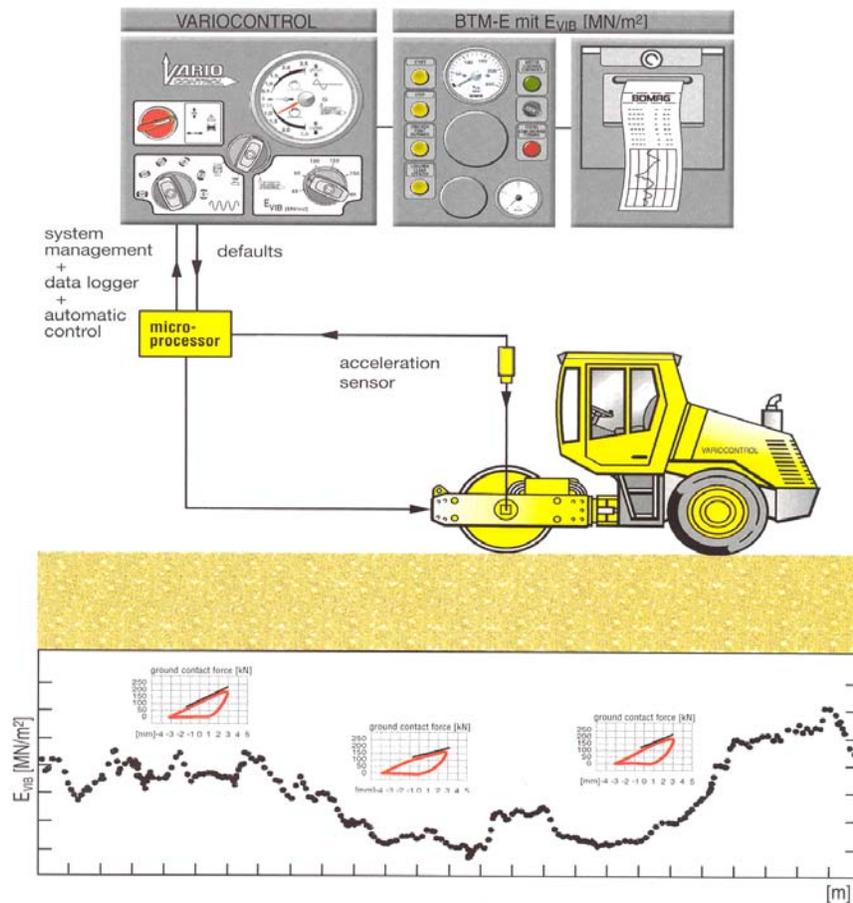


Figure 1.6 Intelligent Compaction System (from BOMAG Brochure)

For the vibrating rollers, high amplitude and low frequency are used to compact soft soils and to reach deeper zones while low amplitudes and high frequencies are used for stiff soils and shallow depth (Figure 1.7 and 1.8). For example on a first pass the roller might use a high amplitude and a low frequency but at the fourth pass it sets itself with a low amplitude and a high frequency. Furthermore such adjustments guarantee that the material will not be “over-compacted”; in this case, the amplitude and frequency are driven down automatically when the targeted percentage of absolute compaction is achieved and the roller passes such a spot without vibration. This new compaction process ensures instantaneous and complete evaluation of the zone being compacted, and remediation of weaknesses on an instantaneous and continuous basis.

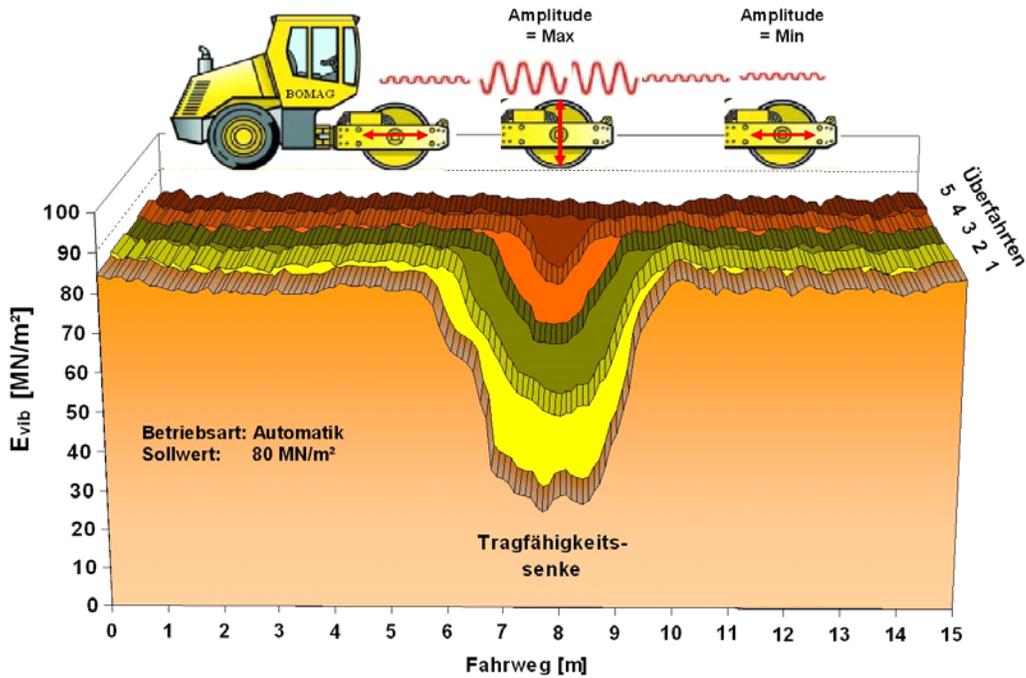


Figure 1.7 Varying amplitude and frequency to optimize compaction (from

Comparison between conventional single drum compactor and VARIOCONTROL roller

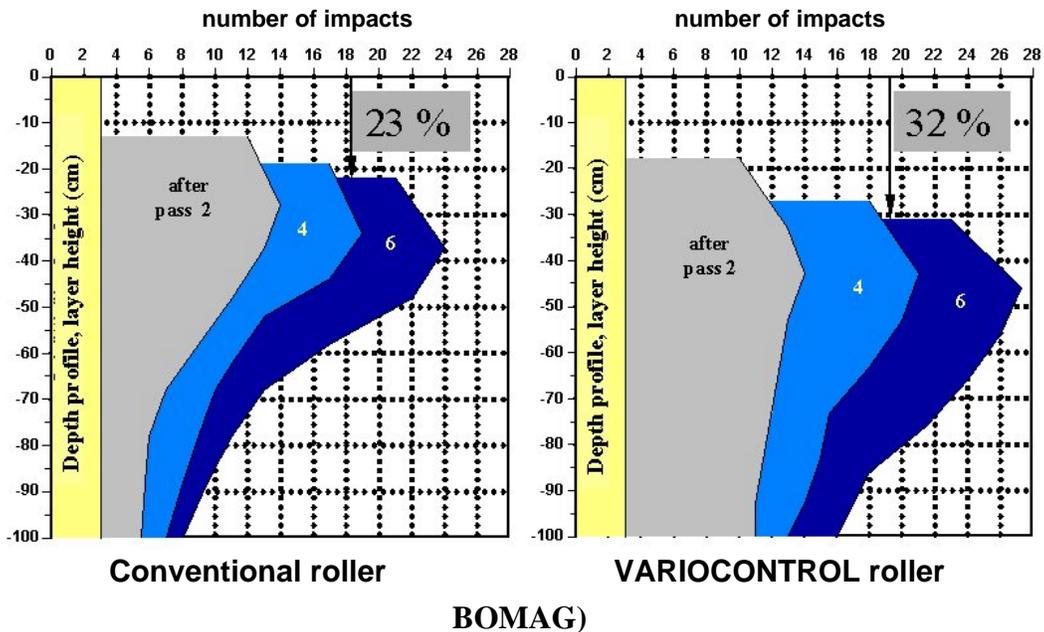


Figure 1.8 Depth of influence of compaction process (from Kloubert's presentation at TRB-2004, BOMAG)

2. HISTORY

It appears that intelligent compaction started in the late seventies with the work of BOMAG in Germany, AMMANN in Switzerland and Geodynamik in Sweden. These three companies seem to dominate the market today. Some of the earlier papers discussing the basic concepts and initial experience were published by Forssblad (1980) and Thunder and Sandström (1980); they dealt with R & D results as well as experience from the field. In 1982, the first measurement system was introduced by BOMAG for soil compactors. Then in 1989, the first documentation system for soil compactors was presented. The first prototype of an “Intelligent Compaction Machine, ICM” was on display at the BAUMA conference in 1992. In 1993, the German Ministry of Highways Construction gave its first recommendations on SCCC (Soil Continuous Compaction Control, a precursor of Intelligent Compaction). The same German Ministry of Highways Construction introduced contract specifications on SCCC in 1994. In 1996 a variomatic roller was introduced for asphalt compaction; in 1998 it was the turn of the variocontrol for soil rollers; in 2000, the modulus Evib was introduced and correlated to plate tests; in 2001, the asphalt manager came out.

3. WHY INTELLIGENT COMPACTION?

3.1. Advantages and Drawbacks of Intelligent Compaction

Intelligent compaction has the following advantages:

1. Higher efficiency and maximized productivity by automatic control of amplitude, frequency, and speed
2. Minimized Number of Passes
3. Higher adaptability (thin/thick layers, soft/stiff subbase)
4. Wider application range
5. Optimal Compaction Results, better quality
6. More uniform compaction
7. Less aggregate crushing
8. Better flatness
9. Complete coverage of compaction surface evaluation
10. Dynamic measurement of soil stiffness
11. No Danger of Overcompaction
12. Compaction Control on the Job
13. Easy to operate
14. Extended life of the roller by minimizing the double jump situation.

Intelligent compaction has the following disadvantages:

1. It requires sophisticated equipment in a rugged environment
2. It requires some operator training
3. It is more expensive than conventional compaction

These drawbacks pale compare to the benefits. In fact the third drawback above is likely to be a short term drawback as the added quality of the final product is likely to represent significant savings associated with less disputes with the clients. Intelligent compaction develop in Europe in large part because of the evolution of contracts and procurements and it might be enlightening to discuss these issues next.

3.2. Type of Contracts

Intelligent Compaction is more popular in European countries than in the U.S.A. in part because of the different types of contracts used for construction project. According

to an FHWA publication (FHWA, 2002), European methods of contract procurement and administration were very similar to those in the United States until the late 1980s. Public transportation agencies retained tight control over the design and construction of the highway systems. Prescriptive specifications and low-bid procurement methods were the public-sector tools of choice for procuring new works in both the United States and Europe. In the late 1980s, European agencies began to make significant changes to contract administration techniques. Some of the most significant drivers of change confronting Europe included:

1. Growing infrastructure needs
2. Inadequate public funds
3. Insufficient and diminishing staff
4. Lack of innovation in addressing project needs
5. Slow product delivery and delays
6. Cost overruns
7. Adversarial relationships
8. Claims-oriented environments
9. Perceived lack of maintenance efficiency
10. New European Union (EU) directives
11. User frustration
12. Political discontent

The most notable differences between European and U.S. procurement methods are that best-value awards are widely used in all types of procurements and design-build are the contracting method of choice for many types of projects in European countries. Design-build contracts are typically awarded on a best-value basis. The other difference is that performance contracting is in its infancy in the U.S. transportation sector, but the tools and techniques are well established in Europe. Performance contracting allows the contractor to employ whatever means it determines are most appropriate (and economical) to satisfy the performance specifications provided by the owner. Performance contracts allow innovation through creative design and construction methods and are thought to lower the overall price of a given project. Performance specifications are critical elements of performance contracting. In The Netherlands,

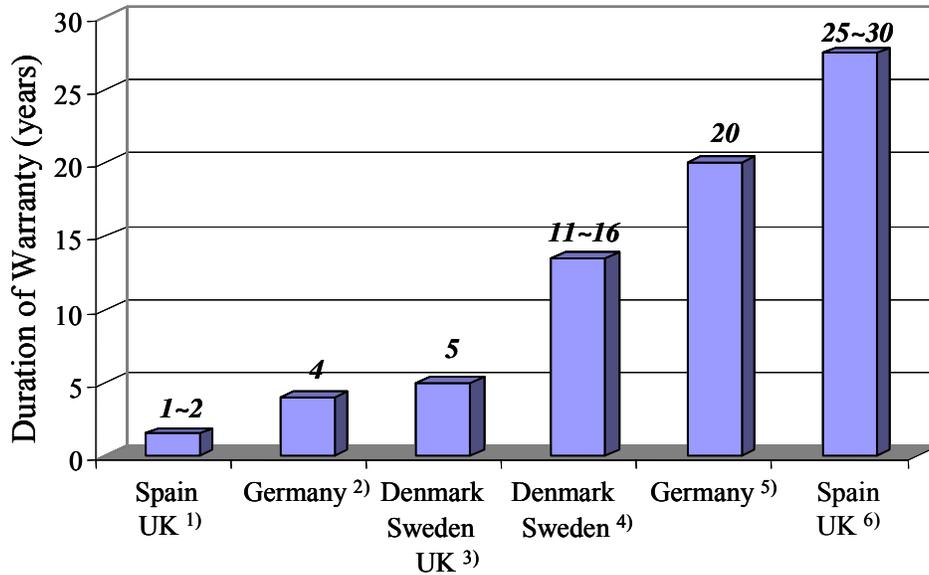
performance specifications are divided in five levels of requirements that range from road-user wishes to requirements for basic materials and processing. Performance specifications detail both the operating level and minimum condition of the facility at the time it is returned to public ownership.

An area of concern in performance contracting in the United States is quality assurance/quality control (QA/QC). Traditional QA/QC roles and responsibilities in the United States can impede the effectiveness of performance contracting. Performance contracts place the responsibility for quality control solely with the contractor, and the owner retains only a minimal quality assurance role. Owner quality assurance is built into the process at various "stop" or "control" points on projects. There are also unique processes for penalty points and quality audits in lieu of heavy owner inspection.

These differences increased the use of intelligent compaction in European countries. CCC has been implemented in national standards of Austria (RVS 8S.02.6), Germany (ZTVE StB94), Sweden (VÄG 94), and Finland. Also countries like France, Ireland and the Netherlands are planning for an introduction of national CCC standards. So far, details of the national standards differ somewhat, especially concerning calibration routines, but one can expect that the European Community will lead to a rather uniform structure of these standards.

3.3. Warranty

The European countries shown in Figure 3.1 have a long history of warranties for pavement construction. These countries have employed material and workmanship warranties with varying warranty periods for 10-40 years. Although their warranty programs have developed independently through either government specification or industry promotion, all these countries believe that warranties have improved the quality of their highway systems. Figure 3.1 provides an overview of the warranty types (after FHWA, 2002).



- Note: 1) Material and Workmanship
 2) Material and Workmanship
 3) Performance
 4) Pavement Performance Contracts
 5) Pavement Performance Contracts “Functional”
 6) Design-Build Finance Operate

Figure 3.1 Warranties Observations (After FHWA, 2002)

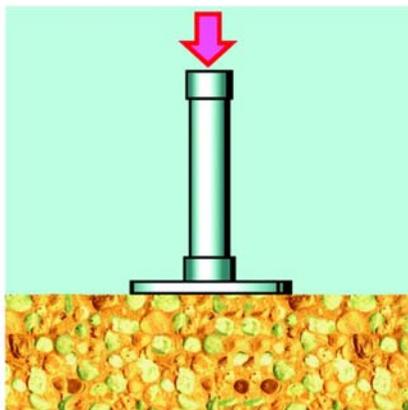
At a minimum, all of the countries use material and workmanship warranties on their traditional contracts. These warranties ensure that the contractor will build the pavement as specified by the owner and fix any defects resulting from the use of improper materials or inferior installation. Depending upon the country, the highway agencies may seek a remedy of defects from either the asphalt contractor or the prime contractor, if the prime is not the asphalt contractor.

3.4. Target modulus values and specifications

A soil modulus is a true soil parameter; yet it is a soil parameter which depends on many factors. Appendix A gives an introduction to the soil moduli including the various factors influencing it. Because of all these factors it is very difficult to go from one modulus to another without proper calibration and comparison. The roller gives a modulus which is associated with a certain stress level, strain level, rate of loading,

number of cycles, and water content among other factors. It is difficult to reproduce all those same parameters in the laboratory. Instead, the countries which have switched to modulus based compaction control have typically compared the roller modulus to standard field plate test modulus. This plate modulus (Figure 3.2) may have been the basis for design in that country for a long time and the comparison has given the authorities of that country confidence that a certain roller modulus indicates satisfactory compaction. However different requirements are set for low traffic roads ($E_{\text{roller}} = 45$ MPa) and for freeways ($E_{\text{roller}} = 120$ MPa). Such value have become part of specifications. Figures 3.3 to 3.6 are presented as examples of recommendations (Germany).

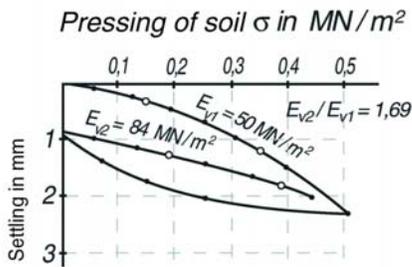
Plate loading test, German standard DIN 18134



Objective: assessment of deformation and strength characteristics of soils, use in earth works, highway and airfield construction and foundation engineering

deformation modulus (strain modulus)

$$E_v = 1,5 \cdot r \cdot \Delta T / \Delta S \text{ [MN/m}^2\text{]}$$



Formular is based on the theory of the surface deflection of an elastic isotropic half-space under a vertically loaded rigid circular plate.

plate diameter: 300 mm,
measuring depth: 400 - 600 mm

Figure 3.2 Plate test as a calibration for target values (from Kloubert's presentation at TRB-2004, BOMAG)

Asphalt road design solutions depending on road classification

Road classification	I				II				III				IV				V				VI			
Traffic load number (VB)	> 1800				900 – 1800				300 – 900				60 – 300				10 – 60				< 10			
Thickn. of frostres. pavement	50	60	70	80	50	60	70	80	50	60	70	80	50	60	70	80	40	50	60	70	40	50	60	70
Asphalt base on subbase																								
Surface course																								
Binder course																								
<u>Asphalt base</u>																								
Subbase (frost resistant material)																								
Thickness of subbase	-	30	40	50	-	34	44	54	28	38	48	58	32	42	52	62	26	36	46	56	30	40	50	60

Thickness in cm, modulus of deformation EV_2 in MN/m^2

V_B = number of vehicles on one lane with a total weight of > 2.8 t (according to German specifications)

Figure 3.3 Recommendations for Asphalt compaction (from Kloubert's presentation at TRB-2004, BOMAG)

Interrelationship between deformation modulus EV_2 and Proctor density for granular soils according to German Federal Earthworks Specification

Type of soil	standard Proctor density [%]	deformation modulus EV_2 [MN/m^2]	EV_2 / EV_1
GW, well graded gravel	≥ 100	≥ 100	$\leq 2,3$
	≥ 98	≥ 80	$\leq 2,5$
	≥ 97	≥ 70	$\leq 2,6$
GE, uniform gravel	≥ 100	≥ 80	$\leq 2,3$
SE, SW sand	≥ 98	≥ 70	$\leq 2,5$
	≥ 97	≥ 60	$\leq 2,6$

Figure 3.4 Comparison between modulus and density (from Kloubert's presentation at TRB-2004, BOMAG)

Compaction requirement according to German ZTVE-StB94

	Degree of compaction D_{Pr} [%]		Air void content n_a [%]	Deformation modulus EV_2 [MN/m ²]	
	granular soils	cohesive soils		granular soils	semi-cohesive soils
▽ Formation	▽ 100	▽ 100 (97)		120 (100)	45
▽ - 0,50 m	▽ 100 98	▽ 100 (97) 97 (95)	12		
▽ - 1,0 m	98	97 (95)	12		
▽ Fill base					

Figure 3.5 Recommendations according to German specifications (from Kloubert's presentation at TRB-2004, BOMAG)

Laying and compaction specification for road construction in Germany

Soil layers	Density (Standard Proctor)	Bearing capacity (load bearing test, EV_2)	Eveness (4 m straight edge)
Subbase	100 - 103 % *	100 - 150 MN/m ² *	20 mm
Capping layer	100 - 103 % *	100 - 120 MN/m ² *	40 mm
Formation	97 - 100 % *	45 - 80 MN/m ² *	60 mm

* depending on road classification and road design

Figure 3.6 Further recommendations for compaction of soil layers (from Kloubert's presentation at TRB-2004, BOMAG)

4. ROLLERS

Rollers are widely used for compacting soil, fill, and asphalt. Rollers are selected depending on the thickness of the layer to be compacted, and the properties of the material to be compacted. The machine parameters include the roller weight (total and drum), the compaction type (static or vibratory), and the diameter and surface of the drum (sheepfoot or smooth). The material properties include grain size distribution, grain shape, maximum grain size, water content, water and air permeability.

4.1. Static Rollers

Rollers with static drums (Figure 4.1) use the effective dead weight of the machine to apply pressure on the surface. Thus, soil particles are pressed together and the void content is reduced. Adequate compaction with static rollers is normally achieved only in the upper layers of the material because the effective depth of static compaction is limited.

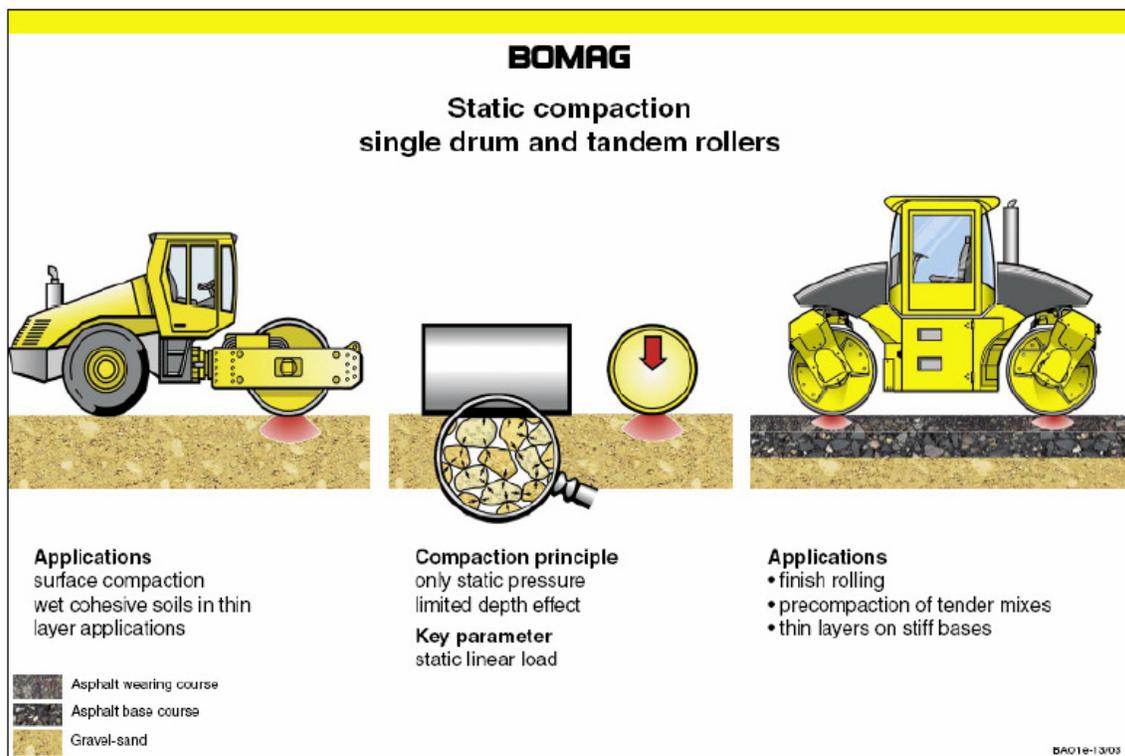


Figure 4.1 Static Roller (from BOMAG Brochure)

Cohesive fine grained soil can be compacted sufficiently with static sheep foot roller. The low permeability of fine grained soils leads to pore water pressures when applying (dynamic) compressive stress. Pore water pressures reduce the compaction effect significantly or prevent compaction all together. However, a statically passing sheepfoot drum “kneads” the soil near the surface resulting in a reduction of pore pressures and void ratio respectively. Nevertheless, only thin layers can be compacted at a time.

4.2. Dynamic Rollers

As reported by Adam and Kopf (2000), dynamic rollers make use of a vibrating or oscillating mechanism, which consists of one or more rotating eccentric weights. During dynamic compaction, a combination of dynamic and static load is used. The dynamically excited drum delivers a rapid succession of impacts to the underlying surface where the particles are set in motion by the transmission of compressive and shear waves. These vibrations eliminate periodically the internal friction between particles and facilitate, in combination with the static load, the rearrangement of the particles into positions that result in a lower void ratio and a higher density. Furthermore, the increase in the number of contact points and planes between the grains leads to higher stability, stiffness, and lower long-term settlement behavior.

4.2.1. Vibratory Roller

The drum of a vibratory roller is excited by a rotating mass which is connected to the shaft of the drum axis (Figure 4.2) The rotating mass sets the drum in motion and the direction of the resultant force is tied to the position of the eccentric mass. Compaction is achieved mainly by the series of compression waves penetrating the soil or the asphalt in combination with the effective static weight of the drum. The resulting compaction force is almost vertical.

The behavior of a vibratory roller drum (amplitude, frequency) changes depending on the soil response. Numerous investigations have revealed that the drum of a vibratory roller operates under different conditions depending on roller and soil parameters. Five operating conditions can occur (Table 4.1).

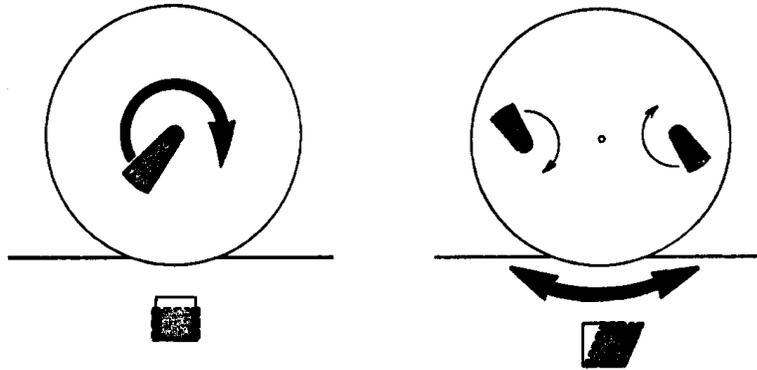


Figure 4.2 Vibratory roller drum (Compaction by cyclic compression) and oscillatory roller drum (Compaction by cyclic shearing) (Adam and Kopf, 2000)

Table 4.1 Operating condition of a vibratory roller drum (After Adam and Kopf, 2000)

Behavior of Motion	Interaction drum-soil	Cycle	Operating Condition	Application of CCC	Soil Stiffness
Periodic	Contact	1	Continuous Contact	Yes	Low
	Partial loss of contact	1	Partial uplift	Yes	
		2 (4)	Double jump	Yes	
		2 (4)	Rocking motion	No	
Chaotic		-	Chaotic motion	No	High

Continuous contact only occurs when the soil stiffness is very low; this is the case of uncompacted or soft layers with fines. Partial uplift and double jump are the most frequent operating conditions. Figure 4.3 shows the vertical movement of the soil at different depth under the drum of a vibrating roller in the double jump condition. The occurrence of the partial jump and the double jump condition depends on the number of excitation per seconds. When the soil stiffness increases, the motion of the drum axis is no longer vertical; the drum starts rocking. Very high soil stiffness in combination with disadvantageous roller parameters can cause chaotic motion of the drum. When rocking and chaotic drum motion happens, no effective compaction is possible. Vibratory rollers are the most commonly used rollers world-wide because they can be used for a wide range of granular mostly cohesionless soils.

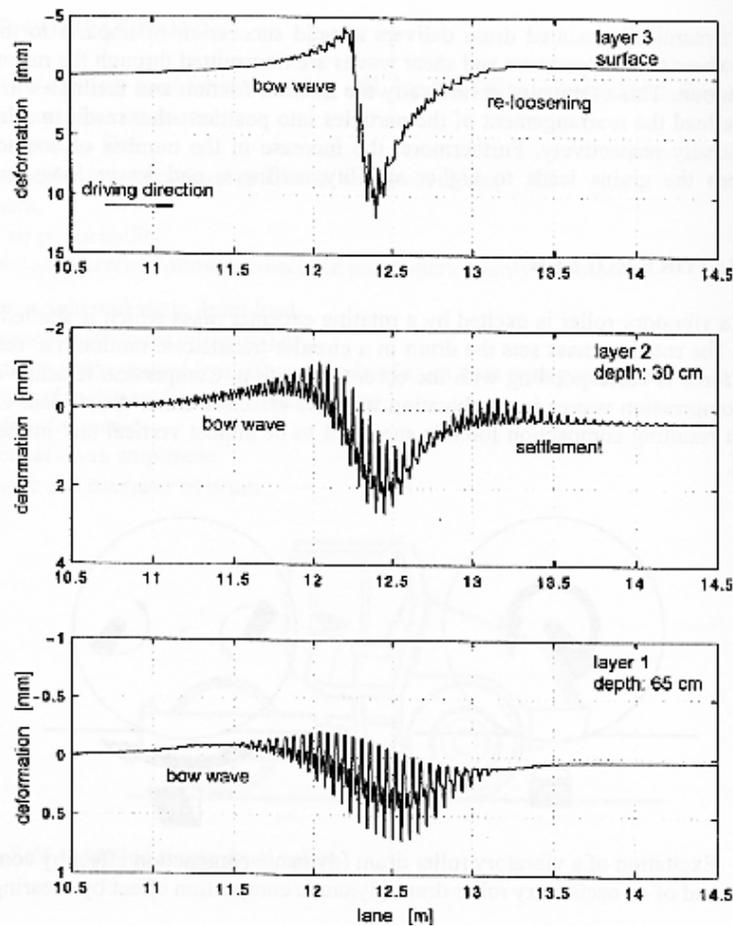


Figure 4.3 Soil deformation at three depths below the impacts of a vibratory roller drum operating in the double jump condition (Adam and Kopf, 2000)

4.2.2. Oscillatory Roller

The drum of an oscillatory roller oscillates torsionally. The torsional motion is caused by two opposite rotating excentered masses, which shafts are arranged excentrically to the axis of the drum. Thus, the soil is shaken horizontally in addition to the vertical dead load of the drum and the roller frame. These cyclic horizontal forces result in soil shear deformation. In this case, compaction is achieved mainly by transmitted shear waves through the material. Oscillatory rollers are used mainly for asphalt compaction and for cohesive soils. Furthermore, oscillatory rollers are used in the vicinity of sensitive structures, because the emitted vibrations are typically lower in amplitude and in zone of influence than the vibratory rollers.

4.2.3. VARIO Roller

The VARIO roller is a development of the BOMAG Company. In a VARIO roller two counter-rotating exciting masses cause a directed vibration. These masses are concentrically placed on the shaft of the drum. The direction of vibration can be adjusted by turning the complete exciter unit in order to optimize the compaction effect for a given soil type (Figure 4.4). If the exciter direction is vertical or inclined, the compaction effect of a VARIO roller can be compared with that of a vibratory roller. However, if the exciter direction is horizontal, the VARIO roller compacts the soil much like an oscillatory roller. There is some difference between a Vario roller and an oscillatory roller because the shear deformation of the soil is caused by a horizontal translation in the VARIO roller, whereas the drum of an oscillatory roller is working torsionally. Nevertheless, a VARIO roller can be used both for dynamic compression compaction, for dynamic shear compaction, and for a combination of these two conditions. Consequently, VARIO rollers are applicable to more soil types and the optimum direction of vibration can be found by a site investigation for each project.

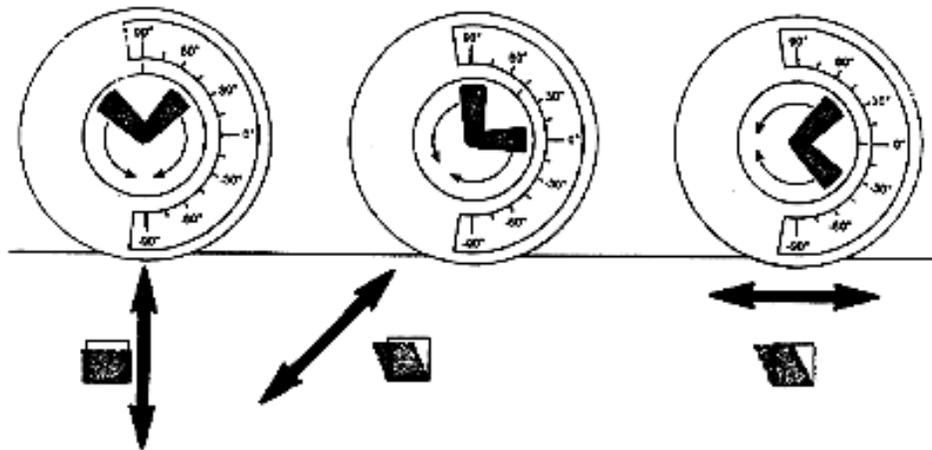


Figure 4.4 Adjustable excitation direction of a VARIO roller drum and compaction effect (Adam and Kopf, 2000)

4.2.4. VARIO Control Roller

Based on the findings related to the operation of different dynamic rollers, BOMAG developed the first automatically controlled roller: the VARIO CONTROL roller (Figure 4.5). In this roller, the direction of vibration can be varied automatically

from vertical to horizontal by using defined control criteria (Figure 4.6), which allow an optimized compaction process (intelligent compaction) and consequently, a highly uniform compaction. These criteria are:

Operating criterion: If the drum passes to the operating condition called “double dump,” the excitation direction is immediately changed, so that the drum goes back to the operating condition of partial uplift.

Force criterion: If the specified maximum compaction force is reached, the excitation direction is changed by the automatic control system, so that the applied force does not exceed the maximum force.

Two accelerometers, which are mounted on the bearings of the drum, record the drum acceleration continuously. The soil contact force, the energy delivered to the soil, and the displacements are calculated in a process unit taking into consideration the roller parameters such as masses, exciter force and frequency. The data are transmitted to an integrated system, which manages the parameters automatically (Figure 4.7).

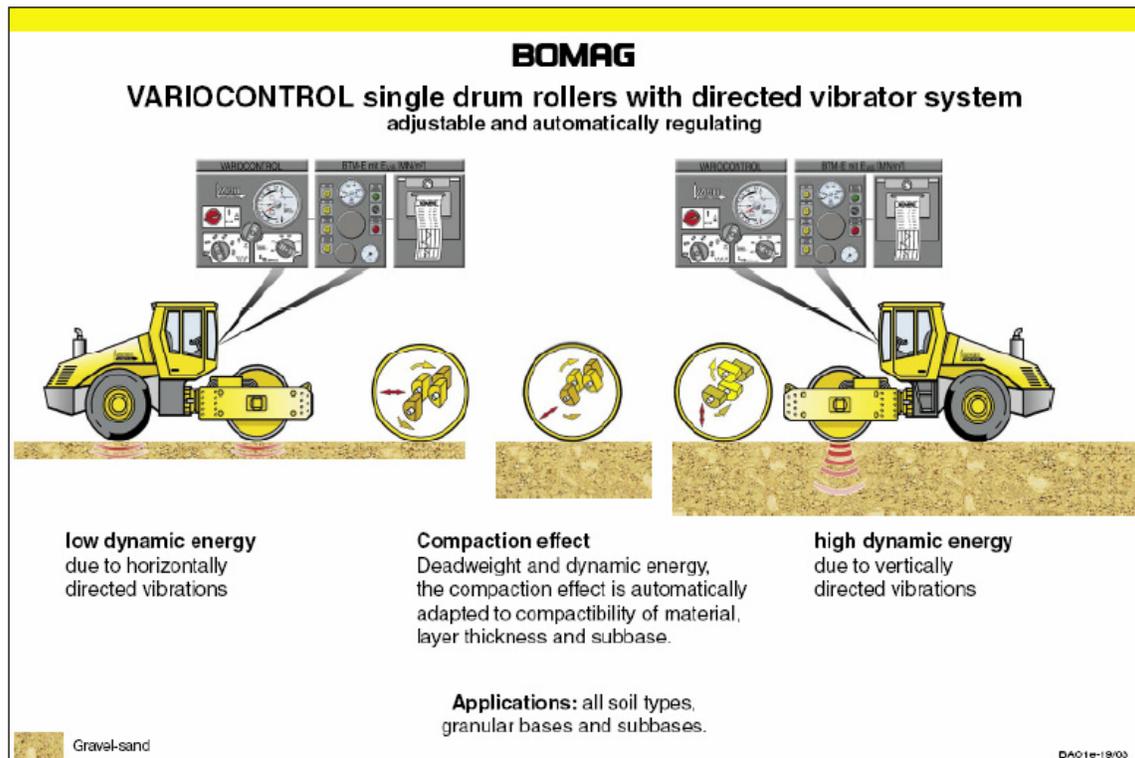


Figure 4.5 ARIOCONTROL Single Drum Rollers (from BOMAG Brochure)

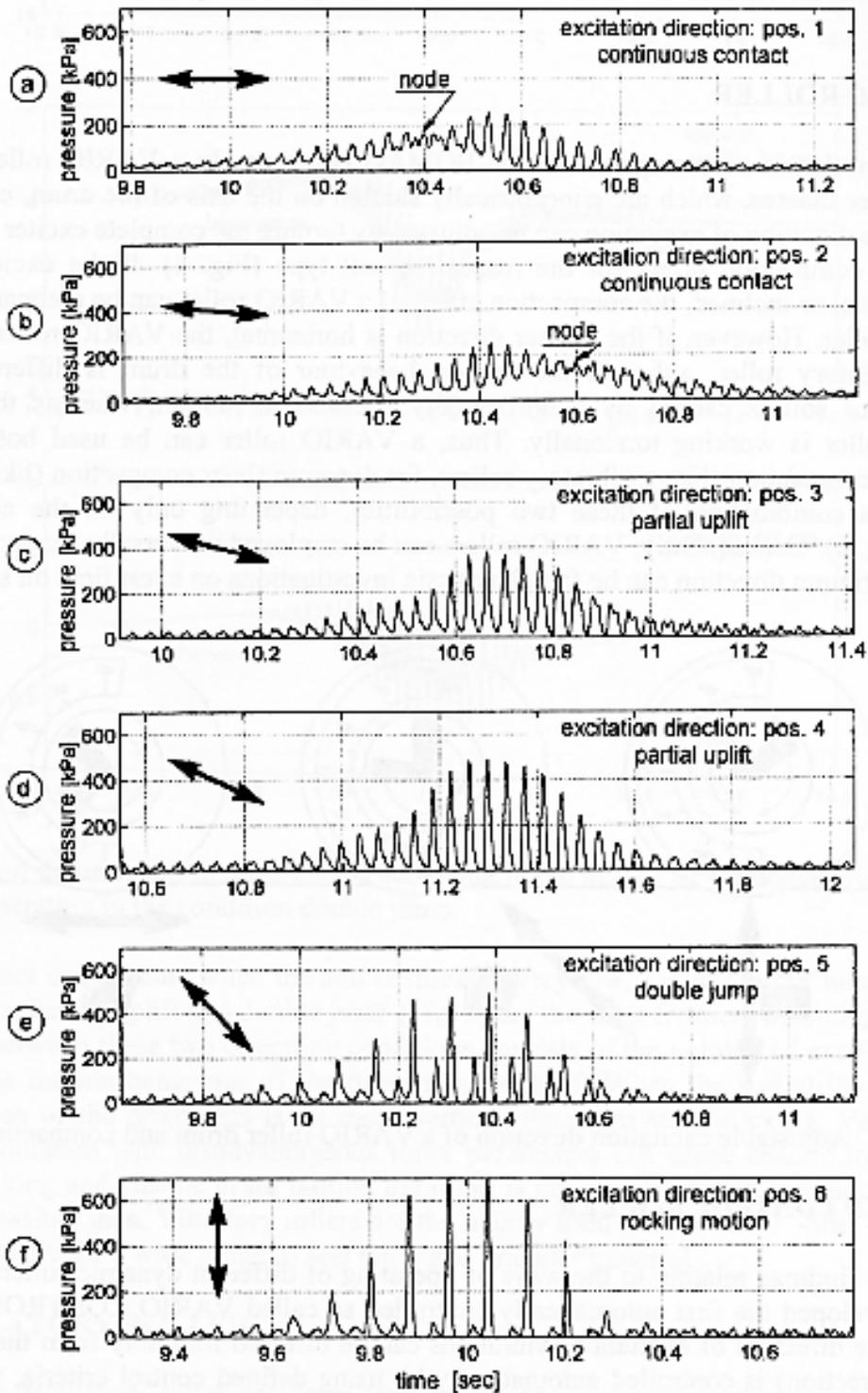


Figure 4.6 Soil pressure and operating condition depending on the VARIO exciter direction (Adams and Kopf, 2000).

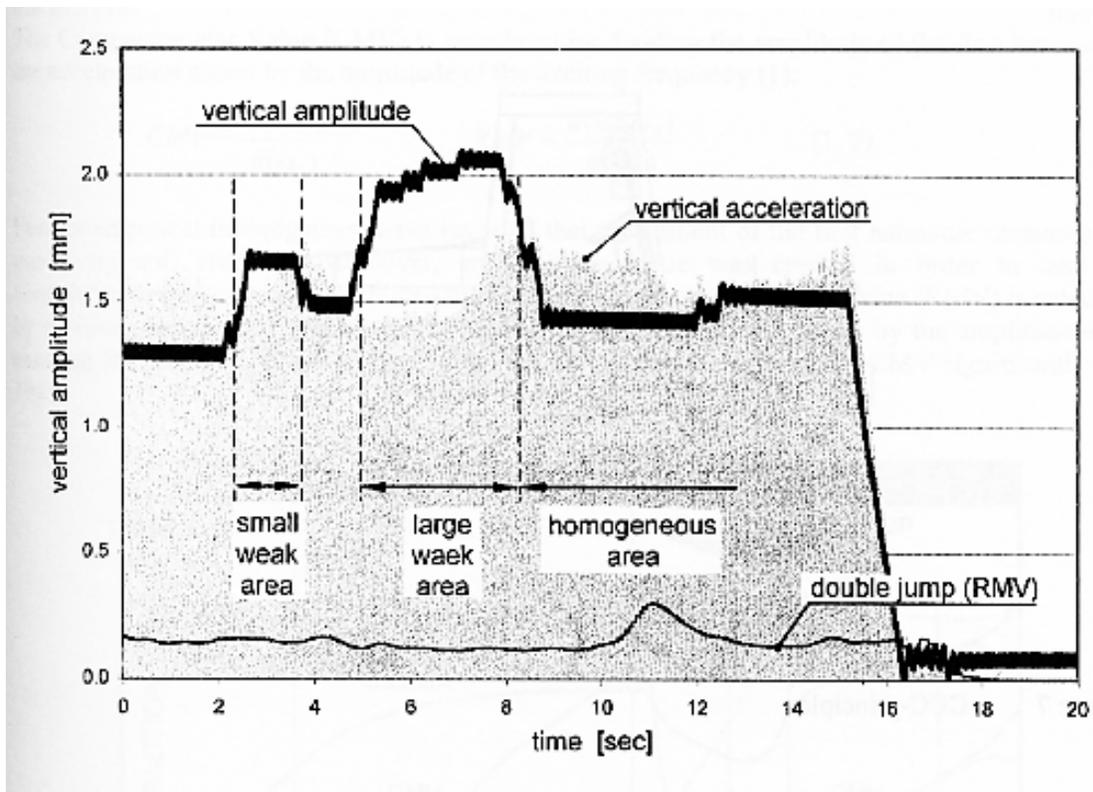


Figure 4.7 VARIO CONTROL automatic control of excitation direction (Adam and Kopf, 2000).

5. THEORY

5.1. Obtaining the stiffness k_B

A dynamic soil compactor produces nonlinear oscillations whose characteristic properties may be described analytically (Anderegg, 2000, Krober, Floss, Wallrah). Figures 5.1 and 5.2 show a theoretical, lumped parameter model of the interaction between a vibratory roller and the ground. The soil-drum-interaction-force (F_B) is defined as follows:

$$F_B \cong -m_d \ddot{x}_d + m_u r_u \Omega^2 \cdot \cos(\Omega t) + (m_f + m_d) \cdot g \quad (5.1)$$

where

m_d = mass of the drum (kg)

x_d = vertical displacement of drum (m)

\ddot{x}_d = acceleration of drum (m/s^2)

m_f = mass of the frame (kg)

m_u = unbalanced mass (kg)

r_u = radial distance at which m_u is attached (m)

$m_u r_u$ = static moment of the rotating shaft (kg.m)

$\Omega = 2\pi \cdot f$

t = time elapsed (sec)

g = acceleration due to gravity (m/sec^2)

f = frequency of the rotating shaft (Hz)

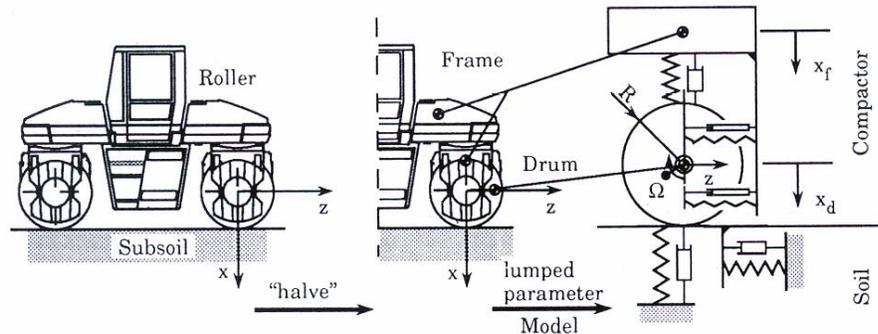


Figure 5.1 Theoretical, Lumped Parameter Model of the Interaction between a Vibratory Roller and the Ground (Anderegg, 2000)

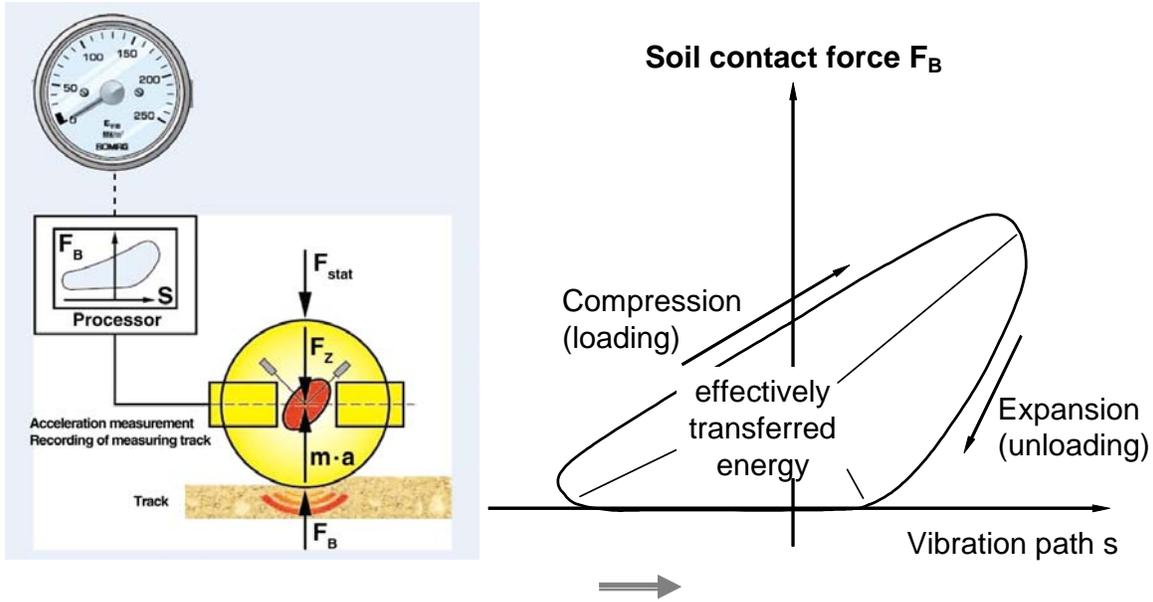


Figure 5.2 Forces on the drum and associated loading loop (from Kloubert's presentation at TRB 2004, BOMAG).

In Equation (5.1), the force acting in the positive direction (downward) has a plus sign. The inertial force $m_d \ddot{x}_d$ should always be directed negatively with respect to the corresponding coordinate because it is an inertia force. If the subsoil is described as a spring and dashpot system, the equation for the soil-drum interaction force is also given by:

$$F_B \cong k_B x_d + d_B \cdot \dot{x}_d \quad (5.2)$$

where

k_B = stiffness of soil (kN/m)

d_B = damping coefficient (kN.s/m) (a damping ratio of 0.2 is usually assumed, Anderegg, 2000)

\dot{x}_d = velocity of drum (m/s)

The acceleration of the drum and the phase angle between excitation and oscillation can be measured. With this information, it is possible to calculate F_B by using equation (5.1). Indeed, all quantities are known on the right hand side of the equation. If we set equation (5.1) equal to equation (5.2), the soil stiffness k_B can be obtained since all

other parameters are known except the damping coefficient. This parameter is usually assumed by using a damping ratio equal to 20%.

Alternatively the force settlement curve can be plotted and the slope of the curve on the loading portion can be calculated as the dynamic stiffness of the material being compacted. Figure 5.3 shows such a diagram.

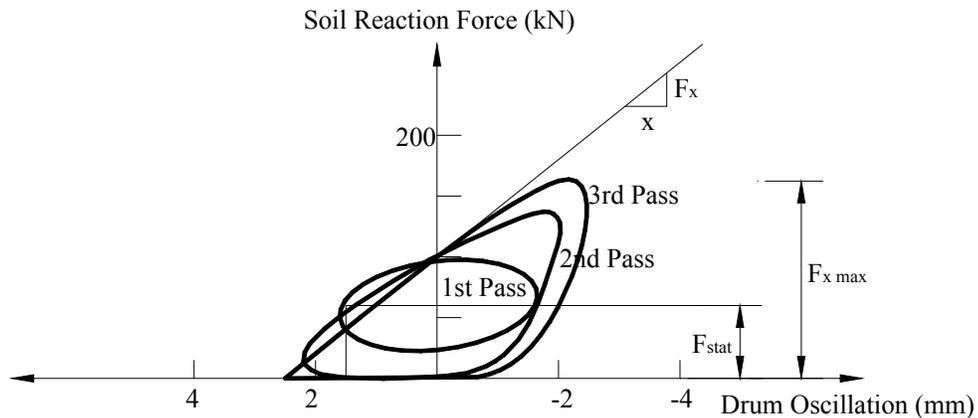
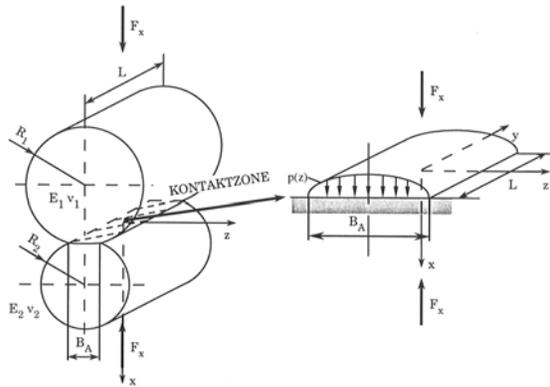


Figure 5.3 Soil Reactions vs. Amplitudes (After Floss and Kloubert, 2000)

5.2. Obtaining the modulus E

The soil stiffness k_B is not an independent soil parameter; indeed the stiffness is the ratio of the load divided by the settlement and is dependent on the area over which the load is applied. The soil modulus E is a true independent soil parameter and it is necessary to obtain the soil modulus E from the measured stiffness k_B .

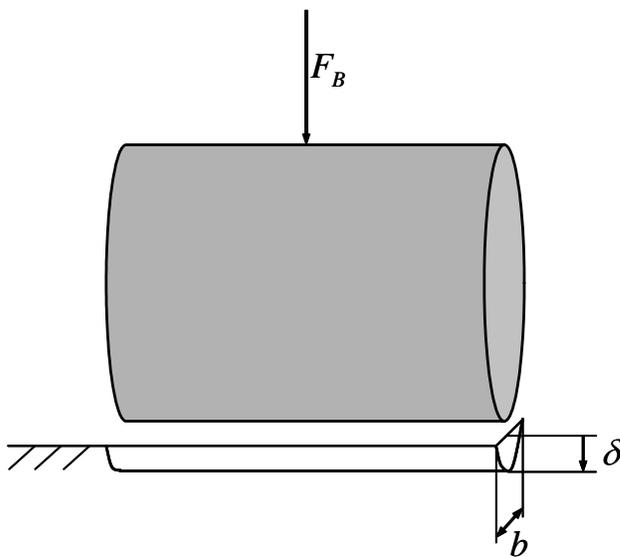
This problem was solved by Hertz in 1895 and further developed by Lundberg in 1939 (Figure 5.4 and 5.5). Hertz and Lundberg gave the relationship between the load on a roller and the imprint area created by the roller on an elastic half space. The solution can be used to find the relationship between the stiffness k_B and the modulus E of the material below the roller.



**Contact Problem
- Lundberg (1939)**

- E : Young's modulus [MN/m²]
- L : Drum width [m]
- R : Radius of the drum [m]
- ν : Poisson ratio [-] ($\nu=0.35$)

Figure 5.4 Drum on elastic soil problem (Lundberg, 1939, from Anderegg presentation at TRB 2004, AMMANN))



H. Hertz, 1895 :

$$b = \sqrt{\frac{16}{\pi} \cdot \frac{R(1-\nu^2)}{E} \cdot \frac{F_B}{l}}$$

G. Lundberg, 1939 :

$$\delta = \frac{1-\nu^2}{E} \cdot \frac{F_B}{l} \cdot \frac{2}{\pi} \cdot (1,8864 + \ln \frac{l}{b})$$

$$\delta = f(F_B, E)$$

Figure 5.5 Hertz (1895) and Lundberg (1939) solutions (from Kloubert presentation at TRB 2004, BOMAG)

$$k_B = \frac{E \cdot L \cdot \pi}{2 \cdot (1 - \nu^2) \cdot \left(2.14 + \frac{1}{2} \cdot \ln \left[\frac{\pi \cdot L^3 \cdot E}{(1 - \nu^2) \cdot 16 \cdot (m_f + m_d) \cdot R \cdot g} \right] \right)} \quad [\text{MN/m}] \quad (5.3)$$

Where L is the drum width, ν is Poisson's ratio, \ln is natural logarithm, m_f and m_d are the masses contributed by the frame and the drum of the roller, R is the radius of the drum, and g is the acceleration due to gravity. Knowing k_B , Equation 5.3 gives E .

The relationship between the stiffness k_B and the modulus E can also be established on an experimental basis by performing the roller tests and plate tests in parallel. Ammann reports on a study from ETH Zurich in Switzerland which was conducted to establish such a relationship. Figure 5.6 shows a relationship between the stiffness k_B obtained from the roller and the moduli M_{E1} and M_{E2} from the first load and reload of the plate test. The figure shows a reasonable relationship with some scatter.

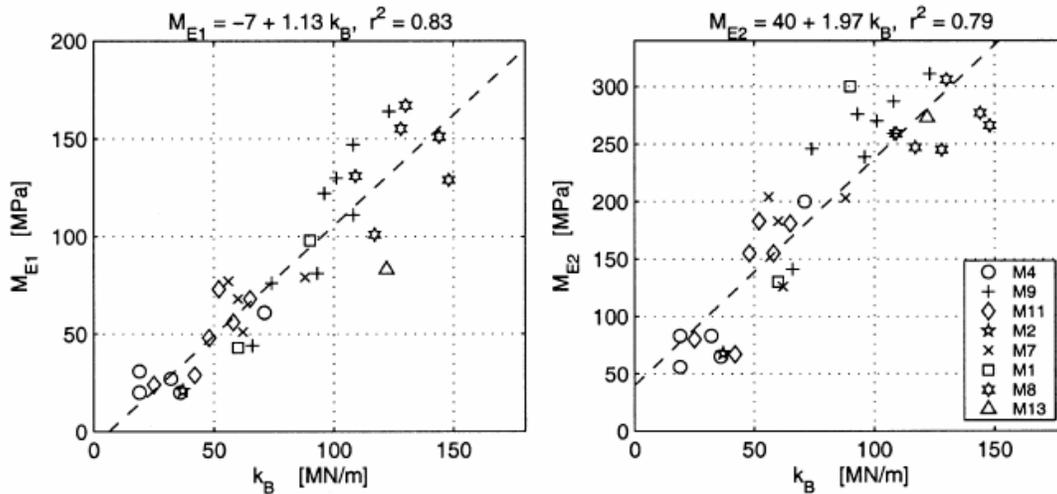


Figure 5.6 Stiffness vs. Modulus (Preisig, Caprez, and Amann, 2003)

The relationship between the measured stiffness k and the elasticity modulus E of the soil can also be obtained by using the results of measurements with the portancemetre (Quibel et al., 1998). The portancemetre (Figure 5.7) is a narrow vibrating roller used to measure the stiffness of the soil below it. It is not a compactor but a testing device which is rolled on the ground while vibrating like a roller. The output is the soil stiffness which

has been calibrated to a modulus by parallel testing with a plate test. Quibel found that the relationship is of the form:

$$E(MN / m^2) = \alpha \cdot K(MN / m) \quad (5.4)$$

Quibel et al. (1998) found $\alpha = 8.0$ for the portancemeter (Figure 5.7) and $\alpha = 2.2$ for a larger roller (1.6 m drum width). The coefficient α is different for each roller geometry and weight (Anderegg, 2000).



Figure 5.7 The Portancemeter (Quibel, CETE, France)

5.3. Drum behavior

An example of the forces involved during the drum vibration is shown on Figure 5.8 while the compaction energies are on Figure 5.9. To operate the roller in a state of temporary loss of soil contact is more or less ideal for soil compaction. The continuous contact with the drum is ideal for asphalt compaction. A vibratory roller has three major parameters: the amplitude, the frequency, and the roller velocity (Figure 5.10). A fourth parameter, the dynamic interaction force between the drum and the soil or asphalt can be extracted from the amplitude and the frequency measurements.

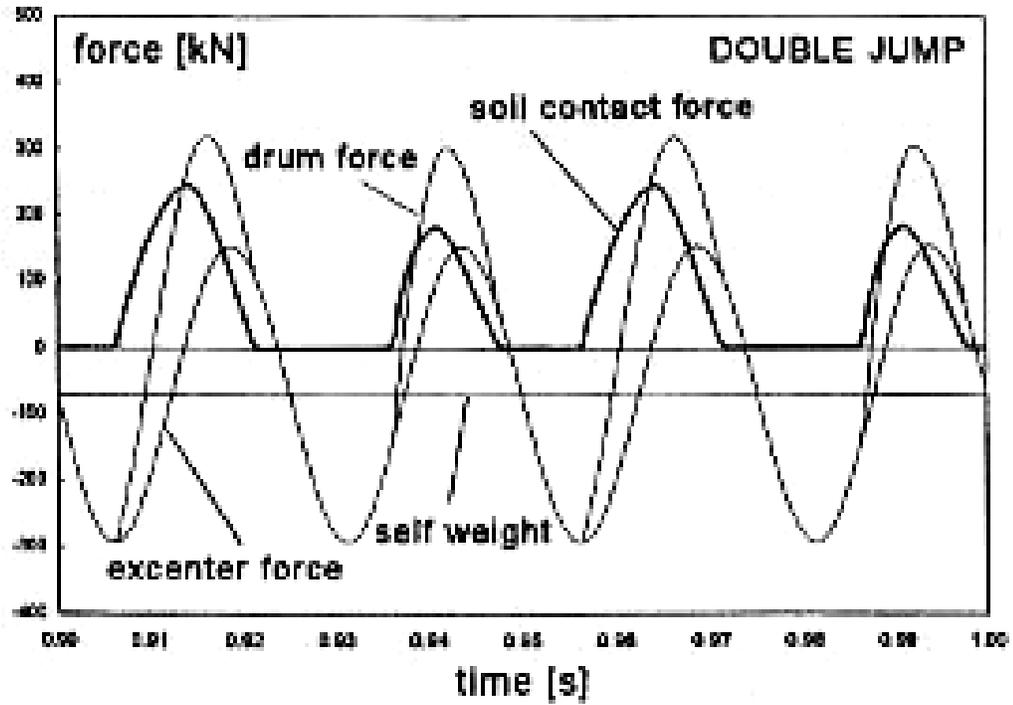


Figure 5.8 Calculated Soil Reactions (Adam and Kopf, 2000)

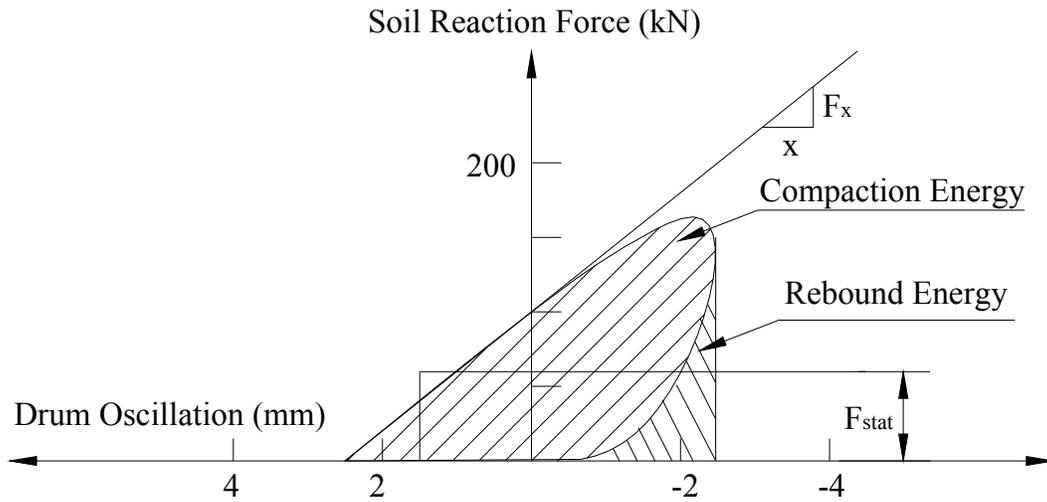


Figure 5.9 Compaction Energy Concept

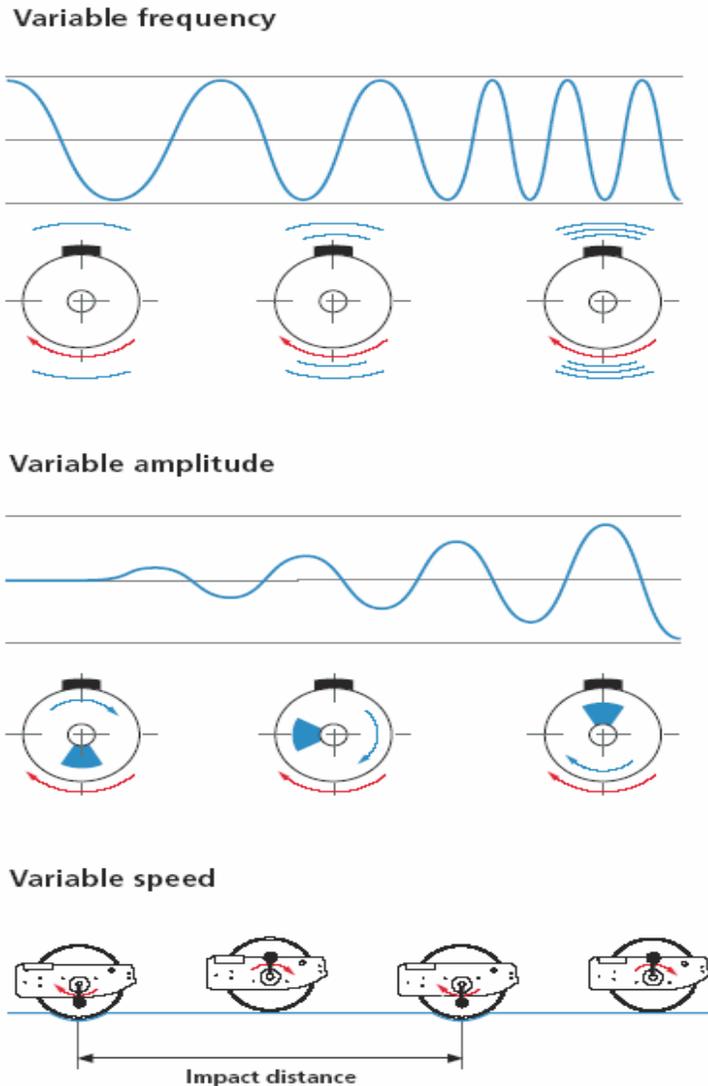


Figure 5.10 Control Parameters for the Automatic Compaction (from AMMANN Brochure)

The rotating eccentric shaft has two parts, each part carrying one half of the eccentric weight. Both concentrically arranged shafts rotate in the same direction, the angle between both parts can be varied by a gear box. This system makes it possible to change the resulting vibration amplitude from 0 to a maximum while the roller is compacting.

The frequency of the vibratory shaft is automatically chosen by the system close to the soil-drum-resonance frequency and the acceleration is recorded. The roller speed guarantees a constant impact space. The space between two impacts of the vibratory

drum has to be about 20 and 40 millimeters. The roller is programmed to update its behavior according to the vibratory path shown in Figure 5.11.

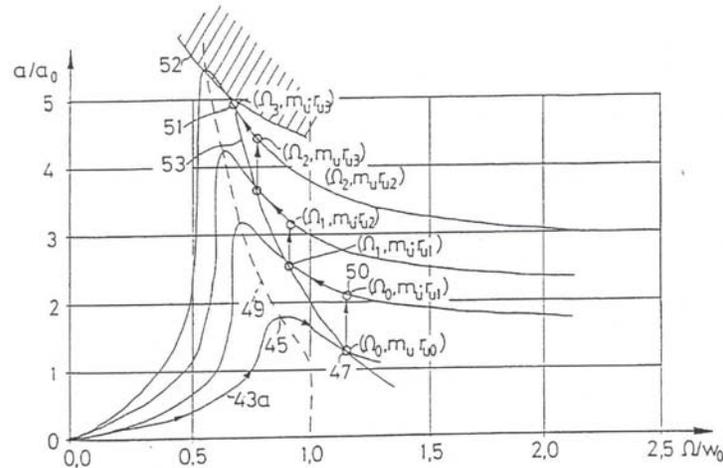


Figure 5.11 Start-up Procedure of Roller (AMMANN, 2002)

There are three different vibration status of a roller (Figure 5.12);

- a) If the drum surface always maintains contact with the ground, the rotational frequency f of the surface is determined with the Fourier analysis. This compacting process is called load operation.
- b) If the surface periodically lifts off the ground, which is more effective compacting, the Fourier analysis is used to determine harmonic oscillation with drastically decreasing maximum amplitude. The lift-off of the surface from the soil is characteristic of the optimal mode of operation because in this case the forces transferred upon the soil are more effective than in case a), which results in more effective compacting.
- c) If the machine shows signs of jumping, which means the machine chassis is beginning to exhibit vibrations around its steady position, the upper harmonic waves are joined by oscillations with half the exciting radian frequency Ω of the unbalance mass. This condition is not stable, and may potentially loosen the soil. Moreover, the machine chassis may begin to vibrate around its longitudinal axis.

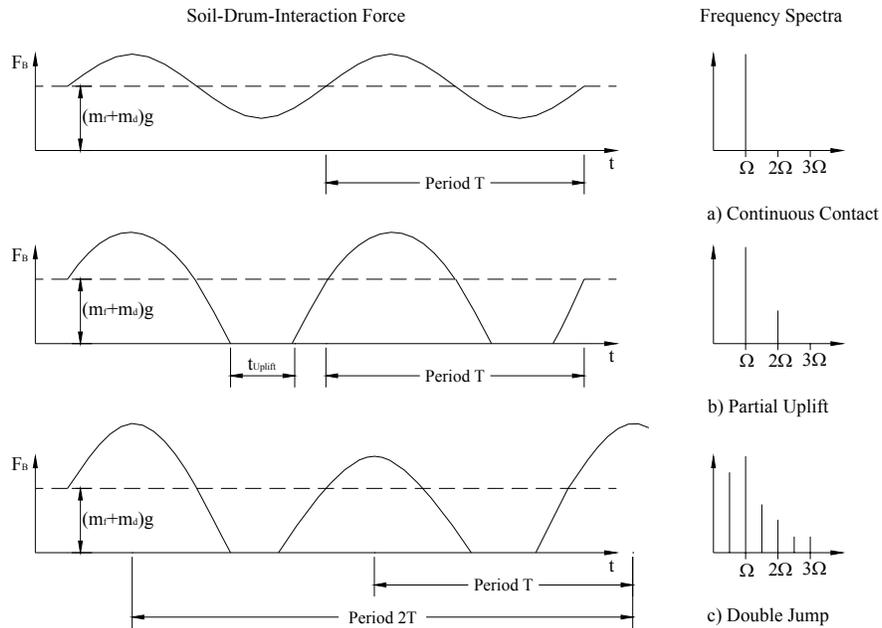


Figure 5.12 Three Elementary Vibration States of a Roller (After Anderegg, 2000)

The speed variable depends on the type of layer that is to be compacted. Due to a low rotation radian frequency Ω , a non-consolidated layer requires a slower travel speed v than a consolidated layer. For example, for a non-consolidated layer the travel speed is $V_u=3$ km/h with a rotation frequency of $f_u=30$ Hz, and for a consolidated layer, the travel speed is $V_g=3$ km/h with a rotation frequency of $f_g=45$ Hz.

6. EQUIPMENT

Rollers vary in weight from 5 tons to 25 tons. They operate at frequencies around 30 Hz. The contact force can reach 40 tons and the displacement under the roller varies from 30 mm (first pass) down to a fraction of a millimeter (last pass). The modulus measured after compaction can range from 30 MPa to 200 MPa. The following gives examples of equipment for intelligent compaction sold by BOMAG, AMMANN, and Geodynamik.

6.1. BOMAG (Germany)

BOMAG VARIOCONTROL (BVC) is a piece of equipment placed on a single drum roller to interact with the roller and automatically optimize compaction. Several rollers equipped with this equipment can be used depending on the size of the project (BW 177 DH-3 BVC, BW 213 DH-3 BVC, and BW 255 D-3 BVC). The direction of vibration is adjustable from horizontal to vertical. The BVC allows the use of much larger amplitudes than is possible with conventional vibration systems. The vertical direction vibration gives the maximum energy available from the roller and the horizontal direction gives the minimum energy from the roller. This flexibility improves compaction close to the surface and reduces loosening.

BTM-E is a modified version of the BOMAG Terrameter, a computerized display of many important compaction parameters with display of dynamic modulus E_{vib} in MN/m^2 (Figure 6.1, 6.2). The operator can pre-select 6 minimum values ($E_{vib} = 45, 80, 100, 120, 150 MN/m^2$ and maximum). During the compaction, the dynamic modulus and speed are continuously measured, recorded and displayed. When the pre-set minimum value is reached or at maximum compaction the VARIOCONTROL system reduces compaction forces and a green light on the E_{vib} display indicates the end of compaction. For compaction control, the BTM-E system is provided with a printer for instant printout of all data from one rolling track. The intelligent system is shown in Figure 6.3. For hot mix, a tandem roller and VARIOMATIC system is used.

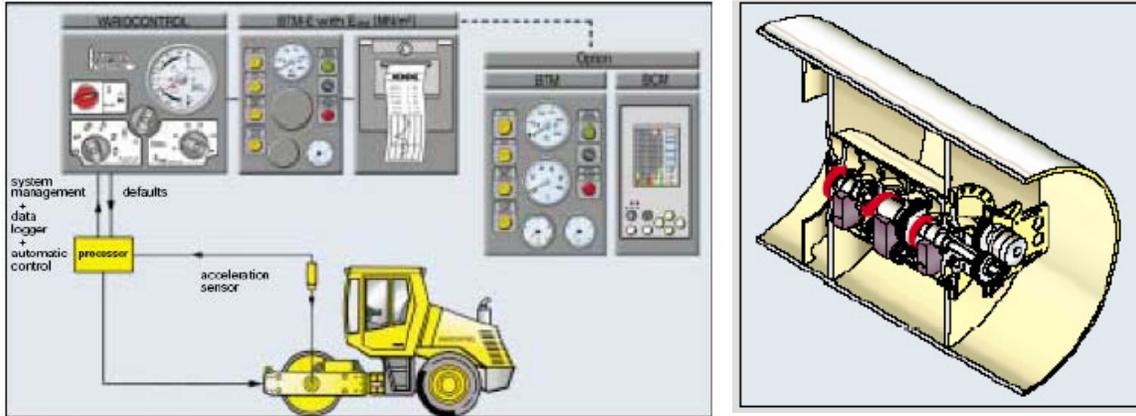


Figure 6.1 BTM-E and BVC (from BOMAG Brochure)

Principle of EVIB measurement system - BTM-E

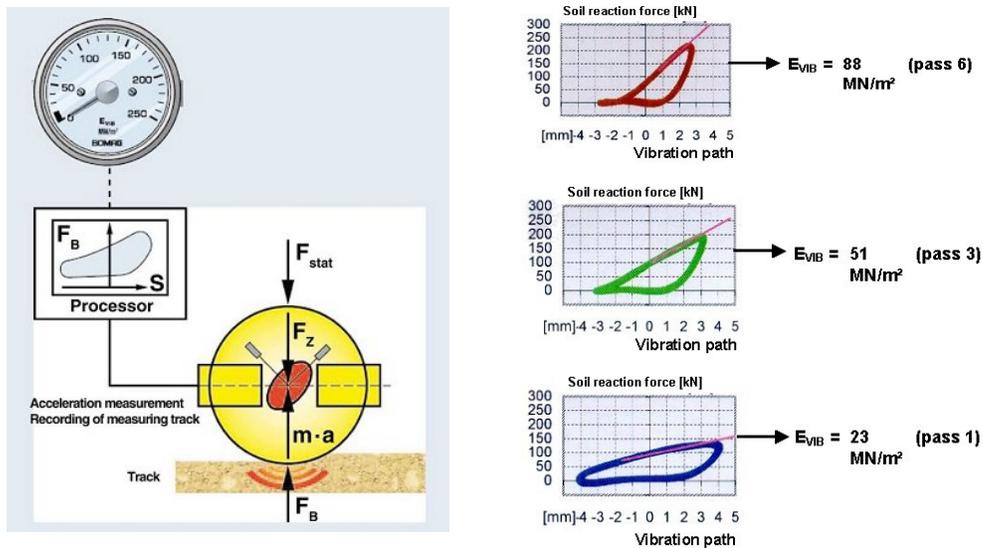


Figure 6.2 Principle of the Evib measurement system - BTM-E (from Kloubert's presentation at TRB-2004, BOMAG)

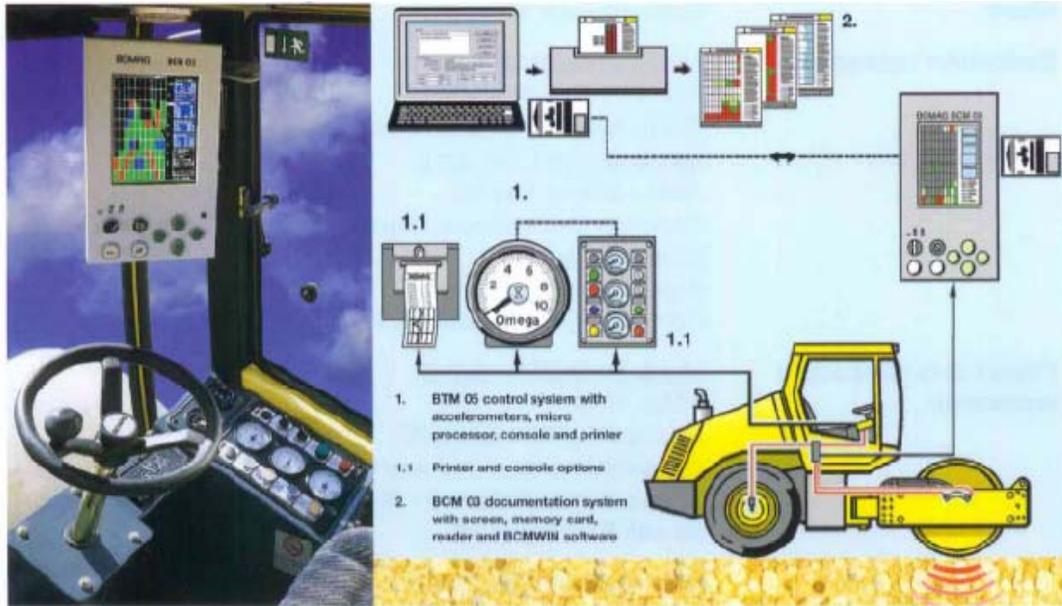


Figure 6.3 BOMAG Intelligent Compaction System (from BOMAG Brochure)

6.2. AMMANN (Switzerland)

AMMANN Compaction Expert (ACE) is also a piece of equipment which is incorporated in the roller. ACE displays the soil stiffness. The vibratory roller compacts the ground until a certain value of the soil stiffness is reached. After that value is reached, the roller continues to measure the stiffness without compacting the soil. The soil data and its attributed location on the job site are stored in the Continuous Compaction Control (CCC) computerized system (Figure 6.4 and Figure 6.5).

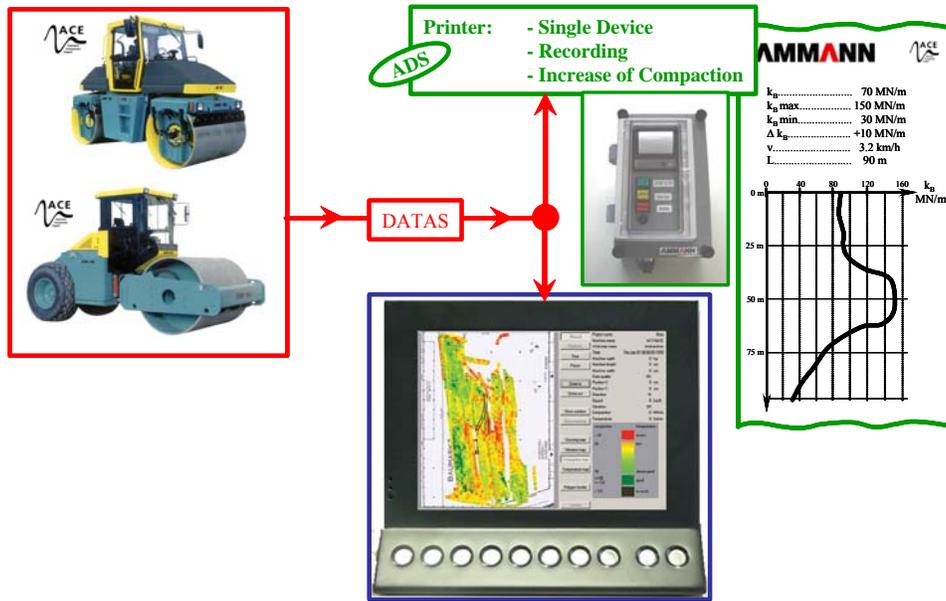


Figure 6.4 Continuous Compaction Control, CCC-Concept (AMMANN, 2003)

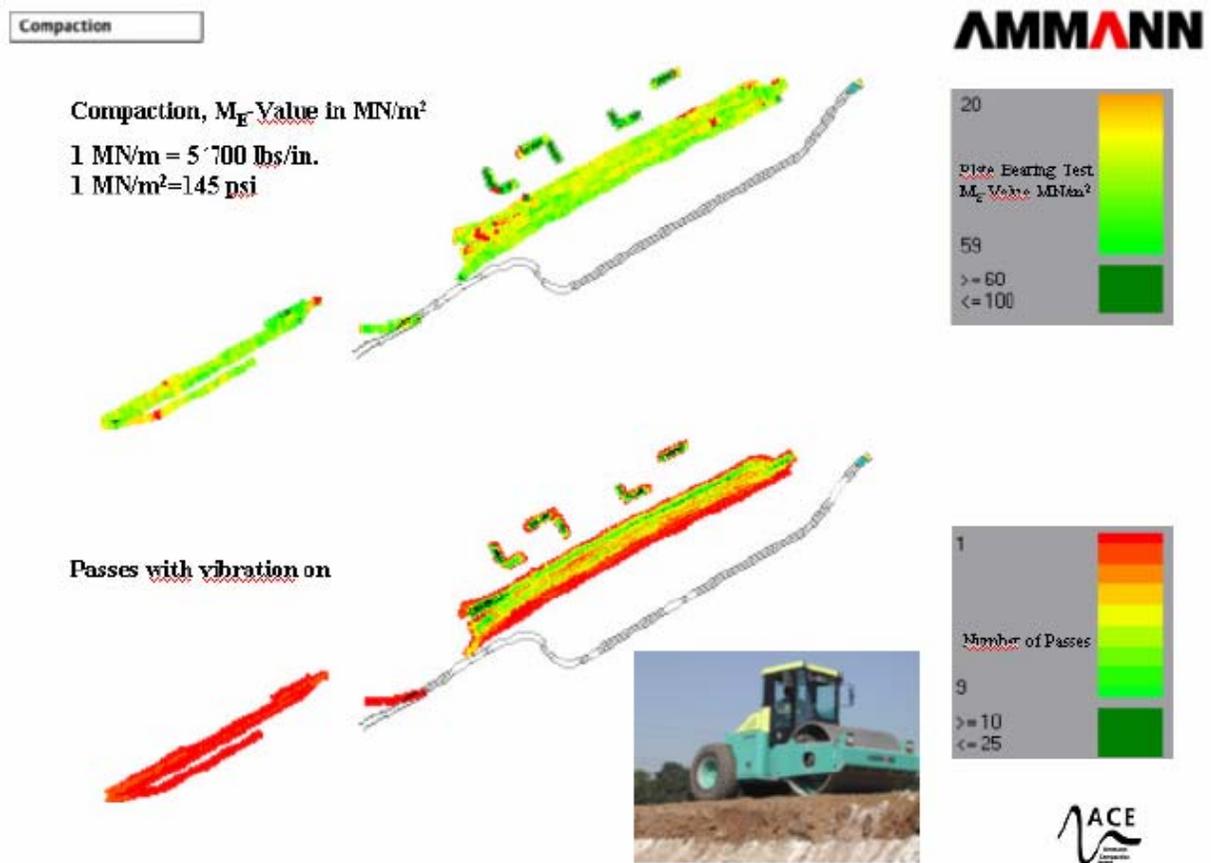


Figure 6.5 CCC Result (AMMANN, 2003)

6.3. GEODYNAMIK (Sweden)

Geodynamik manufactures the Compactometer for use in Continuous Compaction Control (Figure 6.6). A sensor mounted on the drum bearing continuously measures accelerations when the vibrating drum is in operation. Signals from the sensor change as the ground becomes harder and more compacted. These signals are converted into values which indicate a relative measure of the bearing capacity of the ground (CMV).

The Compaction Documentation System (CDS-012J™) displays the compaction results (CMV) while the roller is actually at work. At the end of a complete pass over the entire area the CDS System displays the CMV for all recorded strips side by side (Figure 6.7). This guides compaction more effectively and reduces the number of roller passes to a minimum.

For hot mix, the Continuous Asphalt Compaction (CAC) and Asphalt Compaction Documentation System (ACD) are used. The value used in the ACD-system is analogous to the CMV in soil compaction.

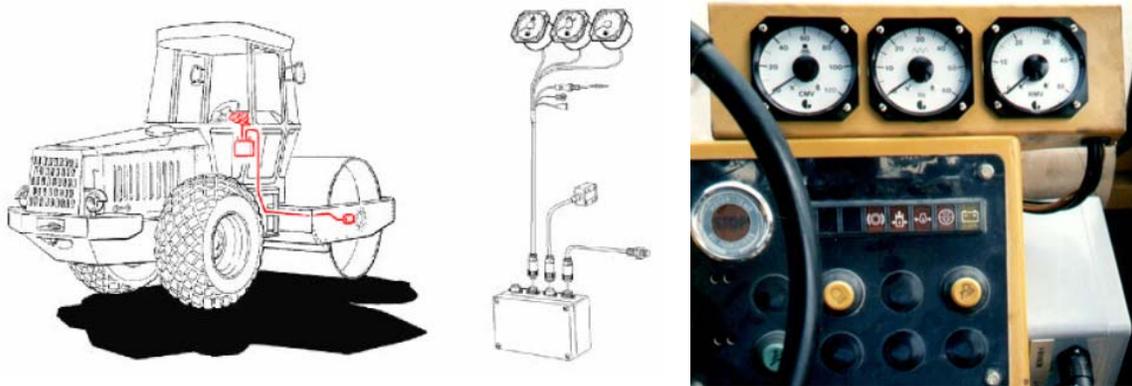


Figure 6.6 Compactometer (from Geodynamik Brochure)



Figure 6.7 Screen Information to the Roller Operator (Thurner and Sandstorm, 2000)

7. CASE HISTORIES

7.1. New Cologne-Rhine/Main Line (Germany-BOMAG)

Deutsche Bahn's new line from Cologne to Rhine/Main is a high speed railway situated at the heart of Europe; it forms a centerpiece of the developing European high speed rail network. Representing an estimated total investment of approximately 4.6 billion dollars, it ranks amongst the largest single European projects.

The 177 km long line (204 km including branches) is divided into three sections, north, middle and south with the middle section further divided into three sections, A, B and C. In all, 4 million m³ of soil had to be removed, 2 million m³ of embankments completed and 250,000 m³ of sub-base laid. The fill material includes extremely inhomogeneous claystone and argillite, which are classified as mixed grain stony soils with strong cohesive properties. The fine grained portion amounts to between 17 and 35 percent. The compaction requirement is for at least 97% Proctor density with an air void content of less than 12 percent.

Because of the difficult nature of the material and in order to achieve further compaction of the coarse grain structure, the material is laid in relatively thin layers (0.23-0.30 m) followed by two passes of BW 219 PDH-3 padfoot rollers. After adding water, the main compaction is carried out with 4-6 passes using BW 219 DH-3 smooth vibrator rollers. The BTM (BOMAG compaction measurement) and BCM (BOMAG documentation systems) provided valuable assistance for quality assurance. Using the measurement values, a weak-point analysis could be conducted. A check was carried out on the compaction requirements using conventional test methods.

7.2. Logistics Center (Germany-BOMAG)

The new logistics centre houses one of the major German food retailers in Butzbach, Hessen. It is located near the A5 Frankfurt – Gießen highway and in the vicinity of the Gambach motorway intersection.

A total area of 102,000 m² needed soil improvement by in-place stabilization of a 0.40 m thick layer to improve the existing loess and clay soils. Specifically, spreading, grading, and compacting in accordance with the ZTVE-StB94 specifications was

required. According to the specifications, compaction had to be performed in 6 overlapping passes. After completing the soil improvement, the contractor had to document compaction of 97% of standard Proctor density and also provide evidence of a deformation modulus E_{v2} of 60 MN/m². Last but not least, the elevation of the surface had to be achieved within a tolerance of ± 40 mm.

After removing the top soil the area had to be leveled. On the sloping cross section, the excess upper layer was removed and used at the bottom to create a level surface. In lower sections, the material was filled and compacted for up to two meters. To protect living areas, 25,000 m³ of soil was placed as a noise protection barrier.

To reduce the compaction costs of approximately 80,000 m³ of soil, Weimer decided to use larger rollers. A self optimizing compaction system was specified for the heavier rollers to provide the maximum possible compaction force; over-compaction and under-compaction had to be eliminated. Since the company had already successfully used BOMAG BW 213 D-3 or BW 213 DH-3 BVC with attached hydraulic vibratory plates, it selected the 25 t roller BW 225 D-3 BVC (VARIOCONTROL). High compaction values were already achieved with BOMAG single drum rollers on test areas: The Proctor densities were generally above 98% against a specified minimum of 97%.

In some areas with high ground water levels smaller single drum rollers were used: two BW 213 DH-3, one with "VARIOCONTROL" and attached hydraulic vibratory plates and one normal version. For the "normal" areas with high lift heights the 25-t single drum roller was the right choice; the noise protection wall of 25,000 m³ was compacted with the BW 225 D-3 BVC. Lift heights of 0.80 m require concentrated centrifugal force of up to 402 kN. For the level application of soil, the lift heights are only 0.60 m thick when dumped and are compacted down to approx. 0.50 m with only four passes, in compliance with the specified Proctor values. Despite the enormous force provided by the BW 225 D-3 BVC, the single drum roller still uses intelligent compaction. During each vibratory movement of the drum the E_{VIB} -value, which is directly correlated with the deformation modulus E_V from the load plate test, is determined in MN/m². In the E_{VIB} -display the driver can follow the increase in compaction. This can be recorded and printed out to document the status of compaction. Any over-compaction is avoided, passes are saved – costs are cut.

With the use of BOMAG single drum rollers including VARIOCONTROL Weimer claims to have saved two passes on each area. Specified compaction was achieved after only four instead of six passes; this is a reduction of more than 30 % - also in labor time and fuel costs.

7.3. Shuibuya Project (China-BOMAG)

The shuibuya project is near Yichang, China. It is located in the upper Qingjiang River Basin. Two dams exist near the Geheyan Hydropower Station; the lower part of the dam is an arch dam and the upper part is a gravity dam. The Shuibuya Concrete Face Rockfill dam has a height of 233 m, and is the highest CFRD in the world at present. The Station adopts the diversion-type underground powerhouse. In January 2000, the State approved the project proposal and construction started in 2002.

Two different rollers were used to compact the well graded fill soil. The layer thickness was 0.80 m and the rolling speed was about 2 km/h. The results are shown in Table 7.. The BW 225 D-3 which has a BVC for intelligent compaction was more effective than the BW 219 DH-3.

Table 7.1 Compaction Result of Shuibuya Project, China

Type of roller	Output m ³ /d (10 h/d) Efficiency factor 0.8			Number of rollers necessary for 15,000 – 20,000 m ³ Daily Production Rate
	4 passes	6 passes	8 passes	
BW 219 DH-3	-	4000-5000	3000-4000	4 – 6
BW 225 D-3 BVC	5500-7000	5000-6000	-	3 – 4

7.4. Raleigh Application (NC, US-AMMANN)

Ammann America, Inc. introduced the first AV 95 Tandem Vibratory Asphalt roller with the ACE – System in the United States in 2001. The rollers are in the 10-ton class and cover a working width from 1.5 to 2.9 m. The machines used were Tandem Combination Rollers which means that they are equipped with a steel drum in the front and a rubber tired axle in the rear.

The core of ACE is the regulation system that uses the measurements to adjust the compaction parameters frequency, amplitude and impact distance to meet the requirements of the permanently changing ground conditions (Intelligent Compaction). The system guarantees optimization of the process to achieve the targeted compaction density without any risk of over-compaction. In cooperation with the regional distributor for Ammann's Compaction Equipment, Park East Sales LLC, the ACE units were taken on different job sites for demonstration and testing. One test-project was a road widening job and the second project was a Superpave operation where the North Carolina Department of Transportation (NCDOT) was performing independent tests to check the performance of the Ammann rollers. The results taken by the DOT in the form of core tests proved that ACE achieved the required density with only one overpass. Because the AV 95 roller were combination models, the job could be done without using a so called Intermediate Roller which is a Rubber tired static roller that is responsible for soft-spots and the finishing job for the Asphalt layer.

7.5. Deponie Mitte Unterfrauenhaid in Burgenland (Adam and Kopf, 2000)

This waste disposal facility in Unterfrauenhaid was extended between 1998 and 1999. It is situated in the area of a former sand pit. The exploited pit was refilled with excavation material but not compacted. Soil exploration has revealed that the artificial underground was not suited as base for a waste disposal facility. Thus, the material was excavated and placed in 0.30 m thick layers again. The demand for a work integrated and continuous control required the application of CCC. Furthermore, CCC was used for acceptance testing of mineral base liners. Conventional control methods like permeability testing and the use of the TROXLER-nuclear gauge for density could be minimized. Thus, time could be saved and cost reduced significantly.

7.6. Schutzdämme Schwaz-Eiblschrofen in Tyrol (Adam and Kopf, 2000)

The rock fall disaster on the Eiblschrofen near Schwaz in July 1999 brought a village in danger of rock impacts. Thus, it was decided to construct rock-fill embankments in order to protect the village. The embankment had to be situated in areas

of major danger, so that construction works could be carried out only after taking special precautions.

It was also necessary to construct the dams as fast as possible while ensuring the maximum quality. The quality management was based on the roller-integrated continuous compaction control with the Terrameter. The calibration of the CCC-value OMEGA and the layer thickness were determined on a test site outside the danger zone. The quality control was performed while compacting. The criteria for sufficient compaction were:

- minimum value for small and large amplitude (indicator of material quality and water content)
- the increase of measured data between two roller passes (indicator of usefulness of further passes)

If the measured value exceeded the required minimum value and if the increase of the OMEGA value was less than 5% away from the mean value of the last pass on this lane, compaction work could be terminated on this lane.

7.7. Kraftwerk Donaustadt in Vienna (Adam and Kopf, 2000)

In the Donaustadt power plant in Vienna, a new power station block had to be constructed. The extension was situated in an area with unsuitable ground properties. The over-consolidated tertiary soil, located at a depth of about 12 to 14 m below the surface is overlain by loose quaternary gravel. The upper layers 3 to 4 m below the surface consist of loose sands and fill. Because of the high structural loads, the irregular load distribution, and the existence of dynamically loaded machine foundations, highly uniform soil properties were required. Thus, the following steps were taken:

- Removing of sands and artificial fill down to 4 m below the surface
- Application of vibroflotation soil compaction for 4 to 13 m from excavation level to tertiary soil
- Soil exchange from excavation level up to base of slab. Sandy gravels from Vienna subway excavation pits served as fill material.

Quality control of the compaction process for the fill material was based on CCC with a Terrameter. The CCC-result became more and more uniform with increasing material thickness.

7.8. Ammann's Projects in the USA

The Intelligent Compaction Projects performed by Ammann in the USA are shown in figure



Figure 7.1 Ammann's IC Projects in the USA

8. RESEARCH NEEDS

8.1. Modulus or Density or Water Content?

The answer to that question is not simple for the following reason. It is possible to have a high modulus without having particles which are close together. A high modulus may exist if a very soft clay dries out. The particles may be relatively far apart (un-compacted structure) yet the clay may be quite strong because the suction which develops between the particles upon drying generates high compression stresses between the particles. This apparent stiffness is destroyed as soon as the clay gets wet again. This is why it is not possible to control compaction on the basis of modulus measurements alone. The density on the other hand gives the compactness of the soil because the density is directly related to how many particles are within a given volume. However, there is no solid evidence to show that the soil density is directly correlated to the soil modulus. Also, one can obtain the same density for at least two different water content (either side of the Proctor compaction curve). This is why it is not possible to control compaction on the basis of the dry density alone. There is a need to carefully evaluate the process of going to a modulus based compaction control independently of any device used to measure the modulus. More specifically **there is a need to better understand the interdependence between the modulus and the water content, and the shape of the modulus versus water content curve for the various types of soils used in pavements and embankments.**

8.2. Target Values

In the case of density based compaction control, the target to be achieved and to be verified in the field is clear. An example would be a target of 95% of modified Proctor maximum dry density with recommendations on the range of water content which are likely to lead to that dry density. These targets are based on a laboratory test performed on the same material ahead of time. Such practice has been in effect for decades.

If we move to a modulus based compaction control, there is a need to have a modulus approach parallel to the dry density approach. **There is a need to develop a simple laboratory test to obtain ahead of time the target modulus from a modulus versus water content curve; this target modulus would be verified in the field using**

the same test. In the field, compaction control would consist of measuring the modulus and ensuring that the water content is within tolerances (not too dry to avoid the beneficial but temporary effect of suction. Because moduli are influenced by many parameters (Appendix A) this approach will require that the same modulus be measured in the lab and in the field including same stress level, strain level, rate of loading, number of cycles, water content. This target modulus would be verified in the field using the same test. The modulus given by the roller would be correlated to this test modulus. On the other hand, the modulus used in design (resilient modulus for pavements) could be also correlated with the laboratory modulus. It is possible that for granular soils with less than say 10% fines it will not be necessary to measure the water content in the field as suction may not have an important effect on the modulus.

8.3. Homogeneity Comparison between Conventional Compaction and Intelligent Compaction

There is a need to simply demonstrate that intelligent compaction leads to better compaction than conventional compaction. This can be done at the research level or at the project level. Either way the comparison should document the modulus, density, and water content of the lift being compacted using the old and the new technology. If this takes place at the project level **some assessment of costs differences associated with the two techniques should be documented.**

8.4. Depth of lifts

The depth range of the compaction effect is governed by a number of parameters including the roller parameters and the soil parameters. The stress level must exceed a certain threshold value in order to cause a rearrangement of the grains. The compaction depth therefore is influenced by the size of the roller, the vibration frequency, speed of roller, the force level that it can generate. **There is a need to document the depth of compaction and the associated depth of lift that can be achieved by various rollers for various soils.** These findings will impact the specifications including the thickness of each lift.

8.5. Review of Specifications

Intelligent compaction or “Continuous Compaction Control, CCC,” is part of national specifications in Austria, Germany, Finland, and Sweden. These methods are based on roller integrated compaction meters that continuously measure the acceleration of the roller drum and calculate a compaction meter value from the acceleration signal. These specifications are different from country to country. **To develop Intelligent Compaction in the US, existing specifications need to be studied and standard specifications need to be drafted.**

9. PROPOSED APPROACH

The following is a possible global solution for modulus based compaction control and the associated use of intelligent compaction. It is given as an example of what could be done at this time with very little effort on a given project.

9.1. Test the Material in the Laboratory

In the laboratory, soil samples are prepared for a standard Proctor test. For each Proctor test, the usual density and water content measurement are made. With the soil still in the compaction mold, a modulus E_{BCD} is measured with a device called the Briaud Compaction Device or BCD (Figure 9.1). This laboratory modulus test is performed in parallel with each of the density and water content test. The results are a dry density versus water content curve and a modulus versus water content curve (Figure 9.2).

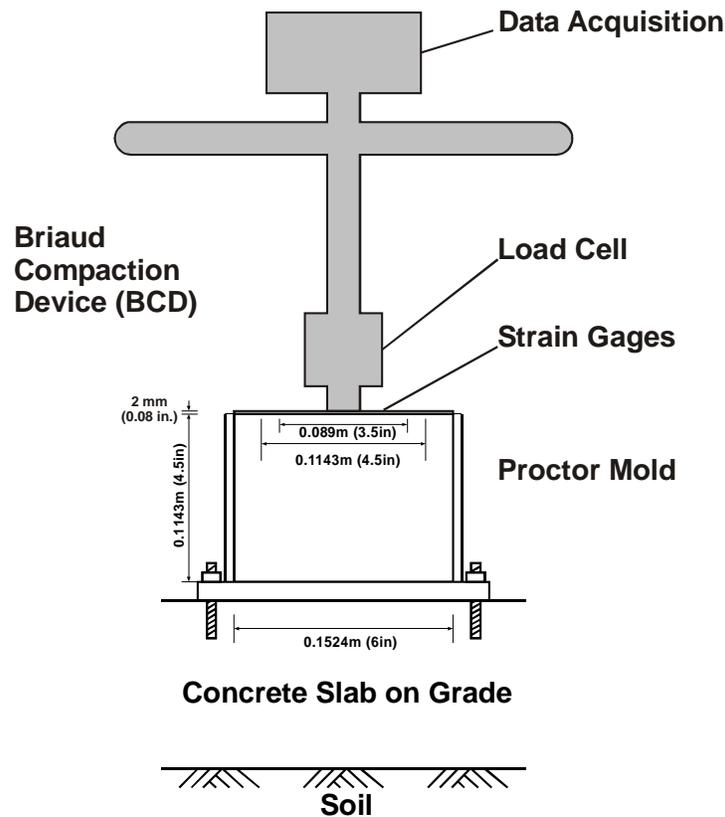


Figure 9.1 Modulus determination in the laboratory using the Briaud Compaction Device (BCD)

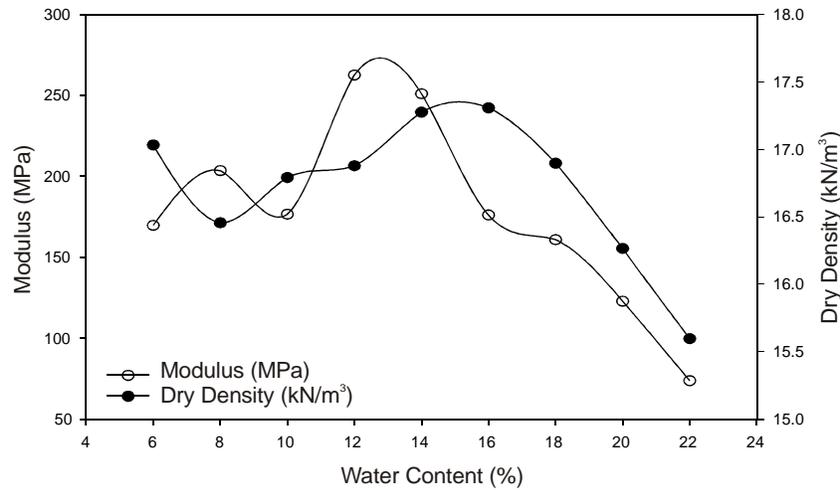


Figure 9.2 Density and modulus curves from the Standard Proctor test.

9.2. Establish Target Values and Specifications

The parallel curves of Figure 9.2 give the target maximum dry density, the target water content, and the target modulus. These target values are incorporated in the specifications. The target water content is established on the basis of the dry density. 95% of the modulus corresponding to that water content is used as minimum target value in the specifications.

9.3. Test the Material in the Field

Once the material has been tested in the laboratory and once the target values have been incorporated in the specifications, the field work can proceed. The intelligent compaction roller is used and gives the roller modulus E_{roller} . This modulus is checked against the same modulus E_{BCD} which was used in the laboratory, for example the Briaud Compaction Device (Figure 9.3). The advantage of using the same test in the laboratory and in the field is that the same modulus is obtained thereby making comparison possible. Now that a common modulus links the laboratory and the field it becomes possible to use the resilient modulus as a target in the field; indeed all that is needed is a correlation to the BCD modulus. In the same fashion, the correlation between the roller modulus and the BCD modulus can be done in the field and helps the contractor set the roller modulus target. In addition to the modulus, the density and the water content are still measured in the field.

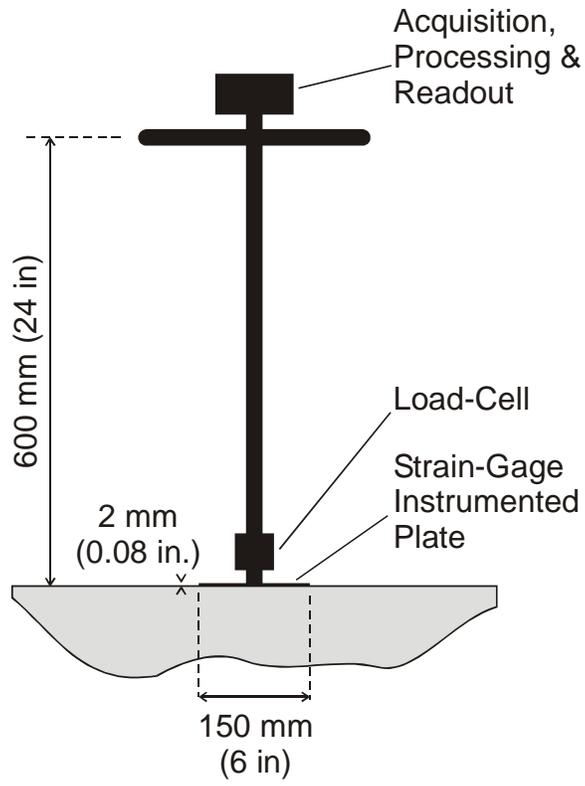


Figure 9.3 Modulus test with the BCD.

9.4. Long Term Approach

The approach proposed above is for the short term. Indeed in the long term it is likely that experience with testing will make it possible to bypass density requirements altogether. Also it is possible to imagine that it may not be necessary to measure the water content for soils that are not very sensitive to moisture such as free draining granular materials. It is also likely that soils with fines will continue to require the dual measurements of modulus and water content. This is the current practice in Germany.

10. CONCLUSIONS

10.1. What is intelligent compaction?

Intelligent Compaction is achieved by a smooth drum vibratory roller with a measurement/control system. This measurement system uses the information collected to adapt the equipment performance continuously, to optimize compaction and meet required conditions. This system controls the different compaction parameters for the roller such as: drum vibration, amplitude, frequency and working roller speed (impact distance). The output parameter is a soil modulus which is calculated continuously on the basis of the monitored drum acceleration.

10.2. Advantages and drawbacks of intelligent compaction.

Intelligent Compaction gives an instantaneous and complete evaluation of the zone being compacted. It helps to remediate weak spots and avoids over-compaction. It reduces the number of roller passes, the number of conventional proof tests, and provides a soil modulus at all locations where the roller has traveled. It gives a more uniformly compacted layer. The drawback is that the equipment is more expensive than ordinary rollers. Intelligent Compaction evolved in Europe starting in the late seventies.

10.3. Contract types accelerated the development of intelligent compaction.

Today, Intelligent Compaction (IC) is more popular in European countries than in the U.S.A. One of the reasons is the difference in the type of contracts which exists in these countries for construction projects. In Europe, best-value awards are widely used in all types of procurements and design-build is the contracting method of choice for many types of projects. Long warranty periods are often in effect. Specifications requiring certain soil moduli values are in use (45 MPa for low volume roads, 120 MPa for freeways).

10.4. Theory is well established.

The theory to obtain the modulus from the measurement of the drum acceleration is clear and well established. It relies on the equations of equilibrium and the solution of a

drum on an elastic half space. The modulus calculated from the combined use of the measurements and the theory corresponds to stress levels as low as 100 kPa (first pass of a light roller) to 5000 kPa (last pass of a heavy roller on a well compacted and well graded soil), strain levels in the range of 1 to 5%, times of loading between 10 and 50 milliseconds, and a very low number of cycles because the vibrations are such that the soil at a certain location is “hammered” once per pass. An airplane has tire pressures approximately equal to the pressure generated by a medium roller (2000 kPa), a heavy truck has tire pressures approximately equal to the pressure generated by a light roller (600 kPa) while a car has much smaller tire pressures (200 kPa).

10.5. Equipment is readily available.

BOMAG, AMMANN, and Geodynamik are equipment manufacturers which provide fully equipped rollers or attachments to place of existing rollers. These pieces of equipment have the ability to optimize compaction according to the intelligent compaction principles and give various displays of the parameters recorded. A number of case histories in the USA and in Europe have been documented.

10.6. Research needs exist.

There are a number of research needs which have to be addressed before intelligent compaction can reach its full potential in the USA.

- a) Demonstrate that intelligent compaction leads to better compaction than conventional compaction. Costs differences between the two techniques should be documented.
- b) Understand the interdependence between the modulus and the water content, as well as the shape of the modulus versus water content curve.
- c) Develop a simple laboratory test to obtain ahead of time the target modulus from a modulus versus water content curve; this target modulus must be verified in the field using the same test.
- d) Study the depth of compaction that can be achieved by various rollers for various soils.
- e) Study existing specifications and draft standard specifications for the USA.

10.7. Proposed approach.

An approach is proposed which satisfies the ideas developed above. It consists of running the usual Proctor test in the laboratory but adding the Briaud Compaction Device test (BCD lasts 2 seconds) on top of the soil in the mold to get the modulus. This gives the target values. In the field, the instrumented roller performs intelligent compaction and is checked at chosen intervals with the BCD. The advantage of the BCD is that the same test can be run in the lab and in the field in very little time.

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APPENDIX A

A. BACKGROUND ON SOIL MODULI

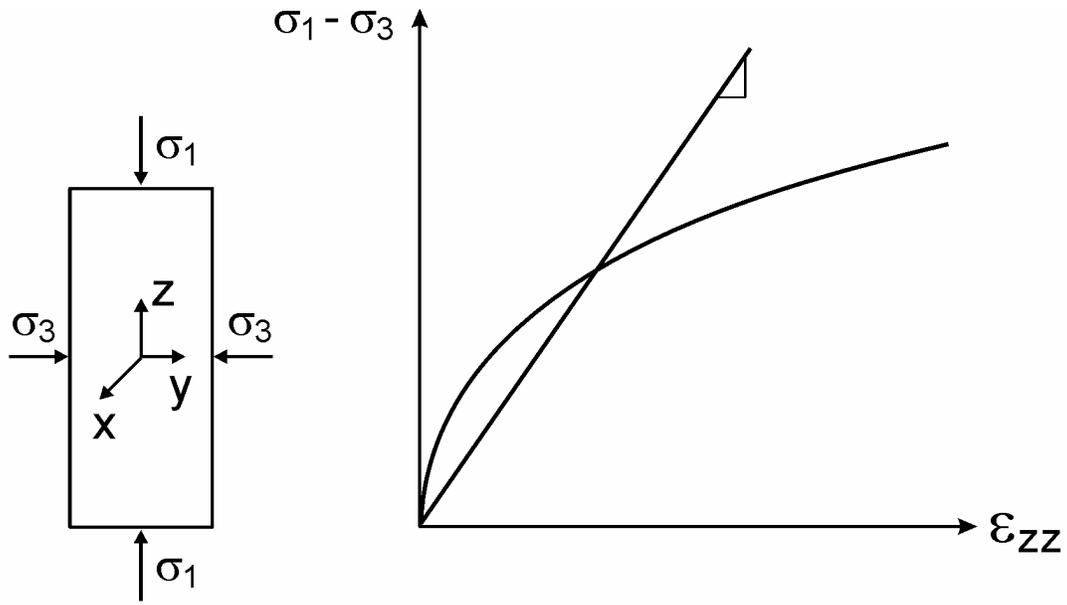
The modulus of a soil is one of the most difficult soil parameters to estimate because it depends on so many factors. Therefore when one says for example: "The modulus of this soil is 10,000 kPa", one should immediately ask: "What are the conditions associated with this number?" The following is a background on some of the important influencing factors for soil moduli. It is not meant to be a thorough academic discourse but rather a first step in understanding the complex world of soil moduli. In a first part, the modulus is defined. In a second part, the factors influencing the modulus and related to the state of the soil are described. In a third part, the factors related to the loading process are discussed. Fourth, some applications of soil moduli are presented. In a fifth and sixth part, the soil modulus is compared to the soil stiffness and to the soil coefficient of subgrade reaction respectively.

A.1. Definition

How does one obtain a modulus from a stress strain curve? In order to answer this question, the example of the stress strain curve obtained in a triaxial test is used. The sample is a cylinder; it is wrapped in an impermeable membrane and confined by an all around (hydrostatic) pressure. Then the vertical stress is increased gradually and the non linear stress strain curve shown on Fig. A.1 is obtained. Elasticity assumes that the strains experienced by the soil are linearly related to the stresses applied. In reality this is not true for soils and there lies one complexity. The equations of elasticity for this axi-symmetric loading relate the stresses and the strains in the three directions as shown in Fig. A.1. Because of the axi-symmetry, equations 1 and 2 in Fig. A.1 are identical. In equations 1 and 3 there are two unknowns: the soil modulus E and the Poisson's ratio ν . In the triaxial test, it is necessary to measure the stresses applied in both directions as well as the strains induced in both directions in order to calculate the modulus of the soil. Indeed one needs two simultaneous equation to solve for E and ν . Note that **the modulus is not the slope of the stress strain curve**. An exception to this statement is the case

where the confining stress is zero as it is for a typical concrete cylinder test or an unconfined compression test on a clay. In order to calculate the Poisson's ratio, it is also necessary to measure the stresses applied in both directions as well as the strains induced in both directions. Note also that **the Poisson's ratio is not the ratio of the strains in both directions** (equation 5 on Fig. A.1). An exception to this statement is again the case where the confining stress is zero.

Which modulus? Secant, tangent, unload, reload, or cyclic modulus? Because soils do not exhibit a linear stress strain curve, many moduli can be defined from the triaxial test results for example. In the previous paragraph, it was pointed out that the slope of the stress strain curve is not the modulus of the soil. However the slope of the curve is related to the modulus and it is convenient to associate the slope of the stress strain curve to a modulus. Indeed this gives a simple image tied to the modulus value; note however that in the figures the slope is never labeled as modulus E but rather as slope S . Referring to Fig. A.2, if the slope is drawn from the origin to a point on the curve (O to A on Fig. A.2), the secant slope S_s is obtained and the secant modulus E_s is calculated from it. One would use such a modulus for predicting the movement due to the first application of a load as in the case of a spread footing. If the slope is drawn as the tangent to the point considered on the stress strain curve then the tangent slope S_t is obtained and the tangent modulus E_t is calculated from it. One would use such a modulus to calculate the incremental movement due to an incremental load as in the case of the movement due to one more story in a high-rise building. If the slope is drawn as the line which joins points A and B on Fig. A.2, then the unloading slope S_u is obtained and the unloading modulus E_u is calculated from it. One would use such a modulus when calculating the heave at the bottom of an excavation or the rebound of a pavement after the loading by a truck tire (resilient modulus). If the slope is drawn from point B to point D on Fig. A.2, then the reloading slope S_r is obtained and the reload modulus E_r is calculated from it. One would use this modulus to calculate the movement at the bottom of an excavation if the excavated soil or



$$\epsilon_{xx} = \frac{1}{E} (\sigma_{xx} - \nu(\sigma_{yy} + \sigma_{zz})) = \frac{1}{E} (\sigma_3 - \nu(\sigma_1 + \sigma_3)) \quad (1)$$

$$\epsilon_{yy} = \frac{1}{E} (\sigma_{yy} - \nu(\sigma_{xx} + \sigma_{zz})) = \frac{1}{E} (\sigma_3 - \nu(\sigma_1 + \sigma_3)) \quad (2)$$

$$\epsilon_{zz} = \frac{1}{E} (\sigma_{zz} - \nu(\sigma_{xx} + \sigma_{yy})) = \frac{1}{E} (\sigma_1 - \nu(\sigma_3 + \sigma_3)) \quad (3)$$

$$E = \frac{\sigma_1 - 2\nu \sigma_3}{\epsilon_{zz}} \quad (4)$$

$$\frac{\epsilon_{xx}}{\epsilon_{zz}} = \frac{\sigma_3 - \nu (\sigma_1 + \sigma_3)}{\sigma_1 - 2\nu \sigma_3} \quad (5)$$

Figure A.1 Calculating a Modulus

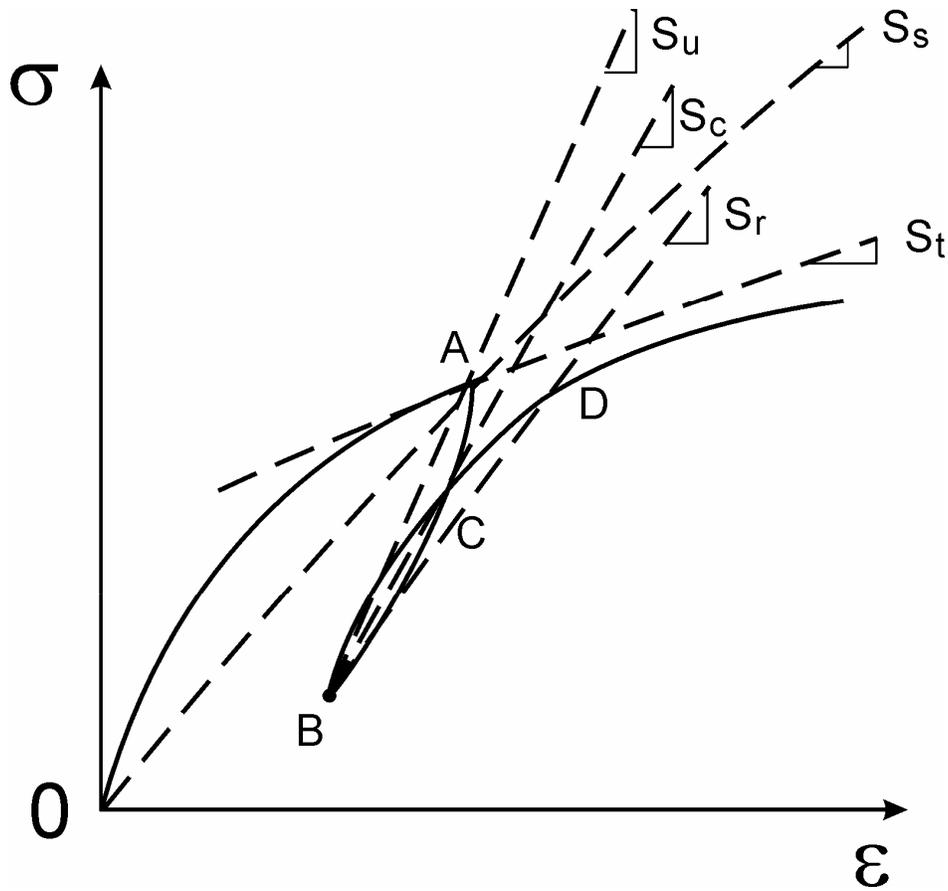


Figure A.2 Definition of Soil Modulus

a building of equal weight was placed back in the excavation or to calculate the movement of the pavement under reloading by the same truck tire. If the slope is drawn from point B to point C on Fig. A.2, then the cyclic slope S_c is obtained and the cyclic modulus E_c is calculated from it. One would use such a modulus and its evolution as a function of the number of cycles for the movement of a pile foundation subjected to repeated wave loading.

Which ever one of these moduli is defined and considered, the state in which the soil is at a given time will affect that modulus. The next section describes some of the main state parameters influencing soil moduli.

A.2.State Factors

The state factors include the following.

How closely packed are the particles? If they are closely packed, the modulus tends to be high. This is measured by the dry density (ratio of the weight of solids over the total volume of the wet sample) of the soil for example; it can also be measured by the porosity (ratio of the volume of voids over the total volume of the wet sample).

How are the particles organized? This refers to the structure of the soil. For example a coarse grain soil can have a loose or dense structure and a fine grain soil can have a dispersed or flocculated structure. Note that two soil samples can have the same dry density yet different structures and therefore different soil moduli. This is why taking a disturbed sample of a coarse grain soil in the field and reconstituting it to the same dry density and water content in the laboratory can lead to laboratory and field moduli which are different.

What is the water content? This parameter has a major impact because at low water contents the water binds the particles (especially for fine grained soils) and increases the effective stress between the particles through the suction and tensile skin of water phenomenon. Therefore in this case low water contents lead to high soil moduli. This is why a clay shrinks and becomes very stiff when it dries. At the same time at very low water contents the compaction of coarse grain soils is not as efficient as it is at higher water contents because the lubrication effect of water is not there. Therefore in this case very low water contents lead to low moduli. As the water content increases, water lubrication increases the effect of compaction and the modulus increases as well. However if the water content rises beyond an optimum value, the water occupies more and more room and gets to the point where it pushes the particles apart thereby increasing compressibility and reducing the modulus.

What has the soil been subjected to in the past? This is referred to as the stress history factor. If the soil has been prestressed in the past it is called overconsolidated. This prestressing can come from a glacier which may have been 100 meters thick 10,000 years ago and has now totally melted. This prestressing can also come from the drying and wetting cycles of the seasons in arid parts of the world. If the soil has not been prestressed in the past, in other words if today's stress is the highest stress experienced by

the soil and if the soil is at equilibrium under this stress, the soil is normally consolidated. An over-consolidated (OC) soil will generally have higher moduli than the same normally consolidated (NC) soil because the OC soil is on the reload part of the stress strain curve while the NC soil is on the first loading part. Some soils are still in the process of consolidating under their own weight. These are called underconsolidated soils such as the clays deposited offshore the Mississippi Delta where the deposition rate is more rapid than the rate which would allow the pore water pressures induced by deposition to dissipate. These clays have very low moduli.

What about cementation? This refers to the “glue” which can exist at the contacts between particles. As discussed above, low water contents in fine grained soils can generate suction in the water strong enough to simulate a significant “glue effect” between particles. This effect is temporary as an increase in water content will destroy it. Another glue effect is due to the chemical cementation which can develop at the contacts. This cementation can be due to the deposition of calcium at the particle to particle contacts for example. Such cementation leads to a significant increase in modulus.

These are some of the most important factors related to the state of the soil and influencing its modulus. In the following part the factors associated with the loading process are discussed.

A.3.Loading Factors

In this section it is assumed that the state factors for the soil considered are fixed. In other words the discussion of each of the factors below can be prefaced by saying “all other factors being equal”. Also in this section the secant modulus is used.

What is the mean stress level in the soil? The loading process induces stresses in the soil. These stresses can be shear stresses or normal stresses or a combination of both. At one point and at any given time in a soil mass there is a set of three principal normal stresses. The mean of these three stresses has a significant influence on the soil modulus. This is also called the confinement effect. Fig. A.3(a) shows an example of two stress strain curves at two different confinement levels. As common sense would indicate, the higher the confinement is, the higher the soil modulus will be. A common model for quantifying the influence of the confinement on the soil modulus is given on Fig. A.3(a)

and is usually attributed to the work of Kondner. According to this model, the modulus is proportional to a power law of the confinement stress. The modulus E_0 is the modulus obtained when the confinement stress is equal to the atmospheric pressure p_a . A common value for the power exponent a in Fig. A.3(a) is 0.5.

What is the strain level in the soil? The loading process induces strains in the soil mass. Because soils are nonlinear materials, the secant modulus depends on the mean strain level in the zone of influence. In most cases the secant modulus will decrease as the strain level increases because the stress strain curve has a downward curvature. Note that an exception to this downward curvature occurs when the results of a consolidation test is plotted as a stress strain curve on arithmetic scales for both axes. Indeed in this case the stress strain curve exhibits an upward curvature because the increase in confinement brought about by the steel ring is more influential than the decrease in modulus due to the increase in strain in the soil. In the triaxial test, the stress strain curve can be fitted with a hyperbola and the associated model for the modulus is shown on Fig. A.3(b). This hyperbolic model is usually attributed to the work of Duncan. In this model (Fig. A.3(b)), E_0 is the initial tangent modulus also equal to the secant modulus for a strain of zero. The parameter s is the asymptotic value of the stress for a strain equal to infinity. In that sense it is related to the strength of the soil.

What is the strain rate in the soil? Soils like many other materials are viscous. This means that the faster a soil is loaded, the stiffer it is and therefore the higher the modulus is. In some instances the reverse behavior is observed. Fig. A.3(c) shows an example of two stress strain curves obtained by loading the soil at two drastically different strain rates. The strain rate is defined as the strain accumulated per unit of time. The modulus usually varies as a straight line on a log-log plot of modulus versus strain rate. The slope of that line is the exponent b in Fig. A.3(c). In clays, common values of this exponent vary from 0.02 for stiff clays to 0.1 for very soft clays. In sands common values of b vary from 0.01 to 0.03. The modulus E_0 is the modulus obtained at a reference strain rate. Much of the work on this model has been done at Texas A&M University.

What is the number of cycles experienced by the soil? If the loading process is repeated a number of times, the number of cycles applied will influence the soil modulus. Again referring to the secant modulus, the larger the number of cycles the smaller the

modulus becomes. This is consistent with the accumulation of movement with an increasing number of cycles. The model used to describe this phenomenon is shown on Fig. A.3(d). The exponent c in the model is negative and varies significantly. The most common values are of the order of -0.1 to -0.3. Much of the work on this model has been done at Texas A&M University.

Is there time for the water to drain during the loading process? Two extreme cases can occur: drained or undrained loading. The undrained case may occur if the drainage valve is closed during a laboratory test or if the test is run sufficiently fast in the field. The time required to maintain an undrained behavior or to ensure that complete drainage takes place depends mainly on the soil type. For example a 10 minute test in a highly plastic clay is probably undrained while a 10 minute test in a clean sand is probably a drained test. The Poisson's ratio is sensitive to whether or not drainage takes place. For example if no drainage takes place during loading in a clay it is common to assume a Poisson's ratio equal to 0.5. On the other hand if complete drainage takes place (excess pore pressures are kept equal to zero), then a Poisson's ratio value of 0.35 may be reasonable. The difference between the two calculated moduli is the difference between the undrained modulus and the drained modulus. Note that the shear modulus remains theoretically constant when the drainage varies. Note also that the Poisson's ratio can be larger than 0.5 if the soil dilates during shear associated with compression.

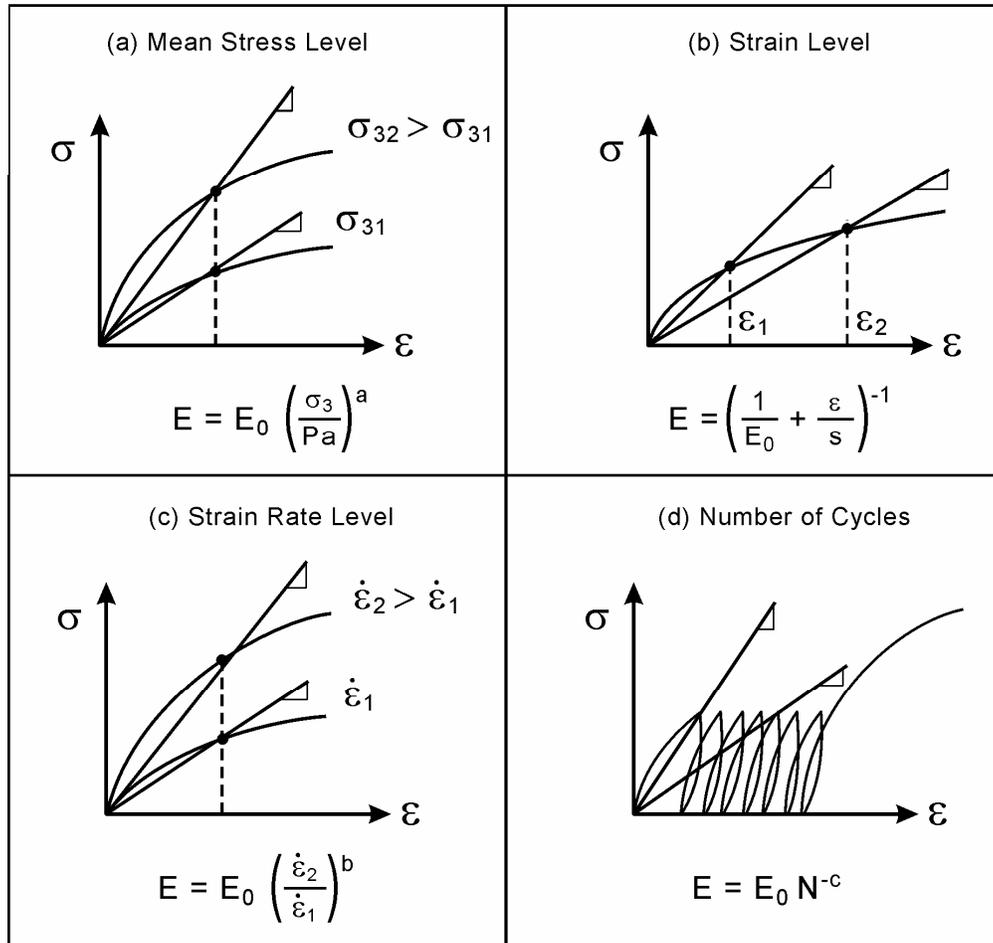
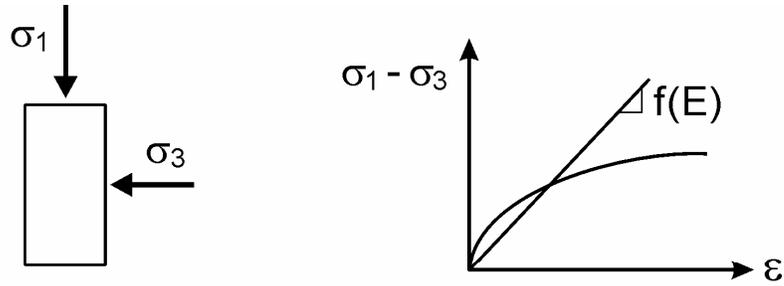


Figure A.3 Loading Factors for Soil Moduli

A.4. Moduli for Various Fields of Application

The modulus is useful in many fields of geotechnical engineering. It is clear by now that the modulus required for one field may be significantly different from the modulus for another field.

In the case of shallow foundations, the mean stress level applied under the foundation is often between 100 and 200 kPa. The normal strain level in the vertical direction is about 0.01 or less and is typically associated with a movement of about 25 mm. The rate of loading is extremely slow because that strain occurs first at the construction rate and then the load is sustained over many years. The number of cycles is one unless cycles due to seasonal variations or other cyclic loading (such as compressor foundations) are included. Example values of the modulus in this case are 10,000 to 20,000 kPa.

In the case of deep foundations, the mean stress level varies because the side friction on the piles occurs over a range of depth, while the point resistance occurs at a relatively large depth. The strain level at the pile point is usually smaller than in shallow foundations because a percentage of the load dissipates in friction before getting to the pile point. The strain rate is similar to the case of shallow foundations with rates associated with months of construction and years of sustained loads. High strain rates do occur however in the case of earthquake or wave loading. Cycles can be a major issue for earthquake loading of buildings and bridges or for wave loading of offshore structures. Because deep foundations are used in very different types of soils and for very different types of loading, the moduli vary over a much wider range of values than for shallow foundations.

In the case of slope stability and retaining structures, movements are associated with the deformation of the soil mass essentially under its own weight. Therefore the stress level corresponds to gravity induced stresses. The strains are usually very small and the strain rate is again associated with the rate of construction at first and the long term deformation rate during the life of the slope or of the retaining structure. Cycles may occur due to earthquakes or other cyclic phenomena. For properly designed slopes and retaining structures, the moduli tend to be higher than in foundation engineering because the strain levels tend to be smaller.

In the case of pavements, the mean stress level in the subgrade is relatively low. The pressure applied to the pavement is of the order of 200 kPa for car tires, 500 kPa for truck tires, and 1700 kPa for airplane tires. However, the vertical stress at the top of the subgrade under a properly designed pavement may be only one tenth of the tire pressure

applied at the surface of the pavement. The strain level is very low because the purpose of the pavement is to limit long term deformations to movements measured in millimeters if not in tenths of millimeters. Typical strain levels are 0.001 or less at the top of the subgrade. The rate of loading is very high and associated with the passing of a traveling vehicle. The loading time is of the order of milliseconds for a car at 100 km/h but is measured in hours for an airplane parked at the gate. The number of cycles is tied to the number of vehicles traveling on the pavement during the life of the pavement. This number varies drastically from less than a million of vehicle cycles for small roads to tens of millions for busy interstates. Typical modulus values range from 20,000 kPa to 150,000 kPa.

A.5. Modulus or Stiffness?

The modulus E has been defined in Fig. 2.3. It has units of force per unit area (kN/m^2). The stiffness K is defined here as the ratio of the force applied on a boundary through a loading area divided by the displacement experienced by the loaded area. It has units of force per unit length (kN/m). The loaded area is typically a plate which can be square or circular or in the shape of a ring. There is a relationship between the modulus and the stiffness. For the case of a circular plate having a diameter B , the relationship is of the form:

$$E = f(K/B) \quad (2.6)$$

This relationship shows that, if the modulus is a soil property, **the stiffness is not a soil property and depends on the size of the loaded area**. Therefore, for an elastic material, the stiffness measured with one test will be different from the stiffness measured with another test if the loading areas are different. Yet, for the same elastic material, the modulus obtained from both tests would be the same. In that sense the use of the stiffness is not as convenient as the modulus and the use of the modulus is preferred.

A.6. Modulus or Coefficient of Subgrade Reaction?

The modulus E has been defined in Fig. 2.3. It has units of force per unit area (kN/m^2). The coefficient of subgrade reaction k is defined here as the ratio of the pressure applied to the boundary through a loading area divided by the displacement experienced by the loaded area. It has units of force per unit volume (kN/m^3). The loaded area can be a footing (coefficient of vertical subgrade reaction) or a horizontally loaded pile (coefficient of horizontal subgrade reaction). There is a relationship between the modulus and the coefficient of subgrade reaction. For a footing or a pile, that relationship is of the form:

$$E = g(k \times B) \quad (2.10)$$

where B is the footing width or the pile width or diameter. This relationship shows that, if the modulus is a soil property, **the coefficient of subgrade reaction is not a soil property and depends on the size of the loaded area**. Therefore, if a coefficient of subgrade reaction k is derived from load tests on a footing or a pile of a certain dimension, the value of k cannot be used directly for other footing or pile sizes. Indeed in this case, careful considerations of size and scale must be addressed. The modulus is not affected by this problem. In that sense the use of the coefficient of subgrade reaction is not as convenient as the modulus and the use of the modulus is preferred.