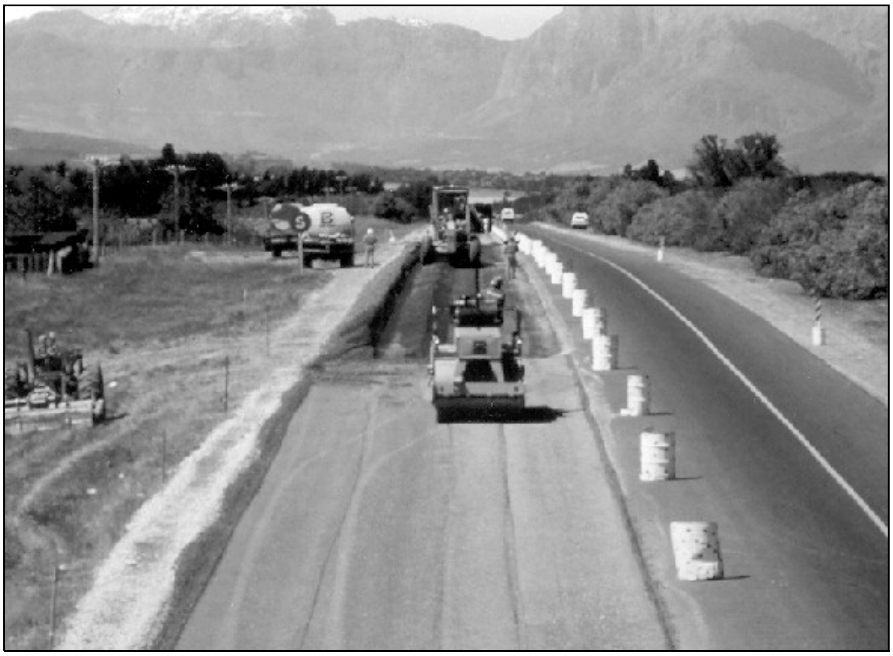
THE COMPACTION OF SOILS FOR ROAD CONSTRUCTION



A COURSE PRESENTED BY THE

SOUTH AFRICAN ROAD FEDERATION

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THE COMPACTION OF SOILS FOR ROAD CONSTRUCTION

(First edition, January 2004)

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Cover Photograph : Rehabilitation of National Route 1, Section 1 (N1/1) – Kraaifontein to Paarl, 1984 – Photograph by Carl van der Merwe.

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FOREWORD

This document has been produced specifically for a course arranged by the SOUTH AFRICAN ROAD FEDERATION. It deals with the compaction of soils for road construction, i.e. soils as defined in paragraph 2.2 of this document. The papers have been compiled and edited by Carl van der Merwe C.Eng., Consultant to UWP Consulting; Renaldo Lorio Pr. Eng., Materials Engineer to UWP Consulting and Lucas Ebels, Ph.D. Candidate at the University of Stellenbosch : They have written sections 1, 7, 8 and 9 and they added yellow pages to sections 2, 3, 4, 5 and 6 highlighting key points and also giving additional information on the relevant subjects. Barry James of Power Construction provided the photographs in Section 1.9. Final editing was by Renaldo Lorio.

Sections 2, 3, 4, 5 and 6 are largely a reproduction of a series of articles which appeared in the Civil Engineering Contractor in 1983. They were a comprehensive series which thoroughly covered the important aspects of compaction. Although these articles were published 20 years ago the theories and principles illustrated, are still valid today. They were written by: Dave Wolmarans who at the time was National Sales Manager of Triplejay and Ashley Dunn who at the time was with Dynapac, United Kingdom. The articles have now been revised by Professor Phillip Savage, Consultant. He also wrote “Relative Compaction and the Significance of Error” included in Section 8.

In the first Section, “The Importance of Compaction in Road Construction” it is shown that a tremendous increase in soil strength can be derived from compaction and that it can be achieved at a surprisingly low cost. It is also shown what detrimental and costly effects poor compaction can have on a pavement. There cannot be a better motivation for good compaction.

In the compaction process virtually every soil type is encountered from crushed stone to clay, each soil having its own characteristic water demand and its own characteristic reaction to the forces of compaction. An understanding of soil types and soil classification is thus essential and this is dealt with in detail.

The role of moisture in soil compaction is critical and the scientific methods which have been developed to understand and quantify water demand are described.

The development of vibratory compaction equipment has made a huge impact on compaction. It has contributed substantially towards the ease and standard of compaction and vibratory rollers play an important part in every construction unit. A large part of the notes are therefore devoted to the theory of vibration and vibratory rollers. Compaction by impact rolling is also illustrated.

It is necessary to address some practical issues since unpredictable problems sometimes occur in the field. In the section “Some Practical Aspects” the views of experienced contractors are given with several examples of problems which occurred in practice and their solutions.

In the evaluation of compaction by test methods a good understanding of the theory of statistics is essential to ensure fair and consistent decision making. Procedures for statistical evaluation are described together with some examples illustrating the analysis of typical results.

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1. THE IMPORTANCE OF COMPACTION IN ROAD CONSTRUCTION

1.1 Historic developments

1.1.1 Early history

Compaction has been used to improve the load bearing capacity of soils for as long as mankind had the need to provide pathways for carrying meaningful traffic volumes irrespective of whether the traffic volumes were pedestrian, animal or motorised vehicles. Although it can be accepted that the early road builders did not understand the principles of soil mechanics they knew intuitively (and empirically) that if a heavy load was applied to the soil (whether static or dynamic) its characteristics improved such that it provided a more mechanically stable pathway that was more suitable for their purposes.

An example of intuitive soil improvement dates back to before the wheel was invented (prior to 3500 BC). At that time the Incas in South America already had a 16 000 km road network that was built entirely by human labour. In what is now Mexico a 5 tonne solid stoneroller was found, which is believed to have been used by the ancient Mayans for compacting these roads, see Figure 1.1. The roads they built included both earthen and stone paved walkways [1].

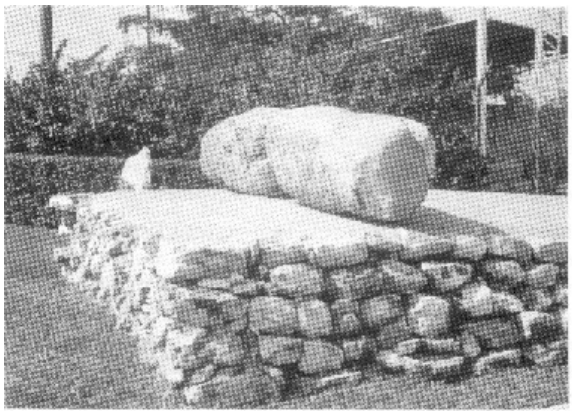


Figure 1.1 : Ancient Mayan stoneroller [1]

Before the wheel was invented, roads in Europe and the Middle East were hardly more than dirt tracks, created by repeated animal and human traffic – in effect compacting the soil (traffic compaction) and thereby creating a more stable pathway. If one could speak of modern road construction, clearing and grubbing would probably have been the only construction process at that time. However, with the advent of the wheel (circa 3500 BC) and the increased use of animal drawn wagons, the need for improved roads increased.

Evidence of the first stone block roads in Babylon date back to approximately 2000 BC to 1500 BC. These roads were constructed by means of placing even sized stone blocks in various layers within a box cut. Figure 1.3 shows a cross section of the Babylonian pavement structure. It is believed that these blocks were not cemented together [12].

Between 500 BC and 500 AD the Romans constructed their widely acclaimed road network. This road network was constructed primarily for strategic and military purposes. Figure 1.3 shows a cross section of the “standard” pavement structure and it can be seen that it has some resemblance to the Babylonian pavement structure but there are two significant differences namely:

- The Roman’s used a mixture of clayey soil (containing pozzolanic material) and lime as a filling between the stone block foundation and the final stone block surface layer. This filling cemented and formed a hard core and thus in effect the entire pavement structure comprised one solid unit as the stone block layers were also set in a mortar comprising a

similar mixture of clay (pozzolanic material) and lime. It is believed that the Romans used some form of compaction to densify the material in the filling.

- The Romans provided their pavements with cambered surfaces and also raised them above natural ground level in order to improve the drainage. From this practice stems the term “highway” which is still in use today.

It is reported that the Romans compacted the subsoil underneath the first stone block layer to create a firm foundation on which to lay of the stone blocks. This compaction was undertaken by means of ramming although some writers do refer to wooden or stonerollers, but there is no proof that such equipment was indeed used. If the subgrade was considered to be too weak, a pioneer layer of timber poles was placed prior to the laying of the first stone block layer. Some of the roads constructed by the Romans are still in use today (Via Appia in Rome built by Appius Claudius 300 BC).

After the collapse of the Roman Empire, trade and travel quickly diminished and the established road network started to deteriorate. Throughout the medieval ages there was little road building activity. However during this time the Chinese used an innovative form of dynamic compaction to compact the material on which they constructed their building structures (mainly temples) [9].

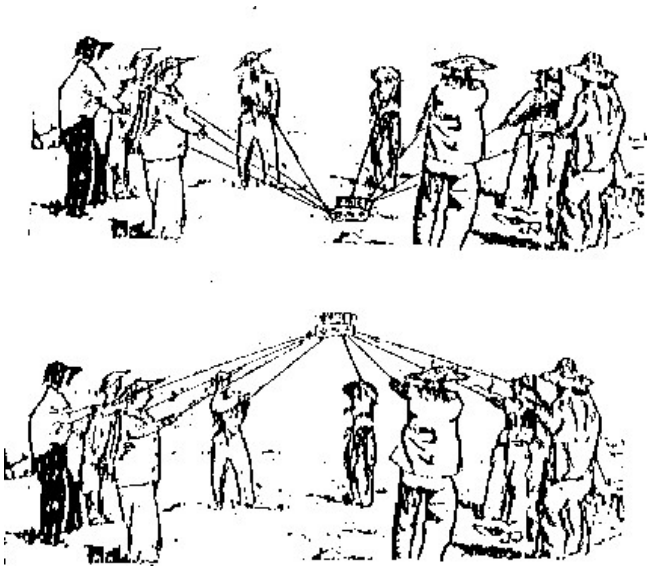


Figure 1.2 : Illustration of Chinese men performing dynamic compaction by means of lifting a weight - pulling and releasing on ropes in unison [9]

1.1.2 The renaissance in road building

After the dark ages there was a revival in road construction practices. This revival was as a result of increases in trade. Early in the 18th century the strategic importance of inter city travel was recognised and the need arose for higher standard roads. In France the “Corps des Ponts et Chaussess” (Engineering Corps) was established in 1716 where soldiers were trained in engineering skills to fulfil the need for the development of infrastructure. The civil counterpart of this institution, the ‘Ecole des Ponts et Chaussess’ (School for Bridges and Roads) was established in 1747. This was the first school for civil engineering and commercial interests drove it.

Around 1765 Pierre-Marie Tresaquet (1716 – 1796), a graduate from this school, continued the developments that were pioneered by the Romans and designed pavement structures with a subbase layers composing of large rocks. The spaces between the large rocks were filled with smaller rocks until the layer of large rocks were completely covered. Following this rock layer was a

base course layer of smaller sized rocks, still essentially a uniformly graded material. See Figure 1.3 [1].

At the same time Thomas Telford (1757 – 1834) from Scotland and John Metcalf (1717 –1810) from Britain applied similar design philosophies but independent from Tresaquet (see Figure 1.2). This type of road construction was expensive and difficult to execute because of the large size rock required for the sub-base layer.

A breakthrough in road building was achieved by John Loudon Macadam (1756 – 1836) from Scotland. Macadam came to the realisation that the large stone foundation was not required when a dense layer of smaller sized rocks, which had sufficient inter-particle friction, was used. Probably without fully understanding why his pavement layers were so strong, Macadam used the principle of particle interlock to create strength and the principle of increased density to further enhance the strength of the layer. The layer was constructed by first placing the coarse aggregate to create the particle interlock and then followed by working the finer material into the voids of the coarse aggregate. See the cross section of the Macadam pavement structure in Figure 1.3. There were two processes by which the fine material could be worked into the voids in the coarse material, namely a wet and a dry process. The dry process comprised working the fine material into the voids in the layer of coarse material without wetting it. The wet method, also referred to as waterbound Macadam, comprise the preparation of a sand slurry (fine material mixed with water), which was then worked into the voids between the coarse aggregate [13].

This process reduced the cost of the road construction significantly as the required thickness of the pavement structures reduced as well as obviating the need to work with large size rocks which were not always plentiful. Due to its lower cost the Macadam roads were favoured by many public authorities in Britain as well as other countries. With the advent of the first mechanical crusher (in 1858 by Blake), as well as the steamroller (in the 1860's) the quality of the Macadam roads improved significantly and consequently became more appealing in Europe and America [2].

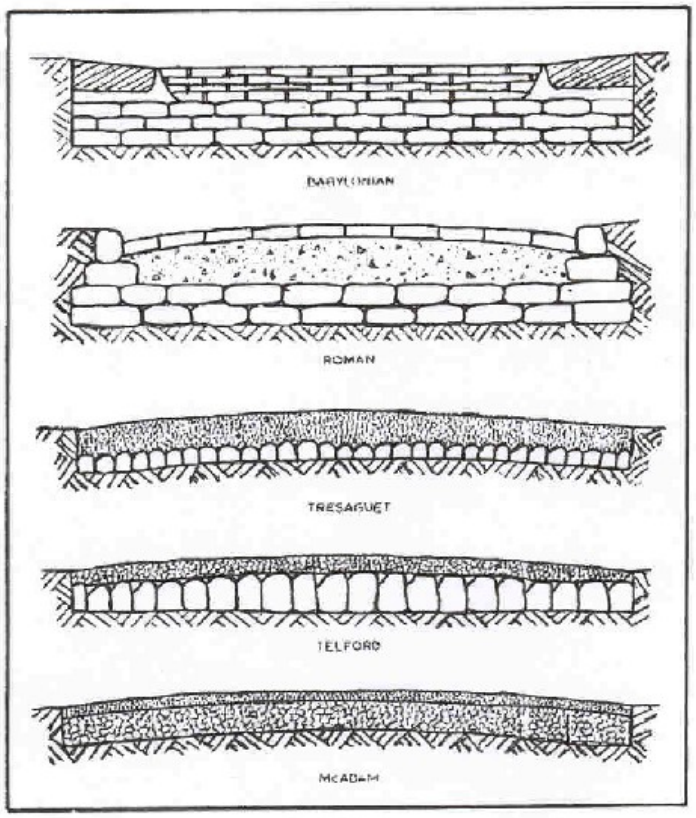


Figure 1.3 : Cross-sections of pavement constructions up to 1900 [8]

1.2 History of motorised compaction equipment

Up until the 1830s the construction of pavement structures was such that the material was generally placed, followed by some limited compaction effort, where after it was opened to traffic. Thereafter additional compaction occurred under traffic. It was around 1830 that horse drawn rollers were first developed in France and used in road construction processes.

By 1860 steamrollers became available and changed the face of road construction. The introduction of steamrollers had a significant impact on the level of compaction that could be achieved. An example of a steamroller that was used in those days is the Ross ramming machine, developed by the American engineer Ross, see Figure 1.4.

Since soil compaction is not only confined to road construction it is not surprising that equipment development also stemmed from other fields of construction. One such field is dam construction. In dam construction it is essential that the material forming the dam wall be compacted in order to increase the stability of the structure and reduce the permeability thereof in order to retain the water. At that time it was acknowledged that trampling cattle and sheep was effective in compacting soils. Around 1820 cattle and sheep were used in England to compact the material in earthfill dams. This led to the development of the so-called sheepsfoot roller.This is essentially and ordinary roller with a studded drum to increase the contact pressure under the studs. The initial size of the studs on the drum was larger than that used on present day sheepsfoot rollers but of similar shape. This roller was first patented in the USA in 1906. The sheepsfoot roller is very effective for compacting cohesive materials without creating a smooth surface that could lead to delaminations, through which water could leak. This is an important consideration in dam construction. However sheepsfoot rollers are often used in road construction to compact cohesive materials especially in fills and selected subgrade layers.

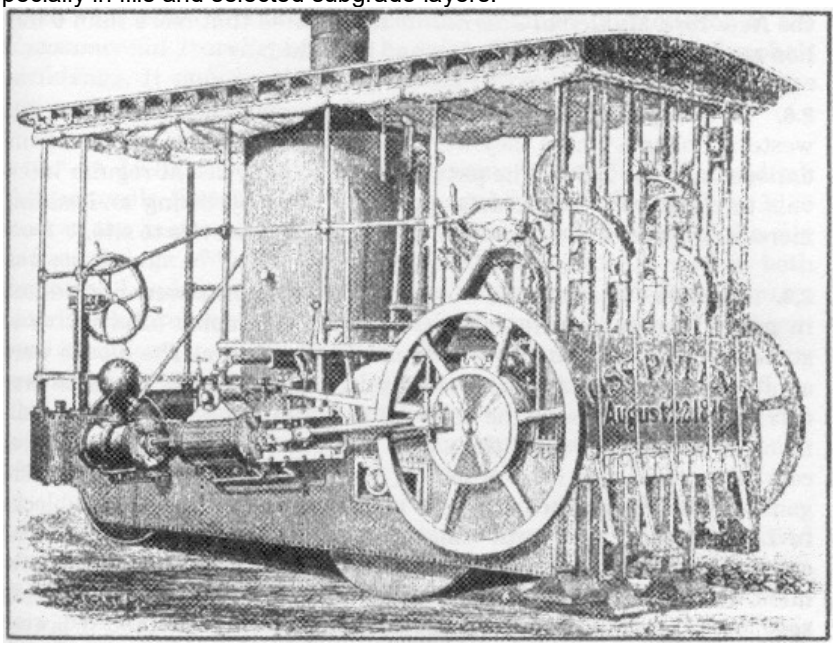


Figure 1.4: The Ross ramming machine (1871) [3]

From the turn of the 19th century the developments in compaction equipment accelerated and below is a succinct summary of the important developments [8].

- By 1920 the sheepsfoot roller had been developed to a size, which is now termed “light”. The mass was in the range of between 6 000lbs (2,7 ton) and 10 000lbs (4,5 ton) and footprint pressure of 60 (414 kPa) to 100 p.s.i. (690 kPa).

- By the late 1920’s the flat wheel steamrollers had been converted to petrol engines and had reached masses of up to 30 tons.

- During the 1930's the small rubber tyred roller of 6 to 8 tons, often called the wobble wheel roller, came into use for compacting thin layers of base course and in smoothing and knitting the surfaces of compacted layers.

- The present "heavy" sheepsfoot roller was developed in the 1930's for highway and earth dam construction. The roller had a mass of up to 30 000lbs (13,6 tons) and could be used at footprint pressures of 300 (2070 kPa) to 600 p.s.i. (4140 kPa).

- Subsequent developments in sheepsfoot rollers have been concerned more with the shape of the foot and mechanical developments rather than the size of the roller.

- Compaction by vibratory and dynamic techniques was developed in Germany during the 1930's. A self-propelled caterpillar type vibrating plate compacter with a total weight of 25 tons was developed as early as 1933. Developments in Germany produced the so called "frog" tamper just before World War II.

- During the Second World War the technology required for airfield pavements resulted in the development of "heavy" compaction equipment. During 1943-1944 the U.S. Army Corps of Engineers constructed several rubber tyred rollers equipped with twenty four 810mm wide tyres capable of being loaded to 75 to 100 tons. The first self-propelled and tractor towed vibratory rollers were constructed during the 1940's.

Since the Second World War the most significant development has been in the field of vibratory compaction. At present vibrating rollers feature very prominently in road construction and they are very effective in producing high density materials.

1.3 The need for change in road building in the early 20th century

The introduction of the Model T Ford in 1908 signalled the beginning of the motorised vehicle era. Since the internal combustion engine is relatively efficient and generally easy to reproduce it allowed the mass production of various types of vehicles (light and heavy). Between the First and the Second World War the number of motor vehicles increased as a result of increased demand and lower unit prices.

As the heavy vehicles were continually improved and equipped with stronger engines they were enabled to carry heavier loads resulting in them becoming more attractive as goods transporters.

As is to be appreciated the motor vehicles brought tremendous freedom of movement to its owner / operator and in order to exercise this freedom pressure was exerted upon various bodies to provide new roads and expand the road network. It thus has to be accepted that the growth in the motor vehicle market and the concurrent growth in the world economy fuelled the need to expand the road network worldwide – roads were built because there was a need for them. However it has also been argued that the roads built the economy and not the other way around.

The provision of roads is generally accepted as the responsibility of the government. The reason for this is that roads are generally built using public funds and they are therefore to be seen as national assets.

Since the rate of construction accelerated it became clear that the more recently established road building practices of Tresaquet, Telford and even Macadam were too expensive and time consuming to construct and even if these pavement structures were to have been employed on a large scale the cost to the various economies of the world would have been extremely high. It was therefore clear that more effective use had to be made of existing natural gravel sources to provide cost effective pavement structures that would be able to carry the required traffic loading whilst remaining in an acceptable condition. Figure 1.5 shows what the travelling conditions were like at that time when roads had to be provided at a relatively low cost without having the necessary technology and understanding of road building materials [1].

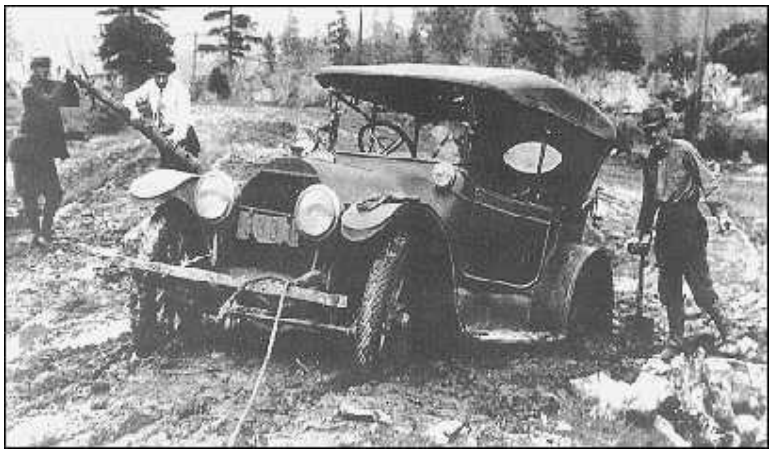


Figure 1.5 : Typical road condition circa 1920

1.4 The basis for scientific research and development of technical standards

With the increased expenditure of public funds on the construction and maintenance of roads, it became clear that more coordinated efforts were required to enhance the quality of road construction in order to provide better facilities. In this regard full scale road tests were undertaken. An interesting example is the Bates Road test which was undertaken between 1922 and 1923. The purpose of this road test was to determine which pavement structure would be the most suitable for the planned expansion of the road network in the state of Illinois. The Bates Road test comprised the construction of test tracks, comprising different pavement types (concrete, bricks and ordinary flexible pavements, which were trafficked by World War One trucks between 1922 and 1923, see Figure 1.6 [11]. Subsequent to this study it was found that concrete pavements performed the best and was consequently selected as the preferred pavement structure and implemented in the road expansion programme. It has to be kept in mind that the use of concrete pavements was an example of over design on roads where low traffic volumes occurred, but it was the only pavement structure in which they had confidence. The cost of pavement construction only reduced once sufficient advances were made in the understanding of soil mechanics in order to use the natural gravel and graded crushed stone more efficiently. That became the challenge of the following two decades when it became clear inter alia that compaction of the pavement layers was of extreme importance.

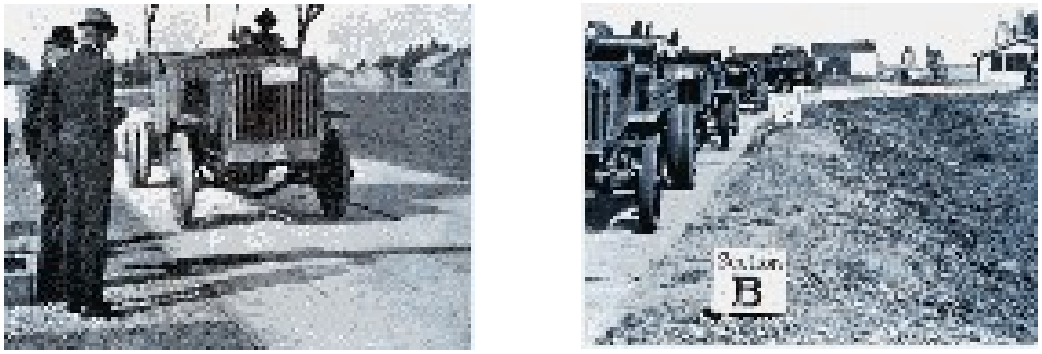


Figure 1.6 : Bates Road Test using old WWI army trucks [11]

It is remarkable that up until the late 1920s there was little scientific research into the development of technical standards for pavement design and construction. At that time roads were generally built based on the practical experience of road engineers or to the current best practice in a particular area. Although it was already known for a long time that increased compaction improved

the performance of unbound materials the theoretical understanding thereof was not yet sufficiently developed in order to enhance pavement design and provide suitable standards.

During 1928 and 1929 the California Highway Department undertook a study of their road network in order assess the type of distress as well as the possible causes thereof. This study identified the following defects :

- Horizontal displacement of the surface material (especially after the material absorbed water and became soft)
- Differential settlement
- Excessive deflection of the material under the pavement structure as a result of repeated loading.

From this study it was determined that insufficient and variable compaction during construction was the primary cause of the first two types of distress. It was further determined that the degree of compaction had a greater effect on the performance of the road than any other factor apart from materials which swelled significantly upon wetting (high clay content materials). It was determined that the cause of the third distress was insufficient pavement thickness upon materials that had insufficient shear strength (inadequate cover on subgrade).

The main findings of this study – that the density and the shear strength of the material are important factors in pavement design – created an impetus to search for ways in which these two properties could be better understood [7].

After some unsuccessful attempts were undertaken to describe the shear strength of unbound materials Porter developed the CBR (California Bearing Ratio) penetration test during the early 1930s, which provided some indication of the shear strength of materials. During the initial stages of the investigation the soil samples were compacted using static compaction. It was found that a pressure of 13,78 MPa had to be applied to the sample in the mould to achieve the same density as the material in the field after it had received traffic for a number of years. This compaction method proved to be too cumbersome and its availability was limited [3].

During the same time but independent from the California Highway Department's activities R.R. Proctor, a field engineer with the Bureau of Waterworks and Supply in California, conducted research into the relationships between the density of soil, the compaction energy, and the moisture content. The Bureau was in the process of developing a series of earthfill dams and desired to establish a method to control the compaction of the material in the embankments. Proctor published a series of four articles in the Engineering News Record concerning soil compaction in 1933. In these publications he introduced his so-called Proctor curve, the relation between moisture content and maximum dry density which formed the basis of the Bureau's compaction control procedures, which is still in use today.

This compaction procedure was adopted by Porter to replace the static compaction procedure used in the CBR test. Due to the simplicity of the equipment and procedure the Proctor test became widely used, such that the American Association of State Highway Officials (AASHO) adopted it and redefined it as the "AASHO standard laboratory method of test for the compaction and density of soil" in their Standard Specification published in 1942. Soon thereafter the US War Department adapted the AAHSO compaction test by increasing the standard compaction energy and termed it the "Modified AASHO test". This increased compaction effort arose from the need to provide stronger pavement structures for airfields during World War Two. Since then the Modified AASHTO test become the most frequently used reference density specification in road and airfield construction. In 1973 AASHO changed its name to AASHTO, which is now the general designation of their test methods [3].

From the combined Mod AASHTO density specification and the CBR test, Porter and later the US Army Corps of Engineers developed pavement design curves, which was the first widely recognized pavement design method for unbound materials. The use of this design method and the use of the Mod AASHTO density specification lead to the construction of significantly improved pavement structures. See Figure 1.7 [7].

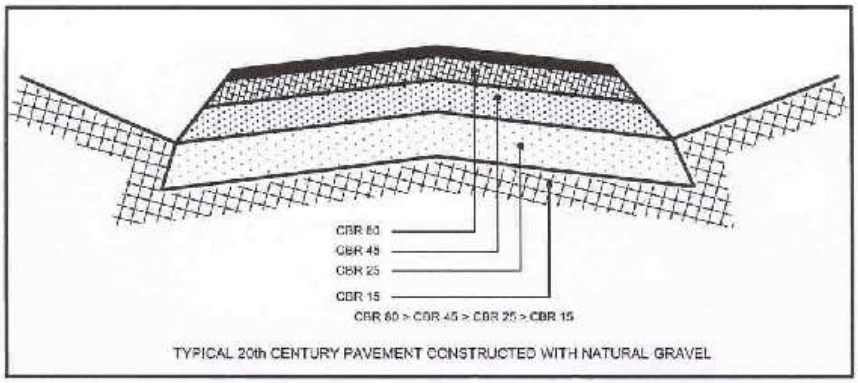


Figure 1.7 : Cross section of typical pavement structure constructed with natural gravel

A concurrent but very important development was the development of the triaxial test by Casagrande during the early 1930s. This test was developed to assess the shear strength of soils more accurately. The Texas Department of Transport was one road authority that adopted the triaxial test to form the basis of their pavement design procedure. The Texas Triaxial Test as it became known was later adopted by other road authorities and used with great success. One such road authority was the Zimbabwean Ministry of Roads who constructed its entire road network using natural gravels.

1.5 The effect of compaction on the shear strength and stiffness of material

Although it was already established by 1929 that compaction is one of the most important factors affecting the shear strength of unbound material it was only until the CBR test and related tests became available that its real effect on the strength of the unbound material could be investigated.

Figure 1.8 shows the relationship between the density of good quality natural gravel and the CBR value.

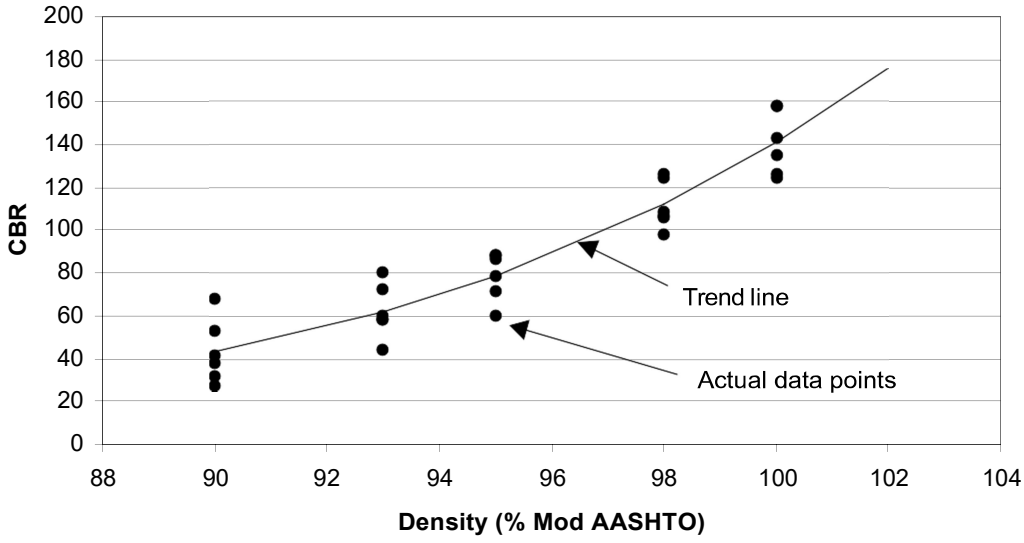


Figure 1.8 : Typical CBR vs density relationship for G4 natural gravel

Figure 1.9 shows a similar trend but for crushed stone base material. It is important to note that for crushed stone the increase in shear strength as indicated by the CBR test is more rapid as the maximum achievable density is approached (note that the density in this example is expressed as a ratio to the maximum density of the aggregate and not with respect to a standard density like the Mod AASHTO density). This has important implications in practice insofar as the high strength is essential for modern traffic intensity and high tyre inflation pressures – and the compaction process must therefore be carried out efficiently to achieve the high densities.

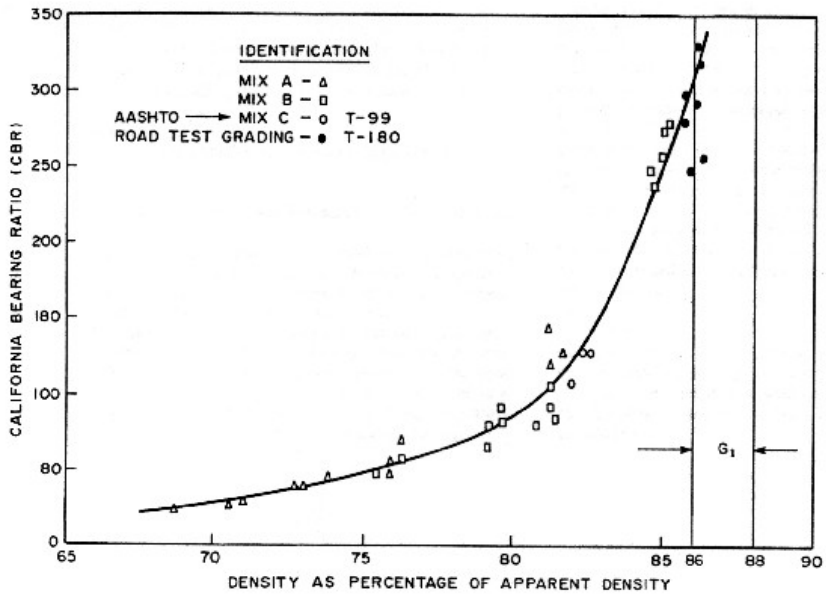


Figure 1.9: CBR vs density of G1 crushed stone

Due to the advances that have been made in analytical pavement analysis since Boussinesq first published his theory regarding the determination of the stress condition in a semi-infinite elastic half space in 1885 it is now possible to quantify the beneficial effect of increased density and shear strength on the overall stress condition in a pavement structure.

Figure 1.10 shows how the pavement response is improved when the density of the materials is increased and consequently its stiffness (an approximation). Figure 1.10 shows the vertical deflection at the top of each layer for two typical pavement structures. The one containing standard compacted material in the various layers and the other a higher compaction level (extra over compaction) in the various layers. Figure 1.10 shows that the improved compaction improves the pavement's response in that the vertical deflection is significantly reduced. Thus the increased density, which leads to increased stiffness and better load spreading ability improves the load carrying capacity of the pavement. The increased load carrying capacity of the pavement structure due to the increased compaction is between approximately 50% to 100% (increase from $3{,}0 \times 10^6$ E80s to approximately 4,5 to $6{,}0 \times 10^6$ E80s).

It can further be deduced that the same protection can be provided to the subgrade layer with less cover if stiffer material is used. It can therefore be seen what beneficial effect increased density could have on the load spreading ability of pavement materials and how it can provide increase protection to the underlying material. Apart from the protection it provides to the subgrade the increased strength allows materials in the various pavement layers to carry heavier loads without shear failures occurring.

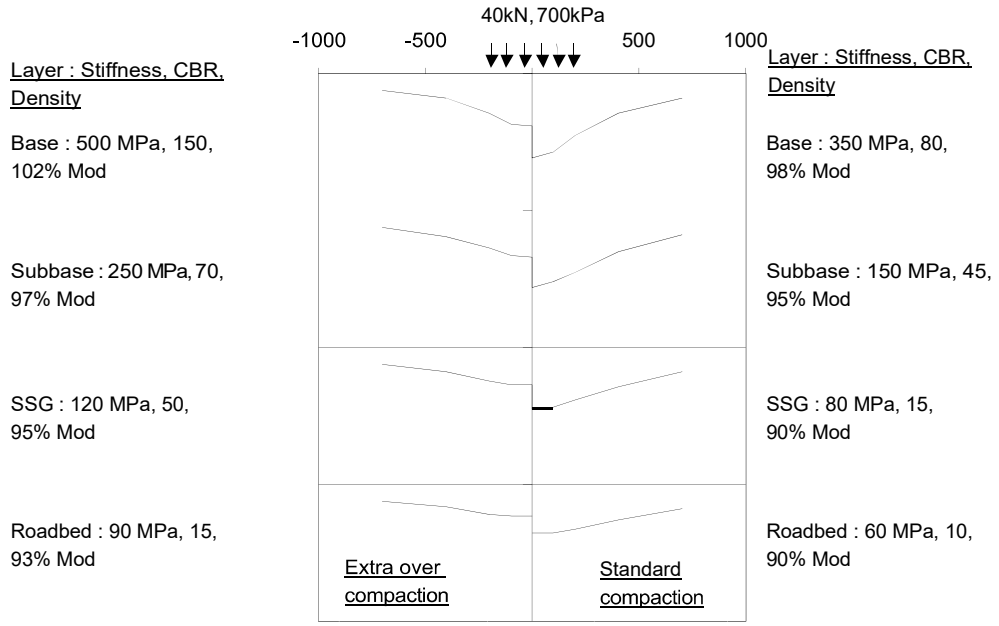


Figure 1.10 : Magnitude of the vertical displacement on top of each layer in the pavement structure - comparing influence of compaction (an approximation)

1.6 The development of modern pavement structures

The incredible value of compaction to a road and its contribution to the stability and structural capacity of the pavement is not always appreciated nor is it realised at what a low cost this is achieved. Consider the increase in strength that is achieved with corresponding cost. The current rate for G1 quality base course (probably the most vital layer in most high quality pavements) varies around R280,00 / m^3 and includes all costs inter alia procuring, placing and compacting the material. The cost of compaction is related to the production rate but generally varies around R 20,00 / m^3 or 7% of the total cost. At this cost the density of the material is increased from approximately 70% ARD to 88% ARD which corresponds to a seven fold increase in the strength of the material, see Figure 1.9. This is mind boggling! If it is kept in mind that increased strength generally leads to increased pavement life it can be seen how effectively the investment in the pavement structure can be enhanced at relatively low cost. The cost ratios obviously change significantly in the lower and less expensive pavement layers but the aforementioned example demonstrates the issues dramatically enough. However the value of compaction is still very often overlooked.

These principles lead to the development of the modern pavement structures being used in South Africa to carry relatively high traffic loadings whilst remaining economical to construct, See Figure 1.11 below. Figure 1.11 shows the use of the cement stabilised subbase layer and the crushed stone base layer. The cement stabilised subbase layer provides additional strength and improved the load spread ability over what can be achieved from the unstabilised natural gravel. The crushed stone base layer comprises well graded material which allows the closest possible packing of the aggregate which assist in achieving to the highest possible density. From Figure 1.9 it can be seen that the crushed stone base material provides good strength (as indicated by the CBR values) in the pavement structure when the material is compacted to a high density. In the construction of these pavements compaction is an essential requirement.

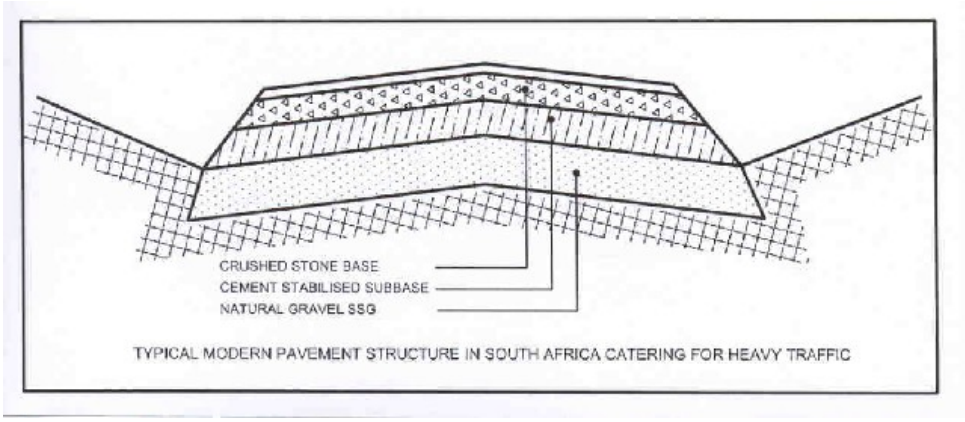


Figure 1.11 : Typical modern pavement structure used in South Africa to cater to heavy traffic

1.7 Consequences of poor compaction

It has been shown above that poor compaction compromises the shear strength of the material. Reduced shear strength generally lead to the occurrence of shear failures that manifest in the road surface as ruts, see Photograph No. 1.1. The width of the rut in the road surface gives an indication of the depth of the weakness. The width of the rut tends to become wider the deeper the weakness lies and vice versa. Some slight rut formation can also be due to further compaction of the material under traffic. This applies to material that was initially poorly compacted. These ruts are less severe than ruts formed due to a lack of shear strength. However both the aforementioned mechanisms are generally present since the shear strength of the material and its density are closely related.

Photograph No. 1.1 : Severe rutting as result of insufficient shear strength (severe ruts such as these pose a safety hazard due to possible aquaplaning during wet weather)

In addition to shear failures (rut formation) poor compaction can also lead to differential settlement (undulations) in the pavement surface which negatively affects the riding quality of the road.

Apart from the deformation that can occur in the pavement surface as a result of poorly compacted material it could also affect the structural performance of the pavement as the lower stiffness of the various pavement layers can lead to increased surface deflection which places additional strain on the asphalt surfacing. If the applied strain is higher than the strain level that the asphalt material can accommodate premature cracking could occur in the surface of the road.

The abovementioned distress can easily compromise the road safety and therefore require corrective maintenance. Since a poorly compacted pavement structure is an inherently weak structure it is likely that the distress would continue to manifest in the road surface and require ongoing corrective maintenance that is often undertaken at high cost.

From the above it can be seen that compaction is a very important process in road construction and due to its apparent simplicity it is often misunderstood and neglected.

In the following paragraphs a technical description of compaction is given as well as a description of what happens to the material when compacted.

1.7 Defining soil compaction

It is important to stress that the terms compaction and consolidation are not synonymous.

Compaction refers to the densification of unbound materials without expelling moisture from the pores whilst being subjected to mechanical manipulation. Thus the densification that takes place during compaction is instantaneous and is not reliant on removing moisture from the pores in the soil matrix.

Consolidation refers to the densification that occurs in a particular material whilst moisture is expelled from the pores. Thus essentially for consolidation to take place the material needs to be saturated. This process generally occurs very slowly whilst the material is subjected to static loading. Consolidation generally occurs in fine-grained soils (clayey material, silt and peat).

The term relative compaction is defined as the ratio of the achieved field density to a reference density. The reference density for natural soil is generally the maximum dry density at 100% Mod AAHSTO compaction, see Figure 1.13. Thus the term compaction is dimensionless and is expressed as a percentage, e.g. required compaction for G4 base material is 98% of Mod. AASHTO density. For crushed stone material the required field density is generally related to the density of the aggregate, i.e. the solid particles (expressed as 88% of bulk relative density).

Over the years specific density limits were developed empirically that should be achieved in the various layers in the pavement structure in order to obtain acceptable pavement performance. These limits are described in more detail in Section 2 and Section 8.1.

1.8 Describing soil density

Soil can be described as a three-component material. The three components are, soil particles, moisture (water) and air. The three-component material is shown schematically in Figure 1.12.

The masses and the volumes of these three components are used to describe the density and the volumetric composition of the material.

Assuming that the mass of the air is zero the total mass of the combined material comprises the mass of the solid particles and the mass of the moisture. To determine the mass of the soil particles and the mass of the moisture a sample of the material has to be dried until the mass of the sample remains constant, indicating that all the moisture has been removed from the sample. By measuring the mass of the sample before and after drying the mass of the moisture and the mass of the soil particles can be determined.

The volume of each component is determined by dividing the mass of each component by its density. The density of the soil can be measured using a standard laboratory test and the density of the water is sufficiently close to 1000 kg/m^3 to accept this figure.

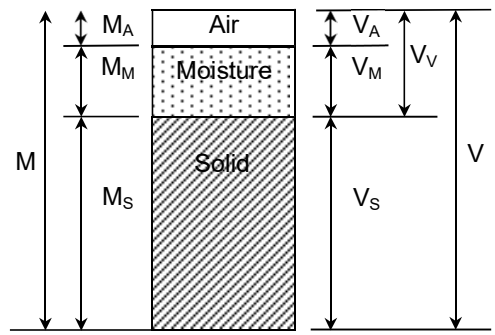


Figure 1.12 : Schematic description of the three components of the soil

The volume of air is determined by subtracting the volume of the moisture and the volume of the soil particles from the total measured volume (usually a known quantity determined from the size of the mould).

The bulk density of soil is defined as the total mass of the sample divided by the total volume:

$$Bulk\ density = \frac{M_S + M_M + M_A}{V_S + V_M + V_A} = \frac{M}{V} \quad \left[\frac{kg}{m^3}\right]$$

The bulk density can also be termed the wet density of the material. The dry density of a particular material is the mass of only the soil particles (M_s) divided by the volume of the entire sample (V).

$$Dry\ density = \frac{M_S}{V} \quad \left[\frac{kg}{m^3}\right]$$

From the above two equations it can be seen that the dry density of a particular material is always lower than the wet density of the same material. It can also be seen that the dry density can be determined if the bulk density of the material and the moisture content is known (moisture content = M_m / M_s).

The aim of soil compaction is to pack the soil particles closer together in order to reduce the porosity (V_v / V) of the material to a minimum, thus pack the maximum amount of soil particles into a fixed volume and thereby achieve the maximum dry density.

It can be appreciated that moisture provides a lubricating effect on the soil particles, which allows them to be packed closer together. However increasing the moisture content only provides benefit until such a point where the moisture content is sufficiently high to prevent the particles from packing close together without first removing moisture from the pores. This trend is depicted in Figure 1.13. From Figure 1.13 it can be seen that for a particular compaction energy there is a particular moisture content at which further addition of moisture negatively affects the achieved maximum dry density. At this point (the optimum moisture content OMC) any further increase in the moisture has the effect of pushing the soil particles apart, thus reducing the dry density and hence reducing strength.

Figure 1.13 further shows that if the compaction energy is increased the required quantity of moisture to lubricate the soil particles during compaction is reduced. However the principle as

described above is not affected, i.e. if the moisture content increases beyond the optimum moisture content the dry density starts to decrease. In addition it can be appreciated that the porosity of the material reduces as the compaction energy increases and consequently the maximum dry density of the material provided of course that the moisture content is at its optimum for that particular compaction energy.

It is important to realise that any material has an ultimate maximum dry density which is influenced by inter alia factors such as shape of the soil particles, particles size distribution, surface texture of the soil particles, clay content etc, and that increased compaction energy will not further increase the dry density of the material. In order to determine the ultimate maximum dry density of a particular material the reader is referred to the work of Semmelink.

The ultimate maximum dry density of any material can never coincide with the zero air void line as shown in Figure 1.13 as the individual particles cannot form perfect (zero void) contact areas.

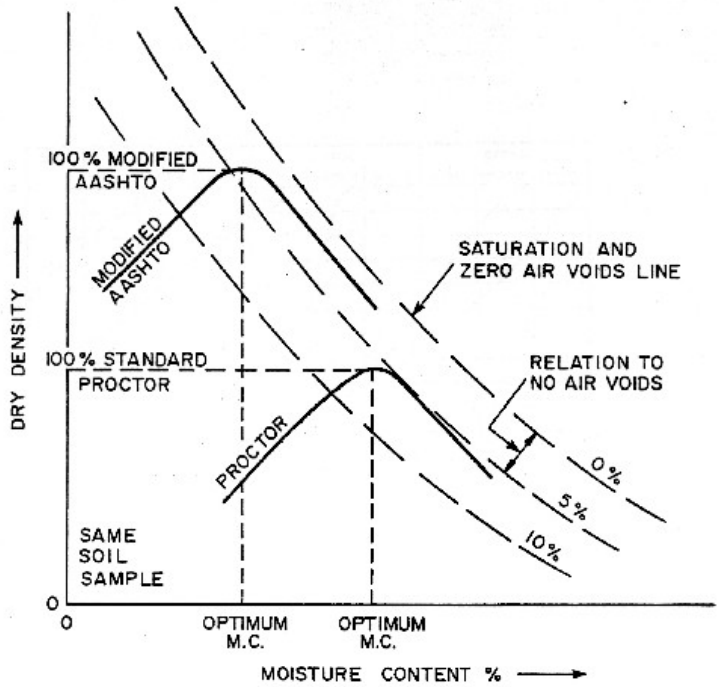


Figure 1.13 : Dry density - moisture content relationship (note the Modified AASHTO compaction is a higher energy compaction than Proctor compaction)

1.9 The compaction process

In this section the compaction process is briefly described by means of a series of photographs depicting the compaction process of crushed stone base material (Photograph No. 1.2 to Photograph No. 1.13). Although the compaction process of crushed stone base material is different from the construction of pavement layers comprising other materials the essential parts of the compaction process is essentially the same (the slushing process only applies to the crushed stone base material).

In addition photographs are also shown of the various types of rollers that are generally used in road construction nowadays (Photograph No. 1.14 to Photograph No. 1.21).

Photograph No. 1.2 : Crusher in quarry – chrushing, screening and stockpiling the base course material.

Photograph No. 1.3 : Loading the base course material in the quarry.

Photograph No. 1.4 : Weighing at weighbridge – measuring quantity of material.

Photograph No. 1.5 : Tipping base course material on the completed subbase layer.

Photograph No. 1.6 : Determining the spacing between the various loads of material using a purpose made calliper. Note that the material in this photograph is not base course material.

Because of the following reasons it is very important that the correct spacing between the loads be carefully controlled :

- Achieving the correct loose layer thickness allowing for subsequent densification to end up with the required compacted layer thickness (approximately 30% volume reduction).
- Achieving a consistent layer thickness that would obviate the need to add or remove material in certain places once the loads have been flattened and distributed by grader prior to compaction (this can be a costly exercise). The current COLTO specification with regard to base course layers requires that the <u>average</u> layer thickness be within 5mm of the specified layer thickness and that the maximum deviation does not exceed 27mm.

Photograph No. 1.7 : Wetting and mixing the base course material after flattening the heaps with the grader.

Photograph No. 1.8 : Wetting the material in order to reach the correct compaction moisture content – using the boxing process.

Photograph No. 1.9 : Mixing the base course material using a tractor drawn plough.

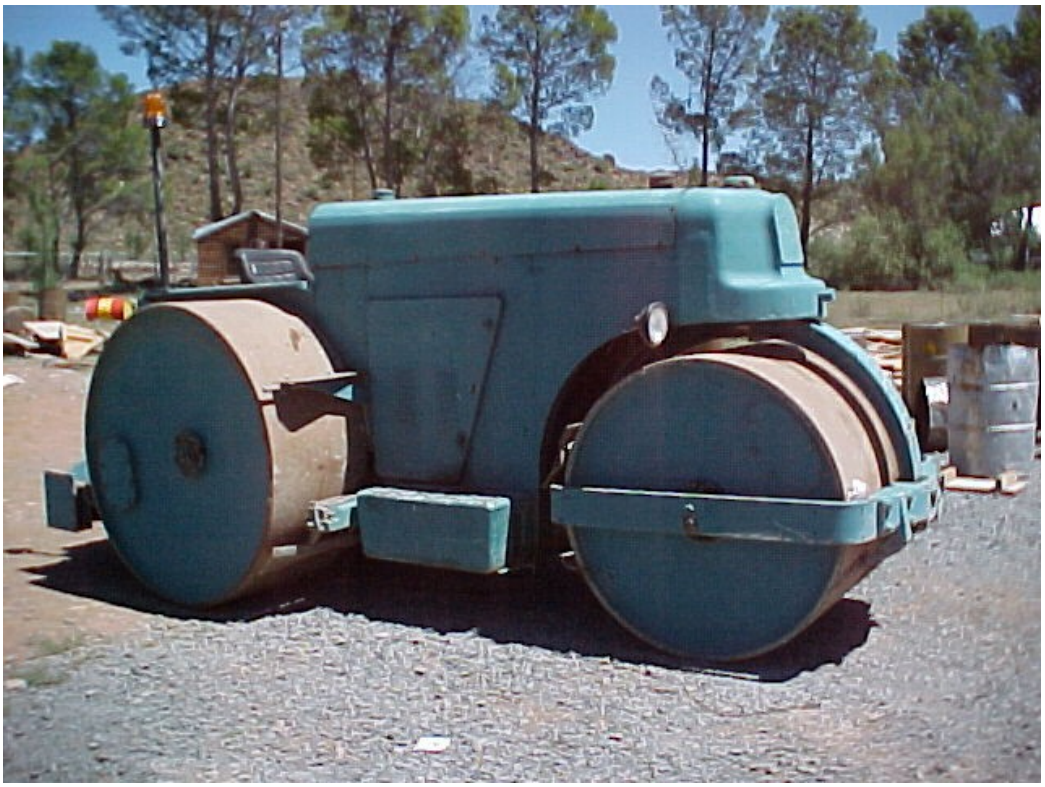
Photograph No. 1.10 : Undertaking the first passes with the vibratory compactors - after the correct field moisture content has been achieved and the material properly mixed.

Photograph No. 1.11 : Compacting the base course material using two vibratory rollers in tandem.

Photograph No. 1.12 : Slushing process in progress – water bowser wetting the base course followed by the pneumatic-tyre roller.

Photograph No. 1.13 : Mosaic of completed base course material.

Photograph No. 1.14 : Pedestrian hand-held roller.


Photograph No. 1.15 : Old static three-wheel roller (steel-wheeled).

Photograph No. 1.16 : Modern pneumatic-tyre roller.

Photograph No. 1.17 : Modern vibratory roller in action.

Photograph No. 1.18 : Padfoot roller in action – compacting cohesive soils in an earth-fill embankment.

Photograph No. 1.19 : Padfoot and steel drum vibratory rollers in action (applying the initial compaction on a 300mm in-situ recycled layer thick layer).

Photograph No. 1.20 : Gridroller – generally used to break down cobbles and other large particle sizes – see Photograph No.1.6.

Photograph No. 1.21 : Five-sided impact roller – used for deep compaction in the roadbed prior to constructing the pavement layers.

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1. Rodriguez, R., H. del Castillo and G.F. Sowers, “Soil mechanics in highway engineering”, 1988

2. Horak E., “Watergebindemacadamplaveisels” (in Afrikaans), MSc thesis, University of Pretoria, 1983

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2. SOIL CLASSIFICATION IN RELATION TO COMPACTIBILITY

2.1 Key points in this section

- Particle size
- Grading
- Consistency limits
- Fines content

2.2 Additional information

Definition of soil

There appears to be no generally accepted definition for the term “soil”. To most people soil does not include rock but technically rock should be included. For the purpose of this document soil is regarded as “Fresh rock fragments as well as the various decomposed products of rock e.g. clay”. Thus soil can be described as particulate material ranging from rock boulders (up to 1,0m in size) down to the smallest clay particle (0,002mm in size). Gravel, sand and silt form the intermediate particles sizes and they were derived from the parent rock be means of chemical or mechanical weathering. In some cases rock need to be crushed for the purposes of road building and this crushed material also falls under the definition of soil.

The definition of soil as given in the Oxford dictionary is as follows “*the upper layer of earth in which plants grow, consisting of disintegrated rock usually with an admixture of organic remains*” From an engineering perspective this definition is of little use and emphasises the difficulty of providing a universally acceptable definition for the term soil.

Soils can be classified according to the following properties:

- Grading (particle size distribution)
- Plasticity (Atterberg limits)
- Strength

There are many different approaches to classifying soils and road authorities have developed systems using the abovementioned properties specifically for road engineering purposes. Examples of these classifications are:

- Unified Classification System
- Extended Casagrande Classification System
- TRH14 Classification System (commonly used in South Africa)
- AASHTO Classification System (Highway Research Board System, A1 – A8)

TRH 14 uses the following classification system :

TRH14 Classification system

Code	Type of material	Liquid limit (max)	Plasticity index (max)	Linear shrinkage (max)	CBR (min)	Required compaction
G1	crushed stone	25	4	2	-	88% of ARD
G2		25	6	3	-	85% of BRD
G3		25	6	3	-	98% or 100% Mod AASHTO density
G4	Natural occurring soils	25	6	3	80 at 98% Mod AASHTO density	98% Mod AASHTO density
G5	Natural occurring soils	30	10	5	45 at 95% Mod AASHTO density	95% Mod AASHTO density
G6		-	3 x GM + 10	-	25 at 93% Mod AASHTO density	95% Mod AASHTO density
G7		-	3 x GM +10	-	15% at 93% Mod AASHTO density	93% Mod AASHTO density
G8		-	-	-	10 at in-situ density	93% Mod AASHTO density
G9		-	-	-	7 at in-situ density	90 to 93% Mod AASHTO density
G10		-	-	-	3 at in-situ density	90 to 93% Mod AASHTO density

Note 1 : $GM = \frac{P_{2.00} + P_{0.425} + P_{0.075}}{100}$

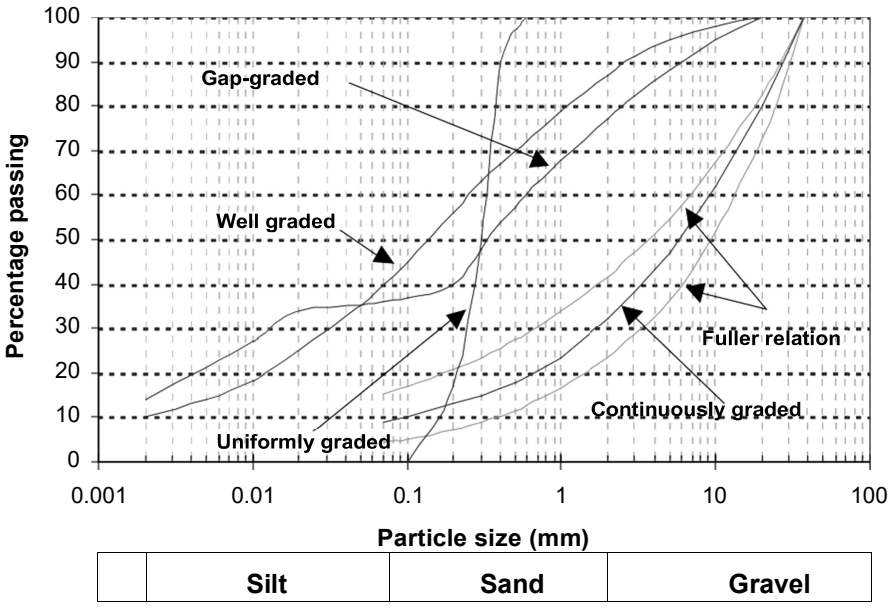
Note 2 : The density requirements are progressively increased towards the upper layers to accommodate the higher stresses that occur closer to the surface of the pavement. Also refer to Section 8 (Evaluation of compaction) for details regarding the measurement of density.

Grading

The grading of a material is an important property which influences its compactibility. If a material has a uniform grading the material particles have more or less the same size. A well-graded or continuously graded material consists of particles with a wide range of sizes. A so-called uniformity coefficient is defined as:

$$Coefficent\ of\ uniformity = \frac{d_{60}}{d_{10}}$$

where d_{60} and d_{10} are particle sizes at 60% and 10% of the cumulative particle size distribution. Uniformly graded materials have a low uniformity coefficient, whereas well-graded materials have a high uniformity coefficient, see figure below.



The continuously graded material generally follows the Fuller relation : $P = 100(s/M)^n$ where s is the individual sieve sizes, M is the maximum sieve size and n is a model parameter that ranges between 0,3 and 0,5. For perfect spheres the Fuller equation describes the particle size distribution that will provide the closest packing and thus the highest density that can be achieved with a particular material. Based on this principle it can be appreciated why the highest classes of unbound base course material generally conform to this particle size distribution as it is essential that the highest possible density is achieved and thus the highest possible strength.

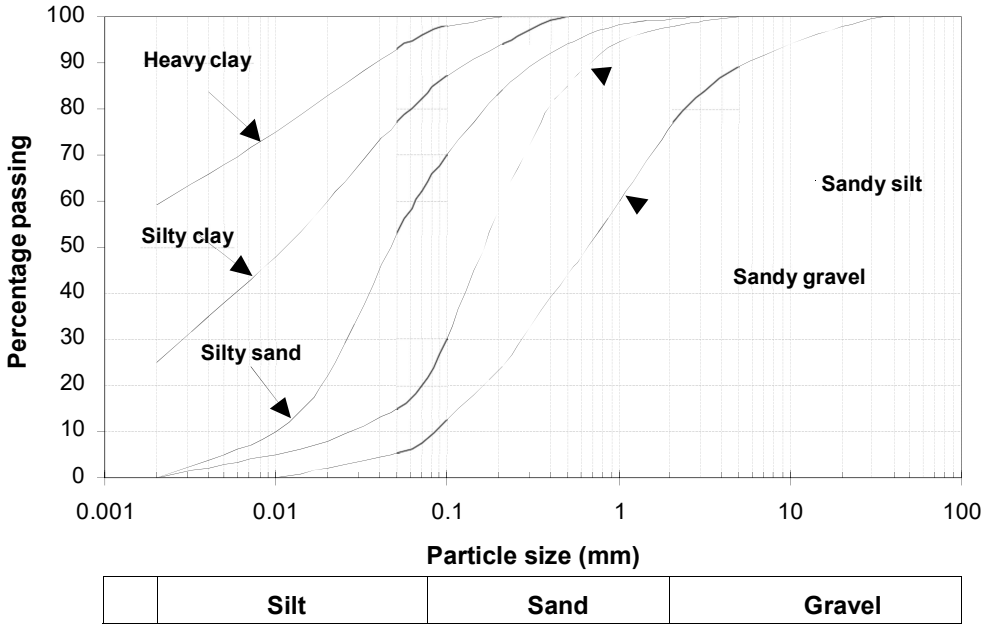
The particle size distribution can also indicate the material type, see figure overleaf.

Hardness of particles

The strength of the material particles can be determined using either the 10% Fine Aggregate Crushing Test (10%FACT) or the Aggregate Crushing Value (ACV). It can be appreciated that the higher the strength of the particles the more durable it is likely to be. The hardness of the material will determine whether the particles will break down (crumble) during compaction. Semmelink determined that the single most important factor that affects the density and the bearing capacity of a material is the grading of the material after compaction.

Shape of the particles

The shape of the particles plays an important role in the compactibility of soil. If the particles are angular and flat it will generally negatively affect the density. However angular, cubical particles contribute towards achieving a higher strength.



Compactibility

Compactibility can be defined as the ease with which the specified density can be achieved in terms of compaction energy. It is largely influenced by the following two factors :

- The sensitivity of the soil to moisture variations in relation to its compactibility
- The cohesive forces holding the soil particles together or resisting movement

This can be illustrated by comparing the compactibility of a windblown sand with a cohesive clay. The compactibility of a windblown sand is not very sensitive to moisture changes and can be compacted over a range of moisture contents. It has very low cohesion and the interparticle friction can easily be overcome by compaction, especially vibratory compaction. So easy is the compaction that specifications for windblown sand require 100% Mod AASHTO compaction. By comparison cohesive soils such as silty clays and sandy clays are difficult to compact. In the dry state relatively large lumps must be thoroughly broken up before adding the compaction water, mixing is more difficult and greater compaction energy is required to overcome the cohesive forces. It is usually impossible to achieve densities greater than 93% Mod AASHTO.

Strength

The actual strength of the material (soil or crushed stone) or rather its load bearing capacity can be assessed with the California Bearing Ratio test. A more accurate estimate of the strength of the material can be obtained using triaxial testing. A suitable standard method is the Texas Triaxial Test method.

2.3 Further reading

The following references are recommended for further reading :

- Department of Transport, *TRH 14: Guidelines for road construction materials*, 1985
- Weinert, H.H., "*The Natural Road Construction Materials of Southern Africa"*, 1980;
- Department of Transport (COLTO), *Standard Specifications for Road and Bridge Works for State Roads Authorities*, 1998

PART I : SOIL CLASSIFICATION IN RELATION TO COMPACTABILITY

The wide variation of soil properties have led to development of several soil classification systems with different purposes.

Grain size and gradation

Grain size and gradation are of great importance for the mechanical properties of a soil.
The following definitions, with reference to grain size, are used:

Blocks (boulders) Stones (cobbles) Gravel Sand	Coarse-grained particles which can be identified by ocular inspection and more exactly by sieve analysis.
Silt Clay	Fine grained particles. The particles cannot be seen visually, individually. Identified and classified by hydrometer analysis and other special tests.

The grain-size classifications systems that are used in different countries show some variations, especially in the coarse grain range, see Figure 2.1. The important border line between sand and silt is found at a grain-size of 0,06 mm, alternatively 0,074mm (US Sieve No.200).

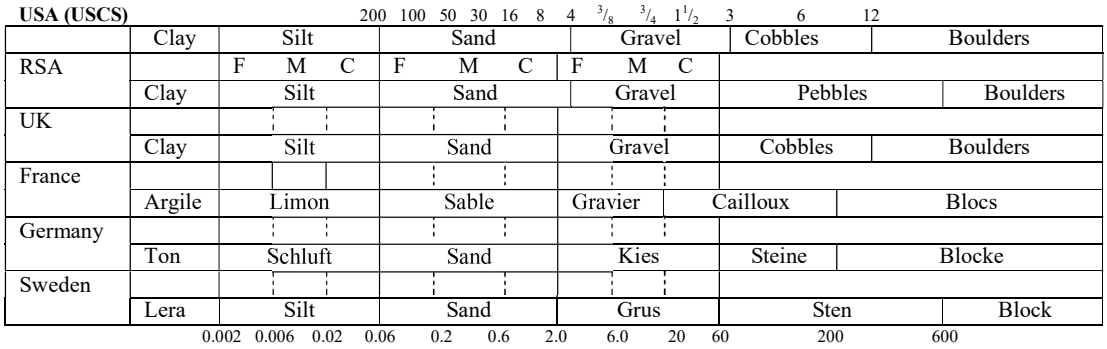
Results of sieve analysis, with a series of standard sieves, are presented in diagrams as gradation curves. The gradation curve shows the grain-size distribution and indicates whether the soil is uniformly or well graded. The uniformly graded soils, with particles of more or less the same size, have a coefficient of uniformity:

$$C_U = (d_{60} / d_{10}) \approx 5$$

where d_{60} and d_{10} are the particle diameters corresponding to values of 6 and 10% on the graduation curves.

Well-graded soils have a coefficient of uniformity C_u larger than 5. In this case the voids between larger particles are filled with smaller particles and a well-graded soil forms a more stable fill than a uniformly graded soil.

US Standard sieves



Particle size mm

Figure 2.1 : Grain-size classification systems used in different countries

Soil classification with reference to compaction

It is difficult to use conventional soil classification systems to determine the most suitable compaction method and type of compaction equipment. One important reason is that the consistency limits (liquid limit and so on) used in several classification systems are not directly related to the compactability. Broms and Forssblad have therefore proposed a classification system to be used with reference to compaction, Table 2.1.

Table 2.1 : Soil Classification

I	Rock fill and granular soils with large stones and boulders[1)]
II	Sand and gravel[1)] a Well graded b Uniformly graded
III	Silt, silty soils, etc. a Silty sand, silty gravel, moraines b Silt and sand silt, clayey sand, clayey gravel
IV	Clay a Clay with low or medium strength[2)] b Clay with high strength[3)]

Notes 1) With less than 5 to 10% of material smaller than 0,06mm.
2) Unconfined compressive strength < 0,2 MPa.
3) Unconfined compressive strength ≥ 0,2 MPa.

According to this system, the gradation of the soils in Group I, II and III are determined. For Group IV, the strength of the soil has to be measured by e.g. laboratory tests for the unconfined compressive strength. A vane or penetrometer field test can also be used. The strength should be measured at the water content which will be applicable during compaction.

Groups I and II, consisting of boulders, pebbles (cobbles), gravel or sand are non-cohesive and free-draining soils. As previously stated they are preferred as fill material and relatively easy to compact. A small amount of fines (silt and clay) can be accepted in soils belonging to Group I and II. The acceptable maximum percentage of fines varies according to particle size and other properties of the fines. A maximum percentage of particles smaller than 0,06 mm of 5 to 10% is indicated in the above classification system.

Groups III and IV have a certain content of fines. The degree of compaction that can be reached, is highly dependent on the water content. If a high degree of compaction is required, the water content should not exceed a degree of saturation of more than 80%. To compact clays (Group IV) compactors generating high static or dynamic contact pressure are required to overcome the shear resistance. It is also necessary to work with a limited layer thickness. The most suitable compaction method should be determined by the strength of the soil, which better indicates the compactability than consistency tests.

3. PRINCIPLES OF SOIL COMPACTION

3.1 Key points in this section

- Internal friction and interlock
- State of motion
- Shear stresses
- Static of dynamic pressure waves
- Capillary action of water
- Cohesion
- Frequency and amplitude
- Vibrating rollers

3.2 Additional information

During the compaction process the air component in the soil is reduced. In order to achieve this, it is required that the solid particles are moved and re-orientated into a more dense packing. To achieve a denser packing of the solid particles certain resisting forces need to be overcome. Such resisting forces may be:

- Internal friction, depending on the particle size, shape and angularity
- Internal cohesion, depending on the plasticity and the capillary action of moisture present
- Water film around the solid particles

The degree of compaction that can be achieved further depends on the amount of compactive energy applied.

3.3 Further reading

- R.D. Krebs and R.D. Walker, "*Highway materials"*, 1971
- C.J. Semmelink, *Compaction*, Course notes Road Infrastructure Course, 1988
- C.J. van der Merwe, C.J. Semmelink and C.P. Marais, *Compactability of crushed stone base course material*, die Siviele Ingenieur in Suid-Afrika, November 1988

PART II: PRINCIPLES FOR SOIL COMPACTION IN RELATION TO VIBRATOR AND OTHER COMPACTION EQUIPMENT

In practice, smooth-wheel rollers, padfoot rollers, rubber-tyred rollers and vibrating rollers are the most common types of soil compaction equipment.

In spite of all the practical experience that exists, (i.e. considerable soil compaction investigations) as well as the vast amount of available literature on the subject, knowledge of the fundamental factors and the theories of soil compaction are still far from complete.

It is therefore important to go a bit deeper into the theories of vibratory soil compaction, and in this study we will summarize some results of the research work undertaken over the last few years.

Smooth-wheel rollers and rubber-tyred rollers work mainly by a high static pressure. Rubber-tyred rollers with a mass of up to 30 tonne (3 tonne per wheel) have been used in South Africa. Because of the internal friction of the soils, a static pressure has a limited compaction effect and a limited depth of action. Especially in granular soils, increased pressure only gives an increased locking effect, as the friction forces increase with the pressure forces acting between the particles.

Explosion type rammers work on the impact principle. When a falling body meets the surface, the impact produces a pressure wave going down into the soil, causing movement of the particles and a better depth of action than static pressure. If a mass falls from a height of only 200 m against the surface of the soil, the impact force is about 50 times greater than the static pressure of the body resting on the soil. In that way, one simply gets an increase of the compaction effect compared with static loading.

In practice, however, the rammers often give too small a compaction capacity.

Vibrating rollers and vibrating plate compactors work with a rapid succession of impacts against the surface of the ground, between 12 and 80 vibrations per second (12 to 80Hz) depending on the type of machine.

From the surface, pressure waves go down into the soil. The particles are set in motion and in this state, the internal friction is overcome.

The special effect of vibration is obtained and the particles, in a state of motion, are relocated and compacted to a high density. During the rather free motion of the particles, small grains can fill up the spaces in between bigger grains and all particles re-align so as to cause the void volume to be as low as possible.

With every pass on the surface, a vibrating roller transmits a number of impacts to the ground. Vibration can, in this way, give faster compaction.

A very important factor is that compared with static rollers, vibration makes it possible to use much smaller and lighter equipment, and less water for lubrication.

Research results

Laboratory investigations and field compaction tests have indicated that the results achieved in vibratory soil compaction are primarily dependent upon two factors:

1) **State of motion** of the soil particles. **The internal friction** is eliminated and conditions created for an effective compaction of the soil.
2) **Pressure and shear stresses** generated during compaction. Pressure may be **static** arising from the mass of the soil and of the compactor. Or **dynamic** in the form of pressure waves generated during the vibration process.

The effect of the vibration on the internal friction in soils of different types is a question of great interest. Laboratory tests carried out with a vane borer showed that the internal friction in the soil was largely overcome by the effect of vibration. This applied to all types of soil investigated –

stones, gravel, sand, silt, moraine and also clay. During vibration the shear strength of gravel and sand was reduced to values ranging from 1 to 8% of those at rest. With the exception of clays, the internal strength increased from the very low value during vibration to the rest value, immediately the vibration stopped.

The internal strength of clay was also considerably reduced during vibration. Moreover, vibration caused a gradual and permanent breakdown of the internal strength of clay – a decrease in strength which remained after vibration has been discontinued. This is connected with the sensitivity of clays to mechanical working. It must be observed that different types of clays have very different properties as regards the degree of sensitivity.

The retardation or deceleration of the drop weight during impact on the top surface of the compacted soil proved approximately to be a direct function of the velocity of the drop weight at the instant of collision with the soil surface.

The maximum force acting on the soil surface (mass x retardation) was, therefore, a direct function of the momentum (mass x velocity) of the drop weight.

Impacts on loose soils resulted in values of retardation which were as low as 15 to 20% of the corresponding values during the impact on the same soil in a compacted state. It was found that a fundamental difference existed between the soil properties before and after compaction.

The pressure waves in the soil which were propagated from the soil surface were recorded by means of velocity, acceleration and pressure measurements at different depths in the soil. The amplitude of the pressure waves in compacted soils was an approximately linear function of the momentum of the drop weight. In loose soils, again a relation was observed between the amplitude of the pressure waves and kinetic energy of the drop weight.

Furthermore, these measurements showed that in compacted soils, which in the main possess elastic properties, the dynamic pressure can be calculated to a relatively high degree of accuracy in conformity with Boussinesq's theories of propagation of stresses due to static loads on the soil surface. The reduction in dynamic pressures with increase in depth was not influenced to any notable extent by the type of soil on condition that the soil was compacted to a high unit weight and hence exhibited an elastic behaviour.

An increase in plasticity tends to an increase in damping of pressure waves and the damping in a plastic clay was greater than in a compacted non-cohesive soil. A low degree of compaction of the soil also entailed an increase in damping of pressure waves.

The influence of the state of motion during vibration is of the greatest importance for the compaction of non-cohesive soils such as sand and gravel. The elimination of the internal friction in combination with the effect of gravity can be sufficient to cause a resettlement of the soil and thereby create a good compaction effect.

This applies to completely dry and completely saturated non-cohesive soils and, in another field of use, to concrete.

For efficient vibratory soil compaction it is, however with few exceptions necessary to combine the **state of motion** with **pressure** and **shear stresses** of a certain magnitude.

The rotation effect of the eccentric moment of vibratory roller tends to produce both pressure and shear stresses.

These stresses are necessary to overcome cohesive forces of a capillary nature which otherwise oppose resettlement of the soil. Water films bound by capillarity act as "elastic ties" between the particles, see Figure 3.1. This **apparent cohesion,** which exists also in sand and gravel, increases rapidly with smaller particle size. Clays have in addition a true cohesive force due to molecular forces or van der Waals forces.

The greater the cohesive forces, the greater will be the pressure and shear forces required to secure resettlement and compaction. In sand and gravel, with water contents between dry and saturated, pressures of 50 to 100 kPa in combination with the state of motion have been found necessary, if satisfactory compaction is to be achieved. The minimum degree of compaction required has been taken as 90% Mod AASHTO density.

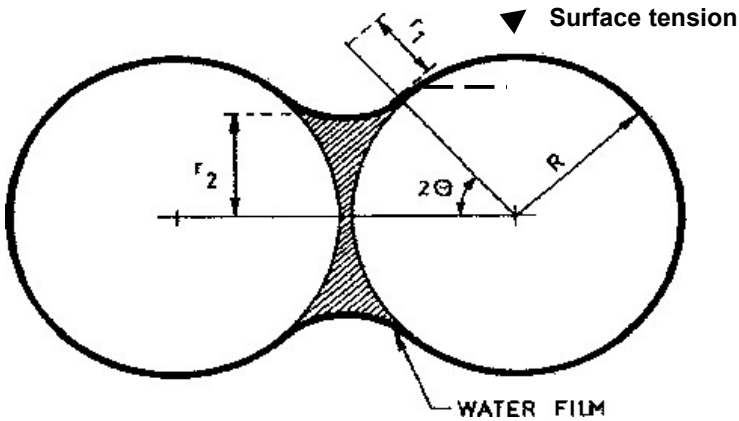


Figure 3.1 : "Elastic tie" of capillary water between two particles

Frequency and Amplitude

In clay soils the magnitude of the pressure forces is the most important factor as regards compaction, and the necessary pressures (static or dynamic) for compaction to 90% Mod AASHTO are, according to laboratory and field compaction tests, of the order 400 to 700 kPa. Pressures of this magnitude are secured with, for example, heavy tractor-towed vibrating rollers with a certain weight and also vibrating tampers.

The importance of static and dynamic pressures in compaction work has meant that the development of suitable methods for measuring such pressures has been an important part of the research work described in this report. Measuring instruments of membrane type have been used, the pressures being recorded with strain gauges.

The relationship between pressures measured in the soil and the frequency and amplitude of the vibrations is illustrated in Figure 3.2. The pressures in the soil have been found to increase noticeably at frequencies up to about 1500 vibrations per minute. At frequencies between 25 and 50 Hz the ground pressures increased less markedly, as frequency increased. The amplitude is of great importance for the generating of large ground pressures. Considering the effect of resonance in the vibrator soil system we obtain the theoretical relationship shown in Figure 3.3 which agrees in principle with the measurement results shown in Figure 3.2. The effect of ground resonance justifies the choice of a frequency for soil vibrators which is somewhat above this resonance frequency.

It can be very misleading to state frequency as a measure of compacting effect. The centrifugal force is also often used as a measure of efficiency. The centrifugal force is approximately proportional to the acceleration (the amplitude multiplied with the **square** of the frequency). This can be a still more misleading figure as regards compaction effect.

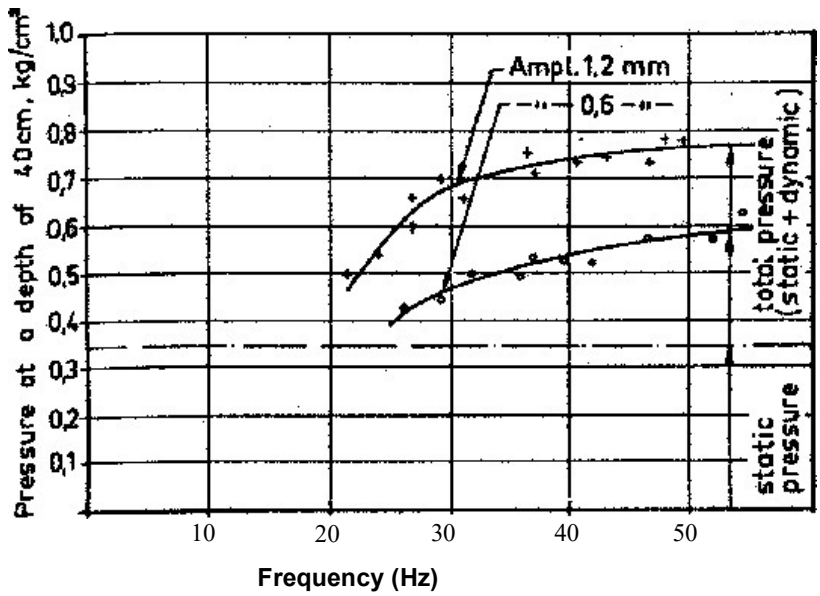


Figure 3.2 : Pressures at a depth of 400 mm obtained with a vibrating roller working with different frequencies and amplitudes

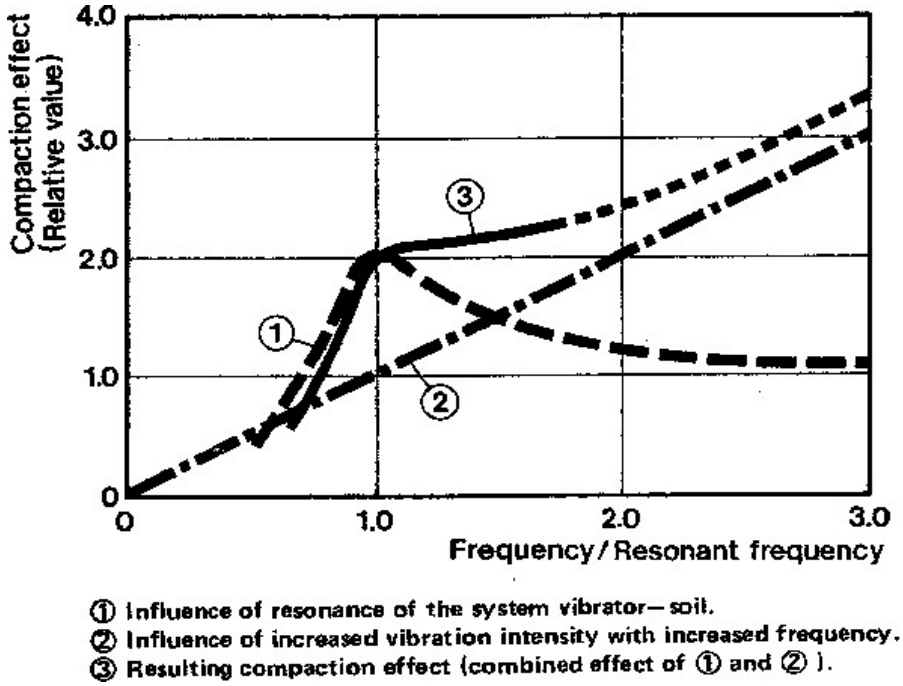


Figure 3.3 : Theoretical relationship between compaction effect and frequency

Practical applications

For soil compaction vibration was first used in Germany as early as the 1930's. It was also found that vibration gave rapid and good compaction. At first, vibratory compactors were made with a

vibrating bottom plate and had a relatively low speed and capacity, which limited their field of use. Vibration was not at that time considered to be a suitable method only for non-cohesive soils – sand and gravel.

The development of vibrating rollers was a great advance and in this way a considerable increase in compaction capacity was possible and also because vibratory compaction could be used on a wider range of soil types. Vibrating rollers nowadays are used for the compaction of rock-fill, crushed rock, gravel, sand, moraine, silt and also for clayey soils.

Vibrating rollers, normally with a mass of between 3 and 20 ton, have a very good compaction effect and a high working capacity. They are used for the large compaction jobs – road and airfield constructions, earth dams, and so on.

Self propelled vibratory rollers have mostly replaced the old 3 wheel rollers for both soil and asphalt compaction. These rollers are generally fitted with driven drums and dual amplitudes. Vibrating rollers today range from small manual push along dual drum or tandem rollers to massive 20 tonne self propelled rollers.

Vibrating Padfoot rollers

Vibrating rollers of this type are primarily intended for the compaction of clayey soils which, especially when the water content is low, demand considerable compactive effort and greater pressure to achieve a satisfactory result. The padfoot design gives a concentration of the static and dynamic forces exerted by the roller against the ground. Compared with static padfoot rollers, the vibrating padfoot rollers give the specified compaction with much fewer passes. The effect of vibrating padfoot rollers can be summarized as follows:

1) Through the concentration of static and dynamic forces achieved with the padfoot all hard lumps are quickly crushed and pulverized. The effect is very favourable on materials such as dry, hard, clay soil. Decomposed or weathered rock can also be mentioned.
2) The mixing and kneading action of the padfood ensures a homogeneous fill, especially if the fill consists of different types of soil.
3) A good binding is secured between the successive layers. Tendencies towards lamination are avoided. This is especially appreciated by dam constructors.
4) A special effect is the pressing in of stones from the surface of the fill, when compacting materials with high stone content with a vibrating padfoot roller.

Design of vibrating rollers

The modern self-propelled single or tandem dual drum rollers are made up of several operating parts:

1) A vibrating or static drum or drums generally of 1,5 to 2,2 m in width; 1,2 to 1,6 m in diameter with a shell thickness of 22 to 60 mm. The drum may be smooth or fitted with trapezoidal pads either permanently welded to the drum or fitted with padfoot shells bolted onto the smooth drum and tack welded to prevent slippage.
2) A steel mounting or module frame of mass to balance the frequency and mass of the drum. This frame is fitted to the drum by means of rubber pads which confine the vibrations to within the drum itself.
3) An engine coupled to an oil pump and the drive wheels all within its framework.
4) An oil driven motor within the drum which drives the drum, in synchronism with the drive wheels.
5) An oil driven motor inside the drum which drives a mass that rotates about an axis offset from the centre of mass thus producing an eccentric moment resulting in vibration of the drum. This motor controls the frequency of vibration and by reversing its rotation can provide dual amplitude.

Smaller, hand operated dual drum vibratory rollers are also used for street repair jobs, side-walk construction and compaction of trenches.

Vibrating plate compactors

Small vibrating plate compactors are also used for street repair jobs, side-walk construction and compaction in trenches.

A machine of this type has a compaction effect in sand and gravel down to a depth between 40 and 400 mm depending on the size of the plate compactor. The working speed of vibrating plate compactors is about 20 m/min.

Vibrating tampers

A vibrating tamper, works with a lower frequency than vibrating plate compactors. The frequency of the tamper is about 10Hz. The stroke height is 40mm. That means that the surface is the object to impact forces which produce rather high pressures in soil, especially in proportion to the low mass of the tamper. This mass is only ±70kg. With this impact effect very good compaction results are also obtained on clays and clayey soils. The low mass of the vibrating tamper makes it very suitable for compaction in trenches and other narrow spots.

Asphalt compaction

Vibrating rollers have, for 40 years or more, been used for asphalt compaction. Investigations on vibratory compaction of asphalt were started in Germany many years ago. Very good compaction results were obtained with a low number of passes. The evenness of the surface was the same as obtained after rolling with static smooth-wheel rollers. Vibratory compaction of asphalt is today very popular and vibratory rollers are almost always used in combination with pneumatic tired rollers (PTR).

PART III: STATIC VERSUS VIBRATORY COMPACTION

In principle there are three separate means by which we densify various materials in order to enhance their physical / structural properties, these are :

1) Static (pressure).
2) Impact (in dynamic applied pressure).
3) Vibrating (particle excitation).

Figure 3.4 gives a graphical description of the three types of compaction.

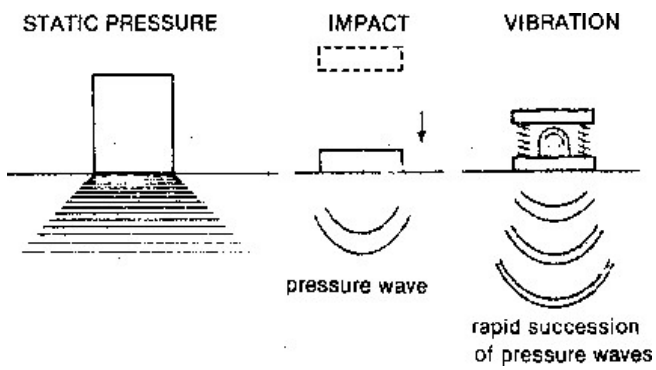


Figure 3.4 : Symbolic representation of compaction types.

It would certainly be of value to give examples of each mentioned factor.

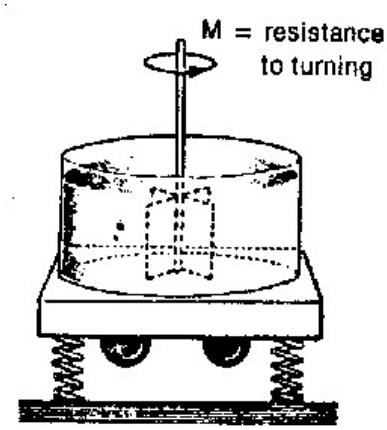
1) Compaction achieved by the “static” or “dead” weight method only, is perhaps the best known and certainly the most widely understood by both those involved in construction and laymen alike. A classic static compactor is the “steam roller” design in common use for well over 140 years.
2) Impact compaction – or ramming – has been employed throughout the ages and today is usually represented by small engine powered hand held rammers. Development of the impact roller, of which three sided, four sided and five sided models exist, has encouraged the use of this type of compaction in road construction, where narrow strips do not hamper turning as these rollers are large and cumbersome to turn. They are drawn over the surface by powerful tractors.
3) Examples of compaction by vibration alone are not easy to give. The most obvious is the state of particle mobility achieved – and, therefore, consolidation - when concrete is treated with a vibrating needle or “poker”. Inter-particle friction is reduced by suitable vibration frequency and amplitude resulting in the concrete becoming so fluid as to pack to maximum density under the influence of gravity alone.

While the above three types of compaction techniques fundamentally separate in their action on construction materials, it is usual for compaction plant to employ, in varying proportions, a combination of all three.

This is especially so in the case of vibratory rollers and therefore, the mechanical aspects of these rollers will be further examined on the assumption that all basic principles of compaction are encompassed in doing so (the exception would be pneumatic rollers which will be dealt with later).

The effect of vibration on the internal shear strength of various materials during vibration has been measured, see Figure 3.5. For example, a well-graded gravel can present 42 times less resistance during the Vane borer test with vibration than without.

From the table in Figure 3.5, it can be seen that, as soils become progressively finer or more clayey, the effect of vibration on the moment of resistance of the Vane borer becomes less marked. It is possible that the shear strength of a soil can in fact lessen as a result of vibration.



Material	$\frac{M_{vibr}}{M_{rest}} \cdot 100$ per cent
Dry or water saturated sand or gravel	2 %
Sand or gravel at natural water content	5 %
Sandy silt at optimum water content	10 %
Silt at optimum water content	20 %

Figure 3.5 : Vane apparatus tests in different types of soils placed in a mould on a vibrating table (tests with and without vibrations)

These factors must be considered when deciding on the best mode of compaction to be employed on various soils. In the case of clay, experience has shown that high static and dynamic forces provide the most favourable results in achieving compaction. The effect of vibration on the relocation of very fine particles is extremely limited due largely to elastic “water-ties” between them. These have a dramatic dampening effect on vibratory pressure waves and thus inhibit particle mobility. It would be of great interest to contractors to consider the implication of the results as shown in Figure 3.1.

While the gravel tested showed remarkable reaction to vibration, it does not necessarily follow that acceptable compaction will occur in practice, especially if the gravel were to be used as a base or sub-base layer. High degrees of particle excitation do not always result in acceptable surfaces or control layers during construction. It is, therefore, necessary to select the type of vibration applied to the material at hand. Returning to clay – as stated it would be desirable to lay emphasis on the static and dynamics modes of compaction rather than the vibratory. If a vibratory roller were to be used on clay, it should have the following characteristics:

1) Be of “pad” of sheepfoot form – thus increasing the static point loading of the available mass and avoiding bridging of the materials surface.
2) The vibration should be characterised by high amplitude, thus gaining maximum impact on the materials surface. This impact should be imparted at a moderate frequency so as to, as near as possible, duplicate the “ramming effect” as illustrated in Figure 3.4.

It is clear that if the above vibratory characteristics of high amplitude imparted at a moderate frequency were to be employed on gravel layers with high particle mobility, a poor surface could result with a loose layer forming on that surface. Opposite modes of vibration than those needed for clay are therefore, desirable – i.e. relatively low static pressures combined with a low vibration amplitude at a high frequency.

Vibratory rollers are now made that have variable frequency and amplitude. However, research has shown that “compaction effect” is influenced most by amplitude and to a far lesser degree by frequency. See Figure 3.6.

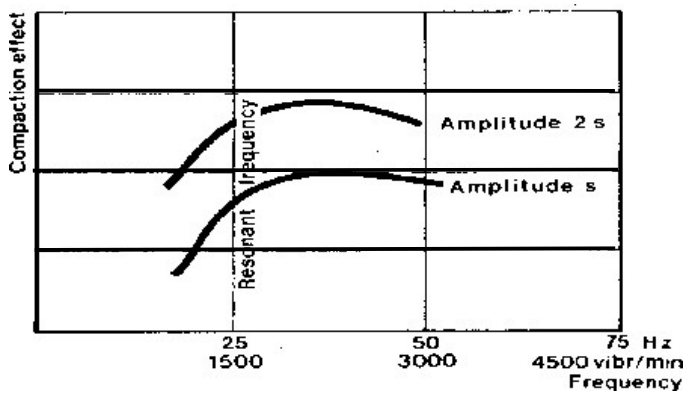


Figure 3.6 : Normal relationships between compaction effect, frequency and amplitude

Those factors relating directly to compaction effect on all types of rollers (except pneumatic) are listed below:

1) Linear static mass (as kg / cm width of roll). This is of particular importance when describing the compaction effect of a static roller as in this case static pressure is the only linear mode of compaction. The diameters of the rolls also play an important part in the area of steel on the material at a given time – thus influencing the applied pressure.
2) Travel speed – this is usually entirely under the control of the operator.
3) Amplitude – in the case of vibrating rollers.
4) Centrifugal force – in the case of vibrating rollers but only under given conditions.
5) Vibratory frequency.
6) Number of compaction passes applied to the material undergoing compaction.

All six points above are seemingly obvious, however as descriptions of a machine's compactive ability are misleading and meaningless.

As in Figure 3.6 – research has shown that an increase in vibrating frequency does not create a direct corresponding degree of compaction. Increasing frequency does, however give an increasing value of centrifugal force.

The formula for centrifugal force shows that an increase in frequency increases the centrifugal force by the square of the additional frequency.

$$CF = 4\ \pi^2.f^2.mr$$

Where: CF = centrifugal force (kN)
f = frequency (Hz)
mr = eccentric moment (kgm)

Figure 3.6 shows clearly that at frequencies between 25 and 50 Hz this no longer holds true. It should be stressed that Figure 3.6 applies to a specific material undergoing tests. Other materials exhibit more suitable reaction to compaction by vibration at somewhat lower or higher frequencies. Figure 3.6 also clearly illustrates the influence of amplitude on compaction effect. From Figure 3.6 it is clear that when the frequency increases from 25 to 50 Hz and the compactive effort or energy force remains unchanged, the eccentric moment (mr) must reduce correspondingly. This implies that the effective amplitude and the mass of soil affected must reduce. It follows that the small amplitude affects a smaller soil mass i.e. works at a shallower depth or thinner layer.

Understanding Eccentric Moment

The eccentric moment of a vibrating roller may be defined as the product of the rotating mass within the drum and its radius of gyration. If friction or other energy losses are discounted, this can be equated to the product of the drum mass and its amplitude. Here the amplitude is termed nominal amplitude as no further energy is applied to the soil mass in effect. When the drum rests on a soil, particles of soil are also vibrated and the product M.A. (mass times amplitude) is the mass of the drum plus that of the soil effected times a reduced amplitude (field amplitude) to equal the energy applied. The field amplitude of the roller drum is thus always less than the nominal amplitude which is based on the roller in suspension.

Less Adequate Parameters For Discussing Vibratory Rollers

Centrifugal force

It has been shown that no relationship exists between the centrifugal force and the vibrating force transmitted to the ground. The centrifugal force increases with the square of the frequency, while the real vibrating force is mainly dependent of the amplitude.

The centrifugal force or better, the nominal centrifugal force, gives, however, a general measure of the vibration intensity (acceleration) of the drum. A vibratory roller should not have a lower drum acceleration than four or five g's (gravitational acceleration). The acceleration value can be calculated with knowledge of centrifugal force and drum mass. At equal static mass and frequencies, the centrifugal force can also be used for a direct comparison between two roller models, since it then shows the relative difference in nominal amplitude.

As the centrifugal force does not represent the real dynamic force, the expression "dynamic force" should definitely not be used as a denomination for centrifugal force.

An acceleration of more than 6 g's is not acceptable in general as the force can damage the drum suspension rubbers or mechanism. As acceleration is proportional to amplitude and the square of frequency high amplitudes are thus confined between 29 to 40 Hz and only small amplitudes are produced when the frequency is greater than 50 Hz.

Total applied force

The total applied force (TAF) defined as the sum of static weight and centrifugal force has earlier been widely used, mainly by American roller manufacturers. As the centrifugal force is not related to the compaction effect, it is evident that the total applied force cannot give a correct measure of the compaction effect. The US Construction Industry Manufacturers Association (CIMA) has recommended that the measure TAF should not be used.

Equivalent static weight

The principle of vibratory soil compaction, the state of motion of the particles combined with dynamic pressure, is quite different from the principle of static compaction. It is clear that this makes it very difficult to find exact relationships between the compaction effect of a vibratory roller and a static compactor. In spite of this, a vibratory roller is sometimes said to have the same compaction effect as a static roller of a certain higher weight. Such a comparison can only be made if the two rollers are exactly specified with regard to weight, dimensions, and so on. The soil conditions must also be exactly specified. An agreement has been made among the European manufacturers of vibratory compactors (Committee for European Construction Equipment, CECE) not to use comparative weight figures as the above.

Terminology and standards

In roller compaction specifications, a generally known and accepted terminology and standard should be used. Standardized terms should also be used by manufacturers of compaction equipment in their technical specifications and catalogues to make it possible for the users to compare, in technically correct way, different models of compactors.

Pneumatic Rollers

Pneumatic rollers and mass

Over the years rubber tyres have proved of value in compaction, but all rubber tyred rollers are not equally efficient even at equal gross mass. It is general practice when speaking about rollers to classify them by their gross mass, but to use this term with multi-wheeled rubber-tyred rollers does not give a true picture of their compactive effort. For example, if we take a 15 tonne pneumatic tyred-roller with seven wheels we have a load on each wheel of 2142kg, but the same weight of roller with nine wheels has a load of 1666 kg on each wheel, and it is obvious from this that different results would be obtained from the two machines. The size of tyre used to carry the load, and the inflation pressure range of the tyre, also control the end results. All these points must be taken into consideration when buying or hiring a roller.

Influence of wheel load and tyre inflation pressure

Wheel load and tyre inflation pressure control the behaviour of the contact pressure and contact area of the section of the tyre which is in contact with the ground, and as work done is applied at this point it is worth while considering the reaction of the tyre to any change in wheel load or inflation pressure. As the product of the contact pressure and contact area are roughly equal to the wheel load; WL = CP x CA, it follows that to reduce the contact pressure with a given wheel load the inflation pressure must be reduced, resulting in an increased contact area of the tyre supporting the load. Inversely if the inflation pressure is increased the contact pressure increases and the contact area is reduced.

Turning now to the wheel load changes, it will follow that an increase of wheel load on a given inflation pressure will give an increase of contact area, and only a slight increase in contact pressure, as seen in Table 3.1 inversely, when the load is reduced on a given inflation pressure the contact area reduced proportionately.

Table 3.1

TYRE INFLATION PRESSURE	WHEEL LOAD 1 000 kg		WHEEL LOAD 2 142 kg	
kPa	CONTACT PRESS. kPa	CONTACT AREA cm^2	CONTACT PRESS. kPa	CONTACT AREA cm^2
137	199	490	213	967
275	330	283	330	638
413	441	225	455	464
515	524	187	565	374
689	606	161	655	316

It must be appreciated that the contact pressures and areas given in Table 3.1 are produced by tyre manufacturers, and are measured on a rigid surface, whereas in the field the compactor first operates on the loose surface of the fill. Immediately the machine travels over the loose fill, all relation to the figures in the table are lost, and it then becomes a matter of experience in selecting the wheel load and inflation pressure to suit the material throughout the compaction range.

Tyre penetration in the fill material occurs on the initial passes over any newly laid fill because the shear strength of the material in its loose condition is exceedingly low, and the tyre contact area obviously increases as the tyre “rides-in” the material. The tyre contact pressure is automatically

reduced by the load being spread over a greater area. Then, as the bearing strength of the material increases by further passes of the machines, so the tyre contact area with the material reduces, and the tyre contact pressure inversely increases. In other words, the load on the tyres remains the same, but the way the load is applied – contact area and pressure – changes automatically as the material increases in strength and density, until finally there is little or no penetration, and the contact areas and contact pressures are then approximately around the figures given by the tyre manufacturers.

Having discussed the behaviour of a tyre in relation to the load and inflation pressure the reaction of the load and contact pressure must now be considered, and for this purpose the area of contact, tyre with ground, is always assumed to be circular, whereas actually it is more elliptical in shape.

Choice of wheel load: influence on layer thickness

While it is possible to produce the same contact pressure with different wheel loads, it is a fallacy to think the same amount of work is being done because of similar contact pressures. For example, take a tyre from the range of pneumatic rollers:

Tyre Size	18,00 x 24	
Wheel load	2142 kg	5500 kg
Inflation Pressure	620 kPa	620kPa
Contact Pressure	606kPa	638 kPa
Contact area	342 cm^2	826 cm^2

It will be seen from the above example that the inflation pressures are equal and contact pressures are nearly the same. The heavier wheel load is approximately two and a half times greater than the smaller one, and the contact areas also differ in the same proportion. One can see from the above figures that extra work is done by the heavier wheel load since it is covering a larger area at the same pressure. There is another factor to be considered, namely, the influence of the size of the loaded area on stress penetration. Theoretical studies by Boussinesq, Jurgensson and others have shown that the greater the loaded area at a given contact pressure, the greater is the stress distribution at a given depth than that of the smaller loaded are as the same contact pressure. In other words the effects of the heavy load go much deeper than the lighter load, even on equal pressure contact, as in Figure 3.7 showing the theoretical pressure bulb under the load area on which the value of the stresses at A. and A.1. are equal, but the depth below the surface of A. is much greater than A.1., showing that the intensity of stress distribution at a given depth depends on the size of the loaded area and that it is directly proportional to it.

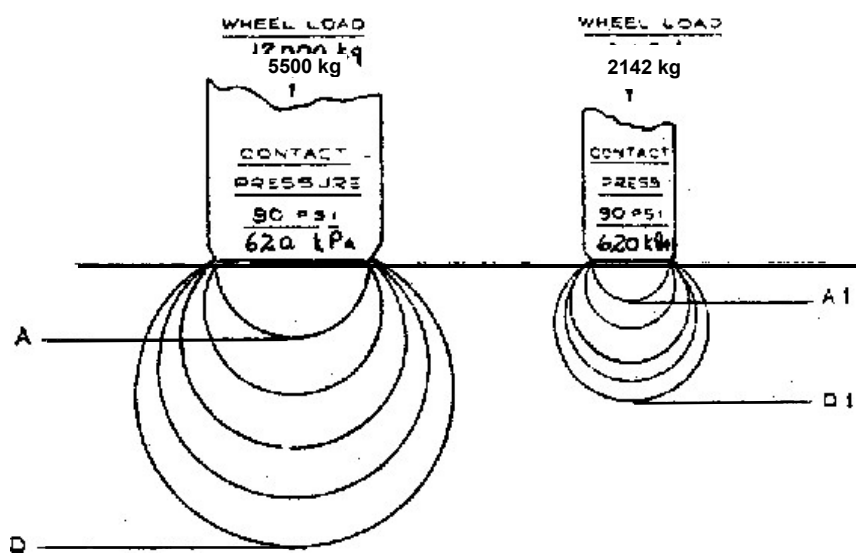


Figure 3.7 : Theoretical pressure bulb under loaded area

Therefore, if compaction at depth is required, heavy equipment is used. With the lighter wheel loads the layer thickness of fill material for compaction is reduced to bring the whole thickness of the layer under the influence of the pressure bulb and stresses created by the smaller loaded area. Therefore, it follows that close attention must be paid to the layer thickness relative to the capacity of the roller being employed.

A final word on wheel load and inflation pressure: where sand ballast is used, a check must be made occasionally for loss of weight caused by the drying out of the moisture and leakages of sand as these losses affect the wheel load. Constant checks must also be made on tyre inflation pressure if constant compaction results are required. Any loss in the selected pressure would give an increase in the constant area with resultant drop in contact pressure.

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4. THE INFLUENCE OF MOISTURE

4.1 Key points in this section

- Optimum moisture content
- Applied energy level
- Maximum dry density
- Moisture – density relation
- Water as lubricant
- Vibratory compaction
- Clay content
- Heaving

4.2 Additional information

The moisture content is defined as the mass of moisture present in a unit volume of soil expressed as a percentage of the mass of the solids in that same unit volume:

Moisture content (%) = (Mass of moisture / Mass of solids) x 100

The relation between moisture content and density was first studied by Proctor, which resulted in the definition of a so-called optimum moisture content (OMC) at which moisture content the maximum dry density as a result of the application of a certain amount of compaction energy has an optimum.

The correct moisture content for compaction under field-conditions differs therefore from the optimum moisture content determined in laboratory compaction tests (such as Proctor and Modified AASHTO test), since the applied compaction energy level in the field is usually much higher than in the laboratory. It is therefore not sensible to specify a moisture content at which the material is to be compacted, if this moisture content is based on laboratory compaction tests, as contractors will experience difficulties in achieving density at this moisture content. It will depend on the moisture susceptibility of the soil, how much lower the optimum moisture content under field conditions is and operator experience. Engineering judgment is required to determine the correct moisture content for field compaction.

The clay content is an important factor determining moisture susceptibility and plasticity of a soil. Clay particles are very small and therefore a unit volume of clay has a very large surface area. The area of the water film surrounding the clay particle is thus also very large. In addition to this, depending on the mineralogical / chemical composition of the clay particle, these particles can hold thick water films (thicker than the particle diameter). Because of these thick water films, clay particles can easily shear over each other without breaking the interconnecting water films. This results in a plastic appearance. At higher moisture contents the shear resistance is reduced to such an extent that the clay can deform under its own weight and obtain a viscous appearance. The moisture content at which the appearance of the soil changes from plastic to viscous is called the liquid limit.

When the moisture content of a plastic soil is reduced, shearing of the clay particle without breaking the interconnecting water films is not possible anymore and the water films will break. Soil will then become friable when it is worked. The moisture content at which this change takes place is called the plastic limit.

The plasticity index (PI) of a soil is defined as the difference between the moisture contents defining the liquid and plastic limit. Standard test methods are available for the determination of these limits (Method A2 and A3 of TMH1).

4.3 Further reading

- D Croney and P Croney, Design and performance of road pavements, 1998
- Road Research Laboratory, *Soil mechanics for road engineers*, 1952
- C.J. Semmelink, *Compaction*, Course notes Road Infrastructure Course, 1988
- Prof P. Everitt, *Moisture in soils*, notes compiled for University of Natal
- SJ Emery, Prediction of moisture content for use in pavement design, PhD dissertation, University of the Witwatersrand, Johannesburg, 1985.

PART IV: THE RELEVANCE OF MOISTURE DURING COMPACTION

During the process of compaction it is important to bear in mind that the moisture content deemed as being optimum is optimum *only for the load applied and under the given conditions for which it is applied.*

It is a well known fact that if we increase the number of blows, for example, during the Mod AASHTO test from 55 to say 105, the optimum moisture content will be lower and the *peak* dry density obtained, higher. Conversely a lowering in the number of blows would produce a higher optimum moisture content and a lower peak, see Figure 4.1.

Furthermore the dry density at A which can only be obtained at the peak moisture content B for 55 blows can readily be obtained with 105 blows at a much lower moisture content C well below its own peaking moisture content D.

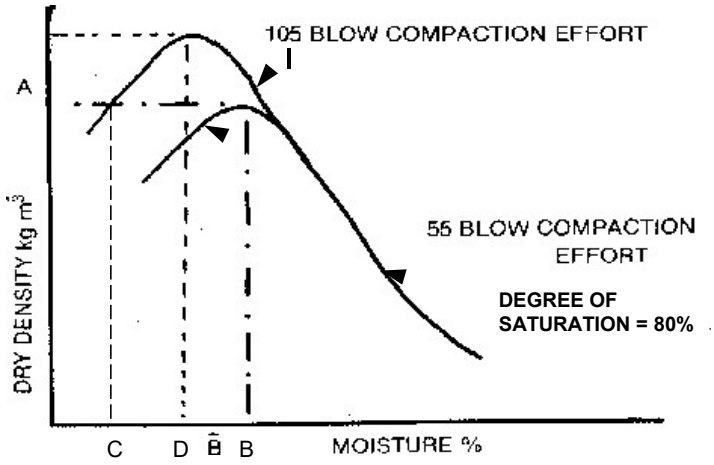


Figure 4.1 : There is a different moisture content for each effort

The advantage of applying a greater load to the material undergoing compaction is that a wider range of moisture contents can be used to obtain a given density. We often see compaction densities well in excess of 100 % even though the soil was not compacted at so-called Mod AASHTO optimum moisture content.

The advantage of a high compactive effort such as we achieve with vibratory compaction is illustrated in Figure 4.2. We can see that to obtain 95% compaction with a high compaction effort we have a moisture range possible between the value X and B, whereas with a lower compaction effort, our range will be much less, i.e. moisture content ranges A to B.

It is well known that it is not ideal to compact at a moisture content that is in excess of the optimum for the required density. So in practical terms, our working moisture content will not be the entire range i.e. X to B or A to B, but be to the left or the dry side of optimum. This illustrates even further the need to impart a high yet non-destructive compactive effort into the material concerned. Figure 4.2.

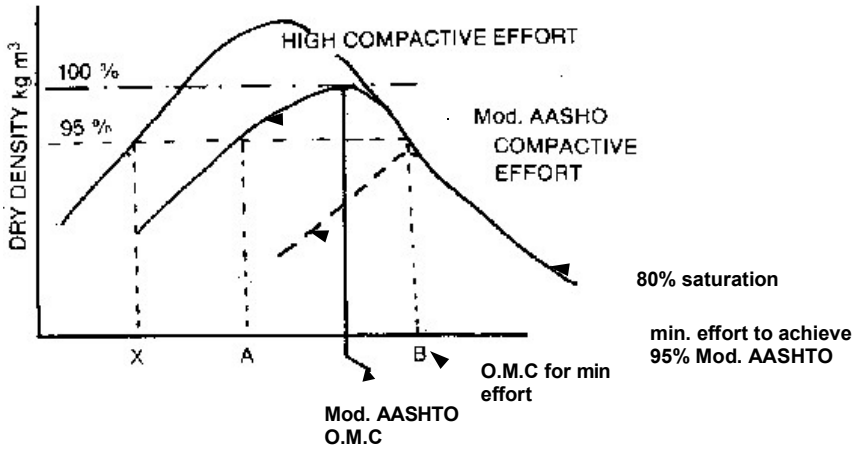


Figure 4.2

Vibratory rollers overcome friction with less moisture and produce a far greater effort than the minimum effort and thus effective compaction on the road can be done at a moisture content lower than Mod AASHTO optimum moisture content. In some cases the relative compaction on a clean sand can be as high as 106% Mod AASHTO.

As a rule of thumb, rolling on the roads with vibration should be done at 75% of “Mod AASHTO” optimum moisture content, i.e. if the graph shows Maximum Dry Density at 10% moisture, one should roll on the material at approximately 7,5% moisture. In hot, dry and / or windy weather the moisture content will have to be increased slightly to allow for the drying out during the compaction process.

The soils we deal with can be broadly classified in rockfills, crushed stone, gravels, sand, silts and finally clays (and any combination thereof). When we compact any of these materials, we endeavour to bring the material to a dense state that will give the required loading capacity.

Water’s main function is to act as a lubricant to enable the particles under compression to rearrange themselves to their most compact form.

Here we may add that vibration performs much the same function (friction is eliminated by the vibration, and the compaction is achieved by forces of static load and gravity).

Water, however, can create difficulties, as in the instance of surplus water in a clayey material where the capillary action of the water coating around individual particles prevents maximum compaction.

Furthermore, it should be clearly understood when considering compacting clay that clay, when subjected to an increased moisture environment, will expand thus reducing density. Specifying that clay must be compacted if the moisture content may increase is folly in the extreme. Clay under a newly constructed road will always expand as subsoil moisture builds up.

As previously stated, the greater amount of energy put into a soil for purposes of compaction, the less moisture is required to achieve required density. Therefore the statement that a particular soil has an optimum moisture content of say 5% is only meaningful when that 5% of moisture content is related to the energy put in during compaction. This is illustrated in the Figure 4.3.

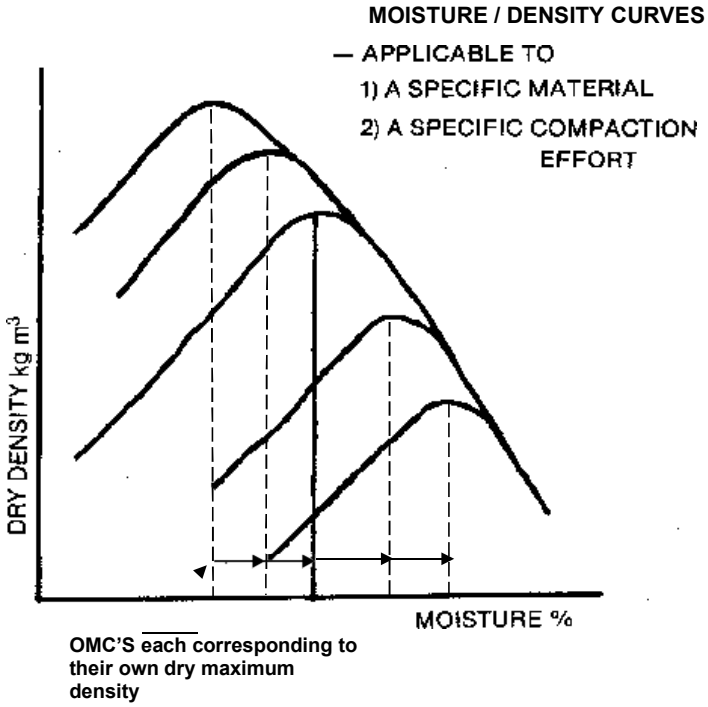


Figure 4.3 : Curves showing OMC's for various maximum Densities.

The reader may have noticed that when certain materials are mixed to what is apparently the optimum moisture content for the density required, the material appears to heave and sponge when placed under the roller. This is because the moisture content is actually in excess of the OMC, or peak moisture content for the required density and the effort is too high for the moisture, which would peak at a lower density. It is therefore, very important to remember that when one speaks of optimum moisture content, one can only relate it to the degree of density concerned.

In cases of clean single sized materials – i.e. Cape Flats Sand – it will often be found that optimum compaction conditions exist where no moisture is present. This is because owing to the single sized nature of sand particles, the particles may find no inhibitions in their movements under vibration and consequently achieve maximum packaging without the assistance of lubricating moisture.

Pneumatic and static steel wheeled rollers

Pneumatic and steel wheel rolling has its main advantage in sealing a finished piece of layer work or in obtaining a sealing on a rough piece of fill. However, in depth, neither type of static roller achieves a significant degree of compaction, unless high pneumatic wheel-loading (up to 30 tonne) is applied.

Vibratory Plant

In recent years, vibratory compaction plant has spread its field of application and now encompasses all aspects of compaction, that is to say all the way from bulk fill to final black-top surfacing. The main criterion here is that for deep lift fill compaction, a heavy vibrating roller with a slow vibration frequency and a high amplitude is applied, whereas on the other extreme, where one requires compaction of let us say 30 mm of premix a somewhat lighter compactor with a higher vibrating frequency and low amplitude is required.

Between these two extremes, vibratory compactors have been produced with suitable amplitude / frequency proportions to fit almost any compaction application. Additionally as mentioned before, vibratory compaction can be carried out at a very much lower moisture condition than with static. Remember, during static compaction, a mechanical interlock can very soon be achieved to such a degree that the static roller cannot overcome the locking effect of the particles in the material. This cannot occur with vibratory compaction, as during vibration this friction between the particles is eliminated. Thus, it is a common occurrence to achieve 104% compaction on a sandfill at 2m depth lifts at only 1 or 2 % moisture content. A crushed stone base with a laboratory optimum moisture content of say 5 % may for example be compacted to the required density at only 3,5 % moisture with the use of a suitably selected vibratory roller. In fact, had the basecourse been compacted at the Mod AASHTO "OMC", it may well have heaved or sponged owing to the fact, as we mentioned earlier, that for the density required, the material would then be over its optimum moisture content.

Just as soil can be rolled at a lower moisture content with vibration, so can hot bituminous mixtures be rolled at lower temperatures. The premix will be in a state of flow under vibration, in spite of the fact that its temperature has dropped to the stage where it is beginning to solidify. Premix can therefore be rolled at a more stable state and a better surface may result (e.g. free of transverse cracks).

Vibratory compaction as a final solution

Bearing in mind the abovementioned facts, it can now be said that almost without exception, there is a vibratory compactor to match any compaction requirement. It is up to the personnel on site to decide what type of vibratory compaction is required and to apply it in the correct manner.

Design of road foundations

Broadly speaking, a road foundation usually consists of the following specifications by layer.

a) <u>High traffic loading :</u>

i)	Fill	90% Mod. AASHTO density required, (in the case of self-draining sand 100% Mod. AASHTO density).
ii)	Selected layer	93% Mod. AASHTO density required, (100% for self draining sand).
iii)	Sub-base	Cement stabilised material compacted to 95 or 97% Mod AASHTO density required.
iv)	Basecourse	Crushed stone base material compacted to 88% of apparent relative density for G1 (although it is believed to be more correct to use bulk density as the reference) or 85% of bulk relative density for G2.
v)	Asphalt	min. of 92% of maximum theoretical density.

b) <u>Low to medium traffic loading :</u>

i)	Fill	90% Mod AASHTO (in the case of self draining sand 100%).
ii)	Selected layer	93% Mod AASHTO (100% for self draining sand).
iii)	Sub-base	95% Mod AASHTO required.
iv)	Basecourse	98% Mod AASHTO required.
v)	Asphalt	95% Marshall density required.

Difficulties in compaction are often encountered on the subbase layer, particularly where a high plasticity index occurs. This is because, material with a high clay content, cementing fines are often brought to the surface during compaction and a crusting results, not allowing penetration of the roller.

Care should be taken to keep this surface moist so as to prevent the crusting. However, over-wetting of the material will always result in "heaving" and this should be avoided. One should note

that whatever compaction work is being carried out, the moisture added to the material should always be added with regard to the compaction plant being employed for that particular phase.

Illustrations of how intelligent application of compaction leads to the best cost engineering solution

i) Pre-compaction of sand trenches prior to excavation can enable the sides to be cut more vertically and thus reduce handling of material and the volume of backfill required.

ii) The bedding of pipes laid in cohesionless sand with concrete poker vibrators under saturation. This operation need only be carried out up to the level of the diameter of the pipe. Thereafter, the trench can be completely filled and compacted almost dry with heavy vibrating plant.

Remember that the pipe cannot possibly subside and thus break if it is bedded correctly. It is only the vertical subsidence of a pipe in sand that causes it to break.

Well-known compaction problem areas and suggested solutions

One area of concern in compaction has frequently been the difficulties with which the specified density is achieved on lime or cement stabilized natural gravels.

Problems, for example, have frequently occurred in obtaining the specified 95% Mod AASHTO compaction on a stabilised sub-base. Peculiar to these problems is the formation of a hardened upper crust within the layer, which inhibits the densification of the lower portion of the layer. Once the abovementioned crust has been formed, no amount of compaction effort is going to induce the material to further collapse in the short term and if anything, the material will lose density due to the crust starting to fragment. Use of the padfoot as in Figure 4.4 eliminates this process. Additionally, it is frequently specified that the compaction activity must cease, within say, four or six hours, after the addition of stabilising agent. This further aggravates the problem and causes the contractor to tend to use static rolling at high speed thereby creating this impenetrable crust on the road surface even faster. Once again, use of the tamping technique would eliminate this and will in fact cause compaction to occur even faster, but as illustrated, from the bottom up thereby causing homogeneous compaction and no impenetrable crust.

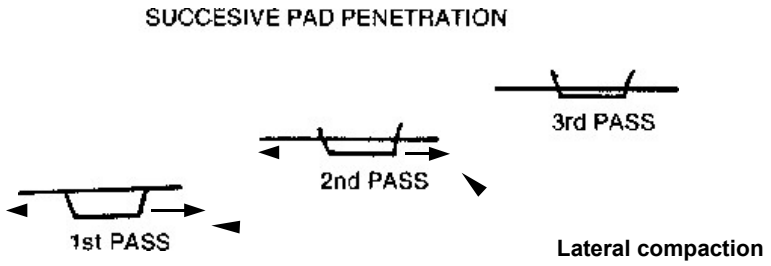


Figure 4.4 : The penetration and rising of the padfoot with passes

Poor quality gravel

Further difficulties in compaction may arise when the gravel is too fine. Here compaction to a high relative compaction may not be possible without considering above normal compactive effort.

Some suggested caution on moisture content control of soil – both before and after compaction

The previously mentioned "heaving" effect often observed on materials should be examined further. The usual direct cause of this phenomenon is obviously that the soil is over the optimum for the load applied, see Figure 4.5.

It is of no practical use to continue attempts at compaction once heaving has commenced, because material now exhibits "hydraulic" movement between its various particles. Any load applied is, therefore, transferred from one area to another, not by aggregate to aggregate contact, but by moisture flow. As a result, particles do not achieve closer proximity to one another, but in many cases actually become wider spaced as energy intended for compaction causes such a degree of moisture flow that the fines in the material are "washed" from between the larger aggregate. This is often observed on thinner layers of fill when the surface immediately under the roller becomes extremely wet and "tacky" a water/fines paste being formed due to the above circumstances.

Three moisture stages, critical to successful compaction are illustrated in Figure 4.5.

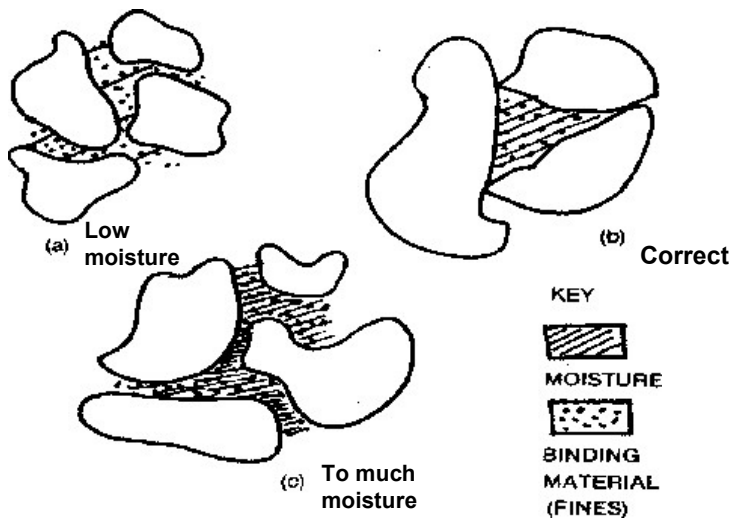


Figure 4.5 : Excess moisture above peak moisture content prevents particle interlock.

Item a): Shows particles within a soil mass with insufficient lubricating moisture between them to enable greater interlock to one another. At this stage the inter-particle friction is equal to the load applied in order to achieve densification and therefore no further densification will occur.

Item b): Shows the correct degree of moisture required to achieve maximum densification. The moisture and fines have combined to form initially a lubricating and finally a cementing paste between the larger aggregate.

Item c): At this point we have reached the abovementioned heaving stage. The fines / moisture combination is no longer a lubricating agent but an effective hydraulic solution. The solution is driven by the action of attempted compaction from one particle interface to the other – in some cases actually causing an increase in the voids between them. That is to say, voids in the sense of areas containing no solid matter.

If we were to draw a moisture / density graph for a material (under given constant compaction loading), item b would represent the maximum density at optimum moisture, whereas item a and c would be on the "dry side" and "wet side" respectively.

We view with these three situations a variance in the internal friction of the material from high (item a) to low (item c). It could be said that the angle of incline and decline of a “Mod” graph illustrates the degree to which internal friction is affected with varying moisture content.

A self-draining soil such as clean sand does not retain more moisture than its peaking condition as any excess is squeezed out through the pores. Hence on the “wet side” the curve of moisture vs. density is flat. The curve following the S-line or degree of saturation line is a non self-draining soil which does not permit excess water to escape under compactive effort, see Figure 4.6.

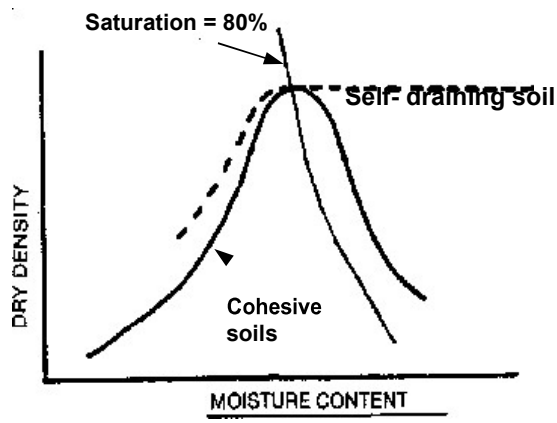


Figure 4.6 : Moisture - density curves, for drainable and cohesive soils

Fine grained soils, because of their large number of contact points (points of friction) give low densities under a given compactive effort. A good gravel on the other hand contains larger solid particles. Voids are replaced by solid mass and fewer friction points exists, see Figure 4.7.

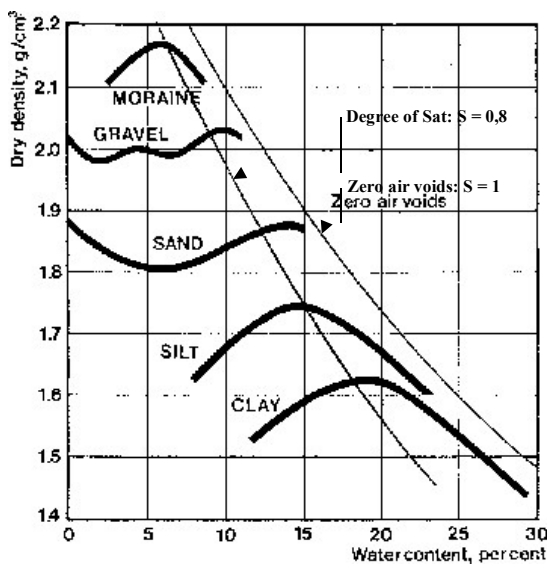


Figure 4.7 : Laboratory moisture – density curves for different types of soil

It will be noticed that regardless of the soil type the peaking moisture or “OMC” occurs at a degree of saturation equal to 80%. All the soils have a particle relative density of equal value in this example soils with different relative densities will exhibit different degrees of saturation lines.

An additional problem in the case of clay is that strong capillary forces exist between the very fine particles and this can be a further inhibiting factor in their relocation. Attempts at vibratory compaction of cohesive materials should be considered in the light of that material's mobility factor.

Efficient compaction makes it possible to increase substantially the bearing capacity and stability of a fill. It improves permeability and in most cases, practically eliminates settlements. Compaction will make the soils stable enough to stand up to traffic and foundation loads and reduce substantially the maintenance costs for highways and other fill constructions. The large variety in properties of materials to be compacted with the very varying job / site conditions, makes it more or less impossible to find a few single general rules for soil compaction. Local variations regarding soil properties, climate and other factors can always be expected.

Moisture Content and Degree of Saturation

As Figure 4.7 illustrates the peaking or "saturation" condition for a given soil density is virtually always at a degree of saturation of 80% but the moisture content varies from density to density. It is thus a pity that "M/C" or "moisture content" is used far more frequently than "degree of saturation" - a far more pragmatic parameter. Furthermore density is dependent on the relative density of the soil particles whereas voids content or voids ratio is also a far more meaningful parameter for degree of compaction. These various parameters are given by:

$$M/C = S(1/D_{dry} - 1/G_b)$$

Where M/C = moisture content (fraction)
S = degree of saturation (fraction)
= fraction of voids filled with water
D_{dry} = dry density of soil (t/m^3)
G_b = bulk relative density of soil particles

Consider two soils A and B where both A and B are compacted to a density of $2.2 t/m^3$ but G_b for A is 2.6 and for B it is 3.0. The voids (n) present in each case is given by :

$$n = 1 - (D_{dry}/G_b)$$

Where for A n = 1-(2,2/2,6) = 15,3
and for B n = 1-(2,2/3,0) = 26,7

Clearly A is the stronger in spite of them both being of equal density per cubic metre. The peak moisture content for A is 5,6% but for B it is 9,7% but in both cases the degree of saturation is 80%, see Figure 4.8.

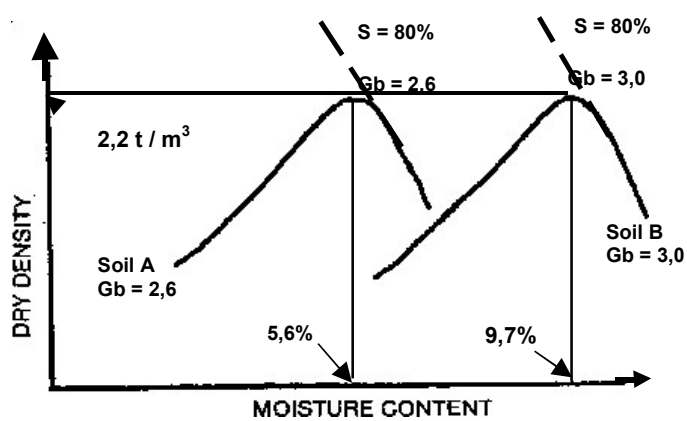


Figure 4.8 : Two soils of equal density but with different particle relative density

The value of density of 2,2 t/m^3 or 2200 kg / m^3 for each soil does not give any indication as to which of the soils has the greater strength.

In Figure 4.9 where the inverse of void ratio is plotted against degree of saturation Soil A is clearly illustrated as showing the greater strength (i.e. less voids or greater degree of compaction).

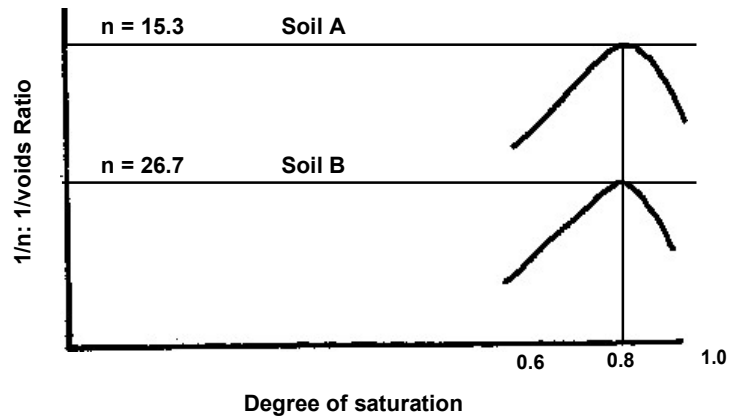


Figure 4.9 : (1 / n) vs. saturation - illustrating the significance of the voids content of soil

General rules and answers

The difficulty to find general rules and answers makes information, exchange of experience and further field and laboratory studies of soil compaction very urgent. It should also be clearly understood that for a given soil, usually there is a choice between several compaction methods.

5. TYPES OF COMPACTION PLANT

5.1 Key points in this section

- Static pressure
- Impact / dynamic pressure
- Vibration
- Particle mobility
- Frequency and amplitude
- Centrifugal force and acceleration
- Pneumatic rollers, wheel load, tyre pressure
- Relative density
- Void ratio
- Resonance
- Shear strength
- Dynamic stresses
- Draining properties of soils
- Liquefaction

5.2 Additional information

Compaction equipment use one or a combination of the following principles:

- Static weight
- Kneading action under pressure
- Vibration
- Impact

The following compaction equipment is commonly used in road construction:

- Smooth drum roller (static and vibratory)
- Grid roller
- Padfoot roller (static and vibratory)
- Pneumatic-tyred roller (PTR)
- Impact compactor
- Plate compactor
- Hand-held vibratory compaction tools

Each of the above equipment serves a specific purpose and the choice of compaction equipment is dependant on the process requirements and application purpose.

If one of the requirements of the compaction process were that large particles in natural gravel are to be broken down into smaller particles, a grid roller would be more effective than a vibratory roller. Or, if the purpose of application of a roller is to obtain a smooth finish of the compacted layer, a smooth drum or PTR will result in a good finish.

Impact compactors are effective for ‘deep’ compaction or compaction in thick layers but have the disadvantage of a poor surface finish or even breaking up the surface. However, this may be acceptable in certain applications (for instance when building up a fill area and successive layers are still to be compacted on top). Impact compactors are also used when in-situ soils with a potential to collapse (collapsing soils) need to be compacted in depth to eliminate their collapse potential.

The choice of compaction equipment may therefore depend on, but is not limited to the following:

- Type of material to be compacted
- Layer thickness
- Density requirements

- Finish requirements
- Cost and availability of equipment

Capacity of rollers

The capacity of a roller is determined using the following simple equation

Capacity = $(P_1 \times P_2 \times R_W \times L_t \times S) / N_p$ [m^3/h]

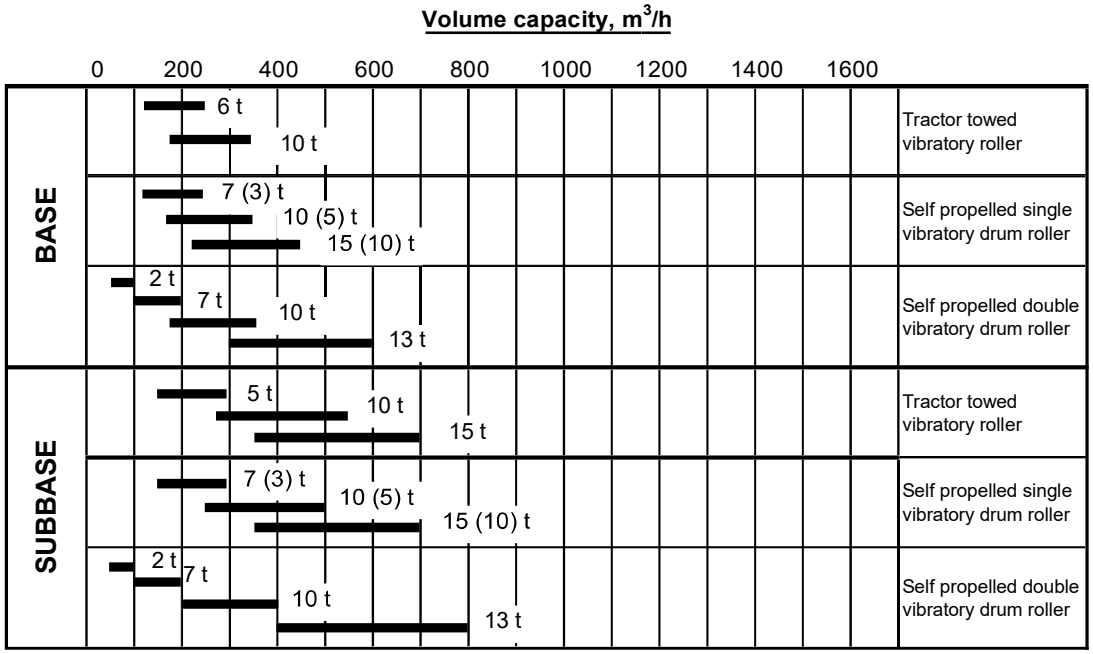
Where : P_1 = Efficiency with regard to time [dimensionless]
P_2 = Eficiency with regard to overlap [dimensionless]
R_W = Width of roller drum [m]
L_t = Layer thickness [m]
S = Roller speed [m/h]
N_p = Number of passes [dimensionless]

Example : A roller width a 2,5m wide drum gives a 15% overlap between each rolled lane. The roller is effective for 55 minutes in the hour and compacts a 150mm thick layer satisfactorily with 8 passes. The roller operates at 3 km/hr. What is the capacity of the roller.

Capacity = (55/60) x (0,85) x 2,5 x 0,15 x 3000 / 8 = 109,6 m^3/h

The figures below give an indication of the capacity of some rollers.

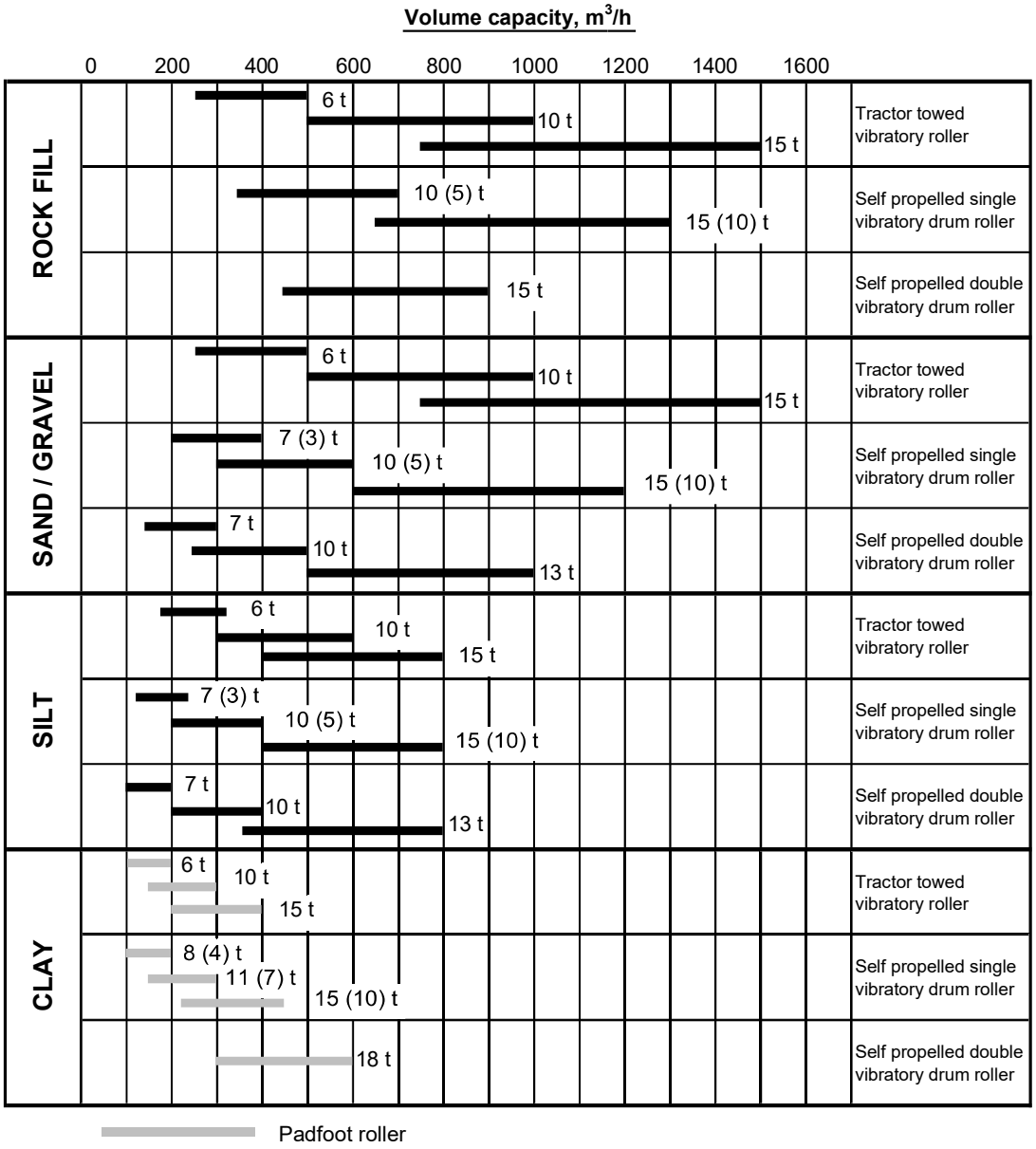
Capacity of rollers – compacting layer materials



After Forssblad

Note : The figures in brackets refer to the mass on the drum in the case of the single drum (tyre wheel propelled rollers)

Capacity of rollers – compacting embankment material



After Forssblad

Note : The figures in brackets refer to the weight on the drum in the case of the single drum (tyre wheel propelled rollers)

5.3 Further reading

- CSIR Transportek, *Practical compaction guidelines (revised edition)*, Divisional publication DP-2003/1, 2003
- Savage, P.F. and A.T. Visser, “Stabilised Materials and Compaction”, University of Pretoria, Course SGC788 notes

PART V: VARIOUS TYPES OF COMPACTION PLANT AND THEIR APPLICATION. A SUMMING-UP IN PRACTICAL TERMS OF VARIOUS POINTS RAISED IN PARTS I – IV

VIBRATORY soil compaction was first studied and applied in Germany during the 1930's (Loos, 1936). A self-propelled Caterpillar-type vibrating plate compactor with a total mass of about 25 tonne was developed by Losenhausenwerk as early as 1933. This vibrator had a variable frequency between 10 and 15 cps and a maximum depth effect of 2 to 2,5 m (Muller & Ramspeck, 1935; Ramspeck, 1937) . Losenhausenwerk also designed at about the same time a self-propelled 1,5 tonne vibrating plate compactor.

The vibroflotation method where heavy internal vibrators in combination with jetting are used to compact cohesionless soils to large depths was also developed in Germany during the 1930's (Schneider, 1938).

Other types of vibratory soil compactors were developed during the 1940's such as tractor-drawn and self-propelled vibrating rollers, multiple plate compactors and vibrating tampers (rammers) as described by Garbotz (1958), Erienbach (1959), Kronenberger (1960), Leussink (1960) and Forssblad (1965).

Vibratory soil compactors can be classified as indicated in Table 5.1. Vibrating tampers are light, hand-guided machines operating at a low frequency, about 10 cps, and with a large amplitude. The types of self-propelled vibrating plate compactors which are most commonly used today weigh between 100 and 500 kg.

Table 5.1a – Different types of vibratory soil compactors.

Surface Vibrators		
Type of Machine	**Mass**	**Frequency**
Vibrating Tampers		
(Rammers)		About
Hand-guided	50-150kg	10 Hz
Vibrating Plate Compactors		
Self-propelled, hand-guided	50-3 000kg	12-90 Hz
Multiple-type mounted on tractors, etc.	200-300 kg	30-70 Hz
Crane-mounted[1]	Up to 20 tonne	10-15 Hz
Vibrating Rollers		
Self-propelled, hand-guided (1 or 2 drums)	250-1 500 kg	40-80Hz
Self-propelled, tandem-type	0,7-10 tonne	30-80 Hz
Self-propelled, driving rubber tyres	4-25 tonne	20-40 Hz
Tractor-drawn	1,5-15 tonne	20-55 Hz

Table 5.1b – Different types of vibratory soil compactors

Internal Vibrators		
Type of Machine	**Diameter**	**Frequency**
Concrete Vibrators		
Manually operated or tractor-mounted	50-150 mm	100-200 Hz
Vibroflotation Equipment		
Crane-mounted	230-380 mm	about 30 Hz

Self-propelled vibrating rollers are of three main types – hand-guided, vibrating tandem rollers, and vibrating rollers with two driving rubber tyres. Tractor-drawn vibrating rollers with weights up to 15 tonne and vibrating rollers with driving rubber-tyres with total weights up to 25 tonne are now in common use.

The use of vibratory soil compaction has increased considerably during the last 45 to 50 years. Vibratory soil compaction was initially used only for cohesionless soils (Lange, 1940; Leussink, 1951). Nowadays vibratory compactors are also used to compact cohesive soils and asphalt surfaces. The main applications are given in Table 5.2.

Table 5.2 – Applications of vibratory soil compaction

Type of Machine	Applications
Vibrating Tampers (rammers)	Street repair. Fills behind bridge abutments, retaining and basement walls, and so on. Trench fills.
Vibrating Plate Compactors Self-propelled, hand-guided	Base and subbase compaction for streets, sidewalks, etc. Street Repair. Fills behind bridge abutments, retaining and basements walls, etc. Fills below floors. Trench fills.
Multiple-type	Base and subbase compaction for highways.
Vibrating Rollers Self-propelled, hand-guided	Base, subbase and asphalt compaction for streets, side-walks, parking areas, garage driveways, etc. Fills behind bridge abutments and retaining walls. Fills below floors. Trench fills.
Self-propelled, tandem type	Base, subbase and embankment compaction for highways, streets, parking areas, garage driveways, and so on. Fills below floors.
Self-propelled, driving rubber tyres	Base, subbase and embankment compaction for highways, streets, parking areas, airfields, etc. Rock fill dams. Fills (soil or rock used as foundations for residential and industrial buildings.
Tractor-drawn	Base, subbase and embankment compaction on highways, streets, parking areas, airfields etc. Earth and rock fill dams. Fills (soil or rock) used as foundations for residential and industrial buildings. Deep compaction of natural deposits of sand.

The knowledge of the factors affecting vibratory compaction of soil is still incomplete in spite of the extensive investigations which have been carried out all over the world since the 1930's. The compaction of cohesionless soils is effected e.g. by average grain size distribution, shape of particles, water content, permeability, compressibility, shear strength and so on. Also the properties of the vibratory compactor such as frequency, nominal amplitude, centrifugal force and static linear loud affect the compaction. The descriptions of the soil and of the compactor in test reports and technical articles are usually very schematic.

The many factors which affect vibratory compaction make it difficult to draw general conclusions. It is therefore important to study carefully the actual test and soil conditions before tests results are applied to conditions other than those under which the tests were carried out.

This study deals mainly with vibratory compaction of cohesionless soils, but some data and results from investigations on rock fills and cohesive soils are given for comparison.

Elements of Soil Compaction

The purpose of compaction is to improve the strength and deformation properties of soils. The shear strength of cohesionless soils increase while the compressibility and the permeability decrease with increasing density. For example, the angle of internal friction of sand or gravel increases up to 12^0 when the relative density increases from loose to dense.

The compressibility of cohesionless soils is also affected by small changes in the density. Compaction of a soil will often increase the equivalent modulus of elasticity as determined by plate load tests five to ten times.

The relative density also affects the permeability appreciably. When the relative density increases from loose to dense the permeability of a uniform sand or gravel decreases by about 50 to 60%.

The change of the permeability increases with increasing coefficient of uniformity and with decreasing grain size (Taylor, 1948).

The compaction of a soil is in most cases expressed by the relative compaction or by the relative density. The relative compaction or the degree of compaction RC (per cent) is defined by the equation:

$$RC = \frac{D_d}{D_m} \times 100$$

Where D_d is the actual dry density (dry unit mass) and D_m is the maximum dry density determined by the Proctor compaction tests, standard AASHTO, modified AASHTO, and so on or by vibratory compaction as illustrated in Figure 5.1.

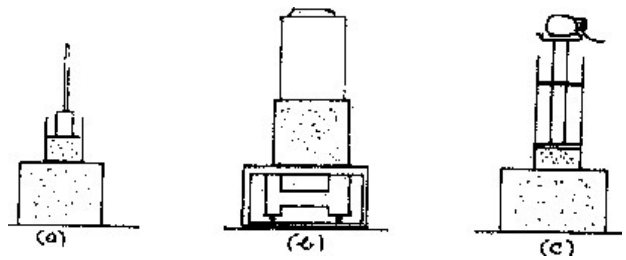


Figure 5.1 : Laboratory Compaction Methods

Item a)	**Proctor Compaction Test or Mod. Compaction Test**
Item b)	**Vibratory Compaction Test with Vibrating Table and Surcharge Load**
Item c)	**Vibratory Compaction Test with Vibrating Tamper or Hammer**

The maximum density can also be determined by laboratory compaction tests using a vibrating hammer or a vibrating tamper working on the surface of the material (Figure 5.1c). The method using a vibrating hammer has been developed by The Road Research Laboratory in England. (Parsons, 1964; Odubanjo, 1968)

The vibrating tamper method has been developed by Vibro-Verken (Forssblad, 1965, 1967). According to this method the soil is compacted in two layers in a mould with 150 mm diameter. Each layer is vibrated for 2 minutes. The resulting maximum dry density is for saturated cohesionless soils about the same as that determined by the modified AASHTO compaction test.

The compaction water content has a different effect on the compaction of cohesionless, free-draining soils than on fine grained, cohesive soils. The water content of a free-draining soil decreases during the compaction if the initial water content is high, while the water content does not change for fine grained soils. As a result, the Modified curve of free draining soils terminates near the maximum value, see curve depicted in Figure 5.2. If a sandy or gravely soil contains some silt or clay, it is not free-draining, and excess water does not drain out during compaction. The resulting compaction curve is regular as shown in Figure 5.2 by Curve 2. Cohesionless free-draining soils very often have comparatively flat compaction curves as illustrated in Figure 5.2 by Curves 1, 3 and 4. In some cases two maxima are obtained probably because of capillary forces in the partly saturated soil.

Due to the comparatively flat compaction curves for free draining soils and the fact that excess water can drain out during the compaction, the water content of the soil is not as important for cohesionless, free draining soil as for fine grained soils. The best compaction of sand is usually obtained when the soil is saturated or completely dry, but a comparatively good compaction can frequently be obtained when the soil is partially saturated (S=0,6 say). Investigations by Bureau of Reclamation (1963) and others as well as practical experience indicate that a soil usually can be characterised as free-draining if the content of fines (soil particles with a diameter smaller than 0,06 or 0,074 mm) is less than 5 to 10%. In special cases, soils can act as free-draining when the

content of fines is as high as 15 to 20%. However, well-graded sandy and gravelly soils (silty sand, clayey gravel, etc) with more than 5 to 10 % of the fines are as a rule not free-draining. These soils have certain cohesive properties and the control of the compaction water content is essential, see Figure 5.2.

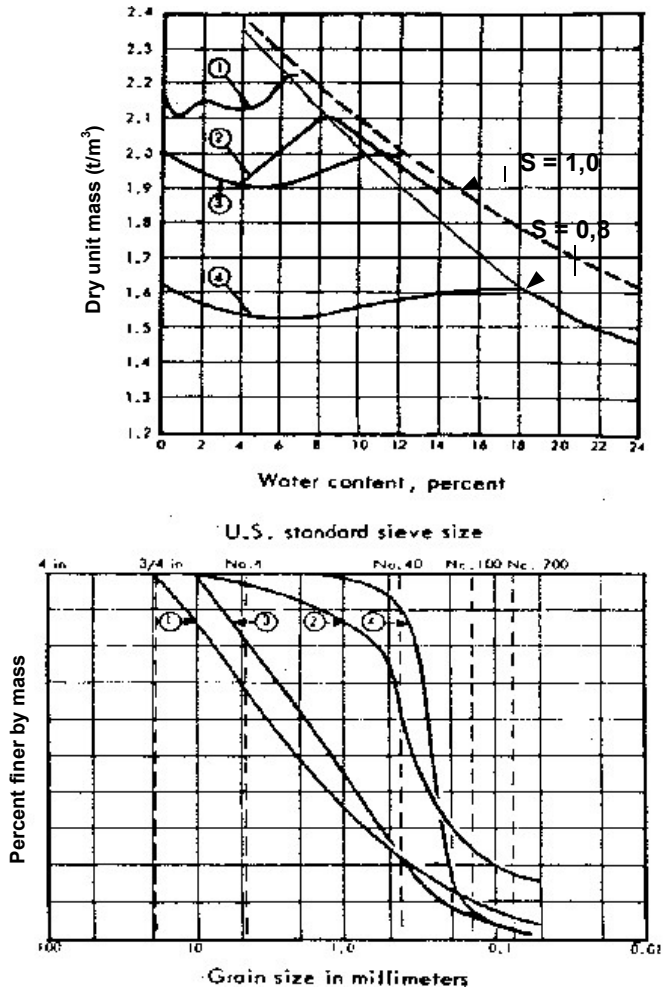


Figure 5.2 : Compaction curves obtained with the modified AASHTO method and grain size distributors for different types of sandy soils

It has been suggested that vibratory soil compaction be correlated with laboratory compaction tests based on vibration.

Impact compaction tests such as the Proctor compaction tests are generally time consuming in comparison with vibratory compaction tests. In addition, impact tests are, to a certain extent, affected by the manual performance.

A further advantage with vibratory compaction tests is that such tests can be adapted to test moulds with large diameters and can, therefore, be carried out on samples containing stones.

Technical data of vibratory compactors

The vibrations of the drum of a vibrating roller or of the bottom plate of a vibrating plate compactor are usually generated by one or several rotating eccentrics. The frequency, amplitude, velocity and acceleration of a sinusoidal oscillation are defined in Figure 5.3. The frequency expressed in cycles per second (Hz) is determined by the rotational velocity of the eccentric(s). The amplitude of the drum or the bottom plate is a function of the eccentric moment of the eccentric as shown in Figure 5.3 and the mass of the vibrating system. The so-called nominal amplitude can be calculated from the following equation :

nominal amplitude = (eccentric moment / mass of drum or bottom plate)

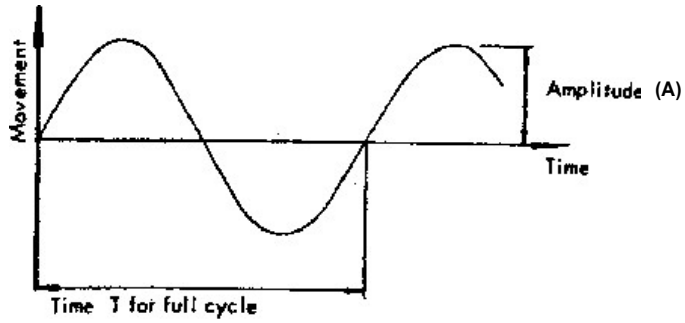
The actual amplitude can, for certain frequency ranges, be up to 50 to 100% larger than the nominal amplitude due to resonance (less than 27 or from 40 to 50 Hz). In operation the actual amplitude is less than the nominal amplitude as the mass of the drum or plate now is increased by the mass of soil vibrated by the roller or bottom plate.

The rotating eccentric initiates a centrifugal force which is a function of the frequency and the eccentric moment is shown in Figure 5.4 (due to the oscillations of the drum or bottom plate there will be a small difference between the real centrifugal force and that calculated from Figure 5.4. The difference is usually less than 1,0 %).

The centrifugal force acts inside the drum or bottom plate. Its magnitude is not equal to the dynamic force transmitted to the underlying soil as is sometimes assumed. Through the oscillations defined by their frequency and amplitude the vibrating drum or bottom plate affects the surface of the ground with a rapid succession of dynamic loads or impacts. Each load cycle generates a stress wave as illustrated in Figure 5.4. It is important to make a clear distinction between the vibrations of the compactor and the characteristics of the stress waves in the soil. The characteristics of the stress waves are to a large extent dependent on the properties of the soil but they are basically of a rotational nature.

The static mass of vibratory compactors has a large influence on the compaction effect since the kinetic energy as well as the momentum (mass x velocity) of the vibrating drum or bottom plate is directly proportional to the mass if the amplitude and the frequency are constant. Tests results indicate, for example, that the maximum depth to which a soil can be compacted is dependent on the total static mass of the compactor. However, the compaction close to the surface is for vibrating rollers also to a large extent influenced by the static mass per linear cm of drum width (S.L.L.). The ratio of drum and frame mass is also of importance with respect to the compaction effect. However, this mass ratio is given in specifications, technical reports, and so on for vibrating rollers and vibrating plate compactors.

It is proposed that at least the technical data in Table 5.2 should be given in specifications and technical reports for vibrating rollers and vibrating plate compactors.



Frequency (number of cycles per sec. (Hz)) = f

Amplitude (maximum deviation from position at rest) (mm) = A

Maximum velocity = v_{max} = $2\pi f. A$ (mm/s)

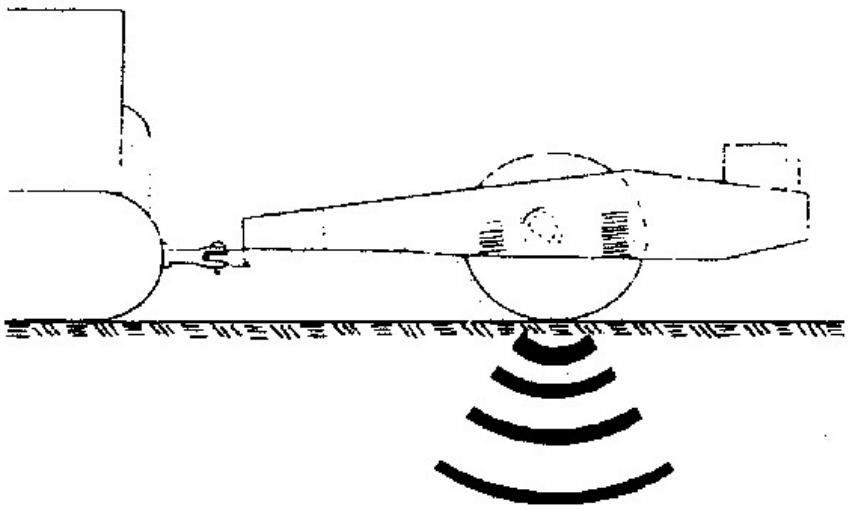
Maximum acceleration = a_{max} = $4\pi^2 . f^2 . A/1000$ (m/s^2)
= $4\pi^2 . f^2 . A/g$ (gravity units)

Figure 5.3 : Definition of frequency, amplitude, velocity and acceleration for a sinusoidal oscillation

Table 5.2 : Technical Data for Vibrating Rollers and Vibrating Plate Compactors

Vibrating rollers	Vibrating plate compactors
Total all up mass (kg)	Total mass (kg)
Static linear load of drum width (kg / cm)	Static load per unit area of bottom plate (kg / m^3)
Drum width (m)	Width of bottom plate (m)
Drum diameter (m)	Length of bottom plate (m)
Frequency (ies) (Hz)	Frequency (Hz)
Nominal amplitude (mm)	Nominal amplitude (mm)
Eccentric moment (kgm)	Eccentric moment (kgm)
Centrifugal force[1)] (kN)	Centrifugal force[1)] (kN)
Working speed (nominal) (km/h)	Working speed (km/h)

1) It can be misleading when comparing compactors of different frequencies.



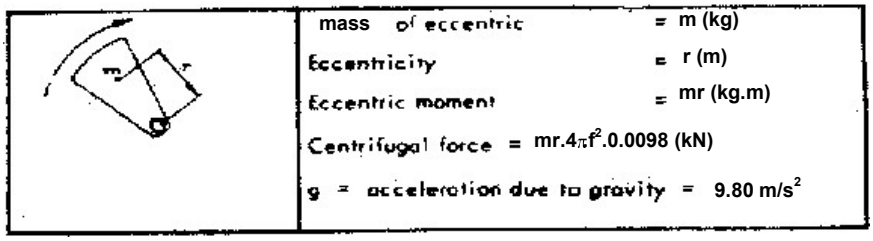


Figure 5.4 : Definition of eccentric moment and centrifugal force

Compaction mechanisms

1. Resonance

According to Hertwig (1931), the highest density of a soil is obtained at the resonant frequency of the vibrator – soil system where the amplitude reaches a maximum. Also Terzaghi (1943), Bernhard (1952, 1959), Converse (1954, 1957) and other recommend that the compaction should be done at the resonant frequency.

The resonant frequency depends on the eccentric moment, the mass and dimensions of the vibrator and on the properties of the soil. Also, Barkan (1960) and others have discussed the factors affecting the resonant frequency of the vibrator – soil system. Resonance frequency falls generally between 20 and 25 Hz.

Compaction tests and practical experience do not indicate however, that vibratory compaction at the resonant frequency will give a higher density than that at other frequencies (e.g. Lorenz 1955, 1960; Forssblad, 1955, 1965; Kutzer, 1962). Usually the density of compacted soil will increase with increasing frequency of the vibrator – soil system. One reason as pointed out by Forssblad (1965) is that the centrifugal force and the momentum and thus the “intensity” of the vibrations also increase with increasing frequency. The resonance effects of the vibrator – soil system will in this case be combined with the effects of an increase of the “intensity” of the vibrations as graphically illustrated in Figure 5.5. The resonance effect is, in this case, “hidden”. Due to this reason, medium and heavy-mass vibrating rollers often are designed with a frequency just above the resonance frequency of the vibrator – soil system.

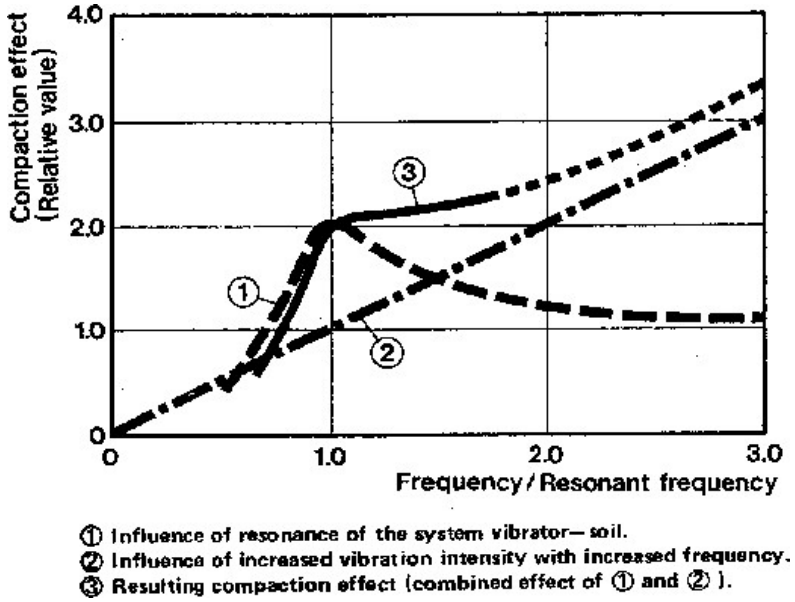


Figure 5.5 : Effect of frequency and amplitude on vibratory compaction of soil

2. Number of load cycles

According to Tschebotarioff and McAlpin (1947), the number of load cycles and the centrifugal force govern the density of the compacted soil when the frequency is low. About 10,000 cycles were required to reach the maximum density of a sand when the centrifugal force was 90 and 180N. The maximum density increased with increasing centrifugal force. When the centrifugal force was 360 and 735 N, approximately 30 000 load cycles were required. These tests were carried out at frequencies between 1,0 and 25 Hz.

Kutzner (1962) also found from vibratory compaction tests with glass beads and dry sand that the density increased with increasing number of load cycles or with increasing vibration time. For the glass beads the increase was small after 500 load cycles. About 9 000 load cycles were required to reach the maximum density of the dry sand at a frequency of 50 Hz i.e. after 3 minutes vibration.

Tests by Whitman and Ortigosa (1968) indicate that the density of a compacted cohesionless soil at accelerations less than 1g is primarily governed not only by the number of load cycles but also by the stress increase in the soil. The density increased as the logarithm of the number of cycles.

The test results thus indicate that the number of load cycles is of special importance at low accelerations. The number of load cycles required to reach a given relative density is also influenced by the gradation of the soil.

3. Effect of water content

The degree of saturation and the water content affect appreciably the compaction properties of sand due to capillary forces between the individual soil particles ("false cohesion"). Figure 5.6 shows the dry density of uniform coarse, medium and fine sands as a function of the water content (Kutzner, 1962). It can be seen that the dry density of the soil compacted either dry or saturated was approximately the same. The lowest density was obtained when the water content was 5 to 10%. Kutzner also carried out compaction tests where a wetting agent had been added to the pore water. In this case, the dry density of the coarse sand after compaction was approximately independent of the water content, see Figure 5.6. These tests indicate that the compaction of partly saturated cohesionless soils is appreciably affected by capillary forces.

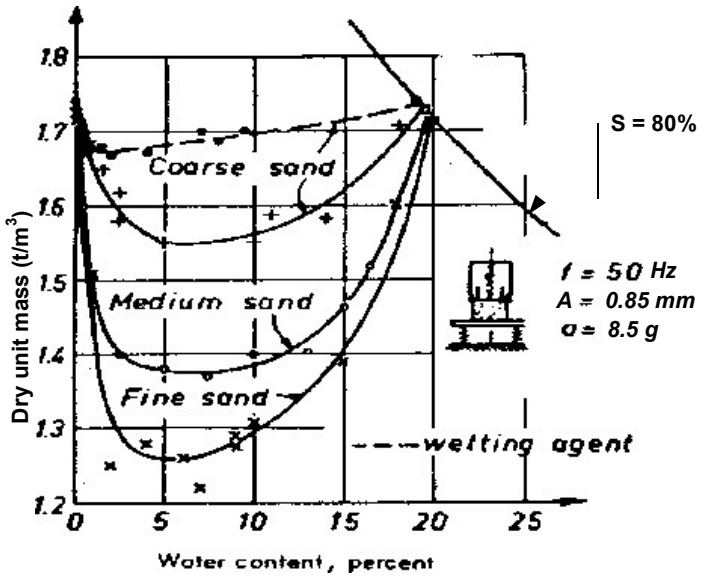


Figure 5.6 : Vibration compaction of sand at different water contents after Kutzner

4. Shear strength during vibration

Mogami and Kubo (1953) observed that the shear strength of dry sand decreased with increasing acceleration. They investigated two types of dry sand in a direct shear apparatus at frequencies ranging from 10 to 50 Hz and at accelerations which varied up to 2g. The test results indicated that the strength decreased approximately linearly when the acceleration increased from 0,5 to 1,0 or 2,0 g.

The shear strength of sands under vibratory loading has also been investigated by Linger (1963) with direct shear and triaxial tests. The vibratory tests gave a lower shear strength than the static tests. The superimposed vibratory loads caused a reduction of the normal load for the direct shear tests and of the confining pressure for the triaxial tests.

Forssblad (1965) used the vane test to study the shear strength of mainly cohesionless soils with and without vibrations at different water contents. The test data indicated that the vibratory shear strength varied between 1 to 6% of the static shear strength. The lowest value was obtained for gravel, sand, gravely silt or crushed stone when the soil was either dry or saturated. The vibratory shear strength of all investigated soils was about the same. (See Figure 3.5 in Part III).

5. Dynamic stresses

The compaction of a soil is, to a large extent, dependent on the type and the intensity of the stress waves, generated by the vibrator. Longitudinal and transverse waves are transmitted through the soil while Rayleigh and Love waves follow the ground surface. In a transverse wave, the soil particles move in a plan perpendicular to the direction of wave propagation while in the longitudinal waves, the particles move into the direction of the wave propagation. The transverse waves propagate with about half the velocity of the longitudinal waves (also called compressional waves).

The amplitude of the longitudinal and of the transverse waves generated by vibrating rollers or plate compactors decrease with increasing distance from the vibrator mainly due to geometrical damping. There is also a reduction of the amplitude due to absorption of energy by the soil.

According to Forssblad (1965) the intensity of the stress waves must be high enough to overcome the shear strength of the soil. In a partly saturated sand and gravel with a relatively low apparent (“false”) cohesion, a vertical dynamic stress of about 50 to 100 kPa was required to reach a relative

compaction of 90% with respect to the modified AASHTO compaction test. For clays a considerably larger pressure – about 500 kPa was required to reach a relative compaction at the peaking moisture content.

Whiffin (1954) has measured the stress distribution below a vibrating roller, a static roller, a power rammer and a crawler tractor in two types of clayey soils. Whiffin found a relationship between peak pressure (static or dynamic) and the relative compaction of the soil which was independent of the type of compaction machine and of the duration of the stress pulse. The dynamic stress generated by the vibrating roller was about 100% larger than the corresponding static stress increase. These results have been confirmed by other authors.

Forssblad (1965) has measured the vertical dynamic stresses below different types of vibratory compactors and for different soil types. As shown in Figure 5.7 the maximum vertical dynamic stress decreased rapidly with increasing depth below the ground surface. Forssblad found that it was possible to calculate approximately the dynamic stress distribution by Boussinesq's equation, for compacted soil. For a loose soil, the dynamic stresses in the soil were less than those calculated by Boussinesq's equation. Christofel and others have measured the stress distribution in the soil below different types of vibratory compactors.

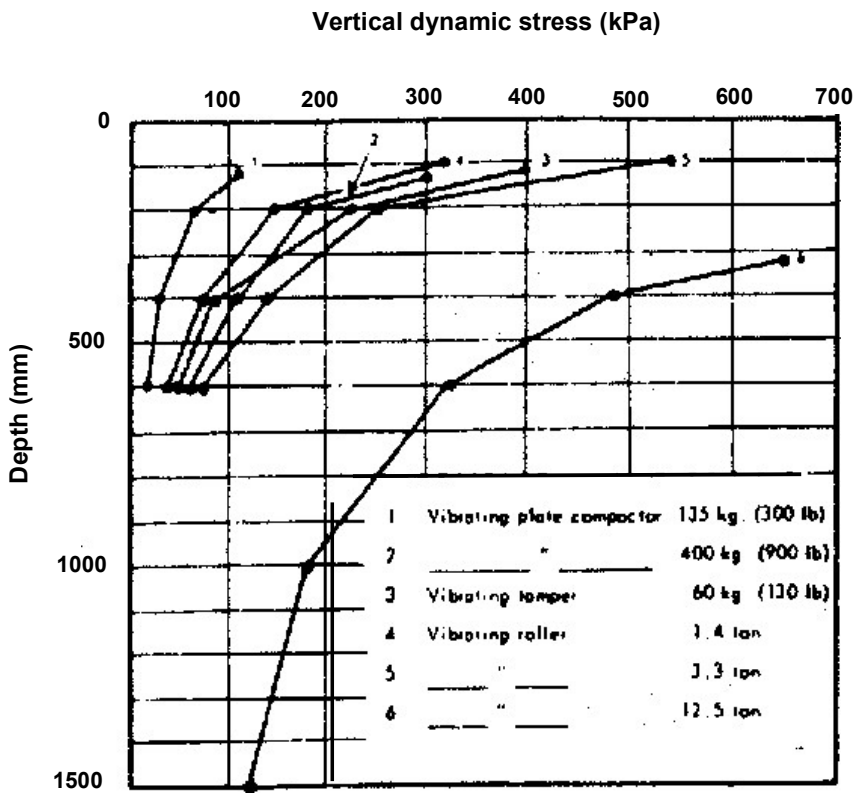


Figure 5.7 : Vertical dynamic stresses at different depth under vibratory compactors of different types and sizes according to Forssblad

Reflection of the stress waves against lower, more compact layers can give special compaction effects. Near such a layer the compaction of a soil will sometimes be better than that close to the surface. Reflection can also have a loosening effect when thin layers of soil are placed on a rigid base.

This thin layer on a rigid base cannot accept the energy of the vibrator which is then reflected back through the layer. The effect not only destroys the layer density but causes bounce. To overcome this problem a decreased nominal amplitude is necessary and a possible increased vibrator speed.

6. Liquefaction during compaction

The pore pressure in saturated sand builds up progressively during the vibration if the relative density is low and the sand is not free-draining. The compactor bogs down when the pore pressure approaches the total over-burden pressure of the soil and the vibrator. This phenomenon is called liquefaction. Liquefaction is dependent on the permeability, the initial density of the sand, the number of load cycles, the over-burden pressure and the intensity of the cyclic load (Seed and Lee, 1968; Lee and Seed, 1967; Peacock and Seed, 1968). The vibratory load and the number of load cycles required for liquefaction increase with increasing relative density and with increasing overburden pressure.

The minimum permeability for the soil to be free-draining is a function of the travel speed, the thickness of the compacted layer and the compressibility of the soil. Approximate calculations indicate that the permeability should be at least 10^{-2} cm/sec. for a layer thickness of 200 mm at travel speed of 4 km/hour, and a volume compressibility of 0,2 m^2 / MN for the soil to be free-draining and to avoid that the compactor bogs down. It has been assumed that the consolidation ration should be at least 50% for each pass.

A Summary of different applications

Rockfill:
For compacting rockfill, heavy vibratory rollers are without any doubt the most efficient type of equipment. They compact layers up to 2 m thick. Well-compacted rock fill has a very high bearing capacity and stability.

But compaction of rock fill requires machines that have been specially designed for the job. One requirement is a thick drum shell.

Sand and gravel:
Stones, coarse gravel and sand are best compacted with vibration. Even light vibratory compaction equipment can reach a high degree of compaction at low layer thicknesses. Medium and heavy vibratory rollers can compact sand and gravel in layers of 0,5 to 1,0m and in special cases even thicker layers. With its good drainage properties, coarse gravel can be compacted even in heavy rain and under water. Sand also has high permeability as a rule. Sand and coarse gravel can be compacted with various water contents and even when completely dry (dry compaction).

Silt:
When compacting silt, you should not let the water content go above the peak value. Pure silt is not plastic, but silty soils often contain clay particles. This makes the silt equivalent to clay as far as compaction is concerned. Lamination is often a problem particularly when the water content is too high. Padfoot rolling is recommended or grid rolling followed up with a PTR.

There are also silty soils which contain coarser particles for example moraine. These can very well be compacted with medium and heavy vibratory rollers even in relatively thick layers.

Clay:
Despite its plasticity, the very small particles in clay have great cohesion. Thus clay requires a high compaction effect and can be compacted only in relatively thin layers, normally not more than 300-400 mm.

Medium and heavy vibratory rollers with padfoot drums are very suitable for clays and also for lime-stabilized clays.

Pure clays particularly those with high swelling potential should not normally be compacted. Apart from the difficulty of overcoming their cohesive property a high density if achieved will be lost when the clay re-absorbs moisture. In order to overcome these clay properties, if it is essential to the work, stabilization with lime or other chemicals should be considered.

Foundation layers.

Bases, subbases and selected layers are usually constructed in thicknesses of 150mm or less. But these courses are nevertheless satisfactorily compacted by vibratory equipment. Few important facts nevertheless apply and their importance noted.

- Large Amplitudes of 0,9 to 1,8 mm are suitable for the bottom portion of a layer but will not compact the top part of the layer. The amplitude must be reduced to about 0,6 mm or less for upper parts of a layer.
- As the top of any layer has no over burden to clamp down particle disturbance a light spray of water will hold particles to a better knit by surface tension.
- Do not over-wet by adding too much water. Remember that vibration overcomes friction without needing water lubrication. Compaction at about S=0,4 or at a moisture content of about 50% peak moisture content will often be quite successful in cohesive gravels and finer soils.
- When compacting crushed rock Base layers such as G1, G2 or G3 types do not exceed the peak moisture content as this results in segregation and pumping. During mixing keep the moisture content to within 2 to 3% to prevent segregation. Generally peak moisture content is seldom more than 4% for most G1 rock types.

IMPACT COMPACTION – CHANGING THE SHAPE OF COMPACTION
Revised form article published in the Civil Engineering Contractor

Traditional approaches to compaction has centred on compaction plant which has relied on either static weight, kneading action, or vibratory rolling of the soil to achieve the desired improvement in strength and bearing capacity.

However the demand for more efficient, economic and reliable compaction has led to the development of a radically new type of compaction plant, the non-circular impact compactor, based on the principle of high energy impact compaction. This revolutionary development has provided the construction industry with an improved compaction capability not easily possible with conventional plant. The distinguishing feature of all impact compactors is their non-circular compacting mass which have a series of points alternating with flat compacting faces. The compactors vary in relation to shape (three or five sided) drums and energy rating.

The capabilities of impact compaction include :

- **Deep in situ subgrade compaction –** the in-depth compaction capability of impact compactors allow deep, in-situ compaction of subgrades to be achieved up to depths of approximately 1,5m to 2,0m, depending on the type of compactor, soil type and moisture regime. This process eliminates the need to excavate, replace and compact material in thin layers as is often required when using conventional plant. A recent project where this capability was utilised concerned the compaction of the in situ subgrade of the northern runway of the massive Chek Lap Kok airport development in Hong Kong. The subgrade material consists of fairly variable marine sands, containing materials varying from a predominantly granular sandy or gravely material of low plasticity to a predominantly silty material of higher plasticity. Due to this variability in material, a method specification was adopted, treating the complete area with 40 passes of 25 kJ impact compactors. The stiffness obtained varied between 20 and 40 MPa on the various materials present, which generally indicates very satisfactory subgrade conditions. A unique benefit realised through the use of the impact compactor is that the complete area was effectively 'proof rolled' by the compactor, assuring the elimination of any weak spots.

- **Thick lift compaction of fills –** in contrast to the relatively thin lift compaction capabilities of conventional plant, impact compactors are capable of achieving thick lift, often single layered compaction of fill, in layers as thick as 600 – 1500mm.

- **Treatment of 'collapsible' soils –** impact compactors are particularly well suited to compacting 'collapsible' soils, which occurs in various parts of the country. These soils contain particles which are lightly cemented together in a highly voided structure and are susceptible to 'collapse' in service due to an increase in either moisture and/or surface loading. Treatment of this material with conventional compaction plant would normally require large amounts of water to be added to the soil, an exercise that can be prohibitively expensive and logistically impossible if vast areas are to be treated and water is scarce or expensive. Projects completed utilising this capability have included the treatment of vast areas earmarked for residential housing development, where the treatment has eliminated the need for the use of expensive raft foundations and enabled the use of standard strip foundations. Extensive trials were recently run on a site near the town of Kriel, to illustrate and quantify this capability. Geology at the site consists of sandstone which is present at depths in excess of 3,0m to as much as 8,0m. The overlaying soil profile is a fairly uniform wind blown slightly clayey fine sandy silt which is highly compressible. Of the most significant results was the fact that a 12ton vibratory roller only managed to induce settlement of 140mm, while the impact compactors induced settlements of up to 600mm for the 25kj compactor. This represents a settlement percentage of 30% over the top 2.0m, as opposed to the 7% settlement percentage achieved by the vibro. As part of the compaction trials a visual settlement indicator was constructed, clearly showing the depth influence of the impact compactor up to 2.0m deep

- **Reduced water requirements -** because of the high energy delivered per blow, the optimum moisture content for a given soil when using impact compaction is generally well below that of Modified AASHTO optimum. This allows impact compactors to be effective in dry regions where water for construction purposes is scarce and expensive. In such circumstance water requirements for subgrade compaction can be substantially reduced.

- **Compaction of rock fill –** the high intensity, in-depth capability of impact compactors makes them well suited for effectively compacting rock fill. This is achieved by subjecting the rock fill to a substantial dynamic compaction force which is able to re-arrange the rock fragments into a dense, interlocking stable mass well able to resist lesser service loads without deformation.

6. GENERAL OVERVIEW OF SECTION 2 TO SECTION 5

6.1 Key points in this section

- Vibratory compactors
- Compaction tests and laboratory tests
- Vibroflotation
- Damage caused by vibrations

PART VI : SUMMING UP IN PRACTICAL TERMS

A LARGE number of different types of vibrating rollers are used as discussed in Chapter 5. The frequency varies between 20 and 80 Hz and the nominal amplitude between 0,3 and 2,5 mm. Some types of vibrating rollers operate at a higher frequency in combination with a low nominal amplitude. Other types have relatively low frequencies and large amplitudes. The frequency can often be changed by changing the engine speed. There are also vibrating rollers with adjustable eccentric moment so that the nominal amplitude can be varied. Tractor-drawn vibrating rollers usually have a centrifugal force which is two to three times the total mass of the roller.

The Road Research Laboratory in England initiated in 1945 (and is still conducting) extensive compaction tests with different types of static and vibratory compactors. An early investigation by Hunt (1946) indicated a higher density was obtained for a coarse sand by a 215 kg vibrating roller than by a 12 tonne rubber tyred roller or by an 8 tonne static smooth wheel roller. The layer thickness was 220 mm. Later compaction tests were carried out on a number of standardised soils (gravel-sand-clay, well-graded sand, uniformly graded fine sands, sandy clay and heavy clay). The first three of these soils contained about 15 % fines. Detailed results from the compaction tests are available in a number of special reports. The test results have been summarised and discussed by Lewis (1954, 1960, 1961, 1967). According to Lewis, vibrating rollers massing up to 1 tonne could only be used to compact granular soils. Tractor-drawn vibrating rollers with mass between 3,75 and 8,5 tonne were suitable for all soil types, also clays. The required number of passes for vibratory rollers was four to six. The maximum layer thicknesses, which were recommended for the test vibratory compactors, are relatively low. The main reason is probably the comparatively high content of fines in the tested soils. Lewis (1961) has published results which show a relationship between the dry density of the compacted soil and the frequency of the vibrations, see Figure 6.1. In the tested frequency range, the effect of changes in frequency was significant only with granular soils.

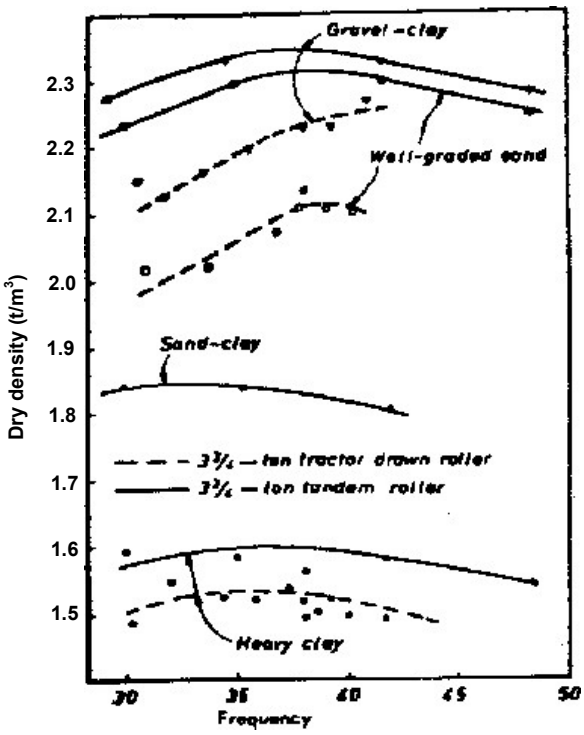


Figure 6.1 : Relationship between dry density of the compacted soils and frequency of two types of roller according to Lewis (1961)

Lewis (1961) has also reported results from compaction tests on a well-graded sand with 3,75 tonne tractor-drawn roller at different towing speeds and different number of passes. These test results indicated that a larger towing speed must be compensated by a larger number of passes in order to reach a given relative compaction. It was found that the relative compaction was mainly dependent on the total vibration time on a given surface area.

The test results obtained at The Road Research Laboratory have been discussed and analysed by Johnson and Sallberg (1960); Morris and Cochrans (1964), and others. Johnson and Sallberg pointed out that a good correlation was obtained for vibrating rollers between the degree of compaction and the static mass per centimetre of roll width (S.L.L). Compaction tests have also been carried out at the National Swedish Road Research Institute (Bruzelius, 1954). Typical Swedish soils (coarse gravel, sand, silt and moraine) were compacted in 0,6 to 1,0 m lifts. A considerably higher relative compaction was obtained by a 3 tonne tractor-drawn vibrating roller and also by a 1,5 tonne vibrating plate compactor than by static rollers. The maximum effective compaction depth ranged from 0,5 to 1,0 m for the vibratory compactors. The surface density of the soil was generally lower than the density at some depth below the surface. The maximum dry density was obtained at a depth of 100 to 300 mm.

Extensive compaction tests

Two manually-guided vibrating rollers massing 250 kg and 1,6 tonne have been tested at the Central Road Research Laboratory in India. The frequency was one of the factors which was varied during the tests (Gokhale and Rao, 1957). Garbotz and Theiner (1959) at the Technische Hochschule, Aachen, Germany, have carried out extensive compaction tests with static smooth wheel rollers, vibrating rollers and vibrating plate compactors. Garbotz and Theiner found that uniformly graded cohesionless materials could not be compacted to a higher density by vibrating rollers than by static smooth wheel rollers. The depth effect of vibratory compactors, however, was considerably larger than for smooth rollers. Vibratory compaction gave the highest degree of compaction for well-graded cohesionless soils.

Garbotz and Theiner also found that the degree of compaction of gravel increased when the frequency of a 1,65 tonne tractor-drawn vibrating roller was increased from 23 to 37 Hz. A further increase in frequency up to 50 Hz caused a small decrease of density. For the compaction of cohesionless soils Garbotz (1964) recommends for vibratory rollers the combination of a relatively high frequency and a low amplitude.

Johnson and Sallberg (1960) have reviewed the factors influencing the field compaction of soils. In this report are summarised results from compaction tests available up to that time.

Forssblad (1965) has published results from compaction tests on sand with 1,4 and 2,3 tonne tractor-drawn vibrating rollers and with vibrating plate compactors massing 40,120 and 400 kg. The influence of the towing speed and of the number of passes were investigated for 3,3 tonne tractor-drawn roller. The required number of passes to reach a specified density increased with increasing towing velocity. Test results indicated that the surface capacity was approximately independent of the rolling speed between 3 and 6 km/h. Lower speeds than normal are recommended when the layer thickness is large and when a very high degree of compaction is required.

D'Appolonia, Whitman and D'Appolonia (1969) have investigated the compaction of a uniform dune sand by a 5 tonne and by a 3 tonne vibrating roller. The layer thickness was large. Two passes of the 5 tonne roller gave the specified compaction (75 % relative density) down to a depth of about 600mm. The compaction was not sufficient, however, close to the surface down to a depth of 250mm. The density at depths larger than about 250 mm increased substantially with increasing number of passes. D'Appolonia *et al* recognised three different compaction zones. The soil near the surface was considered to be overcompacted. Below this zone to a depth of 600 mm the compaction was believed to be determined by the maximum and the minimum dynamic stress generated in the soil by the roller. The minimum dynamic stress must be low enough to allow the particles to move. The compaction of the soil below a depth of about 600 mm was considered to be caused mainly by the repetitions of stress (number of load cycles).

Hall (1968) carried out compaction tests on crushed limestone, sand and lean clay. The sand contained 8 % of material smaller than 0,06 mm. The following types of rollers were investigated.

a) Tractor-drawn vibrating roller, mass 3,2 tonne, low frequency.
b) Tractor-drawn vibrating roller, mass 1,4 tonne, high frequency.
c) Self-propelled vibrating roller, mass 2,4 tonne, low frequency.
d) Rubber-tyred static roller, mass 50 tonne.

Rollers (a) and (d) gave, for the crushed limestone and the clay, the highest relative compaction. The compaction by the two rollers was similar. On sand, compactor (c) gave the best overall results. Roller (b) gave satisfactory results only on the sand.

Considering the size of the rollers (a) and (d) it is very interesting to note that a 3 tonne vibratory roller yielded the same density as a 50 tonne rubber-tyred static roller.

Heavy vibrating rollers are often used to compact rock fill and granular soils containing large stones and boulders. Field compaction tests indicate that rock fill can be efficiently compacted in thicker layers than sand and gravel. The large depth effects which are obtained with vibrating rollers on rock fill is probably due to the high impact forces which are generated when the vibrating drum comes in contact with large stones. Also the damping in rock fill is relatively low.

Vibratory compaction of rock fill usually causes a reduction of the volume by 5 to 7%. Field experience indicates that 10 to 15 tonne vibratory rollers with 5 to 10 passes can compact rock fill in 1,0 to 2,0m lifts. Vibrating rollers with mass between 3 and 5 tonne are used to compact rock fill in 0,5 to 1,0 m lifts.

Heavy vibrating rollers are also used to compact natural sand deposits with a low initial density in order to increase the bearing capacity and to reduce the settlements. The sand may either be fully or partially saturated. If the depth is relatively large, up to 20 or 30 passes may be required. Borings and other measurements have indicated a maximum depth effect of 2,5 to 3m for 10 to 15 tonne vibrating rollers (Baker and Moorhouse, 1968).

Winter compaction of soil and rock fill has been investigated in Canada, Sweden, Norway, Finland, USA and other countries. Frozen soils except clean and dry gravel, crushed rock and rock fill can generally not be compacted to the same dry density as unfrozen soils. The difficulties with winter compaction increase with increasing water content and with decreasing grain size and temperature (Bernell, 1965). Vibrating padfoot rollers have been used successfully to compact gravel during the winter. Due to the high contact pressures lumps of frozen soil can be crushed. Test results indicate, however, that also with this method the dry density of compacted gravel is about 10% lower than that of the same soil compacted during the summer (Bernell, 1965).

During the winter unfrozen soil can often be obtained from excavations. When the unfrozen soil is spread out in cold weather, the surface freezes rapidly. It is therefore, important to compact the soil immediately after it has been placed.

Vibrating plate compactors

The most commonly used plate compactors mass between 100 and 500 kg. With two eccentrics working synchronously it is possible to obtain a directed vibratory force and to move the vibrator during the compaction. By adjusting the eccentric element, the velocity and the direction of the motion can be varied. A plate vibrator can also be moved with only one rotating eccentric if the eccentric is placed at the front end of the bottomplate.

The frequency of heavy self-propelled vibrating plate compactors is usually between 12 to 20 Hz while the frequency of small vibrating plate compactors can be as high as 70 to 90 Hz (Table 3). The amplitudes vary between 0,5 and 5 mm. The centrifugal force is usually 5 to 10 times the total static load (kN) of the vibrator.

Bernhard has carried out compaction tests with a 1,5 tonne vibrating plate compactor and a relative compaction of 95% with respect to the Mod AASHTO compaction test was obtained for uniform silty sand down to a depth of 750 mm. Converse (1952, 1954) has investigated the compaction of a uniform beach sand by a vibrating plate compactor with a mass of about 5 ton. The dry density of the beach sand was at least 95% with respect to the Mod AASHTO compaction test down to a depth of 0,45 and 0,60 m and 93 to 96% between 0,60 to 1,50m below the ground surface.

Aldous and Wills (1952) have investigated the vibratory compaction of crushed rock, gravel and sand. The soils were compacted in 150 to 300 mm lifts by vibrating plate compactors massing either 70 to 170 kg. The maximum density of the gravel was reached after two passes. The unit mass of the soil in the lower half of the 300 mm lift was less than in the upper half.

Compaction tests with 1480 kg, 140 kg and 40 kg plate compactors have been carried out by Streck and Schmidbauer (1954). It was possible to compact a sand to a relative density of 85 % within a depth of 600 to 800 mm with the 1,48 t plate compactor, within a depth of 200 to 300 mm with the 140kg compactor and within a depth of 200 to 250 mm with the 40 kg compactor.

The Road Research Laboratory in England has made comprehensive investigations into vibrating plate compactors (Lewis, 1961). The depth to which soils could be compacted increased with increasing mass and with increasing contact area of the vibrators. It was possible to compact satisfactorily a well-graded sand to a depth of 300 to 450 mm with a 1,5 tonne or a 2 tonne plate compactor. The depth effect at the same mass was larger for the plate compactors than for the vibrating rollers. Lewis also found that fewer passes were required for a plate compactor than for a vibrating roller to reach a given density. The surface and the volume capacities given by Lewis (1967) for plate compactors were, however, much lower than those for vibrating rollers.

Forssblad (1965) has reported results from extensive series of compaction tests with vibrating rollers and plate compactors. A relative compaction of at least 90 % Mod AASHTO compaction was reached for the layer thicknesses given in Table 6.1.

Table 6.1 : Normal layer thickness for vibration of cohesionless soils with less than 5 to 10% of the material smaller than 0,06 mm

Compaction equipment	Layer thickness
Vibrating Plate Compactor, 100 to 200 kg	200 mm
Vibrating Roller, 1 to 2 ton	300 mm
Vibrating Plate Compactor, 400 to 600 kg	400 mm
Vibrating Roller, Tractor-drawn 3 – 4 ton	500 mm

Tests and practical experience indicate that it is possible to compact sand and gravel to a high relative density even in high lifts. Also, relatively fine sand with an average grain size between 0,06 and 0,2 mm can efficiently be compacted in high lifts by vibrating rollers or plate compactors. However, comparatively small amounts of fines (silt or clay size particles) will increase considerably the compaction effort required to reach a certain relative compaction. The layer thickness must therefore be decreased with increasing content of fines.

Vibroflotation

Vibroflotation has been used since about 1935 to compact saturated, cohesionless soils, of the kind found at river mouths, swamps etc. The specially designed large internal vibrators (vibro-floats) generally have a diameter of 37,5 cm and operate at a frequency of 30 Hz. The nominal amplitude is about 10 mm and the centrifugal force is up to 100 kN. The vibrators are provided with jetting devices which facilitate the insertion and the withdrawal of the vibrators. Sand or gravel is added during the vibration to compensate for the subsidence at the ground surface as the soil around the vibrofloats is compacted. With this method the soil can be compacted down to a depth

of 20 to 30 m. Vibroflotation has mainly been used to compact natural strata of sandy soils with a relatively high permeability. A maximum amount of 8 to 16% of particles smaller than 0,06mm can generally be allowed.

Test results by D'Appolonia et al. (1953) indicated that the relative density of a fine uniform sand is at least 70% to a distance of 1 m from the vibrofloats. A spacing of 2 m between the insertion points gave a minimum relative density of 70% throughout the compacted area. The distance from the vibrofloats to which the soil can be compacted appears to increase with increasing grain size and with increasing uniformity of the soil.

Abu-Wata and Said (1958) used vibroflotation to compact a uniform dune sand which had been placed under water. The spacing of the insertion points varied between 1,65 and 2,5m. The compaction of the soil was checked by soundings with a Dutch Cone Penetrometer. A substantial increase in the measured penetration resistance was observed close to the insertion points. The average initial penetration resistance before the compaction was about 60 MPa while the minimum penetration resistance after compaction at a spacing of the vibrofloats of 2,5 m was 120 MPa below a depth of 4,5 m. The penetration resistance increased with decreasing spacing of the vibrofloats. The increase in penetration resistance and of the compaction was small close to the surface.

Internal vibrators which are normally intended for concrete have also been used to compact saturated cohesionless soils (Wenner and Saare, 1954; and Bureau of Reclamation, 1963). According to Wenner and Saare (1954), sand and gravel can be compacted to 90 to 100% Mod AASHTO compaction test to a distance of 0,25 to 0,50 m from the insertion points.

Forssblad (1965) has reported results from compaction tests on sand and gravel with internal concrete vibrators. The size of the vibrator, the frequency and the amplitude were varied. A density between 90 and 100% Mod AASHTO maximum dry density was obtained. The radius of action of the vibrators was found to be smaller, and the required vibration time was found to be longer than in concrete.

Laboratory compaction tests

Laboratory compaction of cohesionless soils by vibration have been studied by many. The soil has, in the laboratory tests, usually been compacted in a Proctor mould placed on a vibrating table. The influence of frequency, amplitude and acceleration on the vibrating table as well as the influence of surcharge load, soil gradation and moisture content have been studied.

Schaffner (1962) has investigated the vibratory compaction of dry sand. Typical test results are shown in Figure 6.2. It can be seen that the void ratio decreased with increasing acceleration. The highest density was obtained at frequencies up to 100 Hz probably due to the very small amplitude at high frequencies.

Ortigosa (1968) and Whitman and Ortigosa (1968) found from vibratory compaction tests that the acceleration without surcharge should for a dry fine sand be at least 1 g. The maximum density was obtained at 2 g. When the acceleration was larger than 2 g the density decreased due to "over compaction". With surcharge load, the required acceleration was 1 to 3 g depending on the intensity of the overburden pressure. Also Dunglas (1967) has investigated the vibratory compaction of sand. Dunglas used a special test mould where water pressure was used as a surcharge load.

Forssblad (1965) found that the dry unit mass increased for a dry sand with increasing acceleration up to 2 g and up to 3 g when the same sand was compacted partially or fully saturated. The dry unit mass was approximately constant when the acceleration exceeded these limiting values. No surcharge was used during the experiments. Forssblad also found that the dry unit mass of the compacted soil was independent of the direction of the vibrations.

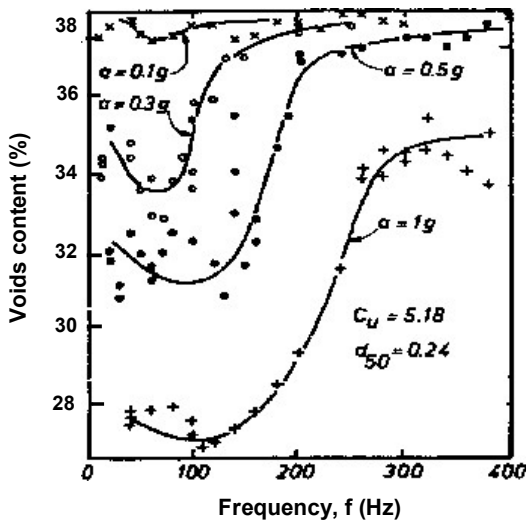


Figure 6.2 : Compaction of dry coarse sand

A vibrating table and a surcharge load or a vibrating tamper or hammer are used in the standardised vibratory tests methods, as illustrated in Figure 5.1. Pettibone and Hardin (1964) have found for the first mentioned method that the density of the compacted soil at a frequency of 60 cps increased with the increased amplitude of the vibrating table. The increase in density was insignificant when the vibration time was longer than 6 minutes.

Damage caused by vibrations

Vibratory soil compactors as well as blasting, pile driving and heavy road traffic may damage adjacent structures. Surface waves generated by medium and heavy mass vibratory compactors can often be observed at large distances from the vibrator. Since vibrations of very low intensity can be felt by man, the risk of damage to buildings and other structures is often exaggerated, however it must not be overlooked.

Measurements indicate that the damage is mainly dependent on the maximum velocity of the vibrations. The risk of damaging adjacent buildings is small if this velocity is less than 0,3 to 1,0 cm/s as indicated by Scheelhaase (1962); Susstrunk (1959); Rieher and Soden (1961) and others. The risk of damage is also dependent on the quality of the building and the type of foundation.

Ground oscillations from vibratory compactors have been measured by Susstrunk (1959) and Forssblad (1965). An oscillation velocity of 1,0 cm/sec was observed by Forssblad at a maximum distance of 5,5 m from a 3,3 tonne tractor-drawn vibrating roller. There were large variations of the ground oscillations due to differences in the soil conditions.

Due to large variations in the soil conditions, in the quality of buildings and in size and type of the compactors, it is difficult to establish general rules about the risk of damage by vibratory compactors. This risk must, however, be carefully considered, especially when heavy vibratory compactors are used.

Buried pipes can also be damaged during vibratory compaction of trench-fills. Minimum thickness of the soil layer above the pipes during compaction by vibration or tamping are given in German specifications. Retaining walls, abutments and basements walls are sometimes damaged by the high lateral pressure which develop during the compaction of the backfill. Measurements indicate that these pressures can be several times larger than those used in the design. Also, the lateral earth pressures, which remain after the compaction are often high and can approach the Rankine passive earth pressure close to the ground surface.

Further research and advances

This report on the state-of-the-art vibratory soil compaction indicates that there is need for further research in this field. The following important points can be mentioned as possible fields for further research :

1. **Compaction mechanism**
   Transmission of the vibrations from vibrator to underlying or surrounding soil. Propagation of stress waves in the soil. Damping properties of soils at different frequencies, amplitudes, gradations, water contents, and so on.

2. **Special features of vibratory compaction**
   Rational relationships between suitable layer thickness and number of passes for different types of soils and vibratory compactors. Crushing of soil and rock particles during vibratory compaction.

3. **Compaction control**
   Development of efficient and rapid methods for compaction control of thick layers and for soils containing large stones. Development of improved laboratory compaction methods based on vibrations.

4. **Damage to structures**
   Damage to buildings by surface oscillations produced by vibratory compactors. Lateral earth pressures on retaining walls, abutment and basement walls during and after vibrating compaction of soils.

5. **Technical data**
   Recommendations about the technical data of vibratory compactors which should be included in specifications, technical reports, and so on.

7. SOME PRACTICAL ASPECTS OF COMPACTION

7.1 Introduction

Problems occasionally occur in the compaction process and as a result the specified densities are not achieved. Such problems can result in a significant reduction in the speed of construction and the concomitant increase in construction costs where compacted sections are rejected – sometimes repeatedly – due to poor densities and thus have to be ripped up and re-compacted.

Some of the precautions which can be taken to prevent density failures are discussed below.

7.2 Training

The personnel who control and supervise the compaction process should be thoroughly trained in every aspect of the process. The most important are: a knowledge of the different soil types encountered, the significance of the compaction optimum moisture content, moisture distribution in the layer, a knowledge of the different types of compaction plant and the management of the plant during the compaction process.

7.3 Compaction Moisture Content

The compaction moisture content is 0,5 to 1,0% lower than the MOD AASHTO optimum moisture content. The compaction moisture content is usually judged visually in the field by simply taking a fist full of soil and squeezing it firmly. If it is too dry the soil will fall apart and as the moisture content is increased the soil will gradually become more cohesive. When it just adheres together to form a ball it will be close to or at the compaction moisture content. If the moisture content is further increased so that the soil ball is easily formed and can be moulded then it is too wet for compaction.

Most experienced supervisors can judge the correct compaction moisture content quite accurately by simple visual assessment as described above. If there is any uncertainty, laboratory samples at the correct moisture content can be prepared for comparison in the field. Rapid field moisture content can also be undertaken. However, in practice the “hand squeeze” method usually works quite well. The attainment of the correct compaction moisture content is of paramount importance. If the field moisture content deviates significantly from this moisture content either on the low (dry) or the high (wet) side the specified density will not be achieved no matter how intense or prolonged the compaction process.

7.4 The Distribution of Moisture before Compaction

It is essential that the water be distributed uniformly in the longitudinal, transverse and vertical directions throughout the layer before compaction commences.

Longitudinal Distribution :
The compaction water is normally applied in several applications of a pressurised water distributor. The approximate volume of water required should be calculated in advance. If the water is added in several applications it is easier to control the moisture content since the moisture content can be judged between applications whilst the water is being added and whilst mixing continues. It is of vital importance that the correct quantity of water be applied uniformly in the longitudinal direction since subsequent mixing does not move the material in the longitudinal direction – for example if there is an over application in one area a wet spot will be created and it will stay wet no matter how long mixing continues.

Vertical Distribution :
The rotovator and plough are excellent for mixing water in the vertical direction. The motor grader is also very efficient in mixing vertically.

Transverse Distribution :
The motor grader mixes very effectively in the transverse direction and irregularities in the transverse distribution can be corrected by cross grading. Good transverse and vertical mixing can be achieved by the motor grader moving the material to and fro.

7.5 Initial Trial Sections

At the start of each construction project or whenever new material is encountered it is advisable to do a trial section to determine the best compaction procedure with particular reference to the correct field compaction moisture content and compaction plant management. If the personnel are very experienced they may dispense with a trial but in many instances a trial section is advisable and more productive than a “hit and miss” approach.

7.6 Layer Thickness and Depth of Compaction

For a considerable time the usual pavement layer compacted thickness was 150mm which is equivalent to a loose thickness of about 200mm. However, the development of improved compaction equipment and in particular vibratory equipment makes it possible to compact greater layer thicknesses. Compacted layer thicknesses of 200 (+/- 260mm loose) have been frequently constructed in recent years. 17 ton vibratory rollers capable of compacting 250mm layers (+/- 320mm loose) to a density of 85% BRD have also become available recently.

In the roadbed compaction in depth is desirable and can be achieved with vibratory and heavy pneumatic rollers. In most cases this is done at the insitu moisture content and is not regarded as controlled compaction.

7.7 Compaction of Crushed Stone Base Material

Crushed stone base materials are classified as either G1, G2 or G3. These materials are essentially the same although the G2 and G3 materials allow the use of natural fines. The G1, G2 or G3 materials also have progressively lower density requirements. G1 material has a tightly controlled grading designed to provide the highest possible density. It is at or very close to the Fuller maximum density grading. It consists of high quality crushed rock that will not break down during the compaction. A compaction process especially designed to compact the G1 material to the highest possible density has been developed. The compaction is carried out in two phases.

First phase: The compaction moisture content is usually between 4 and 6% and the standard compaction procedure is carried out to provide density of approximately 85% apparent relative density. This is initially done by the use of grid rollers and vibratory rollers without vibration, to settle the loose material uniformly. Vibratory compaction is then used with the necessary adjustments in frequency and amplitude – commencing with high amplitude / low frequency and then changing to low amplitude / high frequency as the material densifies. Heavy pneumatic and smooth, steel wheel “static” rollers are also used.

Second phase: The second phase consists of saturating the compacted G1 base by the addition of more water followed by rolling with smooth steel wheel rollers. As the water is forced to the surface the excess fines are expelled and swept off. The remaining fines act as a lubricant which enables the coarser particles to pack closer together to provide a tightly interlocked layer. The excess fines on the surface are swept off to provide a mosaic of tightly packed aggregate. Densities of at least 88% apparent relative density are thus obtained.

7.8 Control of Vibration

The principles of amplitude and frequency have been discussed in SECTION 6 but some examples of their use in practice are given below.

In paragraph 7.7 above, first phase vibratory compaction, is carried out in 3 stages namely: 1st stage : on the loose material the vibration is turned off and the roller is used without vibration. This is to obtain a gentle initial settlement from the loose condition which has been accurately placed and spread to the required levels.

2nd stage : After initial densification the vibration is set at high amplitude and low frequency, the high amplitude providing maximum vibratory compaction throughout the thickness of the layer.

3rd stage : At this stage the material has been compacted in depth and surface finishing is necessary. A high amplitude will disturb the surface and may loosen it. The vibration is therefore changed to low amplitude and high frequency, which effectively reduces the depth influence of the compaction effort. If the vibratory roller available does not have variable frequency and amplitude a different vibratory roller or, more commonly, a pneumatic tyred roller is used for this 3rd stage.

In practice there have been instances where satisfactory densities have been achieved in the base course but the surface is loose and not suitable for the application of a seal. This has occurred because the final rolling was done with a high amplitude which has the effect of disturbing or decompacting the upper 5 to 10mm of the surface. As stated finishing should be done with a low amplitude and can even proceed without vibration, followed by very light watering and pneumatic rolling.

For the compaction of cohesive materials a high amplitude should be used.

Vibrating padfoot rollers are effective in compacting thick layers but they should not be used during the finishing process.

7.9 Comments from contractors

Whilst the application of the theories of compaction accompanied by the correct procedures provide satisfactory results most of the time many instances occur in practice when results are unsatisfactory. Specific densities cannot be achieved or only with great difficulty. The causes of such problems are varied and are often, though not always, associated with unusual soil types. In an attempt to clarify at least some of these problems comments were sought from nine contractors, each with many years experience in compaction. The following questions were submitted to the contractors :

"1) What do you consider to be the most important factor (or factors) in soil compaction?

2) Can you mention instances where :

(a) compaction problems occurred, stating also the cause and
(b) what steps were taken to solve the problems.

If you have comments on any aspect of compaction especially economic aspects, this will also be very welcome."

The answers received from the 9 contractors are given below more or less in verbatim.

CONTRACTOR ONE :

1) What do you consider to be the most important factor (or factors) in soil compaction?

- Grading
- Water content
- Equipment

2) Can you mention instances where :

(a) compaction problems occurred, stating also the cause and

Compaction of a cement stabilized decomposed basalt layer (200mm thick)

- Could not get required compaction with 12ton vibratory rollers, which is normally not a problem.
- The layer had to be constructed in two layers of 100mm, as the top portion of the 200mm layer densified to such an extend that it absorbed all the vibrating energy, leaving the bottom portion uncompacted.

(b) what steps were taken to solve the problems.

- Compaction was achieved by constructing two 100mm layers with two 12ton vibratory rollers.
- Compaction was also later achieved with one 18ton vibratory roller and one 12ton vibratory roller.

CONTRACTOR TWO :

1) What do you consider to be the most important factor (or factors) in soil compaction?

- Correct moisture (Note !!)
- Correct compaction equipment
- Grading modulus of less than 0,5 a problem
- Over compacting
- Material selection
- Speed of various compaction equipment
- Direction of travel on some vibratory rollers

NOTE : Not specifically Mod AASHTO OMC but OMC for compaction effort of equipment.

2) Can you mention instances where compaction problems occurred, stating also the cause and what steps were taken to solve the problems:

- Southern Freeway (Johannesburg) : Material used – blend of 40% Ash and 60% Mine sand – over saturate work, compact with PTR and cover a.s.a.p.
- Sandy / Silty material, correct OMC using a 19 ton vibrator roller not achieving compaction, change to old fashioned tractor and tamper / grid combination.
- Ferricrete (Utrecht area) : Highly voided material causing low densities. To achieve density required, increase OMC and worked like crusher run and pneumatic equipment for compaction.
- N4 – North : Very deep layer of windblown sand with a collapsing potential in the top 2m. Plant used was a 16ton single ton drawn vibrator (ABG).
- Compaction of roadbed in vlei areas, use a pioneer or rock layer or backtip gravel, roll with impact roller building up pore pressures. Allow pore

pressures to dissipate (four days) and repeat until a platform is built (last method not preferred)

- N4 – Middelburg : Felsite gravel has to be wetted well above optimum and left to dry back to optimum before densities could be obtained. (Editor's note: The felsite aggregate was very absorbent and sucked the water from the soil mortar leaving the material too dry for compaction. The water demand of the aggregate had to be satisfied first by pre-wetting.)

NOTE : Beware of over compaction as this is a waste of diesel (Liquid Gold). Do a trial / test section to determine number of passes and then train your staff and operators accordingly.

CONTRACTOR THREE :

1) What do you consider to be the most important factor (or factors) in soil compaction.

- Grading
- Moisture content
- Roller operator

2) Can you mention instances where compaction problems occurred, stating also the cause and what steps were taken to solve the problems:

- Bredasdorp Runway : G1 Base - 3 wheel static roller lifted densities by $^{3}/_{4}$% +/- 6 passes.
- Nduli Ceres : CPA road to Ouder Swaarmoed, slush rolling (G4 Base) with Dynapac, raised densities by 2 to 3%.

CONTRACTOR FOUR :

1) What do you consider to be the most important factor (or factors) in soil compaction.

- Soil type
- Compaction equipment to be used
- OMC / What water content to mix
- What base underneath
- Thickness and compaction to be achieved
- Grading

2) Can you mention instances where compaction problems occurred, stating also the cause and what steps were taken to solve the problems.

- Density Failure: Coarse grading (increased our compaction effort and paid more attention to moisture).
- Better compaction plant (right plant for the right application).
- The use of pneumatic rollers has become a must for the higher specification roads.

NOTE : When we do cold in situ recycling, where we cannot afford density failures or rework, we spend a lot of time on in situ moisture content determination and equipment. This pushes the cost up by at least R4.00/m^3. The state of our plant plays a huge part in cost, as the breakdown time is limited to a minimum.

CONTRACTOR FIVE :

1) What do you consider to be the most important factor (or factors) in soil compaction.

- I feel that the most important factors are moisture and grading. If these are correct then the least effort is required to achieve compaction. It could, however be more economical to use more effort if the right equipment is available and there is a high cost in replacing the material or using water.

2) Can you mention instances where compaction problems occurred, stating also the cause and what steps were taken to solve the problems.

- I had numerous problems with fine or silty material, normally with a high organic content. The laboratory will generally classify the material as sand and one then needs to achieve 100% Mod AASHTO compaction.
- You generally have to mix water into the material using a rotovator or a grader. It is still difficult or impossible to achieve density due to the "Sponginess" of the material. Have at times had it reclassified and only had to achieve 93%, this is obviously easier to achieve. At times we have removed and replaced with clean sand.
- Clay is also difficult to compact as it reaches maximum density quickly and then drops off fast after even one pass extra. One generally would use a Troxler to set up a compaction curve versus passes and establish a maximum amount of passes for the operator.
- I have had various problems with G2, these are generally easy to solve after setting up rolling patterns or pneumatic rolling.
- Laterite has to be compacted on high amplitude first to stop it from compacting in the top layer and not allowing the effort to reach the lower part of the layer due to it's high CBR.

CONTRACTOR SIX :

1) Inability to achieve the specified 85% of BRD compaction – possible causes and remedies.

- Initial rolling pattern using a 12 ton dynapac single drum vibrating roller on high amplitude at a rate of 5 passes in total with 2 or 3 of those passes in reverse was found to be causing shear cracking and a loss of density. Low amplitude passes did little to increase density.
- After changing the rolling pattern and eliminating all high amplitude compaction in reverse we achieved a slight increase in density but this was not 100% conclusive.
- We then increased the 0.075mm rock flour content to 11% without altering the grading requirements of the larger sieve sizes, as the 0.075mm% was at the coarse end of the grading envelope (between 5% and 6%).
- We found that the layer then retained the moisture content to above OMC more efficiently and that the rock flour by coating the aggregate on the smaller stone sizes aided compaction significantly. Compaction was also achieved with fewer roller passes.
- The shape of the larger aggregate, 13mm – 37mm was generally flat and we initially thought that this was the cause of lack of density but after the addition of the rock flour we discounted this as a cause.
- Generally the G2 gradings were on the coarse side with the exception of 0,075mm sieve size, which was very coarse.
- Prior to modifying the G2 the % compaction achieved was 83% BRD and after modification we easily achieved 85% BRD.
- We have since learned that the CSIR carry out a "Shake Down BDR test" which establishes the most likely or maximum field compaction that a material with a certain grading etc. will give. It is possible therefore that the

G2 in question may have only been able to deliver a density of 83% to 84% prior to modification.

CONTRACTOR SEVEN :

Single drum rollers vibratory rollers are frequently used on road layer works which are less than 300mm thick, e.g. SSG subbase, base. These rollers are designed for deep compaction (amplitudes of 1,5mm for example) on earth fills, and in my opinion are the wrong option for layer works. What are needed are heavy rollers with a high amplitude setting of approximately 1mm and a low amplitudes setting of approximately 0,5mm. This to me gives the right range for layer works. Thereafter if grading and MC are correct, compaction is straightforward.

CONTRACTOR EIGHT :

1) What do you consider to be the most important factor (or factors) in soil compaction.

- One is undoubtedly always lead by the factors making up the MOD curve i.e. compaction energy and moisture. In sequence of priority one will always start with the compaction moisture content and play around with it. I say this realizing that moisture and compaction energy is always closely linked. Compaction energy is seldom a factor that one can play with simply because of the economic considerations for the contractor. What one can play with is the energy for a specific type of compaction equipment, i.e. number of passes and ballasting of the equipment. If equipped, one can vary the tyre pressures of the pneumatic tyre rollers and in the case of vibratory rollers the frequencies.
- The type of roller and or the combination of rollers if one has the luxury of playing with this. Most certainly the type of material here becomes very important in selecting the ideal roller and or combination of rollers. Compare for instance the approach when compacting hot asphalt, deep milled material and sand for instance.

2) Can you mention instances where compaction problems occurred, stating also the cause and what steps were taken to solve the problems:

Compaction problems - there are numerous examples of instances but let me mention just three:

- Some years ago we constructed the Rundu to Grootfontein road (Namibia). The fill was a single sized windblown sand and the selected subgrade on top of the fill was calcrete – the only two types of material available in the area anyway. After many test sections we managed to achieve the specified density on the sand only after saturation and using a 30 ton Alberette tow behind roller. Now how does one pull a 30 ton roller on sand? A tyre wheeled tractor or dozer could not do it and we had to use something that “floats” on the sand and opted for a D6 dozer, which again would loosen the top 100mm of sand with its tracks. Again no solution. We then opted for the blanket approach whereby we would place and compact the selected subgrade (blanket) at our risk before testing the top of fill for density. This option worked fairly well.
- A second example was on the Haalenberg to Aus (Namibia) road where the fill was also windblown sand but being in the desert there is of course no water. Again after many test sections dry compaction was opted for using pneumatic rollers and also using the blanket technique – testing always the layer below the completed layer. When the final level is reached one overfills and windrow remove the excess material afterwards.

The trick here was that the sand had to be bone dry else the specification could not be met.

- Another interesting one was on the rehabilitation of the Bloemfontein to Trompsburg road where the base was an in-situ emulsion treated crushed stone material. At the time this contract was the first big conventional ETB rehabilitation contract and not much was known about this technique. Here we had to work way beyond the optimum moisture content and had to add 1% cement to the emulsion treated material to act as a catalyst for the breaking of the emulsion. The work was also carried out in winter, which meant that many a morning one would arrive at the previous day's section to find it completely destroyed because the moisture in the layer froze during the night and was completely loose due to the related expansion of the frozen water.

NOTE : As for economics one is amazed at the ignorance of some of the young engineers leaving varsity without any knowledge of compaction methods and techniques. Compaction plant only makes up about 10% of the total processing costs (your rollers only work about 3 hours per day). We would then still try to reduce this 10% by opting for smaller and less combination of rollers. Our approach has always been that any layers that fail due to compaction after a test section was built to determine methods etc. is the supervisors responsibility and not easily forgiven – this comes in when one considers a bonus at the end of the year or contract completion.

CONTRACTOR NINE :

1) What do you consider to be the most important factor (or factors) in soil compaction.

- Moisture Content (Optimum + 0,5 to 1,0%)
- Gradings and flakiness of stone (failures can be expected if percentage passing the 4.25 mm sieve is greater than 40% and 2,00mm is greater than 60% ???
- Layer thickness
- Speed of roller
- Frequency and amplitude of roller
- Weight of roller
- Condition of underlying layers
- Rolling pattern
- Profile of layer i.e. level
- Construct trial section
- Testing procedure: with sand remove top layer 50 to 100mm before testing

2) Can you mention instances where compaction problems occurred, stating also the cause and what steps were taken to solve the problems?

- N2 Mosselbay : Compacting 300 mm layer with 10 ton roller – used 10 ton padfoot, formed biscuit layers – changed to 20 ton roller smooth.
- Fairtrees (Durbanville, Cape Town) : G2 Base course compacted to 88% bulk density, failure due to low moisture and low 0,425 mm + 2 mm dust.
- N1 (Section in Karoo) : G1 - 150 mm thick, 88% bulk required used pneumatic roller to tighten surface.
- N7 : 300 mm in situ recycling used 10 ton vibratory roller and 20 ton vibratory roller.

3) Can you comment on any aspect of compaction with regards to economic aspects?

- Use of good condition plant that can work efficiently
- Water truck to spray water evenly over the whole area
- Amplitude and frequency of rollers to be checked

- Use of competent laboratory
- Padfoot rollers for clay materials
- Levelness of the surface compacted
- Over compaction
- Trial section
- Size of roller vs. volume / day
- Correct roller for type of material

7.10 Compaction of stabilized materials

The treatment of road building materials with a nominal quantity of stabilizing agent is termed stabilization. Stabilization is undertaken in order to improve the material's strength and load spreading ability. The stabilizing agents that are used most frequently comprise lime, cement and bitumen. The latter is generally used in combination with cement. Blast furnace slag and fly ash are also often used in combination of either cement or lime. A process that is closely related to stabilisation is modification. Modification is achieved is a similar fashion to stabilization but using less stabilising agent. The aim of modification is to improve the characteristics of the material (reduce PI and increase CBR) without necessarily increasing the compressive strength by means of cementation significantly.

Although the strength of stabilised materials is significantly affected by the cementing action of the stabiliser it could appear that the strength contribution provided by the density of the materials is of less significance. This is however not the case as the required density for stabilised materials remains the same as that of the untreated material namely 95% of Mod AASHTO density for subbase material and 90% and 93% for subgrade and selected subgrade layers respectively. There is an exception though : base (upper and lower) is required to be compacted to 97% of Mod AASHTO density, which is one percent lower than the ordinary specification for natural gravel base material.

From the abovementioned it can be inferred that the compaction of chemically treated material is just as important as the compaction of untreated materials.

The compaction process of treated material is essentially the same as the compaction of untreated material, with the following two exceptions :

- Since the stabilizing agents (lime and cement) starts to hydrate upon wetting it provides a limited period of time in which the compaction process must be completed.
- Should the density of the treated material not conform to the required density after compaction it can only be reworked once before the material becomes unsuitable for use. This is due to the fact that the smaller particles start to bond and form conglomerates, which essentially changes the grading of the material as well as rendering the material non-plastic leading to ever decreasing densities upon subsequent reprocessing and compaction.
- It follows that the compaction procedure of (cementitious) stabilised materials must be very carefully and correctly undertaken at the first attempt bearing in mind the time constraint specified for cementitious materials and the costly consequences of a density failure.

The following procedure has been found to be successful when material is to be treated with a chemical stabilising agent, essentially lime and cement stabilisation.

- Once the material has been placed and the layer thickness and levels are correct the material should be wetted to increase the moisture content of the material to the optimum compaction moisture content. It is essential that the moisture be worked through the layer properly to ensure that the moisture content is as consistent as practically possible.
- The following day the moisture content should be checked to ensure that it is still at the compaction optimum moisture content, if not additional moisture should be added and mixed into the layer. Thereafter the stabilising agent can be placed and distributed on the surface. However before the stabilising agent is placed the layer should receive one pass

with a light roller to provide a surface on which the stabilising agent can be distributed evenly. It is important that the correct quantity of stabilizing agent be placed and that the spreading be undertaken such that the most uniform distribution that is practically possible is achieved. Wetting of the material on the previous day before the stabiliser is added is essential since the addition of water to the dry mix results in preferential wetting of the cement which forms small balls thus interfering with the uniform distribution of the cement. However if the previously wetted material is slightly too dry then additional water may be added to bring it up to optimum compaction moisture content.

- After the distribution of the stabilising agent and final adjustment of the moisture content, if necessary, have been completed mixing should commence without delay. The mixing can be undertaken using a grader and a disc-harrow or plough. It may be necessary to add further water to the material during the mixing process as the water demand of the stabilising agent may reduce the moisture content to below the optimum compaction moisture content. Alternatively the affect that the stabilising agent has on the material (flocculation) may increase the material's water demand. This needs to be assessed in the laboratory prior to construction. During the mixing process it is essential that the full depth of the layer be incorporated without damaging the layer below. It is important to ensure that the stabilising agent is properly distributed throughout the layer and not concentrated in the top part of the layer.
- After the mixing has been completed and the moisture content in the treated material is at the correct level, compaction should commence without delay. Figure 7.1 shows the detrimental effect that occurs if there is a delay between the mixing and compaction processes. As can be seen the achievable density reduces rapidly with time, which adversely affects the unconfined compressive strength of the material. It is required that the entire stabilisation process (from spreading the stabilising agent to final finishing) be undertaken within 8 hours if cement is used and within 10 hours if lime is used. For the modification process the time limit is 48 hours regardless of whether lime or cement is used. Note : with some cements the completion time is less than 8 hours – further research into this aspect for South African cements is currently underway.
- Subsequently the layer needs to be cured (kept moist) for 7 days during which only the required construction vehicles are allowed to travel on the layer. Alternatively a curing membrane (bitumen emulsion) should be applied whilst the layer is still damp. The curing membrane can also be the material of the following layer.

Should the required density not be achieved the layer has to be ripped and reworked. During the reworking additional stabiliser needs to be added (approximately 50% of the original quantity used). During this process a new Mod AASHTO density needs to be determined that is to be used in conjunction with the field densities. If the compacted layer does not conform to the required density after being reworked twice it has to be removed and replaced with fresh untreated material. This is obviously very costly and stresses the importance of getting the compaction right the first time.

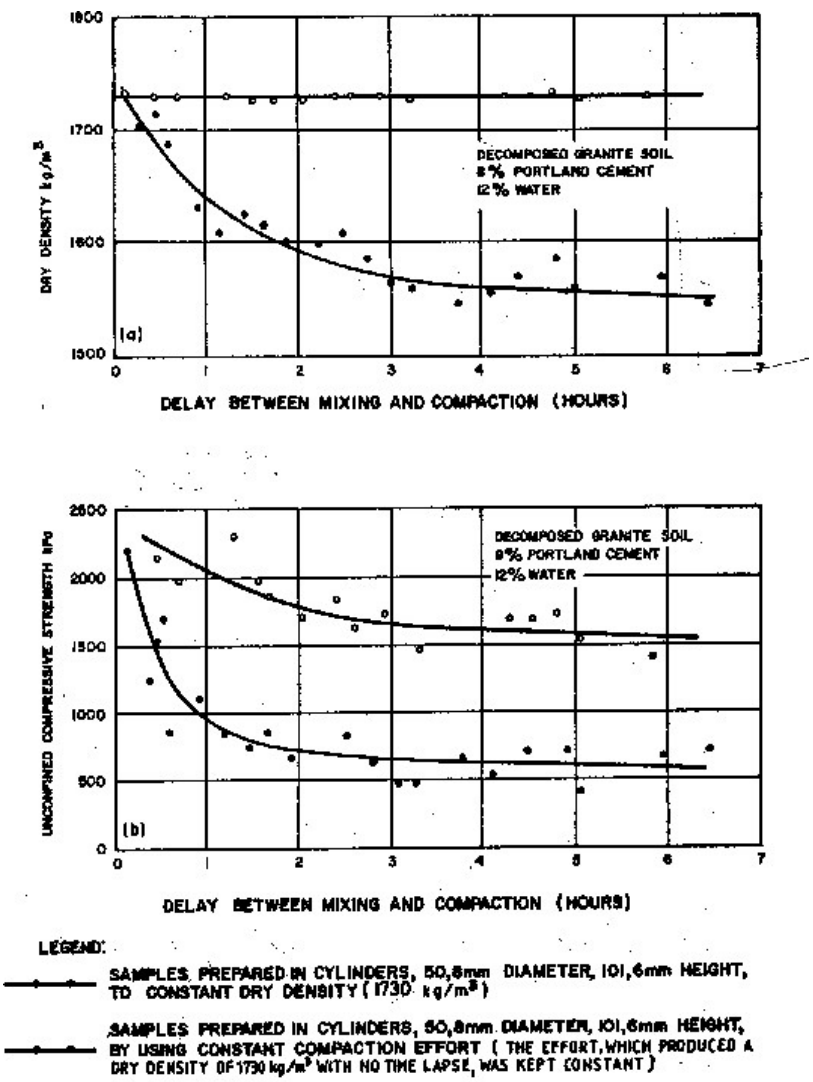


FIGURE 3

The effect of a time lapse between mixing and compaction on dry density and unconfined compressive strength in laboratory testing

Figure 7.1 (from TRH13)

7.11 <u>In situ recycling</u>

Due to the strong emphasis on the rehabilitation of existing roads during the last decade, there was an impetus in the industry to developed in situ recycling machines. These machines have the ability to rework the material in existing roads more effectively as it obviates the need to rip and rework the material using graders disc-harrows and ploughs. The main feature of these machines is the milling drum which is fitted with numerous pedestals each fitted with a pick-holder and milling pick, see Photograph 7.1. This milling drum turns within the milling chamber that houses a spraybar, which is fitted with a series of injection nozzles. Stabilising agent and moisture can be added to the material through these nozzles, see Photograph No.7.2.

At present these machines are predominantly used for the stabilisation of existing materials using bitumen (either bitumen emulsion or foamed bitumen) and cement. As such the

Photograph No. 7.1 : Milling drum of in situ recycling machine

Photograph No. 7.2 : In situ recycling in progress. Note the milling chamber between the wheels. The reworked material behind the recycling machine contains the stabilising agents and the compaction moisture.

cement powder is applied on the existing road surface prior to the milling process during which the bituminous stabilising agent is added. However these machines are also suitable to undertake ordinary cement or lime stabilisation.

The recycling process can be either a one-pass or a two-pass process. The differences are as follows:

One-pass : Due to the ability of these machines to add the compaction moisture and the stabilising agent in a controlled fashion it is only necessary to pass over the existing road once. During this continuous process the material is milled, moisture and stabilising agent added, mixed and placed.
Two-pass : Some contractors feel more comfortable if the existing material in the pavement is milled first and the moisture content increased to the desired level. Following behind the recycling machine is the roller, which pre-compacts the material lightly. Thereafter some of

the surplus material is removed followed by the second pass during which the stabilising agent(s) is worked into the material.

Since the water is added in a single application in both the abovementioned cases it is vital that the water addition is very carefully controlled. Great care should also be taken at the longitudinal joints, see Figure 7.2, to ensure that that there is no overlapping resulting in a double application of water. In this regard an important feature of these milling machines is that the nozzles on the spraybar are individually controlled allowing the machine to overlap previously milled areas without applying additional moisture or stabilising agent, see Figure 7.2 and Photograph No. 7.3.

In both the abovementioned procedures it is essential that the rollers follow directly behind the recycling machine in order to compact the material whilst its moisture content is at the desired level, see Photograph No. 7.3. After the material has been compacted the final finishes are undertaken. These comprise cutting the layer to the correct levels, apply a diluted bitumen emulsion and undertake the final rolling, usually with a pneumatic roller. The most frequently used density specification in such recycling is 102% of Mod AASHTO density. In cases where crushed stone base course materials were recycled a density specification of 85% of bulk relative density has been used. In the majority of cases to date satisfactorily compaction levels were achieved and the contractors appear to have few problems in this regard.

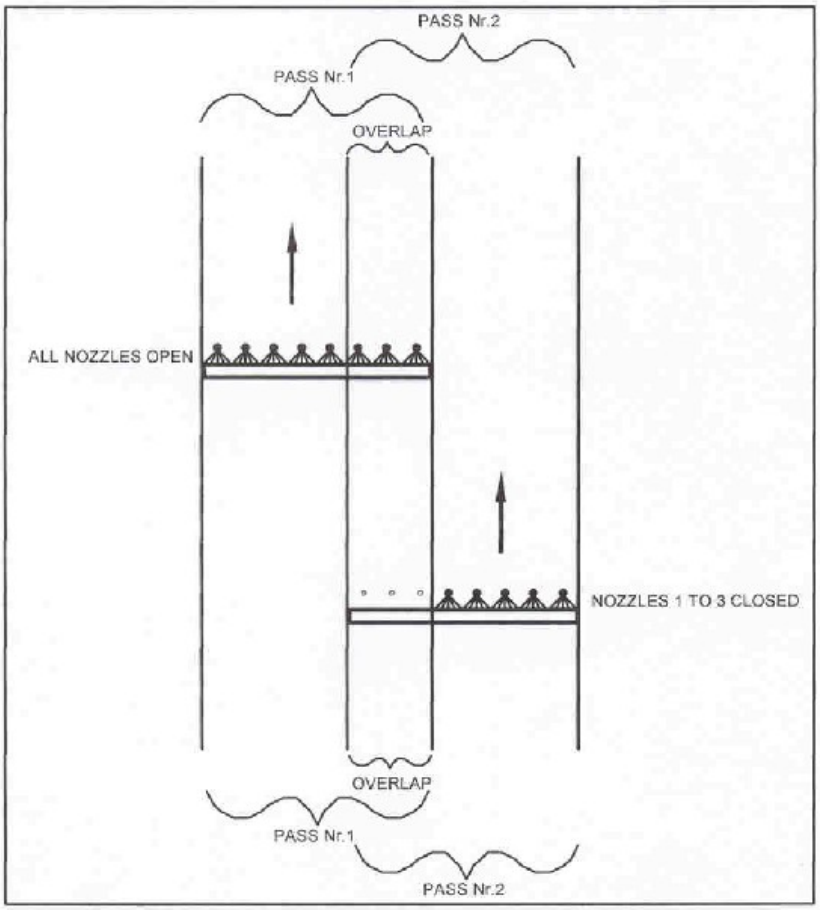


Figure 7.2 : Nozzle configuration and operation where there is an overlap between two areas that are to be recycled.

Photograph No. 7.3 : Two-pass recycling process in action. Note roller in the background following behind the recycling machine. Also notice that the cement powder is spread in front of the recycling machine and that no cement powder is provided where the overlap is to be.

8. EVALUATION OF COMPACTION

8.1 Compaction requirements

As stated in Section 2 relative compaction is defined as the ratio of the achieved field density to a reference density. The reference density can be one of the following standard densities depending on the material to be compacted :

- Proctor density
- Mod AASHTO density
- Bulk density of solid particles
- Apparent density of solid particles

The compaction levels that are required in the standard COLTO specifications were discussed in Section 2 , dealing with soil classification. However for ease of reference it is repeated in Table 8.1 below :

Table 8.1 : Standard compaction requirements

TRH14 Material Classification	Material type	Required compaction (COLTO 1998)
Fill (G10, G9)	Natural gravel Free-draining sand	Natural gravel : 90% Mod AASHTO, Sand :100% Mod AASHTO
Selected Subgrade Layer (G7, G8)	Natural gravel Free-draining sand	93% or 95% Mod AASHTO Sand : 100% Mod AASHTO
Subbase Layer (G5, G6)	Natural gravel	95% or 97% Mod AASHTO 95% or 96 % for stabilized materials
Base (G4)	Natural gravel	98% or 100% Mod AASHTO 97% or 98% for stabilized materials
Base (G3)	Crushed stone	98% or 100% Mod AASHTO
Base (G2)	Crushed stone	85% of Bulk Relative Density
Base (G1)	Crushed stone	88% of Apparent Relative Density

Note : There are opinions that the apparent relative density specification that is applicable to G1 base course materials is too harsh and could lead to impossible density requirements in the cases that aggregate with very porous surfaces are used. See “Relative Compaction and the Significance of Error” at the back of this section.

The determination of the various reference densities is discussed in the following paragraphs.

8.2 Description of the various reference densities

8.2.1 Proctor test (TMH1, Method A7 [1] ; ASTM, Method D698-00A)

The Proctor test comprises the compaction of loose soil in a mould with fixed dimensions using a standard tamping bar. The details of the test apparatus are as follows :

- Cylindrical mould : 152,4mm (6 inch) inside diameter
- Height : 127mm (5 inch)
- Tamping bar weight : 2,495 kg (5,5 lb)
- Tamping bar diameter : 50,8mm (2 inch)
- Drop height of tamping bar : 304,8mm (12 inch)

Air dried soil is first scalped on the 19mm size sieve in order to ensure that all the material passes the 19mm sieve. Thereafter the material is mixed with a predetermined quantity of water to prepare a sample for compaction. This material is then compacted in three individual layers (equal thickness) in the cylindrical mould. 55 blows with the tamping bar are applied on each of the layers in the mould. After the third layer has been compacted the material in the mould is worked flush

into the mould with a flat steel bar in order to ensure that the volume of the material is known accurately. A sample of the material is taken and dried in the oven in order to determine the moisture content of the material accurately. By knowing the mass, volume and moisture content of the sample the dry density can be determined. Dry density is defined as the mass of the dry soil (excluding the mass of the water) divided by the volume that the sample occupies. By repeating this exercise for varying moisture contents Figure 8.1 shown below can be constructed.

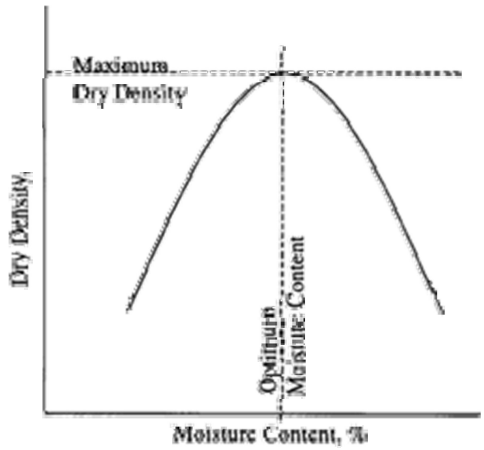


Figure 8.1: Proctor Curve

From Figure 8.1 the maximum dry density and the optimum compaction moisture content of a material can be determined.

It should be noted that the standard number of blows for the Proctor test is given as 25 in the ASTM test method and 55 in the TMH1 test method. This would therefore indicate that the Proctor test as defined in TMH1 and the Proctor test as defined in the ASTM are not the same compaction test.

8.2.2 Modified AASHO test (TMH 1, Method A7 [1])

As stated the Proctor test was developed to monitor the compaction of earth fill dams. A higher density was required in road construction and this led to the development of a test known as the Mod AASHTO test which has the following features :

The weight of the tamping bar was increased to 4,536 kg (10 lb)
The drop height of the tamping bar was increased to 457,2mm (18inch)
The mould is filled in five equally thick layers, each layer receiving 55 blows.

As can be seen the modification was aimed at increasing the compaction energy in order to achieve higher dry density.

The Mod AASHTO test is undertaken in a similar fashion than the Proctor test and from the dry density moisture content relationship the maximum dry density and the optimum compaction moisture content is determined.

The Mod AASHTO density test has been used for many years and has become the industry norm for the evaluation of compaction of natural gravel layers in road construction.

8.2.3 Density of the aggregate (TMH1 [1], Methods B14 and B15)

With the introduction of high quality crushed stone materials, with maximum particle sizes varying between 26,5mm or 37,5mm, it was found that the Modified AASHTO test was no longer a suitable reference density. The reason for this is that in the Mod AASHTO test the samples are scalped on the 19mm sieve thereby excluding all particles with sizes larger than 19mm since particles larger

than 19mm could not be accommodated in the 152,4mm diameter cylindrical mould. In crushed stone material, which is a well-graded material, scalping will remove between 15 to 30% of the material and thus change the material significantly. It was therefore decided to express the compaction for the material relative to the density of the aggregate particles.

When considering the density of the aggregate one has to distinguish between the bulk density and the apparent density as explained below.

Since aggregate particles contain small imperfections (known as fissures) on the surface, which do not affect the size or shape of the aggregate, they can be considered as insignificant with regard to the manner in which the aggregate particles will pack together in the layer. Therefore these fissures will not affect the actual compaction of the material. However when measuring the density of the aggregate particles the fissures need to be considered as follows :

Apparent Density : The apparent density of the aggregate is defined as the mass of the solids divided by the volume of the solids only, excluding the volume of the fissures (permeable voids) in the surface of the aggregate particles. The density is referred to as being apparent since it is impossible to know whether there are bubbles in the aggregate (impermeable voids), which are not connected to the surface of the aggregate.

$$\textit{apparent density} = \frac{\textit{(mass of solids)}}{\textit{(volume solids + volume impermeable voids)}}$$

Bulk Density : The permeable voids can be determined by the weight of the absorbed water on the surface of the aggregate. Subsequently the bulk relative density of the aggregate is determined as the mass of the solids, but now divided by the volume of the solids plus the volume of the impermeable voids and the volume of the permeable (surface imperfections) voids.

$$\textit{bulk density} = \frac{\textit{(mass of solids)}}{\textit{(volume solids + volume impermeable voids + permeable voids)}}$$

As can be seen from the above formulae the apparent density of the aggregate will always be larger than the bulk density of the aggregate. However if there is zero permeable voids on the surface of the aggregate the apparent density and the bulk density will be the same. A similar, more detailed explanation of the above differences between apparent and bulk density is given by Savage [2], this reference is included in full at the end of Section 8.

By dividing the bulk density and the apparent density by the density of water (which is accepted as 1000 kg/m^3) are the bulk relative density and the apparent relative density respectively are obtained.

8.3 Determination of field density

At present the field density of compacted material is determined using the following two methods :

- Sand replacement method
- Nuclear density gauge

See page 8.15 (Figure 2) and page 8.18 (Figure 3) for graphical representation.

However due to the ease with which the nuclear density gauge operates and its improved accuracy it is used more frequently than the sand replacement test. According to the commercial laboratories in South Africa the nuclear density gauge is now used almost exclusively to measure the field density of compacted materials in road construction.

The principles on which these tests operate are discussed briefly in the following paragraphs.

8.3.1 Sand-replacement method (TMH1, Method A10(a))

The principle of the sand replacement test is to determine the mass and volume of a soil sample obtained from the compacted layer. This is achieved by excavating a small hole in the compacted layer and carefully retaining the excavated material. Thereafter the volume of the hole, from which the material was excavated, is determined by pouring sand, with a know density, into the hole in a defined way. By knowing the mass of the sand and its density its volume can be determined. The dry density of the material can be determined once the mass of the dry material and the volume of the excavation are known. Photograph 8.1 shows the sand replacement test apparatus whilst the test is being undertaken.

A limitation of the sand replacement method is that the sand does not always fill the sample hole entirely which leads to underestimating the volume of the hole and consequently overestimating the density of the material. This is especially prevalent in crushed stone materials [2]. Since the sand replacement method is also a slow and somewhat laborious method it is often disallowed in project specifications and the contractor is solely reliant on his nuclear density gauge.

8.3.2 Nuclear density measurement (TMH1, Method A10(b)T)

Around 1960 William Troxler initiated the development of the nuclear density gauge, see Photograph 8.2 below.

The nuclear density gauge works on the principle of transmitting gamma rays through the soil which is to be tested. The photons that are emitted from the source travel through the material to a detector situated in the base of the gauge. The photons collide with electrons in the soil that are in the path between the source of the photons and the detector; the higher the density of the material, the lower the number of photons reaching the detector. Therefore the ratio of the number of photons emitted and the number of photons detected can be related to the density of the material.

Over the last decade the use of the nuclear density gauge has increased significantly and has become the density measure of choice. It is however important to realize that the nuclear density gauge is also subject to error. A typical error that can occur is related to the disturbance in the material, which is created whilst preparing the peg-hole.

The disturbance occurs when the peg is driven into the compacted material and pushes the material particles away from the peg, in effect loosening the material. When the probe of the nuclear density gauge is inserted in the peg-hole and the photons transmitted to the detector, the photons have to pass across the disturbed face of the peg hole that is at a lower density than the remainder of the material in the layer. This localized low density affects the overall density measurement and can lead to underestimating the density of the material.

In order to compensate for this error it has to be quantified first. This is achieved by preparing a controlled pilot hole during compaction. Typically a steel peg is inserted in a known area in the layer whilst the material is still loose and thus a pilot hole is formed. This procedure is repeated after each roller pass, thereby allowing the material to compact around the hole without disturbing the sides of the hole. After the compaction is completed the peg is driven into the compacted material close to the controlled pilot hole such that the base of the nuclear density gauge fits between the two holes. Thus by measuring the density through each of the holes, with the base of the gauge in the same position, the effect of the disturbed faces in the peg-hole can be determined. The difference in the measured density can be used subsequently to correct the density measurement taken through ordinary prepared peg-holes [2].

Errors can also occur if the gauge is placed on a rough surface; fine sand is usually used to provide a smooth bedding surface for the gauge.

Photograph No. 8.1 : Sand replacement test in progress

Photograph No. 8.2 : Nuclear density gauge “The Troxler”

8.4 Evaluation of compaction

Once the field density of a compacted material and the appropriate reference density have been determined the level of compaction that has been achieved can be determined readily.

In any scientific test, which comprise taking measurements and samples, there are unavoidable errors that are inherent to that test. The principle sources of error are : operator error, sampling error and equipment error. However, with careful equipment calibration and operator training the test results can be accepted with a reasonable degree of confidence. Therefore it has to be accepted that the obtained measurements are not precise and will be variable.

In the case of field density testing, significant variations in results could also occur due to inherent variations in the density itself i.e. from one test point to another. In addition there is also a degree of inherent material variability from test point to test point. Moroney [7] emphasizes the aforementioned as follows *" We must remember, also, that all sampling investigations are subject to experimental error. Careful consideration to design enables this error to be kept to a minimum. No sampling investigation can give a result, except in terms of probability. What order of probability will satisfy us? The confidence we are able to extend to our final results depends both on the magnitude of interfering effects and on the size of our sample. Only very careful forethought and hindsight can make any sampling investigation other than an incoherent pile of data."*

Statistical procedures have been developed to determine whether a set of density results is at an acceptable level or whether it should be rejected for a given probability. These procedures are discussed below.

8.4.1 Statistical evaluation

From the above it follows that there will always be variation in the compaction test results that are obtained for a particular layer. For example if the required density of a particular layer is 98% Mod AASHTO density the actual measured values, obtained in randomly selected positions, could be as shown in Figure 8.2.

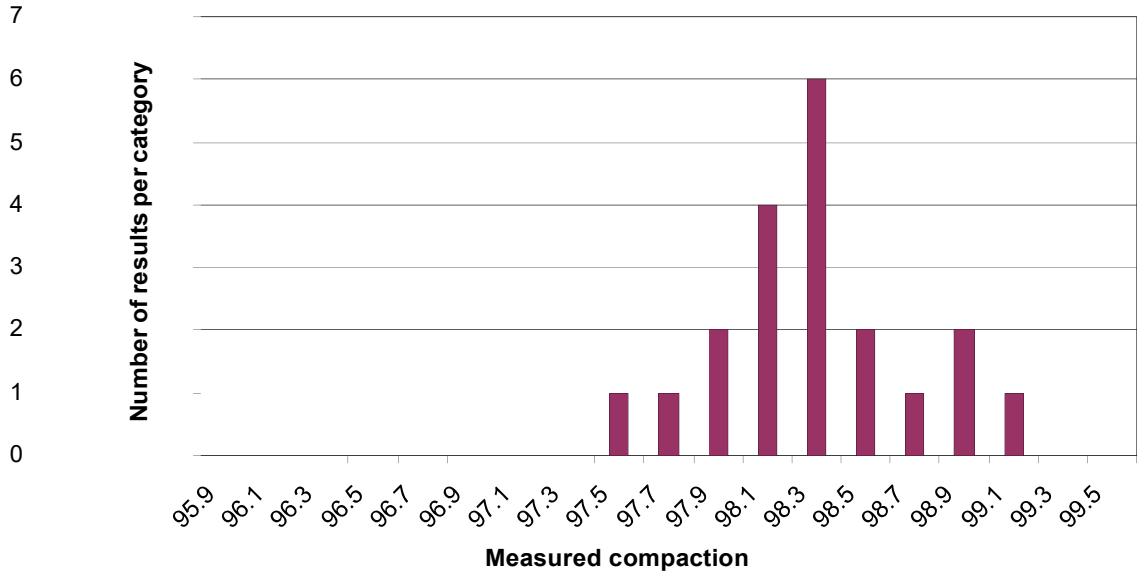


Figure 8.2 : Frequency histogram of compaction measurements

It can be seen that there are individual test results lower than the required 98% compaction level and it follows that there is a probability that in some areas the compaction might be lower than the required 98%, which is unacceptable. In order to cater for the aforementioned, a limit is placed on the probability that a certain percentage of the measurements may fall below the specified level. An

estimate of this can be achieved by normalizing the above frequency histogram, i.e. by dividing the number of test results per category by the total number of test results obtained, see Figure 8.3.

In addition, if it is accepted that the measured compaction test results are normally distributed (distributed evenly around the mean value of the data set) the sample's mean value and standard deviation can be used to estimate the parameters μ and σ respectively in the equation of the Normal Distribution's probability density function, see equation below.

$$f(y) = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(y-\mu)^2}{2\sigma^2}} \qquad \sigma > 0, \quad -\infty < \mu < \infty, \quad -\infty < y < \infty$$

The normalized theoretical distribution for the above example is also shown in Figure 8.3. It can be seen that the theoretical distribution has an even distribution around the mean value of sample data.

Since the distributions in Figure 8.3 have been normalized the frequencies in the various categories sum to unity.

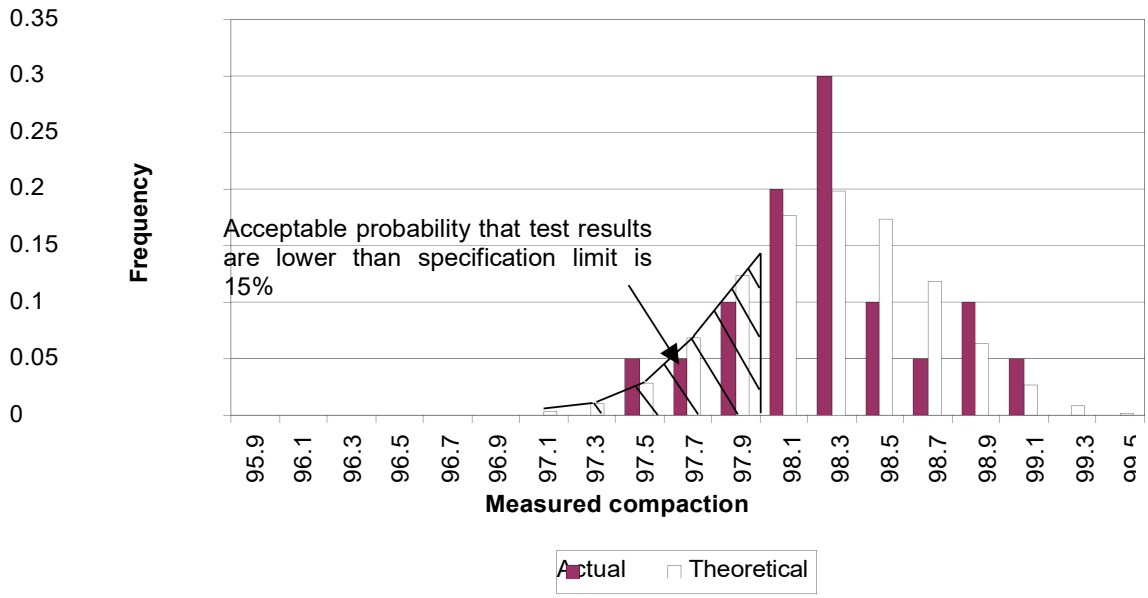


Figure 8.3 : Normalised frequency distribution of compaction measurements

Since the area under the normalized Normal Distribution sums to unity the area (hatched) that is to the left of the 98% specification limit represents the probability that the achieved compaction could be lower than 98%.

To ensure that the compaction is at an acceptable level, a limit on the probability that the compaction is below the acceptance limit must be placed. It must be remembered that a 100% probability cannot be specified in practice. Over the years the industry has accepted a 15% limit,
i.e. a probability that 15% of the density test results could be lower than the specification limit. Therefore statistical acceptance criteria are designed to ensure that there is an 85% probability that the density test results would be greater than the specification limit. In the case of compaction, which is a lower limit specification, this can be achieved by ensuring that the mean value of a particular data set is sufficiently larger than the specification limit. The magnitude of this off-set needs to incorporate the following factors :

The variation of the sample data
The size of the sample (number of test results)

The risk that unacceptable work is accepted

The approach that has been developed in South Africa is described below with reference to the lower limit specification applicable to compaction. For a comprehensive description of the acceptance control procedure the reader is referred to the TRH5 document and the work of Semmelink [4], [5].

For the lower limit specification which is applicable to compaction, the acceptance limit is as follows :

La = Ls + ka.Sn

Where : La – Acceptance limit
Ls – Property specification
ka – Probability factor which is a function of sample size, percentage failure and risk that an unacceptable product will be accepted.
Sn – Actual standard deviation of test results

If a specific property of a material fails to comply with the acceptance limit by a small margin the material can still be accepted conditionally. In order to determine this range of values a rejection limit is determined using the same procedure, which is defined as follows:

Lr = Ls + kr.Sn

Where : Lr - Rejection limit
kr - Probability factor, which is a function of sample size, percentage failure and risk than an unacceptable product will be conditionally accepted.

The above judgement scheme is shown graphically in Figure 8.4 for three hypothetical sets of test results X_{n1}, X_{n2} and X_{n3}. For the purpose of this example all three the hypothetical sets of test results have the same standard deviation. In Figure 8.4 it can be seen that if the actual sample mean is greater than the La limit (X_{n3}) the product would be acceptable as the percentage failure is less than the allowed percentage failure.

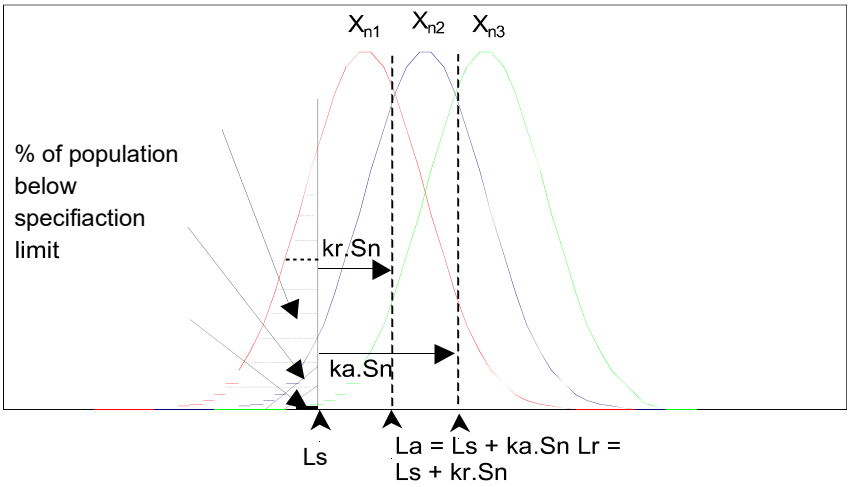


Figure 8.4 : Approach to existing judgement scheme (lower limit specification)

If the actual sample mean falls between the Lr and La limits (X_{n2}) the product would be considered as conditionally acceptable and if the actual sample mean is lower than the Lr limit (X_{n1}) the product is considered to be unacceptable as the probability that too many test results fall below the specification limit is too great. If the material is accepted conditionally a payment reduction factor is incorporated in order to compensate the client for the increased probability that the density test

results would not conform to the specification, i.e. not receiving what he is entitled to. The payment reduction factor (fr) is as follows :

$$fr = 0{,}67 + 0{,}3(Xn - Lr) / (La - Lr)$$

The above is merely a brief description of one element in the existing judgement scheme. There are still aspects like upper limit specifications, double limit specifications, evaluation of outliers, resubmission of test results, etc. These are all very important aspects and therefore the reader is referred to the COLTO standard specifications where the judgement scheme is discussed in its entirety. For reasons of brevity a flow diagram is given in Figure 8.5, which describes the acceptance control procedure. Following the flow diagram three examples are given to illustrate how the statistical acceptance control procedure is used to evaluate compaction. Thereafter, following the references, a paper by Savage [2] is provided to describe the significance of test errors and its influence of the evaluation of compaction.

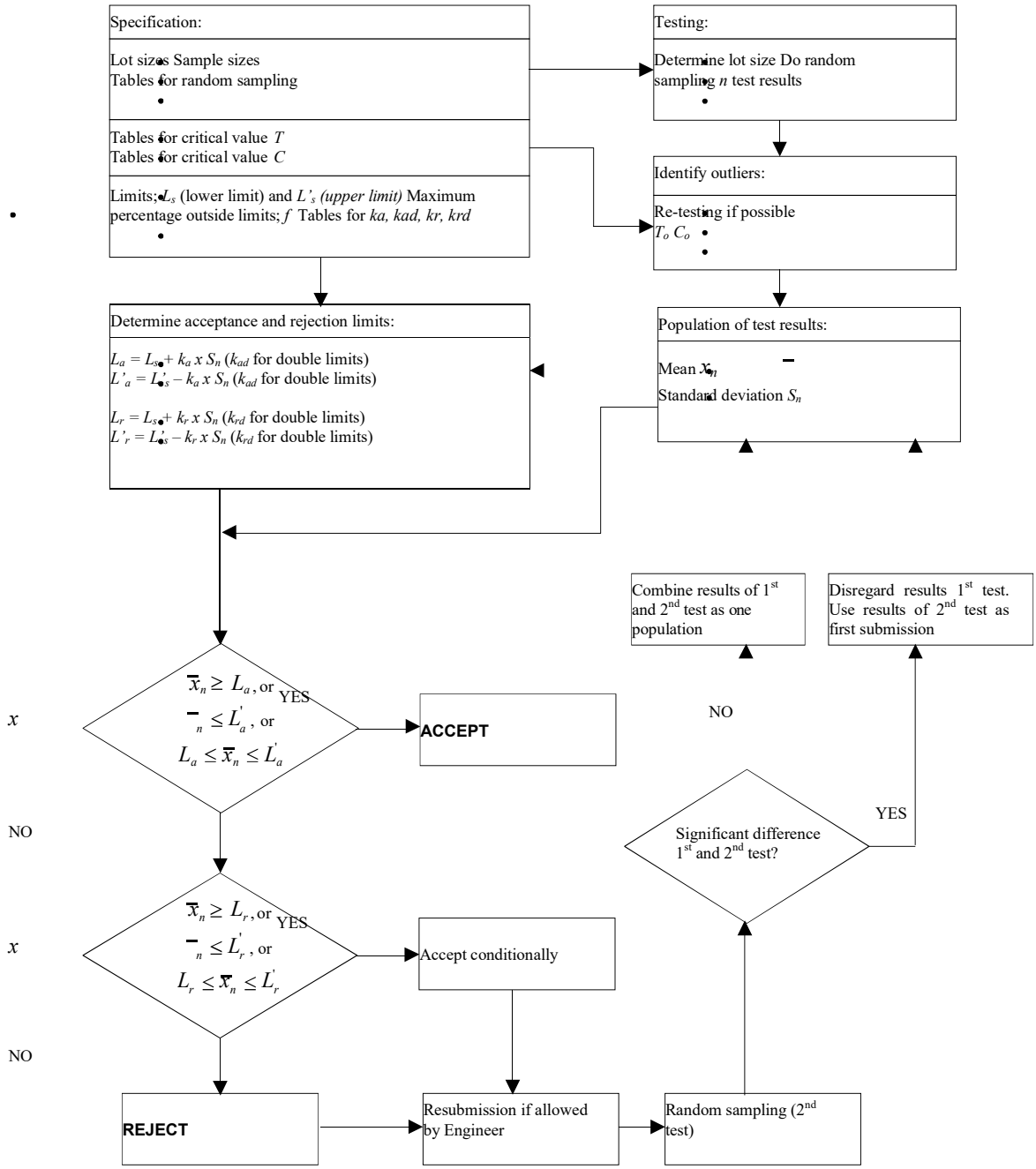


Figure 8.5 : Flow diagram of COLTO 1998 acceptance control procedure

8.4.2 Examples : Evaluating compaction

Example 1 : A contractor completed the compaction of a natural gravel subbase layer, which is required to be compacted to 95% of Mod AASHTO density. Subsequently contractor requests the Resident Engineer to accept the portion of work as being complete. In order to evaluate compaction the Resident Engineer requires the field laboratory to measure the density at 6 randomly selected positions within the demarcated area. The following test results were obtained : 95,3%; 95,1%; 94,8%; 95,0%; 95,2%; 94,9%. Does the Resident Engineer accept the work ?

Answer :

The mean (Xn) of the measured compaction is 95,05% and the standard deviation (Sn) is 0,187%. Since

compaction is a lower limit specification the following applies :

La = Ls + ka.Sn

From Table 8206/3 in the COLTO specification it is determined that the maximum percentage defectives (ϕ) that are allowed for the compaction of subbase material is 15%. From Table 8206/4 the value of ka that corresponds to ϕ=15% and n=6 (number of measurements) is determined, in this case ka = 0,358. Therefore the acceptance limit becomes :

La = 95,0 + 0,358 x 0,187 = 95,067 %

Since Xn < La the achieved compaction is not acceptable, but could still be conditionally acceptable. To determine this, the following applies :

Lr = Ls + kr.Sn

Where kr = 0,089 for n=6 and ϕ = 15%, Therefore the rejection limit becomes :

Lr = 95 + 0,089 x 0,187 = 95,017 %

Since Lr =95,017% < Xn = 95,050% < La = 95,067% the work can be accepted conditionally and the partial payment factor (fr) applied.

fr = 0,67 + 0,3(Xn – Lr) / (La-Lr)

fr = 0,67 + 0,3 (95,05 – 95,017) / (95,067 – 95,017) = 0,868

Therefore in lieu of outright rejection the contractor can receive 86,8% of full payment for this portion of the work and it is accepted in the permanent work.

Example 2 :

A Contractor is required to compact a G1 base course layer to 88,0 % of apparent relative density. The field laboratory undertakes 8 measurements at randomly selected positions within the demarcated area. The 8 results of the density measurements are as follows : 2386,6 kg/m^3; 2392,0kg/m^3; 2394,7 kg/m^3; 2392,0 kg/m^3; 2389,3 kg/m^3; 2386,6 kg/m^3; 2389,3 kg/m^3; 2381,1 kg/m^3. The laboratory further determined that the apparent relative density of the material is 2712,0 kg/m^3. Is the achieved compaction level acceptable?

Answer:

The achieved compaction test results are : 88,0%; 88,2%; 88,3%; 88,2%, 88,1%; 88,0%; 88,1%, 87,8%.

The mean value (Xn) for the tests is 88,088% and the standard deviation (Sn) is 0,155%. The compaction is a lower limit specification and the following applies :

La = Ls + ka.Sn

From Table 8206/3 in the COLTO specification it is determined that the maximum percentage defectives (ϕ) that are allowed for the compaction of base material is 15%. From Table 8206/4 the value of ka that corresponds to ϕ=15% and n=8 (number of measurements) is determined, in this case ka = 0,440. Therefore the acceptance limit becomes :

La = 88,0 + 0,440 x 0,155 = 88,068% Xn =

88,088% > La = 88,068%

Therefore the compaction of the G1 base course material is acceptable.

<u>Example 3 :</u>

A contractor is required to compact a lower selected layer to 90% Mod AASHTO density. The field laboratory undertakes 4 measurements at randomly selected positions within the demarcated area. The 4 results are as follows : 89,3%; 90,0%; 90,1%; 90,1%. Is the achieved compaction acceptable?

<u>Answer :</u>

The mean value (Xn) for the measurements is 89,875% and the standard deviation is (Sn) 0,386%. Compaction is a lower limit specification and the following applies :

La = Ls + ka.Sn

From Table 8206/3 in the COLTO specification it is determined that the maximum percentage defectives (ϕ) that are allowed for the compaction of fill material is 15%. From Table 8206/4 the value of ka that corresponds to ϕ=15% and n=4 (number of measurements) is determined, in this case ka = 0,220. Therefore the acceptance limit becomes :

La = 90 + 0,22 x 0,699 = 90,154% Xn =

89,875% < La = 90,154%

Since Xn < La the achieved compaction is not acceptable, but could still be conditionally acceptable. To determine this, the following applies :

Lr = Ls + kr.Sn

Where kr = -0,148 for n=4 and ϕ = 15%, Therefore the rejection limit becomes :

Lr = 90,0% – 0,148 x 0,220 = 89,967%

Since Xn = 89,875 < Lr = 89,967% the work cannot be accepted and will consequently have to be reworked.

References

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3. Soil mechanics for road engineers, 1952

4. National Institute for Transport and Road Research of the CSIR, TRH5, Statistical concepts and its application in the quality control of road construction, 1987.

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6. Committee of Land Transport Officials, Standard Specifications for Road and Bridge Works for State Road Authorities, published by South African Institution of Civil Engineering, 1998

7. Moroney, M.J., ”Facts from figures”, Penguin Books, 1963

RELATIVE COMPACTION AND THE SIGNIFICANCE OF ERROR

INTRODUCTION

There are few contractors who have not experienced difficulties at some time or other in achieving a specified relative compaction (RC). This problem is most common when crushed stone layers are to be compacted. There are of course a magnitude of reasons why the compaction of road foundation layers can give disappointing results, even after considerable rolling effort but it is not the purpose of this article to discuss these factors; what will be reviewed is the question of the accuracy of the testing and interpretation of results both in the field and the laboratory which can seriously prejudice proper acceptance of work.

Relative compaction is equal to the ratio of field density to that of a laboratory determined standard density expressed as a percent. Expressed mathematically

$RC = (D_{field} / D_{lab}) \times 100$ Where D_{field} - the density of a layer as

measured in the field.

D_{lab} - the laboratory determined standard density done on the field material.

Errors can and do occur in the determination of both the field as well as the laboratory control densities.

Let us consider the significance of these errors, which are often overlooked or ignored when applying this relationship of field / laboratory densities particularly when tests are performed for purposes of approval and payment for completed work.

Laboratory tests

The most popular laboratory standards to which field densities are related are:

1. the Modified AASHTO density and
2. the Relative Density of the so called “solids”.

We will now study these two standards with a view to understand and appreciate the importance of errors that could be associated with their determination.

The Modified AASHTO density

The repeatability of this test on virtually identical samples is often quite low. It is quite common for two tests on material considered to be uniform to differ by more than 50 kg/m^3, see Table 1. The same soil was separated into four sub–samples and each tested separately. It is significant to observe that the same material can yield a range of densities that can differ by as much as 124 kg/m^3. Relating a field density to the highest and lowest of these modified density values can result in relative compaction values differing by as much as 4,3 to 4,6%. Quite staggering when rejection tolerance for field relative compaction may be as tight as 1% below which penalties or even rejection could result.

Table 2 shows the results obtained from a different soil from that in Table 1 but the message is again very clearly one of considerable repeatability error.

Table 1 : Replicate tests on a divided sample

Sub-sample	% Passing					Mod Density kg/m^3	Opt. M/C O/O	Range kg/m^3
	19mm	4,75mm	2,0mm	0,425mm	0,075mm			
1	73	40	33	26	19	2081	7,9	Max. Mod
2	97	45	39	30	19	1964	9,0	Min. Mod
3	84	41	32	22	19	2028	8,3	
4	96	55	37	22	19	2088	9,1	124

Table 2 : Repeated modified density tests on a single soil

Test No:	1	2	3	4	5	6	7
Mod. Kg/m	1947	1904	1897	1909	1920	1887	1945
Op. M/C %	14,0	14,8	14,2	13,7	14,3	13,8	13,6
Range: kg/m	Maximum - Minimum Mod: 60,0						
Error %	G_{blk} = 2,6; 60p x 100 / 2600 = 2,3% ; G_{blk} = 2,9; 60,0 x 100 / 2900 = 2,1 %						

The relative density of "solids"

Fortunately not very often but often enough to be alarming, certain specifications still require the field Relative Compaction to be 88% of solid density. The more enlightened specifications however still use the term apparent relative density instead of bulk relative density. It is essential that the difference between these relative densities of crushed rock is fully appreciated.

Consider the following: The total volume of crushed rock, and even weathered rock gravel for that matter, when compacted is made up by the following volumes, see Figure 1 :

a) The volume of the solid part of the rock particles (V_s)
b) The volume of air bubbles within the rock structure (V_b)
c) The volume of the surface fissures or cracks in the rock particles (V_f)
d) The volume of interparticle space (voids) in the compacted mass (V_p)

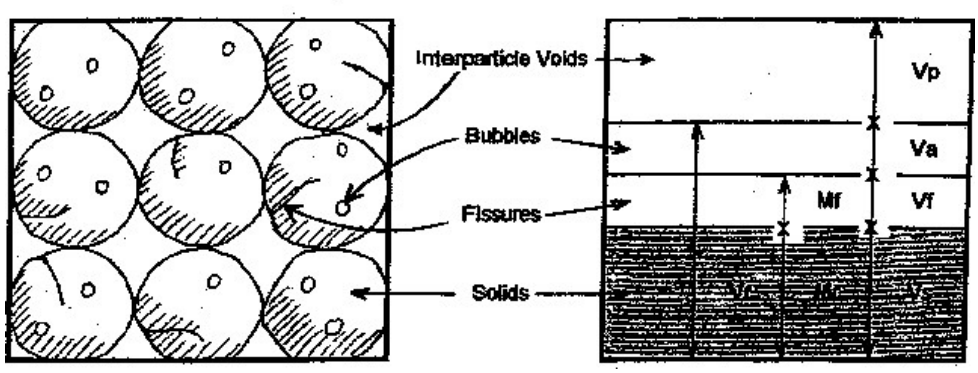

Figure 1: Volumes of interparticle voids, fissures and solids

Relative density of the crushed rock particles may be loosely defined as the mass (M_r) of the particles divided by the mass of an equal volume of water or expressed in terms of density: the mass of the particles divided by the volume of the particles. The interpretation of the "volume of the particles" is important here :

The term Solid Density (G_{sol}) implies that no air bubbles or fissures exist and is given by:

$$G_{sol} = (M_r/V_s)$$

This is a hypothetical value and cannot be measured as the volume of air bubbles entrapped within the rock mass cannot be assessed. As the value of Solid Density cannot be determined it is thus quite meaningless to specify a field compaction terms of this density!

The terms Apparent Density (G_{app}) is a compromise and can be readily determined in the laboratory. It accepts the presence of air bubbles and is given by"

$$G_{app} = (M_r/ (V_s + V_b))$$

As it is easily obtained by simple laboratory technique it is often specified as the standard to which field compaction is to be compared e.g. 88% of apparent density. Again this density ignores cracks or fissures as part of the rock volume and erroneously assumes they form part of the interparticle space. Since compaction, however effective, cannot possibly close or reduce cracks or fissures in the rock particles it is essential that this fissure volume is included as part of the rock particle volume. I.e. $V_r = V_s + V_b + V_f$. Hence the term Bulk Density (G_{blk}) which is given by:

$$G_{blk} = (M_r / (V_s + V_b + V_f))$$

This and only this density, namely bulk density must be used when specifying a porosity of not more than 12% or so called relative compaction of not less than 88% of a Standard laboratory value.

The relationship between G_{app} and G_{blk} is given by :

$$G_{blk} = (G_{app} / (1 + G_{app} \times q))$$

Where q is the water absorption of the crushed rock and is the mass of water (M_f) required to fill the fissures divided by the dry mass of the rock particles.

i.e. $$q = (M_f / M_r)$$

It is quite clear from the above that G_{blk} is only equal to G_{app} when q is zero. After testing at least 17 different crushed rock samples throughout South Africa not a single one showed a value of q ever approaching zero. Generally values for q range from 0.6 to over 1% which can result in a significant error should G_{app} be specified where G_{blk} should apply.

For example: Assume q equal to 1% and apparent relative density of 2650 kg/m^3. If this density was erroneously specified as the standard control density and a 12% maximum porosity was required (i.e. 88% of G_{app}), the Contractor would be required to achieve a field density of not less than 88% of 2650 kg/m^3 i.e. 2332 kg/m^3. The correct specification would require 88% of G_{blk} i.e. :

$$RC = (0{,}88 \text{ X } 2{,}650)/(1 + 0{,}01\text{x}2{,}650) = 2272 \text{ kg/m}^3 _____$$

i.e. (88% of G_{app}) / (1 + q x G_{app}), i.e. (88% of 2582 kg/m^3)

Specifying G_{app} instead of G_{blk} implies that the contractor must compact to a field density of 60 kg/m^3 higher than necessary or to a relative compaction of 90% instead of 88% i.e. (2332/2582)x100 = 90%.

It is of small wonder that difficulty is often experienced when compacting crushed rock. It is of interest to note that a porosity of 10% (100 – 90) is difficult to obtain even in the laboratory.

Field density tests

The two most common field density tests are:

1. the sand replacement method; and

2. the nuclear gauge method using the probe technique for foundation layers.

Both these tests produce errors or loss of accuracy which increase substantially when coarse materials are tested. Crushed rock for example is particularly prone to yield high testing errors. In fact the sand replacement method is often not permitted as a control test for crushed rock layers because of the severe error produced.

Sand replacement density

It may be assumed that sand has an angle of repose of approximately 30^0. Now the removal of very granular material such as crushed rock from the test hole, however carefully this may be attempted, can leave cavities in the wall of the density hole which are improperly filled with the replacement sand, see Figure 2. The net result of course is that the final calculation produces a density result which is too high and thus unacceptable for control testing. This error is not easily measured and in any event results in the sand replacement method being a “No-no” for crushed rock layers.

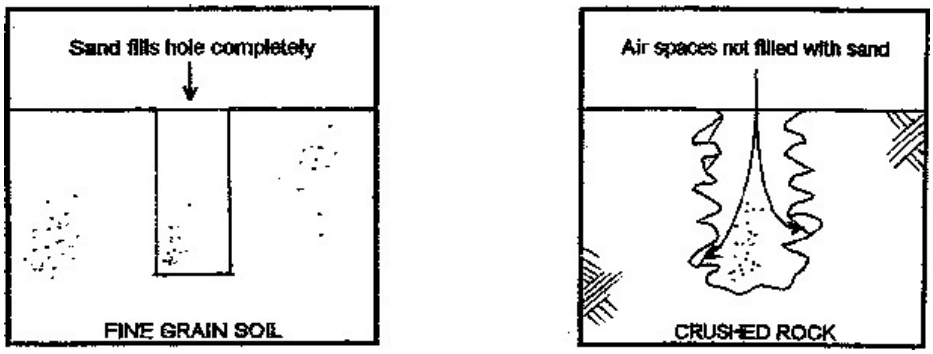

Figure 2: Density hole in (a) fine grain soil and (b) crushed rock.

Nuclear gauge density

It is indeed regrettable that although the error produced by the sand replacement method is recognised very little if any cognisance is given to the error produced when the nuclear gauge is employed on layer work. Assuming the gauge is properly calibrated its accuracy as a density tester for wet density is not in question. It is the establishing of the hole for the probe insertion that is the cause of serious error.

Some 200 or more tests in crushed rock layers have shown quite startling results. The making of a probe hole by hammering a steel peg into the layer to be tested produces a disturbance of the material surrounding the peg hole. It does not produce further densification in granular materials. Although research in this field is still in progress evidence so far appears to indicate that the error produced is dependant upon the size of the coarse aggregate as well as the density of the layer and the existing moisture content. The error of course is quite variable for any given layer of constant density and moisture content but average values appear to be of the order shown in Table

3. From the values shown in this table it would appear that a nuclear gauge reading in excess of say 83% is quite meaningless if a reasonably accurate assessment of density is to be made, unless an error or at least an average error correction is applied.

It is quite unacceptable that in spite of these errors which always gives a too low reading, they are not accounted or allowed for in most specifications for crushed rock layers such as G1, G2 and G3 or even gravels of G4 class. If the sand replacement error is recognised for granular materials the nuclear probe-hole error must likewise be recognised. Furthermore the probe hole error can be readily assessed!

Table 3 : Order of error caused by the peg disturbance in crushed rock

Percentage Of Bulk Dens.	Maximum Size of Crushed stone:	
	37,5 mm	26,5 mm
88	3 - % too low	2 - 4 % too low
86	1,5 - 3 % too low	1 - 2 % too low
84	1 – 2 % too low	1 - 1,5 % too low
82	Negligible	Negligible

Note: These error figures are tentative and were obtained with moisture content of 3 to 4%

An example of serious error

Consider the case of a crushed rock layer where the following data is in fact available (not always the case in practice) :

Required relative Compaction : 88% of standard G_{app}, where G_{app} = 2715 kg/m^3 ,G_{blk} = 2643 kg/m^3 and probe hole error = 55 kg/m^3 too low.

Sand replacement test: not permitted.

Case 1. Incorrect specification : Relative compaction to be 88% of G_{app} with peg hole error not allowed for. The lowest acceptable nuclear gauge is :

Thus RC = 88% x 2715 = 2389 kg/m^3

Case 2. Proper specification: Relative compaction to be 88% of G_{blk} with peg hole error correction applied. The required minimum density is:

RC = 88% x 2643 = 2325 kg/m^3 but the nuclear gauge reading of 325-55=2270 kg/m^3 would be acceptable.

From the above it is evident that a nuclear gauge reading of 2270 kg/m^3 proves that the specification properly applied has been met whereas if the improper specification applies (Case 1) this would yield an erroneous Relative Compaction of :

$$(2270 / 2715) \times 100 = 83{,}6\ \%$$

which is guaranteed rejection with no payment! Alternatively, assuming that the minimum value of 2389 kg/m^3 for the gauge reading was insisted upon, it would result in the Contractor being forced to achieved a Relative Compaction of:

$$(2389 + 55) / 2643 \times 100 = 92{,}3\%$$

A porosity of 7,7%! This is virtually impossible even in a laboratory without crushing the aggregate. During the testing of the 17 different samples of crushed rock as mentioned above, all graded G1 mean size distribution, severe laboratory vibration far in excess of what could be achieved in the field failed in most cases to yield porosity's of less than 10% and only one sample gave a 7% value. Severe crushing of the aggregate also occurred.

It is worth noting that this extra density requirement even if achievable could result in as much as 9 times the energy required to achieve the proper density.

Recommendations

In order to ensure that the Contractor is receiving a square deal when field densities are being controlled the following actions on his behalf are suggested :

1. When modified density is the laboratory standard

If the layer density is to be control tested, obtain at least 5 separate modified density results for the layer material. If the difference between the maximum and minimum values is not greater than 50 kg/m^3 accept the average value as the laboratory control standard. Repeat this process each time the layer material is changed or any change is suspected.

If the difference between the maximum and minimum values of the separate modified density results exceed 50 kg/m^3 the material is likely to be one which tends to be highly variable such as a decomposed Dolomite or Granite. Insist that a sample of about 7 kg is taken at each spot where a density test is performed after the test has been done. To the 7 kg of material the moisture content must be brought up to optimum being the average of 5 separate laboratory modified density tests, (It will be noted that the OMC does not vary excessively, only the density does). A single one-shot modified density test is now done on this field sample and this value is accepted for the control standard for this test position. An experienced laboratory technician can readily assess by eye and feel when the Optimum moisture content is reached. This one-shot test is done immediately in the field and can produce an answer in only a few minutes.

2. When a maximum porosity is specified

This maximum porosity is generally 12 to 14%. The specification may state the field density shall not be less than 86 to 88% as the case may be of :

a) solid density (***incorrect***)
b) apparent density (***incorrect***)
c) bulk density (***correct***)

If a) is specified the specification is spurious as solid density cannot be determined.
If b) is specified, this is only valid if the material compacted has no water absorption which must be proved, otherwise the specification is unreasonable as it requires the contractor to reduce the volume of all particle cracks and fissure which are now deemed to be inter-particle voids.

Specifications a) and b) are clearly unacceptable and they should be corrected to read specification c) which acknowledges the correct standard as Bulk Density.

3. Where the Sand replacement test is not permitted

The sand replacement test is often not accepted as a control test on crushed rock layers as it is inaccurate and produces readings which are too high. The alternative is the nuclear gauge. If no allowance is made in the specification for peg-error correction this matter should be raised at an early stage as the lack of any peg hole error correction makes the nuclear gauge reading quite meaningless for very granular soils and gravels. The peg hole error correction may be determined as follows :

During rolling a peg hole is made, marked and renewed repeatedly after each roller pass until final density is achieved. The pre-formed hole will have virtually no disturbed area surrounding it and the nuclear probe tests reading may be accepted as virtually peg-error free. A new hole is now made by knocking the peg into the layer just clear of the gauge length, see Figure 3. The gauge probe is now inserted with the instrument body over the same zone as before but rotated through 180^0. The new reading is then compared with the “no-error” value. The difference is due to peg hole error and may range from a low figure for low densities to a high figure for higher densities. Several tests have yielded error values in excess of 100 kg/m^3. These errors always show a lower reading for the fresh hole than that for the pre-formed no-

error hole, clearly indicating a destruction of density of the very material being tested for density.

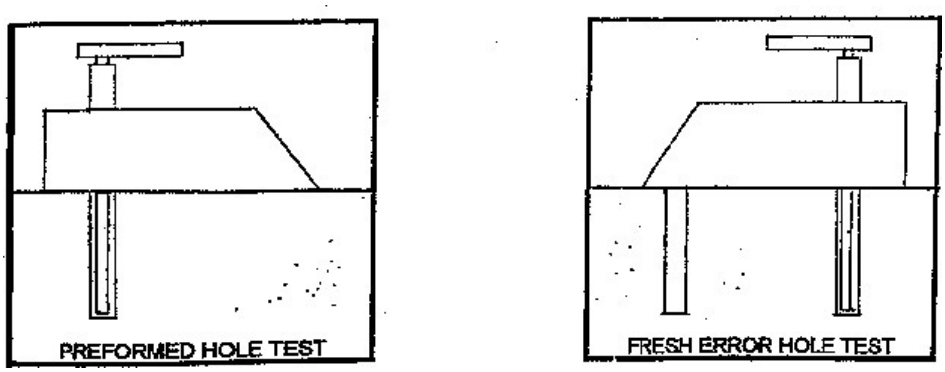

Figure 3: Determination of peg error by testing in preformed hole (a) and then in fresh peghole (b) with gauge reversed

This error from only one test position is unsatisfactory but an average of 10 to 15 similar test holes will give a reliable average peg hole error value. This error must now always be added to all further control test readings of the nuclear gauge where pre-formed holes have not been sited. Naturally each time a change of material occurs a new error correction must be established.

It is quite unacceptable to prohibit the sand replacement test because it is inaccurate but at the same time to ignore the inaccuracy of the nuclear probe test!

9. CLOSURE

COMPACTION - Constant Companion of the Padmaker

Yes, they have been constant companions virtually since the beginning of mankind. “Compaction has been used to improve the load bearing capacity of soils for as long as mankind had the need to provide pathways for carrying meaningful traffic volumes irrespective of whether the traffic volumes were pedestrian, animal or motorised vehicles.” (See first sentence of Section One, page 1-1).

By instinct they have come together and have remained inseparable – there is no point in contemplating a separate existence. It has been a mutually rewarding relationship. With carefully planned and soundly executed compaction the padmaker has created magnificent road networks so essential for economic development.

As road construction developed through the ages stone slabs and large rock pieces were gradually replaced, for economic reasons, by finer granular materials (compare Figs 1.3 and 1.7). Compaction then became more important. This was not realised at first and it is therefore not surprising that, in a survey undertaken by the California Highway Department in 1928, it was found that poor compaction was the main cause of pavement distress. (last paragraph page 1-7). The importance of compaction was clearly demonstrated with a concomitant demonstration of catastrophic road failures resulting from poor compaction. The padmaker took note. He developed a technology which enabled him to quantify the increase in strength that he can derive from soil compaction. (Figures 1.8, 1.9 and 1.10). The bond between the padmaker and his constant companion, compaction, became stronger.

Another challenge appeared in the 20th century – it was the phenomenal increase in traffic, especially heavy industrial traffic. This time the padmaker was better prepared and with slow but steady development of compaction technology accompanied by other developments, the padmaker has constructed pavements capable of carrying up to 50 million E80 axles – an outstanding achievement.

As in any bilateral relationship, constant attention and nurturing are necessary to ensure that the relationship remains sound. So, in such a long lasting relationship it is inevitable that some human failings may appear – complacency, taking things for granted, resulting in neglect. This can be illustrated as neglect in the form of a poor diet : too much water, or too little or too variable water, poor feeding in the form of unsuitable or variable materials and poor handling of plant. Such a diet can result in indigestion in the form of premature and severe rutting, undulations, surface disintegration and potholes in severe cases. This backfires on the padmaker in no uncertain terms in the form of high and premature maintenance and rehabilitation costs inevitably accompanied by headaches and sleepless nights for the padmaker. Persistent neglect may even result in loss of employment.

To maintain a good relationship and to avoid undesirable ailments resulting from neglect, a careful study of the prescriptions given of the foregoing pages is recommended together with their diligent application in practice.

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