# ANNUAL SUMMARY REPORT

Project Title: P60: Best Practice in Compaction QA for Pavement and Subgrade Materials (Year 1 – 2016/2017)

Project No: PRP16036

Author/s: Dr Jeffrey Lee, Dr David Lacey & Dr Burt Look

Client: Queensland Department of Transport and Main Roads

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AN INITIATIVE BY:

# SUMMARY


Currently, the acceptance of construction works (e.g. earthwork embankments, subgrade and pavement granular layers) by Australian road regulatory bodies requires in situ density testing to be completed in order that a relative percentage of the laboratory-determined maximum dry density can be determined. Current earthworks specifications often rely on the assumption that there is a direct correlation between density and modulus parameters (i.e. the greater the density achieved, the higher the modulus of the compacted material). However, this assumption may not be valid and it is affected by many properties of the engineering material.

Over the last two decades, some alternative field assessment methods have been developed which either directly measure the in situ modulus or correlate with the resilient modulus. This project focuses on exploring the use of these new assessment methods. Such innovative testing methods can be grouped into the following four main categories:

- penetration test devices
- surface based impact devices
- geophysical methods
- in situ sensors.

Among these four categories, the ‘penetration test devices’ and ‘surface based impact devices’ were identified to have the potential to be adopted in TMR’s current Quality Assurance (QA) framework.

A literature review conducted in 2016/2017 highlighted that some similar international studies had been completed in recent years. Recent projects in Australia also demonstrated the advantages associated with these innovative, commercially-available field assessment techniques.

The project team attempted to rank the different QA methods using a weighted rating approach.

The traditional density test techniques were also evaluated as reference values in this assessment. The overall comparative assessment of the different QA test techniques is shown in the following table.

| QA technique | Test status | Measured parameter | Ranking (%) | Overall comparative assessment |
|---|---|---|---|---|
| Sand Replacement Test | Traditional – reference value (density) | Relative dry density | 71 | ★★★☆☆ |
| Nuclear Density Gauge | Traditional – reference value (density) | Relative dry density | 66 | ★★☆☆☆ |
| DCP | Traditional – penetration test | Rod penetration rate | 58 | ★☆☆☆☆ |
| PANDA probe | Innovative – penetration test | Cone tip resistance/blow | 74 | ★★★☆☆ |
| Plate Load Test | Traditional – modulus test | In situ modulus (stress-deformation) | 52 | ★☆☆☆☆ |
| LFWD (Prima 100 Model) | Innovative – modulus test | In situ modulus (stress-deformation) | 82 | ★★★★★ |
| LFWD (Zorn Models) | Innovative – modulus test | In situ modulus (deformation only) | 78 | ★★★★☆ |
| Clegg Hammer | Innovative – modulus test | Clegg Impact Value | 78 | ★★★★☆ |
| Geogauge (soil stiffness gauge) | Innovative – modulus test | In situ modulus (harmonic stress-deformation) | 79 | ★★★★☆ |
| Borehole shear tester | Innovative – in situ strength parameter test | In situ Mohr-Coulomb strength parameters | 66 | ★★☆☆☆ |

Full-scale field testing is proposed. An equipment acceptance trial, where potential QA methods will be evaluated, will be conducted in Year 2 of this project, followed by material-specific testing in Years 3 and 4 of the project.

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# 1 INTRODUCTION

## 1.1 Background

Currently, in situ density and California Bearing Ratio (CBR) testing is used in Quality Assurance (QA) during the construction of earthworks, subgrade and unbound granular pavement layers in Queensland. These traditional methods do not directly measure in situ modulus. Another limitation is that the final test results are not available immediately after testing.

Over the past two decades, alternative QA compaction methods have been developed that provide test results immediately after testing. These alternative methods also report modulus values that are used in modern geotechnical and pavement design analysis. This approach negates the need to rely on correlation relationships (which are often material type dependent) to convert measured density or CBR to in situ modulus.

## 1.2 Project Scope

The purpose of this project, which is being conducted under Queensland Department of Main Roads' (TMR) National Asset Centre of Excellence (NACOE) program, is to make recommendations to TMR regarding how they could update the methods that are currently being used in the QA of pavement and subgrade materials. A literature review on alternative QA compaction methods was conducted in the first year of the project and this is included in Appendix A by Foundation Specialist Group (2017). This report summarises the key findings from the literature review which forms the basis of a field validation study to be conducted in future years of this project.

# 2 CURRENT AND ALTERNATIVE QA COMPACTION METHODS

Current earthworks specifications rely on the assumption that there is a direct correlation between density and modulus (i.e. the greater the density achieved, the higher the modulus of the compacted material). As a result, existing specifications used in Australia often require that either sand replacement testing or Nuclear Density Gauge (NDG) testing be conducted to demonstrate that adequate density was achieved within the earthwork layers. However, the assumptions used when converting density to modulus have been shown to be highly idealised, and they can be affected by the properties of the compacted fill, subgrade or base material. More importantly, testing has shown that a higher density does not necessarily indicate a higher strength or modulus (Mooney et al. 2003, Mooney et al. 2010).

Modern geotechnical and pavement designs are based on in situ modulus values. It is therefore reasonable to investigate the feasibility of using alternative methods which are based on measuring in situ modulus directly.

## 2.1 Current QA Methods

There are a number of issues associated with the historical reliance on density testing and CBR results for QA purposes:

- ***Lag indicators*** – it can take several days to complete the laboratory evaluation of sampled materials, and thus to report the ratio between the maximum/optimum material parameter and that achieved in the field. During this time, the contractor typically continues work without waiting for acceptable QA results to become available. If non-conforming QA test results are then made available, there are significant costs associated with removing and replacing both the non-conforming material and the overlying material that has been placed whilst the contractor was waiting for the results.
- ***Density oversize correction*** – this is necessary when more than 20% of the material exceeds 19 mm or 38 mm for Moulds A or B respectively.
- ***Strength and modulus parameters*** – geotechnical and pavement designs are based on strength and modulus values. It is assumed during the design stage that a relationship exists between density and strength/modulus even though density is neither a strength nor a modulus parameter. The simple correlation between CBR and modulus (E) (e.g. E = 10 x CBR) often used in design is generic and there is a significant correlation error associated with its use.

It is worth noting that the CBR test is also not applicable when more than 20% of the material is retained on the 19 mm sieve. Such material is often discarded as part of the test according to the Australian Standards.

## 2.2 Alternative QA Compaction Methods

The increasing interest in moving towards the use of in situ modulus testing for QA purposes has led to the development of several in situ test devices that directly measure the modulus, or strength, of the material. These alternative QA compaction methods do not involve the measurement of density. They can be grouped into the following categories:

- penetration test devices
- surface-based impact devices
- geophysical methods
- in situ (sacrificial) sensors.

A summary of the different QA compaction methods that are available is presented in Table 2.1.A detailed discussion of each device can be found in the literature review included in Appendix A of this report.

**Table 2.1: Summary of alternative QA compaction methods**

| Category | QA methods | Brief descriptions |
|---|---|---|
| Penetration Test | Dynamic Cone Penetrometer (DCP) | A hand-operated device for measuring the resistance of a soil to penetration by a steel cone. The steel cone is connected to a rod and driven into the ground by a 9 kg drop hammer. From this test, the field CBR and allowable bearing capacity can be estimated. |
| | PANDA Probe | A device that determines the cone tip resistance by measuring the driving energy and penetration rate after each hammer blow. Similar to the DCP, the cone tip resistance can be used to infer density, or derive material properties such as shear strength, in situ CBR or modulus values. |
| | Borehole Shear Tester | An in situ shearing device that measures the drained shear strength under different normal stresses. The operation has two phases: the consolidation phase and the shearing phase. A Mohr-Coulomb failure plane can be plotted and drained shear strength parameters for cohesion (c) and angle of friction (Φ) can be calculated. |
| Surface-based impact devices | Static Plate Load Test | Composite elastic modulus (E) values are determined by measuring the average settlement of a rigid plate under known loadings. Usually, multiple loading and unloading cycles are applied to determine the initial and reloading responses.<br>Historically, this method has been used as the reference standard for determining the stiffness/modulus in the field. The main limitation is the size of the plate and the high setup and running costs. |
| | Light Falling Weight Deflectometer (LFWD) | The LFWD measures the deflection induced in the pavement by a circular loading plate under a dropped weight. A composite elastic modulus can be determined based on the measured force and pavement deflection under the loading plate. The capability of the equipment varies with different manufacturers. Some models include built-in load cells. |
| | Clegg Hammer | The Clegg Hammer test involves the dropping of a weight directly onto the surface of the pavement. The stiffness and modulus of the tested area is inferred by measuring the deceleration of the instrumented hammer. |
| | Briaud Compaction Device (BCD) | The BCD utilises a surface base plate that is in contact with the surface. The BCD measures the bending strain, from which the stiffness of the soil is then calculated. |
| | Geogauge | The Geogauge measures the deflection of a soil by applying a vibratory force at different frequencies and measures the deflection of a plate. Its small size and portability makes it an ideal QA device during earthworks construction. However, it only has a limited depth of influence (to about 300 mm). |
| Geophysical methods | Portable Seismic Pavement Analyzer (PSPA)<br>Spectral Analysis of Surface Wave (SASW) | These devises are used to estimate the seismic modulus of the pavement structure based on its response to seismic excitation. They could be used to derive the shear wave velocity properties of the compacted material in the embankment. |
| In situ (sacrificial) sensors | For example: earth pressure cells | Equipment buried within the compacted soil to monitor the change in the compression wave velocities during compaction and changes in material density. |

# 3 ASSESSMENT/RANKING OF DIFFERENT QA COMPACTION METHODS

## 3.1 Assessment Criteria and Known Issues

Unlike the methods used to measure density directly, all the alternative QA compaction methods involve the application of a force to the material by pushing a probe into the subsurface or applying a known force on the top of the surface and measuring the surface response. Known issues for the various methods assessed are summarised in Table 3.1.

**Table 3.1: Known issues for the alternative QA compaction methods**

| QA methods | Known issues |
|---|---|
| DCP | Results heavily affected by particle size and moisture content |
| PANDA Probe | Results affected by particle size and moisture content |
| Static Plate Load Test | Requires external load to be provided and lengthy test duration |
| LFWD | Devises supplied by different manufacturers generate different test results |
| LFWD (Prima 100) | The magnitude of the load that can be applied is limited |
| LFWD (Zorn) | As above; also absence of load cell limits functionality |
| Clegg Hammer | Does not provide a direct measurement of stress vs. deflection |
| Geogauge | Issues with testing of fine-grained soils with high moisture content, and dry sands |
| Borehole Shear Tester | The equipment requires a borehole to be excavated. Testing of dry non-cohesive materials can be difficult. |

## 3.2 Measured Parameters, Measurement Repeatability, and Applicability

It can be seen from Table 2.1 that the various QA methods measure and report different parameters. While some of these methods measure modulus directly, others report different results to infer a modulus value (e.g. rod penetration rate, cone tip resistance, Clegg Impact Value or Mohr-Coulomb strength parameters). The measurement repeatability and its applicability for use on different material types also differ.

A summary of the different equipment characteristics that address the repeatability of the measurement, as well as the applicability of the equipment when used on different material types is presented in Table 3.2. Ideally, equipment which has good measurement repeatability, the ability to assess stress dependency and the ability to test a range of material types would be the most desirable.

For example, the Plate Load Test can reliably measure modulus across all material types (i.e. cohesive soil, sand, and gravel). It also has good repeatability with variation in modulus value less than 10%. The applied loading can be adjusted using a hydraulic jack; this allows different levels of stress to be applied. The test does not rely on an implicit correlation relationship to measure modulus because, once the applied pressure and settlement is measured, a modulus value can readily be defined without the need for further interpretation.

**Table 3.2: Comparison of QA test techniques – measured parameter, repeatability and applicability**

| QA technique | Measured parameter | Required correlation to modulus? | Can assess stress dependency? | Repeatability variation of field test (uniform material) | Ranking of modulus correlations based the applicability for different material types | | | Noted issues |
|---|---|---|---|---|---|---|---|---|
| | | | | | Cohesive | Sand | Gravel | |
| DCP | Rod penetration rate | Yes<br>(DCP → E) | No | < 60% | Poor | High | Medium | Results heavily affected by particle size & moisture content |
| PANDA Probe | Cone tip resistance | Yes<br>($q_d$ → E) | No | < 30% | Medium | High | High | Results affected by particle size |
| Plate Load Test | Modulus (stress-deformation) | No | Yes | < 10% | High | High | High | Reference modulus test |
| LFWD (Prima 100) | Modulus (stress-deformation) | No | Yes | < 15% | Medium | High | High | Load cell can be used to assess a range of test stress conditions |
| LFWD (Zorn) | Modulus (deformation only) | No | No | < 15% | Medium | Medium | Medium | Absence of load cell limits functionality |
| Clegg Hammer | Clegg Impact Value (CIV) | Yes<br>(CIV → E) | Yes | < 15% | Poor | Medium | Medium | No direct measure of stress - deflection |
| Geogauge (Soil Stiffness Gauge) | Modulus (harmonic stress-deformation) | No | No | 0–30% | Poor | Medium | Medium | Issues with testing: fine-grained soils with high moisture contents & dry sands. Uses small strain stiffness. |
| Borehole Shear Tester | In situ Mohr-Coulomb strength parameters | N/A (strength test) | N/A | N/A | High | Medium | Poor | Requires borehole. Difficult to test on dry non-cohesive materials. |

## 3.3 Costs, Ease of Use and Turnaround Time for Test Results

Table 3.3 provides a summary the estimated cost (principal cost and yearly operation cost) and time for testing/reporting of results associated with the different equipment. As already discussed, one known issue with current density measurement techniques is the time delay between measurement and when the results become available. Ideally, equipment with a short measurement time and both low capital and on-going operation cost would be the most desirable.

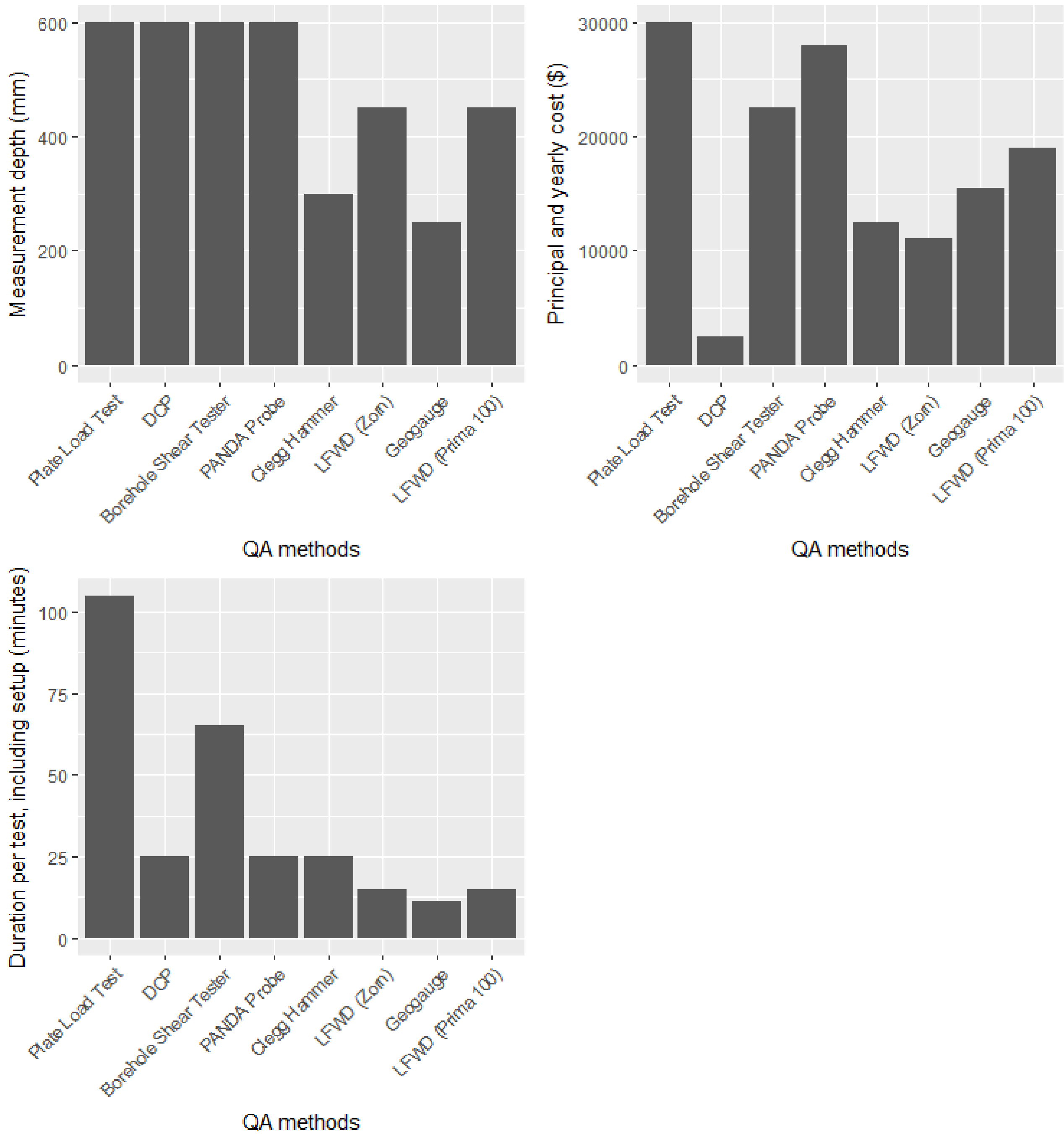
Once again taking the Plate Load Test as an example, the equipment costs about $30 000 and it typically takes about one hour to set up and complete a single test. On the other hand, the initial principal cost of the LFWD is lower and a test can be completed in 10 minutes (including set-up time), which is a major improvement in productivity.

### Table 3.3: Comparison of QA test techniques – cost and test duration

| QA technique | Approx. principal cost of equipment ($AUD) | Yearly calibration/ consumable costs | Duration (per test) | | | | Typical turnaround of test results | No. of tests per day | Comments |
|---|---|---|---|---|---|---|---|---|---|
| | | | Setup | Field test | Laboratory | Interpretation /reporting | | | |
| DCP | $2 000 | < $500 | 5 mins | 10 mins | N/A | 10 mins | < 1 day | ≤ 30 | External recording generally required |
| PANDA Probe | $27 500 | < $500 | 5 mins | 10 mins | N/A | 10 mins | < 1 day | ≤ 30 | PANDA reported detailed soil parameter after each blow increment |
| Plate Load Test | $30 000 (electronic) | nil | 15 mins | ≥ 1 hour | N/A | 30 mins | 1 day | 2–4 | Requires external reaction force to be provided |
| LFWD (Prima 100) | $17 000 | $2 000 | 5 mins | ≤ 5 mins | N/A | 5 mins | < 1 day | ≤ 100 | PDA records full data history |
| LFWD (Zorn) | $9 000 | $2 000 | 5 mins | ≤ 5 mins | N/A | 5 mins | < 1 day | ≤ 100 | Limited Information record |
| Clegg Hammer | $12 000 (9.1 kg hammer) | < $500 | 5 mins | ≤ 5 mins | N/A | 15 mins | < 1 day | ≤ 100 | External information record required |
| Geogauge (Soil Stiffness Gauge) | $15 000 | < $500 | 5 mins | 75 secs | N/A | 5 mins | < 1 day | ≤ 100 | Limited Information record |
| Borehole Shear Tester | $22 000 | < $500 | 30 mins | 20 mins | N/A | 15 mins | 1 day | ≤ 10 | Requires auger borehole to complete |
| Nuclear Density Gauge | $10 000 | $2 000 | 5 mins | 60 secs | 24 hours | 15 mins | ≥ 3 days | ≤ 30 | Laboratory testing for oversize/MDD required<br>Density measurement ≠ Modulus |
| Sand Replacement Test | $750 | < $500 | 5 mins | ≥ 30 mins | 24 hours | 15 mins | ≥ 3 days | ≤ 10 | Laboratory testing for oversize/MDD required<br>Density measurement ≠ Modulus |

Key quantitative parameters (such as measurement depth, total cost and time duration per test) are shown in Figure 3.1.

**Figure 3.1: Comparison of different QA methods based on measurement depth, total cost and duration per test**




A qualitative comparison of the equipment is presented in Table 3.4. This assessment is expressed in terms of cost, speed and total turn-around time in terms of the provision of the results.

**Table 3.4: Summary assessment of QA test techniques – cost, speed of test and duration between testing and provision of results**

| QA technique | Measured parameter | Rating:<br>One star (★☆☆☆☆): least desirable test (highest cost/lowest productivity<br>Five stars (★★★★★): most desirable test (lowest cost/highest productivity) | | | |
|---|---|---|---|---|---|
| | | **Principal cost** | **Operating cost per test** | **No. of tests/day** | **Total test result turnaround time** |
| DCP | Rod penetration rate | ★★★★☆ | ★★★★☆ | ★★★★☆ | ★★★★☆ |
| PANDA Probe | Cone tip resistance/blow | ★☆☆☆☆ | ★★★★☆ | ★★★★☆ | ★★★★☆ |
| Plate Load Test | In situ modulus (stress-deformation) | ★☆☆☆☆ | ★☆☆☆☆ | ★☆☆☆☆ | ★★★☆☆ |
| LFWD (Prima 100) | In situ modulus (stress-deformation) | ★★★☆☆ | ★★★★☆ | ★★★★★ | ★★★★☆ |
| LFWD (Zorn) | In situ modulus (deformation only) | ★★★★☆ | ★★★★☆ | ★★★★★ | ★★★★☆ |
| Clegg Hammer | Clegg Impact Value (CIV) | ★★★☆☆ | ★★★★☆ | ★★★★★ | ★★★★☆ |
| Geogauge (Soil Stiffness Gauge) | In situ modulus (harmonic stress-deformation) | ★★★☆☆ | ★★★★★ | ★★★★★ | ★★★★☆ |
| Borehole Shear Tester | In situ Mohr-Coulomb strength parameters | ★★☆☆☆ | ★★★☆☆ | ★★☆☆☆ | ★★★☆☆ |
| Nuclear Density Gauge | Relative Dry Density (RDD) | ★★★☆☆ | ★★☆☆☆ | ★★★★☆ | ★★☆☆☆ |
| Sand Replacement Test | Relative Dry Density (RDD) | ★★★★★ | ★☆☆☆☆ | ★★☆☆☆ | ★★☆☆☆ |

## 3.4 Zone of Influence

One major concern associated with moving towards an alternative QA method is the depth that the proposed method can measure. This is related to the zone of influence of each measurement technique. A comparative assessment of all the equipment reviewed is presented in Table 3.5. The literature suggested that most of the equipment evaluated would be capable of testing to a maximum depth of 300 mm.

Table 3.5: Comparison of QA test techniques – depth (zone of influence) of test

| QA technique | Measured parameter | Zone of influence | |
|---|---|---|---|
| | | Description | Depth of measurement |
| DCP | Rod penetration rate | Rod penetration length | Infinite |
| PANDA Probe | Cone tip resistance | Rod penetration length | Infinite |
| Plate Load Test | Modulus (stress-deformation) | 2.0 x plate diameter | 600 mm (300 mm plate) |
| LFWD (Prima 100) | Modulus (stress-deformation) | 1.0 to 1.5 x plate diameter | 300–450 mm |
| LFWD (Zorn) | Modulus (deformation only) | 0 to 1.5 x plate diameter | 300–450 mm |
| Clegg Hammer | Clegg Impact Value (CIV) | Varies based on hammer weight and drop height | 200–300 mm (10 kg hammer) |
| Geogauge | Modulus (stress-deformation) | 150–250 mm | 150–250 mm |
| Borehole Shear Tester | In situ Mohr-Coulomb strength parameters | Immediate location of test | Infinite |
| Sand Replacement Test | Relative Dry Density (RDD) | Depth of excavation mMax. 300 mm) | Typically 200–250 mm |
| Nuclear Density Gauge | Relative Dry Density (RDD) | Depth of probe (max. 300 mm) | Typically 200–250 mm |

## 3.5 Comparison of Equipment

In an attempt to rank the different alternative QA methods, a weighted rating approach was used. The following factors were considered (in order of decreasing importance in the opinion of the project team):

- accuracy, repeatability, and reliability of equipment (30%)
- requirement/duration/ease of processing the results (25%)
- duration of test (20%)
- operating cost (15%)
- principal (purchasing) cost (10%).

The results of this assessment are summarised in Table 3.6. It can be seen that all of the equipment reviewed in this study has the potential to provide the in situ parameters in a shorter timeframe compared with the traditional methods (e.g. Plate Load Test, Sand Replacement Test, and NDG).

The equipment with the highest weighted ratings are LFWD, Geogauge, Clegg Hammer and the PANDA Probe.

**Table 3.6: Overall comparative assessment of available QA test techniques – traditional vs. innovative**

| QA test | Test status | Measured parameter | Ranking (%) | Overall comparative assessment |
|---|---|---|---|---|
| Sand Replacement Test | Traditional – reference value (density) | Relative dry density (RDD) | 71 | ★★★☆☆ |
| Nuclear Density Gauge | Traditional – reference value (density) | Relative dry density (RDD) | 66 | ★★☆☆☆ |
| DCP | Traditional – Penetration test | Rod penetration rate | 58 | ★☆☆☆☆ |
| PANDA Probe | Innovative – Penetration test | Cone tip resistance/blow | 74 | ★★★☆☆ |
| Plate Load Test | Traditional – Modulus test | In situ modulus (stress-deformation) | 52 | ★☆☆☆☆ |
| LFWD (Prima 100 Model) | Innovative – Modulus test | In situ Modulus (stress-deformation) | 82 | ★★★★★ |
| LFWD (Zorn Models) | Innovative – Modulus test | In situ modulus (deformation only) | 78 | ★★★★☆ |
| Clegg Hammer | Innovative – Modulus test | Clegg Impact Value (CIV) | 78 | ★★★★☆ |
| Geogauge (soil stiffness gauge) | Innovative – Modulus test | In situ Modulus (harmonic stress-deformation) | 79 | ★★★★☆ |
| Borehole Shear Tester | Innovative – In situ strength parameter test | In situ Mohr-Coulomb Strength Parameters | 66 | ★★☆☆☆ |

# 4 PROPOSED FUTURE FIELD VALIDATION WORK

The literature review highlighted that some methods have the potential to improve the productivity and increase the reliability of measured in situ modulus. More importantly, some of the methods have the advantage of providing immediate feedback during construction, thus minimising time delays and possible rework (if a Lot was later confirmed to be non-conforming).

## 4.1 Past Project Experience

Similar international studies have been conducted investigating the use of alternative QA methods and an increasing number of studies of the use of such alternative methods are being conducted in Australia. The project team has experience on some of these projects located in Queensland. For example, in the Gateway Upgrade South Project, the LFWD was used as a QA tool for the compaction of a gravel-dominated material that would not, due to its comparatively large particle size, be suitable for testing for either density (Sand Replacement or NDG) or penetration testing (DCP).

In 2016, a full-scale trial embankment project was undertaken in Toowoomba, Queensland. The trial included a study to assess the suitability of various penetration and surface-based impact tests for compaction assessment. Side-by-side density (sand replacement and NDG), penetration (DCP and PANDA Probe) and modulus (Plate Load Test, LFWD and Clegg Hammer) testing was undertaken on embankments during their construction. It was found that both the testing speed and the measurement depth of the alternative methods exceeded that of the traditional density testing techniques. Furthermore, the variability of the PANDA Probe was lower than the DCP.

The LFWD was used by TMR to characterise residual soil and weak rock materials. It was found that the in situ modulus of the material could be measured using the LFWD, whereas both density and penetration testing would be unsuitable due to the ‘rock’ nature of the material. The LFWD-derived modulus parameter was able to be used to successfully assess and distinguish between both material types, whereas the laboratory-based CBR values plateaued once ‘rock’ materials were encountered. This did not reflect the increased rock strength associated with decreasing weathering effects. The LFWD testing allowed site- and material-specific correlations to be developed and removed the comparatively large error inherently associated with the adoption of ‘universal’ correlations between density and modulus (or CBR and modulus).

More recently, comparative studies between the in situ modulus of compacted materials (e.g. clays, sands, and gravels) as determined by the Plate Load Tester and the LFWD has also been undertaken at construction projects throughout Queensland. In all cases, the potential for the LFWD to be used as a QA technique has been identified.

## 4.2 Proposed 3-Years Testing Plan

The studies in Queensland reported in Section 4.1 identified the potential to adopt and take advantage of some of these technologies for QA assessment. However, both the materials tested and the range of equipment used were limited.

In order to allow a more comprehensive evaluation of the different technologies, and to select the best technologies for implementation, a testing plan summarised in Table 4.1 is proposed for future years of the project. An equipment acceptance trial will be conducted in Year 2. The purpose of this trial is to directly compare all the available techniques. It is anticipated that one or two material types will be investigated. It is anticipated that three different types of equipment will be selected for the next phase of the study.

Work in Year 3 and Year 4 will focus on conducting a material-specific trial. As found in the literature review, the modulus correlation of different QA assessment technique varies with different material types. The following material types will be investigated (listed in order of priority):

- residual soils
- extremely-weathered rock
- highly-weathered rock
- granular pavement base and subbase
- alluvial clay
- alluvial sand.

A summary of the proposed testing plan for Year 2, Year 3 and Year 4 is shown in Table 4.1 (listed in the order of priority).

**Table 4.1: Proposed three years testing plan**

| Year | Type of field trial | Equipment | Material |
|---|---|---|---|
| Year 2 (2017–18) | Equipment validation trial | All equipment | One material group |
| Year 3 (2018–19) | Material specific trial | Top 3 candidates equipment | Residual soils |
| | | | Extremely-weathered<br>Highly-weathered |
| Year 4 (2019–20) | Material specific trial | Top 3 candidates equipment | Granular pavement base & subbase |
| | | | Alluvial clay |
| | | | Alluvial sand |

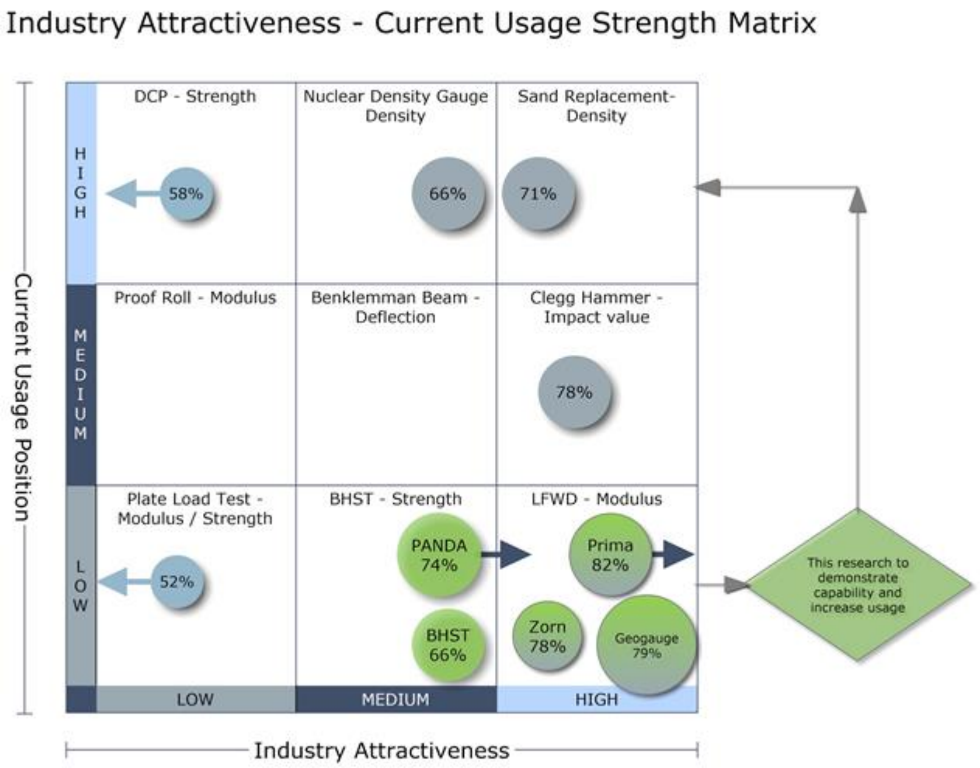
# 5 CONCLUSIONS

Currently, in situ density and CBR testing is used in QA during the construction of earthworks and unbound granular pavement layers. A problem with these tests is that they do not directly measure in situ modulus. Another limitation is that the final test results are not available immediately after testing. Over the past two decades, alternative QA compaction methods have been developed that provide test results immediately after testing. These alternative methods also report modulus values that are used in modern geotechnical and pavement design analysis. This approach negates the need to rely on correlation relationships (which are often material type dependent) to convert measured density or CBR to in situ modulus. The purpose of this NACOE is to make recommendations to TMR regarding how they could update the methods that are currently used in the QA of pavement and subgrade materials.

Year 1 of this project involved a literature review of alternative techniques to assess the in situ properties of earthworks and pavement layers during construction. It was concluded that there is potential for alternative QA methods to be utilised in construction sites. These new methods will not only provide a direct measure of the in situ modulus value but also lead to a reduction in time delays associated with the traditional density measurement methods.

The relative industry attractiveness of the different equipment is summarised in Figure 5.1.

**Figure 5.1: Current industry acceptance/usage of innovative QA/QC tools compared to potential for use (industry attractiveness)**




It can be seen from Figure 5.1 that the available equipment, according to industry attractiveness, lags current usage. The next phases of the study will have the following three broad objectives:

- reliability of alternative equipment types assessed on actual projects
- knowledge transfer to road agencies and industry
- material-specific assessments.

Previous experience and learnings from recent Queensland projects have also been documented as part of this project. A testing plan has been proposed for future years of this project which involves a comprehensive evaluation and selection of the best technologies for use. An equipment acceptance trial will commence in Year 2, where a range of QA methods will be assessed parallel to each other. This will be followed by a field trial in Year 3 and Year 4 where the top three candidate items of equipment will be tested using different types of material to establish the necessary material-specific relationships.

# REFERENCES

Refer to detailed reference list included in Section 7 of Appendix A.

# APPENDIX A FOUNDATION SPECIALISTS GROUP REPORT – PHASE 1 LITERATURE REVIEW

Foundation Specialists Group

# Best Practice in Compaction Quality Assurance for Pavement and Subgrade Materials

Australian Road Research Board (ARRB)

**Phase 1 - Literature Review**

Report No. 2091RDL02C

25 May 2017

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Melbourne
Ground Floor, 1G Marine Parade
Abbotsford VIC 3067 Australia

Brisbane
Level 4, 490 Upper Edward Street
Spring Hill QLD 4000 Australia

Foundation Specialists Pty Ltd
P.O. Box 1177
Collingwood VIC 3066 Australia

P: +61 3 9417 4333
F: +61 3 8648 5693

contact@foundationspecialists.com.au
foundationspecialists.com.au

ACN 167 316 921
ABN 69 167 316 921

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# 1. Introduction

This document presents the results of the initial Literature Review of alternative (non-density) methods of achieving earthworks and pavement compaction Quality Assurance (QA).

## 1.1 Aim

The purpose of this research is to update processes / techniques for Quality Assessment (QA) of earthworks and pavement materials relevant to road construction and maintenance projects.

Current QA practices are anchored in historical earthwork practices rather than appropriate (current) technology. Updating the techniques adopted for material assessment are anticipated to yield QA tools that directly measure relevant design parameters (e.g. deformation / stiffness) *insitu* rather than rely on an assumed improvement with simpler measured properties (e.g. *insitu* density).

## 1.2 Background

The level of compaction of both granular (pavements) materials and subgrades for earthworks has conventionally been verified using density measurements. Over the past two (2) decades alternative methodologies have been developed such as Light Falling Weight Deflectometers (LFWD or PWD), the Briaud Compaction Device (BCD), Soil Stiffness Gauge (SSG) and Intelligent Compaction (IC) instrumentation of compaction equipment.

Notwithstanding the higher coverage achievable with Intelligent Compaction (IC) assessment techniques, they both rely on (i) material verification and documentation provided by the Contractor; and (ii) to be calibrated with specific compaction equipment / brands. Intelligent Compaction would thus be considered to potentially form part of the Contractor's process control, but is considered unlikely to be frequently available for use by independent verifiers. Instead, many other non-traditional testing techniques – i.e. not sand replacement or nuclear density gauge tests – are currently commercially available that would, as per the current typical industry arrangement, typically be utilised by a consultant / soil tester to independently verify the end product. The latter approach is thus the focus of this research project in preference to assessment of IC techniques, as it mirrors the typical current industry arrangement (i.e. continues the expectation of independent verification of the *insitu* condition of earthworks / pavement materials).

Historically sand replacement density testing and California Bearing Ratio (CBR) tests have been used in Quality Control (QC). The Nuclear Gauge density test, when introduced to the wider industry in the 1970's, provided another tool to fast track density test results, as compared to the Sand Replacement test. However, the continued use or reliance upon density / CBR tests for QC have several inherent issues associated with them, which include:

1. They are lag indicators. During the 1 week to several days to obtain results, the contractor has already advanced fill placement over that lift. Significant equipment standby costs (say $25,000 per week) would be incurred if the Contractor was forced to wait on results of testing prior to the continuation of material placement and compaction. However, if any non-conformance does occur within compacted materials, a significant cost to remediate such materials are typically experienced – costs associated with the removal and replacement of both the non-conforming layer and any overlying material subsequently installed.

2. Compaction measurements via *insitu* density and CBR tests are technology from the 1930s. Equipment advances brought a "modified" compaction test in the 1940s. Nuclear density tests, Benkelman Beams, Deflectometers, Dynamic Cone Penetrometers and Clegg Hammers were introduced as other testing tools in the 1970s. The LFWD, BCM and SGG have been used internationally in the past 25 years. It is time to use equipment of the 21st century.

3. Pavement and earthworks engineering design are largely currently based on deformation behaviour, characterised by the modulus ($E$) parameter, however the current industry standard is to evaluate construction materials via CBR test techniques and use a generic CBR:$E$ correlation (e.g. $E$ = 10 x CBR as per Austroads, 2012). However, the CBR is an INDEX of strength, and should be considered neither a strength nor a modulus parameter. CBR is also a highly variable index, with a recent attempt in testing standards to reduce that variability by increasing preparation time by up to 7 days for highly plastic clays (as per AS1289.6.1.1 – 2014). This amplifies the issue of current tests being lag indicators

(refer point 1). Hence CBR is a 2nd order parameter (i.e. CBR requires a correlation to produce the desired parameter), and by definition has both (i) material / equipment and (ii) correlation errors associated with the transformation between CBR → *E*.

4. During the construction phase of the project, achieved density values are typically adopted as the QC parameter. The underlying assumption is that if a specific density threshold is satisfied, then the design strength / stiffness requirement will also be satisfied (i.e. assumes that Density → *E* or Density → Strength relationships are present). However, the measure of density is associated with 1930s / 40s technology, and thus the density measure is an outdated 3rd order parameter (Density → CBR → *E*) and the application of which allows testing / correlation errors to be introduced at multiple points.

Current technology exists which allows the removal of the typically applied intermediate correlation steps. All the "modern" measurement tools previously mentioned measure the modulus parameter directly (deformation based on magnitude of applied load). The use of direct modulus parameter measurements removes up to two (2) levels of assumption / error in comparison to the indirect estimation of a modulus parameter from CBR or density testing. Importantly recent testing has also demonstrated that a higher achieved density does not necessarily indicate a higher strength or modulus, and thus calls into question the underlying assumption that Density → *E* or Density → Strength relationships are present.

Existing technical literature has many reports on issues with using CBR as a design parameter. However, it has a long-standing use, and familiarity to those associated with using this CBR index means it will remain as a "standard" test despite its many limitations.

## 1.3 Recent Learnings and Local Literature

The authors of this literature review have been active in the evaluation of, and development of appropriate standard approaches using, "modern" test techniques for the assessment of the *insitu* condition of the near-surface within Australia.

This literature review document has drawn freely from some of the author's published peer-review reference papers (International and within Australia), including:

- Look, B. (2012). "Quality control specifications for large earthworks projects", in Indraratna, B., Rujikiatkamjorn, C. & Vinod, J.S. (eds.) *Proceedings of the International Conference on Ground Improvement and Ground Control*, Wollongong, NSW, pp 1113 – 1118
- Lacey, D., Look, B. and Williams, D. (2013). "Assessment of Relationship between *insitu* modulus derived from DCP and LFWD testing" in Cui, Y-J., Emeriault, F. and Cuira, F. (eds.) *Proceedings of the 5th International Young Geotechnical Engineers' Conference: 5th IYGEC*, Paris, France, 31st August – 2nd September 2013, IOS Press, pp. 379 – 382
- Mellish, D., Lacey, D., Look, B. and Gallage, C. (2014) "Spatial and Temporal Variability in a Residual Soil Profile," *4th International Conference on Geotechnique, Construction Materials and Environment* (GEOMATE 2014), Brisbane, QLD
- Lacey, D.W., Look, B.G. and Williams, D.J. (2015) "Relative modulus improvement due to inclusion of geo-reinforcement within a gravel material", *Proceedings of 12th Australia New Zealand Conference on Geomechanics (ANZ 2015)*, Wellington, New Zealand, paper 123, pp. 940 – 947
- Lacey, D., Look, B. and Marks, D. (2016) "Use of the Light Falling Weight Deflectometer (LFWD) as a site investigation tool for residual soils and weak rock", in Lehane, Acosta-Martínez & Kelly (eds.), *Geotechnical and Geophysical Site Characterisation 5*, Volume 2, pp. 1099 – 1104.
- Look, B. and Lacey, D. (2017, Accepted), "Dynamic Monitoring and Modulus based specifications with deep lift compaction", *Proceedings of the 19th International Conference on Soil Mechanics and Geotechnical Engineering (19th ICCSMGE)*, Seoul, Korea

Additionally lessons learned from trial embankments (2016) at Toowoomba Second Range Crossing (TSRC) has been incorporated into this report. These trial embankments focused on the evaluation of deep lift compaction techniques, but as a by-product allowed the comparison of modern test equipment to traditional evaluation methods of compacted materials.

# 2. Material Parameters relevant to design / performance

## 2.1 Introduction – Existing QC Procedures vs. Performance Specifications

Existing QA / QC procedures typically utilise *insitu* density measurements for compaction control of unbound materials. Historically, this approach was adopted largely due to the convenience of the test method (speed, cost, simplicity) rather than reflecting the engineering properties responsible for the ongoing performance of such materials.

Instead of the achieved density, it is the *insitu* stiffness and strength of the unbound materials which are responsible for their stability and resistance to deformation under applied loads. The stiffness of the material – evaluated as the deformation magnitude due to an applied load – governs mechanical behaviour and is the key element in preventing material failure, while the achieved strength of material is the stress limit that can be applied before failure occurs.

In road construction, typical current QA / QC procedures evaluate the suitability of materials via density assessment, with the expectation that achieving a nominated density threshold correlates to the achievement of suitable stiffness and strength parameters. However, such a general correlation does not exist and would be material-specific. Large variations of stiffness and strength can occur across small variations in measured *insitu* density. In addition, the stiffness and strength parameters – and thus the behaviour and response of the material – are sensitive to (i) moisture content variation; (ii) the applied stress magnitude during loading; and (iii) the *insitu* stain condition – none of which are measured by the completion of *insitu* density testing.

Figure 2-1 illustrates that the key design inputs (engineering parameters) are not confirmed onsite via direct measurement during construction, but rather are inferred by the completion of *insitu* density assessment and demonstration of compacted material compliance with the governing specifications of the relevant regulatory authority.

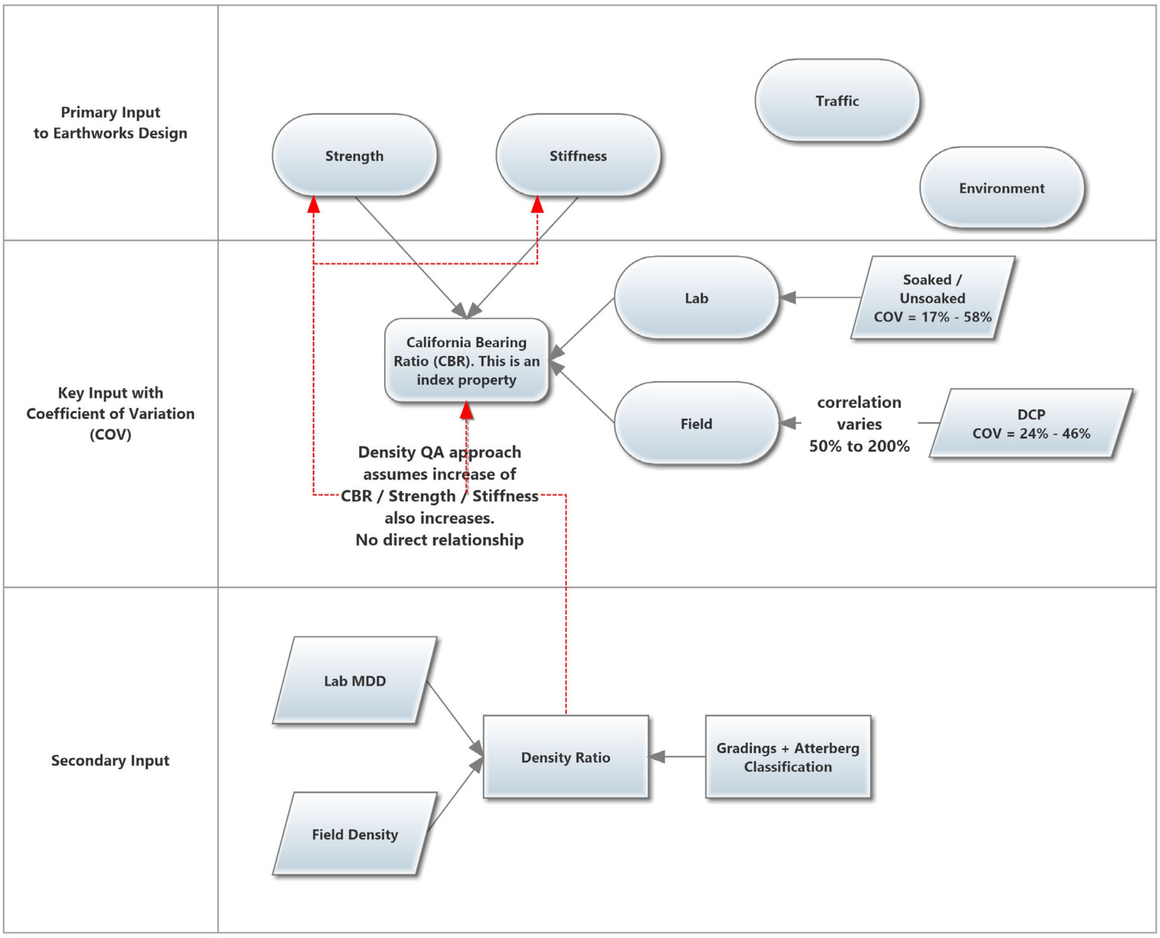



**Figure 2-1 Key input vs implicit errors in current approach of testing 2nd or 3rd order parameters**

Similar to the increase in the adoption of mechanistic-empirical design techniques for pavement design, there is a growing trend to directly measure the relevant engineering properties – strength and deformation – of construction materials during both the design (site investigation) and construction phase QA programs. A study involving the survey of numerous US State Departments of Transport (Puppala, 2008) indicated that it is well reported that stiffness / deformation parameters - surveyed in the form of a resilient modulus value ($M_R$) – were considered to be a better design parameter for incorporation into pavement design than the indirect parameters currently utilised. However, the lack of a standardised, or simple, field or laboratory test to determine a modulus (E) value for construction materials prevents it wide adoption. The same survey also found that the lack of existing correlations between equivalent stiffness parameters and other, more common, material design parameters resulted in the existing strength based design process (i.e. CBR → E correlation) remained the preferred design method among those surveyed.

The following sections of this chapter define the strength and deformation (modulus) parameters relevant to the design and performance of unbound materials, and the variation thereof based on varying stress and strain conditions.

## 2.2 Shear Strength

As described in detail by Hopkins (1991) and Newcomb & Birgisson (1999), the shear strength of the unbound materials controls the mechanical behaviour of material during the application of traffic loadings. The load imposed (by the wheel loading of trafficked surface) must be lower than the shear strength ($\tau$) available to resist the failure (i.e. shear strength governs the bearing capacity).

The available shear strength is a function of (i) the frictional resistance between solid particles; (ii) cohesion and adhesion between soil particles; and (iii) the interlocking of solid particles to resist deformations. The typical simplified measure of shear strength ($\tau$) for design can be quantified by the principle of effective stress (Terzaghi, 1943) via the use of the Mohr-Coulomb failure criterion, as defined in Equation 2-1 and Figure 2-2.

$$\tau = c' + \sigma_n' \tan\phi' \qquad \text{(Equation 2-1)}$$

Where: $\tau$ = shear strength, or stress, of the bearing media;

$\sigma_n'$ = effective normal stress;

$c'$ = effective stress parameter (cohesion intercept);

$\phi'$ = effective stress parameter (angle of internal friction)

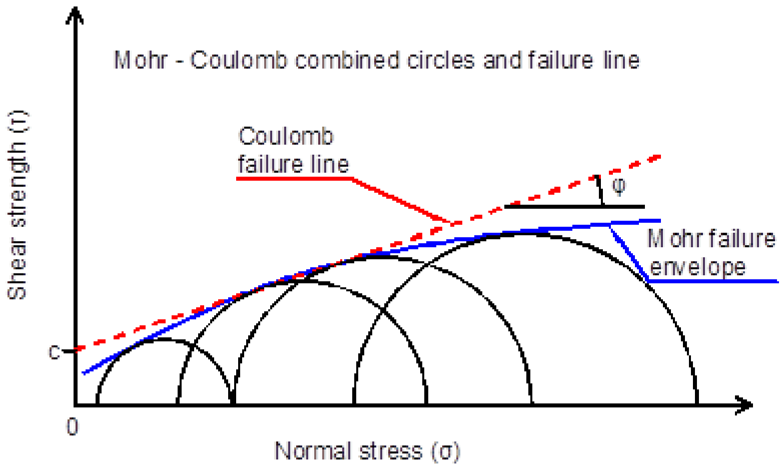



**Figure 2-2 Definition of mobilised shear strength parameters for Mohr-Coulomb failure criterion (after Holtz and Kovacs, 1981)**

From Equation 2-1 it can be observed that the shear strength of a material is stress-dependent. Typically, the c (or c') and $\phi$ (or $\phi'$) parameters and their stress-dependent behaviour are directly measured at a number of stress conditions during the completion of laboratory based tests (triaxial cell or direct shear testing).

Stability of embankment fills is also typically based on the material strength properties. For example, Queensland Main Roads minimum geotechnical design standards (2015) limits the embankment shear strength parameters used for earth-fill design to be no greater than c' = 5 kPa and $\phi'$ = 30 degrees (for 'Class A' and 'Class B' materials). Yet such parameters are seldom (if ever) measured *insitu* during project construction.

## 2.3 Deformation Modulus

The stiffness (modulus) – deformation / settlement – behaviour of the ground once a foundation or loading scenario is applied (e.g. traffic loading for roads, building loading for foundations) is the most important engineering property for geotechnical problems which are related to deformation or settlement issues (Dyka, 2012). As identified by Von Quintus et. al. (2009), as current mechanistic-empirical pavement formation design uses the modulus parameter of each layer as the key input material property - controlling deformation and / or distress (rutting) of the formation for assessment of pavement performance – there is increased interest in the accurate evaluation of such a parameter via direct measurement.

For elastic materials, the modulus of elasticity (Young's modulus, *E*) and Poisson's ratio ($\nu$) define the deformation behaviour. The modulus (*E*) of a material is generally defined as the gradient of a stress / strain curve (within the elastic phase of behaviour) as shown in Figure 2-3, with the modulus defined from a stress-strain curve in Equation 2-2 (from Briaud, 2001).

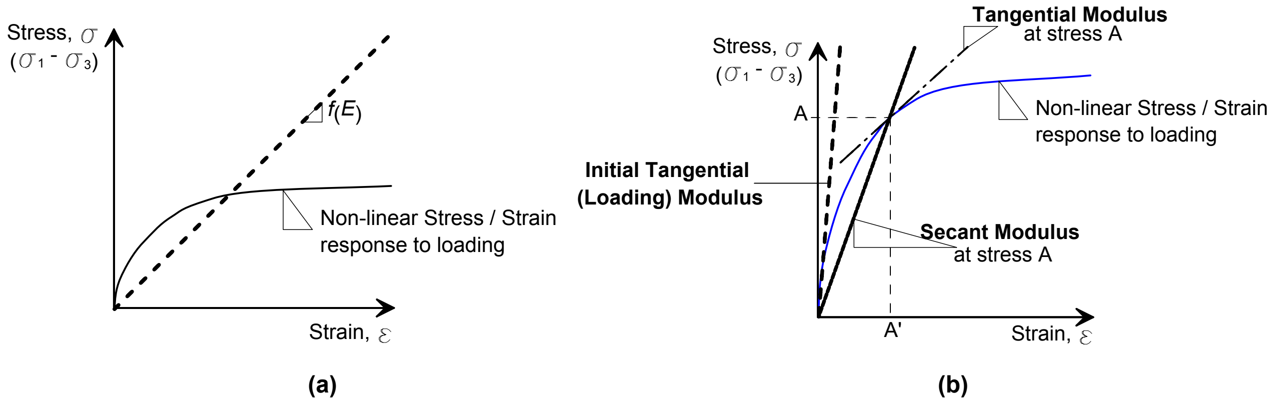



**Figure 2-3 Concept showing (a) elastic phase modulus (Young's modulus, *E*), based on stress / strain curve (after Briaud, 2001); and (b) definition of typical modulus values (from Lacey, 2016)**

$$\text{Young's Modulus } (E) = [\sigma_1 - (2\nu\sigma_3)] / \varepsilon_1 \qquad \text{(Equation 2-2, from Briaud, 2001)}$$

Where:

$\nu$ = Poisson's Ratio

$\sigma_3$ = Lateral (confining) stress

$\sigma_1$ = Axial Stress

$\varepsilon_1$ = Axial Strain

However, Equation 2-2 assumes that the material is continuous, linear elastic, isotropic and homogeneous. Although most unbound materials are not truly elastic – they become permanently deformed under repeated loads - these parameters are typically adopted for subgrade soils due to their comparatively deeper location within the pavement formation resulting in a low strain environment.

Within the total deformation that occurs due to an applied load (stress), a portion of the deformation may be recoverable (resilient deformation) whilst a portion is unrecoverable (plastic deformation). This is shown conceptually in Figure 2-4. The resilient modulus ($M_R$) parameter is defined as the ratio of applied deviatoric stress to the resilient (elastic / recoverable) strain experienced by the material under repeated loadings (i.e. traffic).

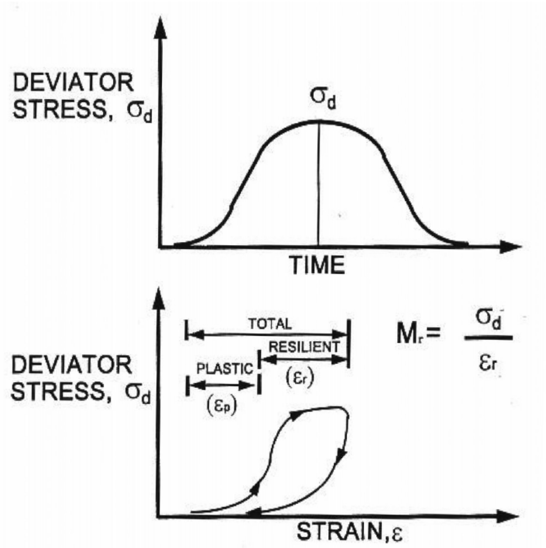



**Figure 2-4 Definition of Resilient Modulus ($M_R$), from Puppala (2008).**

Within the laboratory setting a number of methods can be utilised to define $M_R$, but typically the parameter is measured upon a compacted soil specimens within a triaxial cell within by (AASHTO T307, 2013):

(i) Initial material conditioning via application of a significant number of load/rest cycles (e.g. WSDoT (2009) specify a minimum of 1000 cycles with a standardised load magnitude of 55 kPa); and

(ii) Application of a sequence of repeated (cyclic) loads onto the conditioned sample. The stress dependency property of the $M_R$ parameter – further detailed in Section 2.5 – is assessed by repeating the test at a number of confining stresses (e.g. WSDoT (2009) apply a standard 200 load cycles per confining stress magnitude assessed and average the recorded deformation after each cycle).

Also, as per Figure 2-5, the modulus parameter determined will also vary based on the number of loading / unloading cycles previously applied to the material.

The resilient modulus ($M_R$) is considered analogous to the elastic modulus (E) for soil characterisation and pavement design purposes, as the loads applied in the laboratory test are very small when compared to the ultimate loads imparted at failure and as, due to the large number of cyclic loads applied, the deformation measured during the test is considered completely recoverable (i.e. elastic properties). As identified by Puppala (2008), it appears that the $M_R$ parameter was selected as the 'standard' modulus value for pavement design as it is close to $E_0$ for stiff materials.

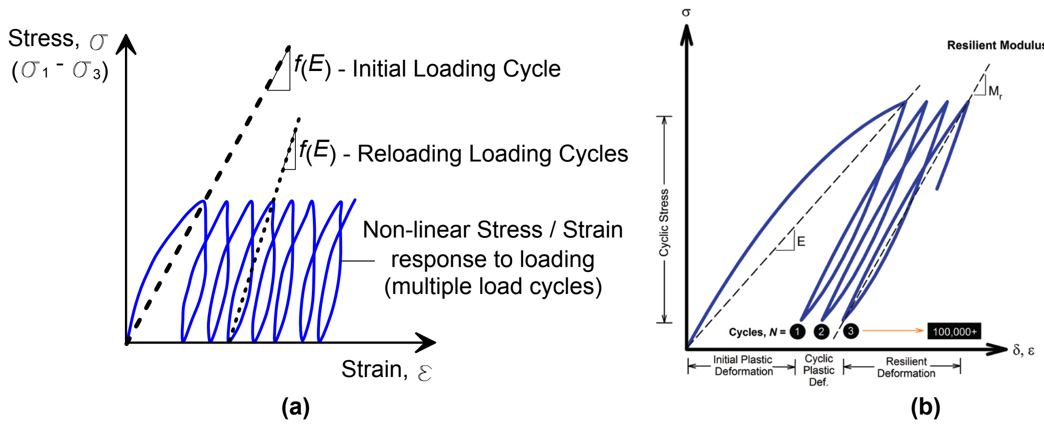



**Figure 2-5 Variation in secant modulus based on cyclical loading (a) Initial Loading cycle ($E_i$) compared to reloading cycles ($E_2$, $E_3$ etc.), after Briaud (2001); and (b) Initial Loading cycle ($E_i$) compared to resilient modulus ($M_R$) (from http://www.ingios.com/)**

## 2.4 Poisson’s Ratio

Poisson’s ratio ($\nu$) is the ratio of the transverse strain to longitudinal strain observed as a result of a change to the normal stress applied to a material. This theoretical value varies between 0.5 (undrained clay) and 0.0, with typical values utilised by industry shown in Table 2-1 (from Das, 2011 and Look, 2007).

**Table 2-1 Typical Poisson’s Ratio ($\nu$) for soil and rock materials**

| Material | Relative Density / Consistency | Poisson’s Ratio ($\nu$) | Reference |
|---|---|---|---|
| Sand | Loose | 0.20 – 0.40 | Das, 2011 |
| | Medium Dense | 0.25 – 0.40 | |
| | Dense | 0.30 – 0.45 | |
| Silty Sand | Various | 0.20 – 0.40 | |
| Sand and Gravel | Various | 0.15 – 0.35 | |
| Sands, gravels and other cohesion-less materials | Various | 0.30 – 0.35 | Look, 2007 (after Industrial Floors and Pavement Guidelines, 1999) |
| Cohesive Materials | Low PI (< 12%) | Undrained: 0.35 Drained: 0.25 | |
| | Medium PI (12% < PI < 22%) | Undrained: 0.40 Drained: 0.30 | |
| | High PI (22% < PI < 32%) | Undrained: 0.45 Drained: 0.35 | |
| | Extremely High PI (>32%) | Undrained: 0.45 Drained: 0.40 | |
| Intact Bedrock | N/A | 0.10 – 0.30 (0.20) | Look, 2007 |
| Cement Treated Materials | N/A | 0.20 | |

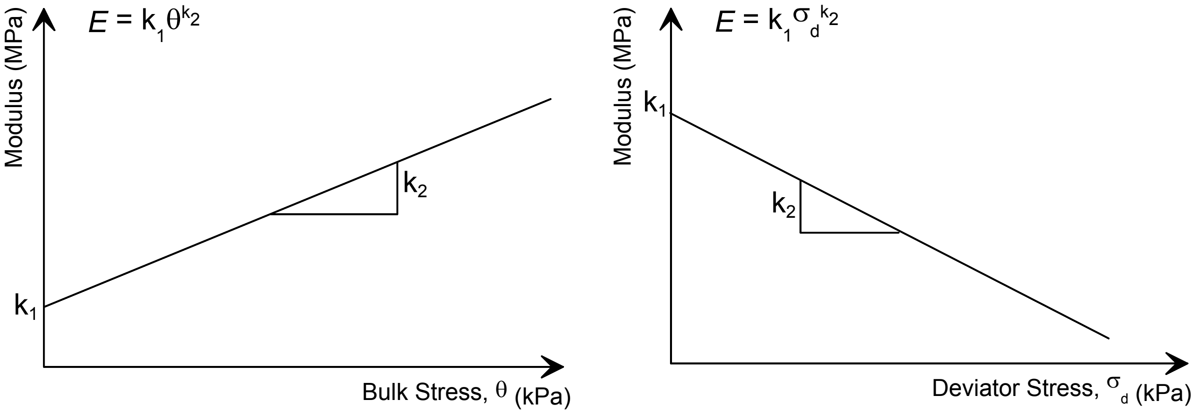
## 2.5 Stress Dependency

The modulus parameter (*E*) is known to be stress dependent, and individual materials will exhibit variation in the observed modulus based on the stress applied during testing. As reported in NCHRP Synthesis 676 (Mooney et. al., 2010) the observation of modulus variation as a function of varying the applied force is “well established in the laboratory (e.g. Ishihara, 1996, Andrei et al., 2004).”

The extent of ‘stress dependency’ behaviour of soils (i.e. the magnitude of variation in calculated *E* values based on the varied applied magnitudes of stress) has been previously reported to be material specific, and largely based on the dominant soil component. This behaviour is similar to that observed in the $M_R$ results obtained from Repeated Load Triaxial (RLT) testing, for which Mahoney et. al. (1991) identified that for the particular soil being tested, the modulus parameter may either increase, decrease or remain relatively steady as he applied stress magnitude is increased. As shown in Figure 3.8, Mahoney et. al. (1991) found that power relationships could be generally fitted to the relationship between modulus (E) and applied stress magnitude.

Modulus parameters are expected to increase as the confining (and thus bulk) stress is increased, whilst the same parameter would likely decrease with increasing shear (deviatoric) stress. For granular soils dominated by fine-grained materials and for all cohesive soils, the magnitude of decrease in modulus with increasing shear stress typically outweighs the increase in modulus due to increasing effective confining stress (Andrei et al., 2004). This is conceptually shown in Figure 2-6. However, it should be noted that the presence of comparatively soft or stiff layers within a specific test’s ‘zone of influence’ may individually alter the response of the modulus parameter at varying stress magnitudes.

The presence of stress dependency behaviour means that all derived modulus parameters should be reported with the associated test stress / range and, where appropriate, standardised to facilitate direct comparison of test results. Similarly, the selection of a suitable modulus test method for use as a QA tool should consider the test stresses to imparted upon the material, and if such stress magnitudes are representative of those to be imparted during the life of the project.



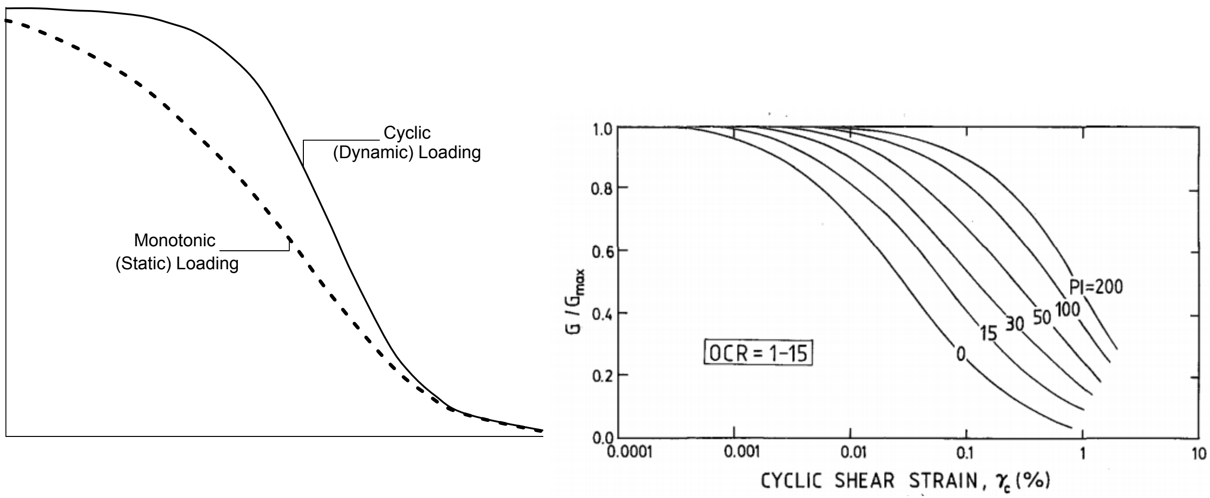

**(a)** **(b)**

**Figure 2-6 Expected 'stress-dependent' behaviour and variation in modulus based on variation of applied test stress. (a) for granular materials and (b) for fine grained / cohesive soils (after Mahoney et. al., 1991)**

## 2.6 Strain Dependency

In addition to exhibiting stress dependency, the measured modulus parameter also varies significantly based on the strain condition at which it is tested. Generally, if a constant stress magnitude is applied, the modulus will reduce as the strain magnitude is increased (refer Figure 2-7a). Although the exact reduction curve is material (and likely site) specific, a number of existing researchers (e.g. Mayne, 2001; Atkinson, 2000) have derived 'modulus degradation curves' that relate the modulus reduction based on the imparted strain level.

The particular rate of modulus reduction / relationship with strain has also been related to other material characteristics; such as plasticity (refer Figure 2-7b, from Vucetic & Dobry, 1991), Over-Consolidate Ratio (OCR), material age, void rato (*e*) or presence of cementation. Moreover, as identified by researchers such as Lo Pestri et. al. (1993), the rate of the modulus parameter reduction can be faster for materials exposed to monotonic (static) load testing than when the same material is exposed to dynamic loading scenarios. This is especially relevant for granular materials, and has been hypothesised to be caused by the progressive crushing of materials under cyclic / dynamic loading, in which the contact area between soil particles is increased during the test procedure.

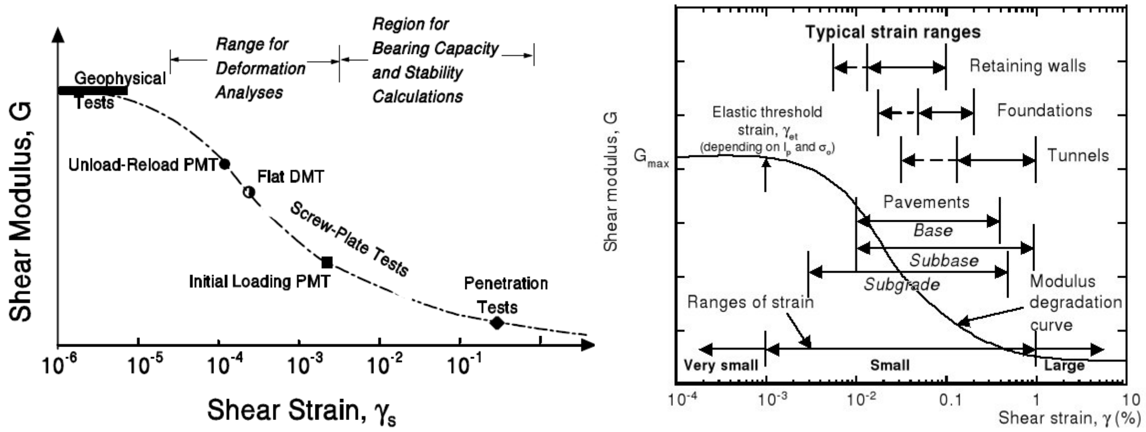



**(a)** **(b)**

**Figure 2-7 Modulus Degradation Curve – (a) comparative rate of modulus reduction based on shear strain imparted during testing, shown both for static and cyclical loading (after Mayne, 2001); (b) Variation in reduction curves based on plasticity of materials, as observed under dynamic testing conditions (from Vucetic & Dobry, 1991)**

The adopted method of site assessment / QA tool will test the soil at a specific strain level. This can vary from geophysical methods returning modulus values observed at very small strain levels (resulting in maximum modulus – $E_0$ or $G_0$ – parameters) to test methods that result in large strains and material failure (e.g. penetration tests such as DCP). The *insitu* strain magnitude induced by site development will vary based on (a) the loading type and magnitude applied to the material (and any variation thereof over the life of the project); and (b) the location (depth) of material below the level of the applied loading. For realistic analysis, it is thus important that the modulus parameter incorporated into settlement and / or deformation analyses allow for the reduction in modulus based on the likely strain magnitude induced by the proposed development.

As reproduced in Figure 2-8, both Sabatini et. al. (2002) and Sawangsuriya (2012) have identified locations upon the modulus reduction curve applicable to the strains imparted by (a) typical *insitu* test techniques and (b) various permanent structure types. For example, the characteristic shear strain ($\gamma_s$) induced for the monotonic loading of foundations and walls are generally in the range of $10^{-1}$ to $10^{-2}$ percent (Burland, 1989).




(a) (b)

**Figure 2-8 Variation in shear modulus with different shear strain levels annotated with different geo-engineering applications and *insitu* tests. (a) from Sabatini et. al. (2004); and (b) from Sawangsuriya (2012).**

# 3. Current (Density) QA Compaction

## 3.1 Introduction to density QA testing

Currently, compaction QA of earthworks / subgrade / unbound pavement materials involves determining the field dry density and moisture content of compacted lifts and comparing them to target density and moisture content values. The *insitu* measured parameters are compared to the maximum dry density or optimum moisture content (OMC) values determined via laboratory-based tests performed on the same material. The ratio between the field density and the target laboratory determined density value is referred to as relative compaction or Relative Dry Density (RDD), whilst the ratio between field moisture content and OMC is referred to as the moisture ratio.

It is noted that the material type and compactive effort applied in both the field and laboratory state should be similar, otherwise the determined RDD / moisture ratios would not be considered valid. To this end, the compactive effort applied in laboratory tests are generally standardised – standard or modified compactive effort – and required to be identified in density test reports. Within Australia, both Australian Standard and relevant testing authorities have standard test methods that stipulate the methodology associated with density tests.

## 3.2 Density Test Techniques

Density testing can be either destructive (replacement methods) or non-destructive (nuclear methods). The traditional sand replacement method typically extends to 250mm depth. Within Australia both nuclear and sand-replacement tests are currently routinely completed as QA/QC tests to conform road embankments to the compaction requirements (and are specified within the standards of the relevant testing authority).

The current state of industry is to utilise either of the following (2) density tests for QA purposes:

- Sand Replacement Tests
- Nuclear Density Gauge Tests

Of these two (2) test techniques, the sand replacement test is accepted to provide the ‘reference’ density value as it involves the physical replacement of materials within the compacted strata undergoing assessment. In contrast, the Nuclear Density Gauge (NDG) tests relies on the comparison of propelled and detected gamma rays. Thus, a relationship (bias) between the compaction level (RDD) reported by the NDG and Sand Replacement Tests must be first established to allow the NDG to provide meaningful results. The frequency or intensity of testing required to establish the bias between density test techniques depends on the material type (e.g. granular, cohesive materials; presence and proportion of large particles) and the uniformity of construction materials.

### 3.2.1 Sand Replacement Test

The often used sand replacement test involves a hole of specific dimensions – normally circular or rectangular with a diameter of 150mm to 250mm and depth of between 150 and 300mm – being excavated from the surface. The recovered soil has its mass and water content accurately determined (oven dried in laboratory setting) and the volume of the hole is determined by full replacement of the excavation with a clean, uniform sand of known dry density (refer Figure 3-1- sand cone). This is a destructive test.

The *insitu* relative density achieved (i.e. relative dry density, RDD or density ratio, DR) of the material being assessed is determined via comparison of the *insitu* density achieved and the maximum dry density (MDD) of the material. The maximum dry density is typically determined in a laboratory via the use of the Proctor compaction methodology.

It should be noted that Proctor (1945) identified that his aim in the development of a ‘standard’ compaction method for determination of MDD was never intended to become a universal method used for QA purposes. He also argued that QA methods should focus on the completion of *insitu*, strength based penetration tests rather than the assumption of *insitu* strength from measured dry density. Similarly, the ‘standard’ compaction method was also derived for fine grained material, rather than the common practice of implementing it to all material types.

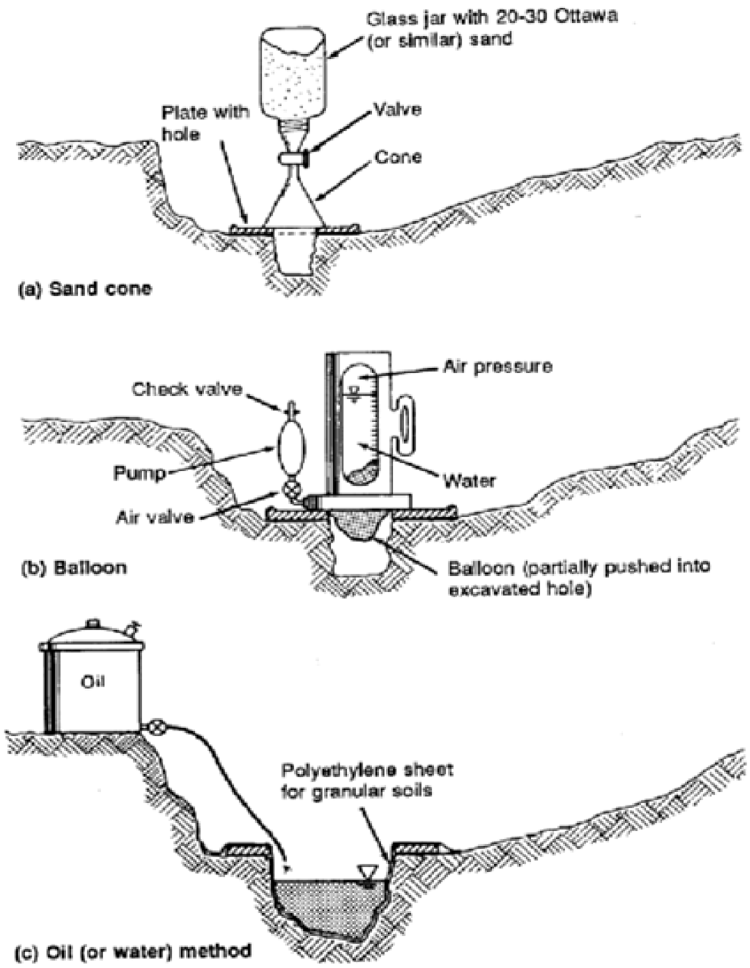



**Figure 3-1 Various replacement (destructive) density test equipment (from Holtz and Kovacs, 1981)**

### 3.2.2 Nuclear Density Gauge (NDG) Test

The principle of operation of the NDG is that gamma radiation is emitted by the device into the compacted material and the reflected rays are recorded (detected) to determine the material's insitu wet density. The denser the compacted material are, the higher the frequency of interaction between the electrons and the gamma radiation's photons and lower the number of gamma ray 'detections' (i.e. denser materials reflect a lower number of photons). Based on calibrated relationships, the number of recorded photon reflections is used to calculate the density of the compacted material.

NDG's have the advantage that they are also capable of determining the *insitu* moisture content of the compacted material by assessing the frequency that the neutrons imparted into the material from the gauge's nuclear source have been slowed by the hydrogen atoms present within the compacted material (i.e. wetter materials cause a higher number of neutrons to become slow-moving than drier materials). The amount of hydrogen atoms – and thus moisture content – present within the compacted materials can be determined by the NDG by counting the number of neutrons moving at slow speed.

As cited by Kim et. al. (2010) the NDG device gained widespread acceptance as a valid compaction QA method in the early 1970s after an industry-wide calibration standard was developed. As presented in Figure 3-2, nuclear gauges can be operated in two modes: direct transmission mode and backscatter mode. Winter and Clarke (2002) reported that direct transmission mode yielded a more accurate density measurement than use of the NDG in backscatter mode.

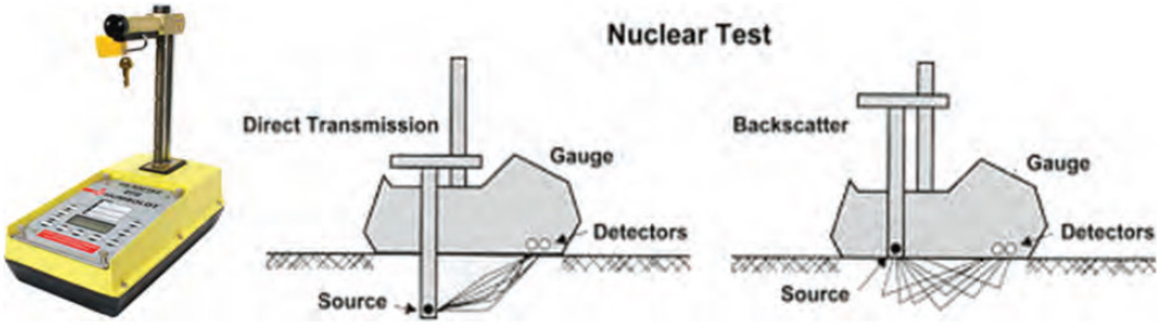



**Figure 3-2 Nuclear Density Gauge (after Troxler, 2000)**

## 3.3 Relevant Australian Standards applicable density QA testing

The following Australian Standard and regulatory bodies have standard test methods applicable to 'traditional' density QA testing, such that a Relative Dry Density (RDD) ratio can be determined upon placed and compacted materials:

### 3.3.1 Sand Replacement Test

- *Australian Standard* – 1289.5.3.1 Method 5.3.1: Soil compaction and density tests—Determination of the field density of a soil – Sand replacement method using a sand-cone pouring apparatus
- *Queensland Department of Main Roads* – Test Method Q141B: Compacted density of soils and crushed rock - sand replacement
- *VicRoads* – Code of Practice, RC 500.05 – Code of Practice for Acceptance of Field Compaction.
- *NSW Roads & Maritime Services (RMS)* – Test method T111 Dry density/moisture relationship of road construction materials

### 3.3.2 Nuclear Density Gauge Test

- *Australian Standard* – 1289.5.8.1 Method 5.8.1: Soil compaction and density tests—Determination of field density and field moisture content of a soil using a nuclear surface moisture-density gauge – Direct transmission mode
- *Queensland Department of Main Roads* - Test Method Q141A – Compacted density of soils and crushed rock - Nuclear Gauge
- *VicRoads* – Code of Practice, RC 500.05 – Code of Practice for Acceptance of Field Compaction.
- *NSW Roads & Maritime Services (RMS)* – Test method T111 Dry density/moisture relationship of road construction materials

## 3.4 Limitations of use of Density Tests as QA tools

Density tests should be considered only indicators of the key parameters (strength, stiffness), and a higher density does not always indicate a higher strength / stiffness of material. Even if such a relationship does exist for a specific material, the relationship between density and other engineering parameters is not necessarily linear.

Additionally, if oversize particles are present within the tested material, correction factors are required to be applied. However, in current industry practice, such correction factors are often not applied due to commercial, time and / or a lack of awareness of this requirement.

### 3.4.1 Density Test vs Modulus Parameter

Although embankment and pavement designs are based on achieving a design modulus (stiffness) the current practice of onsite QA is primarily density measurement based. The measurement of achieved density – and the comparison to maximum achievable density to produce relative compaction values – adopts an underlying assumption that if a certain RDD threshold has been achieved the design stiffness (modulus) has also been achieved.

This approach is flawed in that density measurements are not reflective of, or fundamentally related to, the stiffness of unbound pavement or earthworks materials. The use of density measurement for QA and compaction control do not provide information related to the construction material's stiffness or strength, and thus cannot be considered to be indicators of the material's likely performance in terms of stability or resistance to deformation under loading. Small variations in the placed material's density can have comparatively large implications to the material's achieved stiffness and strength, and thus the sole acceptance of density testing for material QA has the potential to result in cumulative errors that may significantly influence the performance of the compacted subgrade / pavement materials.

Subgrade modulus is typically obtained by assessment of the California Bearing ratio (CBR) and correlating this to a modulus parameter (commonly via generic relationships rather than the assessment of site- or material-specific correlations). As such, the measurement and application of density tests for onsite QA is utilising a 3rd order parameter (and CBR a 2nd order parameter) to assess if the *insitu* modulus parameter reflects that adopted by the designer during the mechanistic–empirical pavement design procedure.

Figure 3-3 conceptually presents the typically applied assumed relationships to determine / validate modulus parameters from field or laboratory testing. As identified in this figure, there are significant locations within this procedure where correlations between parameters is assumed (e.g. density / penetration rate to CBR, or CBR to modulus), and where significant errors can be incurred if generic relationships are adopted.

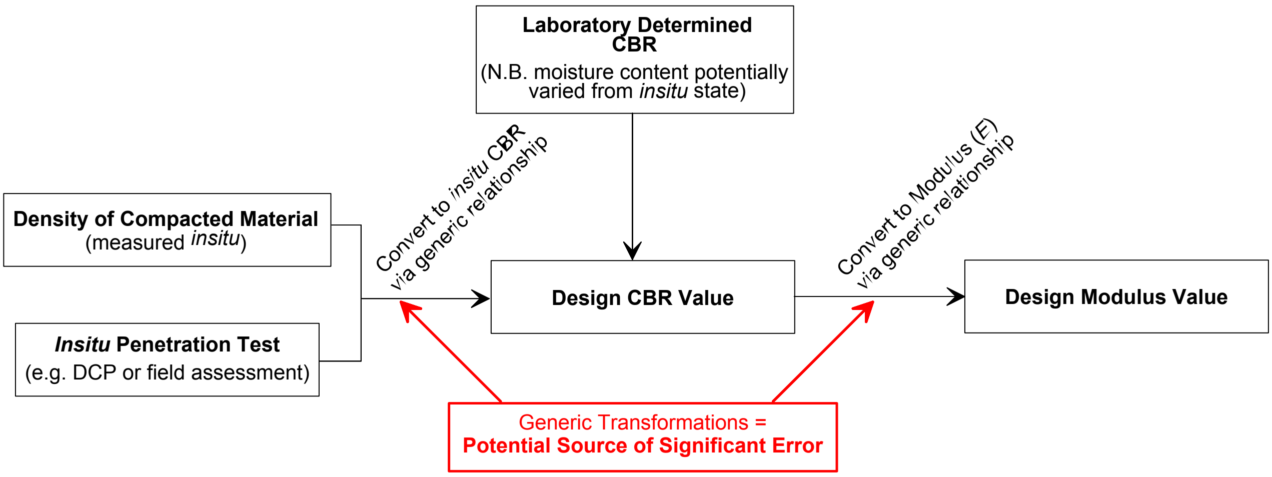



**Figure 3-3 Flow chart demonstrating locations of potentially significant error due to assumption of generic relationships**

### 3.4.2 Issues with using density tests as sole QA parameters

Importantly, recent testing (e.g. Floss et. al., 1991 and Mooney et. al. 2003; as cited in Mooney et. al. 2010) has shown a higher density does not necessarily indicate a higher strength or modulus. Modulus can be influenced via a number of non-density related factors, including moisture content, underlying layer stiffness and the stress at which a test is conducted.

Historically (and currently) density testing and CBRs have been used for QA purposes. However, the use of density / CBR tests have several associated limitations that include:

- *They are lag indicators* – Several days to 1 week is typical to complete the laboratory evaluation of sampled materials, and thus to provide results of the ratio between the maximum / optimum material parameter that has been achieved in the field state. During this time the Contractor typically continues work and advances fill placement above the lift (without waiting for acceptable QA results to be provided). Significant project timeframe and equipment standby costs would be incurred if the Contractor had to wait on acceptable QA results to be provided prior to continuing work, which are considered unacceptable to all parties. However, if non-conforming QA test results are received, significant costs of remove and replace both the non-conforming material and the overlying material are incurred. Accordingly, minimising the lag between site preparation / site testing and receipt of test results would result in the minimisation of costs associated with any required re-work;

- *Density oversize correction* – A correction factor is required to be applied when greater than 20% of material exceeds 19mm or 38mm for Mould A and B size, respectively. This is not consistently being applied across industry, with 22% of 235 samples examined not applying that correction (Look, 2016).
- *Compaction and CBR tests are technology from the 1930s* – Equipment advances brought a "modified" test in the 1940's. Nuclear gauge density tests, Benkelman Beams, Deflectometers, Dynamic Cone Penetrometers and Clegg Hammers were introduced as alternative onsite testing tools in the 1970's. These tools, with the exception of nuclear gauge density testing, have not been widely accepted as alternative QA test techniques within Australia. The LFWD, BCM and SGG have been used internationally in the past 25 years. It is time to use equipment of the 21st century.
- *Mechanistic – Empirical pavement design is based on strength and modulus parameters* – Yet we test CBR and continue to adopt a simple correlation (linear or power relationship) to relate the CBR test result to a modulus parameter (e.g. $E$ = 10 x CBR). Such correlations are generic and would be expected to have a significant correlation error (of $\pm$ 50%) if site- and material-specific relationships are not determined. CBR is an INDEX of strength; and is thus neither a direct measure of strength or modulus parameters. Density is also neither a strength nor modulus parameter and is simply assumed that such a relationship exists for the purposes of QA (due to the comparative simplicity of density testing).
- CBR is a highly variable index, with recent attempt in testing standards to reduce that variability by increasing preparation time by up to 7 days for highly plastic clays (refer Table 3-1). This amplifies the issue of current tests being a lag indicators. AS1289.6.1.1 requires reporting of the CBR value to the nearest 5% and 10% for values above 21% and 50%, respectively. This suggests the variability of a set of test results for a single material unit is expected to be at least 20% of the average CBR test result. Rallings (2014) reported that for a typical set of CBR test results, less than 60% of the test results would report a CBR value within ±30% of the median CBR value.

**Table 3-1 CBR Curing Time (as per AS1289.6.1.1, 2014)**

| Plasticity | Conditions of prepared sample | |
|---|---|---|
| | Within 2% of OMC | Greater than 2% from OMC |
| Sands and granular material* | 2 hrs | 2 hrs |
| Low (LL $\leq$ 35%) | 24 hrs | 48 hrs |
| Medium (35% $\leq$ LL $\leq$ 50%) | 48 hrs | 96 hrs (4 days) |
| High (LL > 35%) | 96 hrs (4 days) | 168 hrs (7 days) |

*These can include naturally occurring sands and gravels, crushed rocks and manufactured materials with fines content typically less than 12%

- In an attempt to improve repeatability of results, Australian Standards have now introduced additional testing time when industry is currently demanding a faster turn-around of test results. This constrains delivery on construction projects, in which a Contractor typically has tight completion timeframes and testing delays could incur significant equipment standby costs. A blind eye for the increased curing time requirements is already occurring in practice among road authorities, contractors, consulting engineers and the testing authorities. This is evident in that test results are still being provided in the same timeframe as prior to the updated CBR test Australian Standard (2014) being issued, and such results are still being accepted and utilised without question. Instead, the typical testing time – from sample receipt till the test result / report being made available – should have extended from 1 week (minimum) to over 2 weeks if the material is to cured for typically 4 to 7 days prior to a 4 day typical soaked CBR (as per the requirements of the 2014 Australian Standard).

- *Variation in 'standard' test procedures* – As per the relevant Australian Standards (AS1289.6.1.1, 2014) the CBR test is not applicable for materials in which more than 20% is retained on 19mm sieve, as such material is discarded and the sample confined within the CBR mould becomes non-representative of the material. In contrast, the QTMR testing standard *Q113A – CBR of a Soil - Standard* (2016) allows material particles of any size to be crushed to pass through 19 mm sieve. Such differences in material preparation would result in different CBR test values being determined and reported. Similarly, the Australian Standard and QTMR standards for nuclear density gauge testing have significantly different definitions regarding the quantity of 'oversize' materials allowed to be present within a material for the test technique to be considered valid.

### 3.4.3 Case Study

A few of the issues and the inappropriateness associated with the sole use of laboratory CBR test results for evaluation of *insitu* material conditions (i.e. relating laboratory CBR to *insitu* stiffness parameter) can be illustrated by the results of a QLD site based case study. As summarised in Table 3-2 (Lacey et. al., 2016), the results of this side-by-side testing program demonstrates:

- Weathered rock material reports a maximum laboratory determined soaked CBR of be 12% to 13% across XW to HW rock, regardless of the strength improvement associated with the decrease in weathering state. This maximum CBR soaked value is likely due to the exclusion and disposal of oversize material during the sample preparation methodology typically adopted for the CBR test. Whereas the other three (3) test techniques used to measure *insitu* CBR and stiffness parameters clearly show a significant change in material parameters for each rock weathering state considered, this is not reflected in the reported laboratory determined CBR values;
- The DCP values is widely used in industry to derive *insitu* CBR with generic correlations, yet the literature shows any such correlation is material- and site-specific. Thus the adoption of a generic correlation to DCP test results would be expected to yield inexact CBR values; and
- Different commercially available Light Falling Weight Deflectometer (LFWD) equipment can produce different *insitu* measurements of the same parameter (modulus). While useful in a relative sense, there is a need also to be able to use a reliable (standardised) stiffness parameter. Refer to Section 4.3.2 of this document for further discussion of this variation in LFWD derived parameters due to instrument configuration.

**Table 3-2 Comparative laboratory determined and *insitu* derived CBR and modulus parameters for a range of soil and rock materials encountered on a QLD site (Lacey et. al., 2016)**

| Weathering state of Material | | Soaked CBR (%) | *Insitu* CBR (%) | *Insitu* Modulus – LFWD Testing (MPa) | |
|---|---|---|---|---|---|
| | | Laboratory Tested | Derived from DCP | Zorn ZFG Instrument | Prima 100 Instrument |
| Fill | | 1.5 % | 11.9 % | 11.9 MPa | 20.0 MPa |
| Residual Soil | Granular | Not Tested | 23.3 % | 24.0 MPa | 41.2 MPa |
| | Cohesive | 3.0.% | 16.5 % | 16.6 MPa | 37.2 MPa |
| XW Rock | | 12 % | 39.4 % | 31.1 MPa | 69.8 MPa |
| XW / HW Rock | | 13 % | 51.7 % | 35.0 MPa | 85.1 MPa |
| HW Rock | | 12 % | 63.5 % | 40.9 MPa | 134 MPa |

# 4. Alternative Compaction QA Methodologies

## 4.1 Introduction to alternative compaction QA testing

The widespread adoption of geotechnical design procedures that incorporate the material strength and / or material stiffness (modulus) parameter as a key input requirement has resulted in an increased industry awareness / interest in alternative QA techniques that provide direct measurement of stiffness and strength parameters (i.e. to allow direct comparison with design adopted parameters, and assurance that such parameters have been achieved in the field).

Current iterations of many mechanistic–empirical pavement design procedures incorporate generic relationships between commonly undertaken material tests (e.g. CBR or DCP test results) and the desired modulus design parameter. This has arisen due to the lack of common testing methods that could provide direct measurement of the modulus parameter of construction materials. However, the use of directly measured site- and material-specific modulus parameters has been identified as desirable as the use of such test results can produce more efficient pavement and earthworks design solutions (rather than the reliance on a generic relationship that may either under- or over-estimate the *insitu* modulus achieved, and thus potentially result in either unnecessary project costs or unsafe / inappropriate solutions respectively).

Industry desire for the direct measurement of *insitu* modulus (stiffness) parameters has thus driven the development of several innovative field test devices that directly measure the stiffness or strength of compacted earthworks / unbound pavement materials. Test devices currently commercially available to provide non-density compaction QA can be divided into four (4) main groups:

- *Penetration Test Devices* – Index testing techniques that utilise hammer blows to drive rods through the compacted material, including the Dynamic Cone Penetrometer (DCP) or PANDA. Insitu strength assessment techniques that also utilise small diameter boreholes that penetrate into the compacted materials to allow ‘at depth’ assessment are also included in this grouping of field tests;
- *Surface based Impact Devices* – Devices that apply static, vibratory, or impact load to the ground, then estimate the stiffness based on the load and displacement measurements (using velocity transducers or accelerometers). These include the Light Falling Weight Deflectometer (LFWD or LWD), Clegg Hammer, Briaud Compaction Device (BCD), static Plate Load Test (PLT) and the GeoGauge;
- *Geophysical Methods* – In which surface waves are generated and detected in the tested layer to determine its modulus. Specific compaction geophysical devices exist such as the Portable Seismic Pavement Analyser (PSPA), or common geophysical methods (e.g. MASW, SASW) could be used to derive shear wave velocity profiles of the compacted material (when present in sufficient thicknesses);
- *Insitu (sacrificial) sensors* – Equipment buried within the compacted soil to monitor the growth in amplitude of compression waves during compaction and changes in material density. An example of such equipment is Earth Pressure Cells (EPC).

In addition, roller mounted Intelligent Compaction (IC) and Continuous Compaction Control (CCC) techniques are simultaneously being developed, whereby compaction rollers are instrumented with a real-time kinematic (RTK) and location (GPS) sensors along with a roller drum measurement system. Combined, this provides an on-board display of real-time compaction measurements, allowing the roller operator to identify localised locations of comparatively low stiffness and directly apply remedial action prior to QA material testing.

The following sections of this chapter describes the specific test equipment and techniques within both the ‘penetration test’ and ‘surface based impact device’ groupings of compaction QA test devices, as the employment of such tools in place of existing density tests are considered the most probable in the near-future, not require preparation / installation prior to earthwork / pavement construction, and would still maintain a post-compaction test program undertaken by an independent verifier (i.e. largely maintain status quo).

For each considered QA technique the following information is provided:

- Description of test equipment

- Description of test methodology
- Standard documents (Australian and/or International Standards) applicable to test equipment or test technique
- Specific material parameter / Index value reported by test technique
- Repeatability of test results
- Advantages of test
- Disadvantages of test
- Review of existing literature relating to use of test as a QA method

## 4.2 Penetration Test QA Techniques

### 4.2.1 Dynamic Cone Penetrometer (DCP)

#### Test Equipment and Methodology

The Dynamic Cone Penetrometer (DCP) is a simple, portable and low cost penetration tool used as an indicator of strength and variation within a sub-surface profile.

Figure 4-1 presents the key elements of the DCP test equipment, which has different configurations (hammer weight, drop height and angle of cone) in various countries. For example, in many non-Australia jurisdictions (e.g. North America) the cone angle adopted for a standard DCP test is 60˚ rather than the 30˚ cone stipulated by the Australian Standard, *AS1289.6.3.2*.

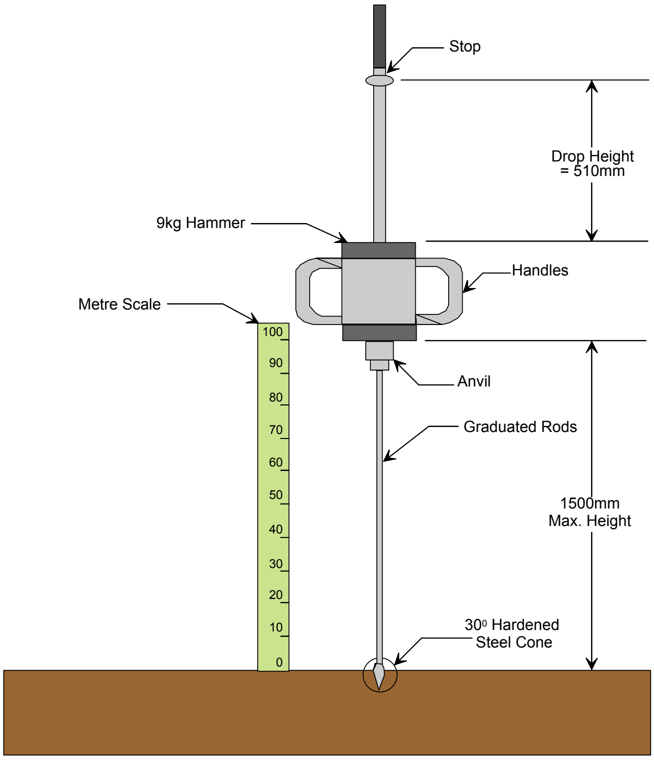



**Figure 4-1 Key components and general equipment setup of DCP Penetration test (after Look, 2014)**

Initially developed in Australia (Scala, 1956) and refined within South Africa (Kleyn, 1975), the DCP has gained widespread use for site characterization of pavement layers and subgrades within the United States, the United Kingdom, Australia, and New Zealand.

In Australia, the DCP is a penetration technique is governed by Australian Standard *AS1289.6.3.2*. This test method stipulates a 20mm diameter, 30˚ cone is manually driven by the repeated dropping of a 9 kg hammer vertically onto an anvil from a regulated drop height of 510mm (i.e. to produce a consistent driving stress from each hammer blow) to induce the penetration of a solid cone into the subsurface. Table 4-1 summarises the key dimensions and weights associated with the standard DCP test method observed by various jurisdictions worldwide.

**Table 4-1 Dimensions and weight of key equipment within DCP apparatus for various national standards**

| Element | | Australia (AS 1289.6.3.2:1997)<br><br>New Zealand (NZS 4402.6.5.2:1988) | United States (ASTM D6951)<br><br>South Africa (Method ST6 in TMH No. 6) | Europe / United Kingdom (BS EN ISO 22476-2:2005) – Dynamic Probe Light (DPL) |
|---|---|---|---|---|
| Hammer / Weight Drop | Mass | 9 kg | 8 kg | 10 kg |
| | Standard Drop | 510 mm | 575 mm | 500 mm |
| Theoretical Energy per blow | | 45.0 J | 45.15 J | 49.0 J |
| Cone Dimensions | Angle | $15^{\circ}$ (from centreline of cone) | $30^{\circ}$ (from centreline of cone) | $45^{\circ}$ (from centreline of cone) |
| | Diameter | 20 mm | 20 mm | 34 mm (min.) |
| | Mantle Thickness | 3 mm | 3 mm | 37.5 mm |
| | Surface Area (Lateral + Mantle) | 12.7 $cm^2$ | 6.9 $cm^2$ | 25.0 $cm^2$ |

### Applicable Standards

- AS 1289.6.3.2-1997 (R2013) – *Methods of testing soils for engineering purposes – Soil strength and consolidation tests - Determination of the penetration resistance of a soil - 9kg dynamic cone penetrometer test*
- ASTM D6951 – *Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications*
- NZS 4402.6.5.2 (1988) – *Methods of testing soils for civil engineering purposes - Soil strength tests - Determination of the penetration resistance of a soil - Test 6.5.2 Hand method using a dynamic cone penetrometer*
- QLD Department of Transport and Main Roads – *Test Method Q114B: Insitu California Bearing Ratio - dynamic cone penetrometer*
- The South African National Roads Agency (1984) – *Method ST6 – Measurement of the In Situ Strength of soils by the Dynamic Cone Penetrometer (DCP)*
- BS EN ISO 22476-2 (2005) – *Geotechnical Investigation and Testing–Field Testing–Part 2: Dynamic probing*

### Parameters provided from test

The results of the DCP test can be reported as either:

(a) Penetration Resistance (PR or DCP-PR or $N_p$) – the number of blows required to produce a rod penetration of a standard length (normally 100mm or 300mm); or

(b) The length of rod penetration produced per single hammer blow, in millimetres / blow (denoted DCPI or $DCP_I$).

DCP test results (PR or $DCP_I$ values) are then, via generic or site-specific correlations, used to infer relative density / consistency categories of the subsurface profile or to derive material parameter profiles (e.g. shear strength, *insitu* California Bearing Ratio (CBR) or modulus values).

### Repeatability of Test Technique

As cited in the NCHRP Synthesis 456 (Nazzal, 2014) the repeatability of DCP results is related to the material type being assessed. Although many researchers have identified a 'good' repeatability is achievable within uniform materials (e.g. Peterson and Peterson, 2006; Dai and Kremer, 2006), a number of other projects have identified that DCP test results should be viewed with caution due to high variability.

Von Quintus (2008) reported a CoV of DCP test results that varied between 2.9% and 27.4% for tests completed on 10 types of soil at seven (7) roadworks test locations. Such a CoV range was identified to be due to the DCP penetration rate being affected by the varying amount and size of coarse aggregate particles present within the subsurface being assessed.

Mellish et. al. (2014) assessed both the spatial and temporal repeatability of the DCP by undertaking recurrent testing at a single site over a period of 12 months, whilst simultaneously monitoring the moisture content present within the subsurface. The results of the study demonstrated that for the gravelly clayey sand being characterised the inherent variability of the DCP test results (material and measurement error) was up to 27.4%, whilst temporal effects – associated with a natural moisture content change of up to 10% during the field monitoring period – added an additional average 22% variation. In total, the CoV of the results of the DCP tests were shown to be up to 58%.

Within three (3) trial embankments constructed as part of Queensland's TSRC project, pairs of DCP tests completed upon compacted fill embankment materials displayed a characteristic ($80^{th}$ percentile) variation from the averaged PR index value of +/- 22.6% (Sandstone), +/-24.6% (Interbedded Siltstone / Sandstone) and +/-27.4% (Basalt).

Similarly, high CoV values associated with DCP test results were also reported by Hossain and Apeagyei (2010). In their comparative study completed in Virginia, USA, an averaged CoV of 38.5% was observed for DCP data collected at 6 project sites (39 test locations, with individual CoV ranges of 13 to 68%), covering subgrade and 'gravel road' test environments.

In summary, the variability within DCP test results – and thus the repeatability of the DCP test technique – is known to significantly vary based on the material type being assessed. The DCP penetration rate within a single material is known to be affected by moisture content, density, material uniformity and the presence / frequency of large sized particles. From the CoV values returned by various researchers, the results of the DCP – PR or $DCP_I$ parameters – should not be relied upon for high accuracy or unchanging, and instead should be considered as representative of the material / condition at the time of testing. Both the CoV of test results and consideration for the potential for material conditions to change (e.g. seasonal moisture content variation) should also be incorporated during the derivation of characteristic parameters for geotechnical design.

Due to the potential variation in equipment utilised for DCP testing – and any developed correlations - it was also recommended by Pappula (2008) in NCHRP Synthesis 382 that test records should always present the potential energy applied with the DCP device when used in the field conditions (refer Table 4-1).

**Advantages**

- Quick and economical
- Simple, self-contained equipment that requires no calibration
- Requires minimal training
- Can be used to continuously profile comparative material state throughout the thickness of material unit (as long as refusal does not occur), and confirm thickness of individual layers (as long as comparative density is sufficient to make a distinction of interface depth).
- Widely used with standard specifications available for test methodology
- Has been demonstrated to have high strength correlations with strength and stiffness properties – e.g. CBR, shear strength and elastic modulus (*E*) – when applied to uniform materials

**Disadvantages / Limitations**

- Index test that does not directly measure a strength or deformation material parameter
- Generic conversions between DCP blow count and material parameters (CBR or modulus, *E*) are known to be dependent on the type of soil (e.g. Webster et. al., 1992) – site-specific correlations with other test techniques are required for best results.

- Hammer imparted energy is assumed to be constant, with no component to measure the imparted measurement
- An insensitive tool with the comparative tip movement per blow not consistently monitored after each hammer blow.
- Results can be insensitive, with single change in hammer blow count representing large change on materials. Coupled with the repeatability of the test, the interpreted material parameter can have large spread and be unsuitable for use to target compaction (RDD) thresholds.
- Penetration test is prone to refusal within gravel / granular materials (i.e. not suitable to assess compaction of granular unbound pavements). Presence of larger particles sizes within tested profile may also decrease the rod penetration rate, with no associated change in material density or strength.

**Existing Literature regarding use as compaction QA test**

A number of existing research studies have been undertaken to correlate the results of the DCP – the penetration rate of the rod based on repeated hammer blows – with level of compaction, California Bearing Ratio (CBR) and, often indirectly, Young's modulus (*E*).

However, as cautioned by Pappula (2008), such research studies lack a standardisation within the testing devices used – with different sized cones, hammer weights and drop heights being specified, resulting in varied test energy developed for rod penetration. Similarly, the method of rod penetration measurement / input into the developed correlation may vary based on jurisdiction (e.g. SI vs. imperial units; or depth interval over which averaged rod penetration is averaged). Accordingly, specific correlations should be viewed as being site- and equipment-specific, and the suitability of a specific correlation should be evaluated prior to adoption.

**DCP Penetration Rate correlated to CBR**

As cited in Borden et. al. (2010), the original intent of the DCP – as described by Scala (1956) – was for a field tool to be created that could be correlated with CBR. The suitability of the DCP to CBR relationship is noted to be effective, as the influence of both moisture content and dry density variation have been identified affect the results of both tests in a similar way (Siekmeier et. al., 2000; Harrison, 1987). However, it is also noted that if variation from the field moisture content condition occurs during subsequent testing – e.g. due to laboratory conditioning (e.g. soaking for CBR tests) – any DCP to CBR correlation should allow for such disparity between the moisture condition at time of respective testing.

A number of correlations between the results of DCP and CBR tests have been subsequently proposed, many of which are currently being adopted as generic correlations, as summarised in Table 4-2. The general form of the DCP to CBR relationship presented in Equation 4-1.

$$\text{Log (CBR)} = a - [b \times \text{Log}(DCP_I)] \qquad \text{(Equation 4-1)}$$

Where *a* and *b* are constants that vary by researcher based on the data points / materials utilised for the correlation study.

**Table 4-2 Examples of historical published relationships between DCP test results and CBR**

| Author | Study Location | Year of Publication | Relationship* | No. of Datapoints | Comments / Materials Tested |
|---|---|---|---|---|---|
| Scala | Australia | 1956 | Log (CBR) = 0.881 + 1.16 x Log (25 / $DCP_I$) | Unknown | Fieldwork |
| Kleyn | South Africa | 1975 | Log (CBR) = 2.62 – 1.27 x Log ($DCP_I$) | 2,000 | Pavement materials, Laboratory |
| Smith & Pratt | Australia | 1983 | Log (CBR) = 2.555 – 1.145 x Log ($DCP_I$) | Unknown | Fieldwork |
| Harrison | Indonesia | 1987 | Log (CBR) = 2.56 – 1.16 x Log ($DCP_I$) | 40 | Laboratory Based – For Clay-like soils with $DCP_I$ > 10mm / Blow |

| Author | Study Location | Year of Publication | Relationship* | No. of Datapoints | Comments / Materials Tested |
|---|---|---|---|---|---|
| Harison *(Cont.)* | Indonesia | 1987 | $\text{Log (CBR)} = 2.70 - 1.12 \times \text{Log} (DCP_I)$ | 32 | Laboratory Based – For Granular soils with $DCP_I > 10$mm / Blow |
| | | | $\text{Log (CBR)} = 2.81 - 1.32 \times \text{Log} (DCP_I)$ | 72 | Laboratory Based – All Material Types |
| | | 1989 | $\text{Log (CBR)} = 2.55 - 1.14 \times \text{Log} (DCP_I)$ | N/A | Collation of data from many previous studies |
| Livneh & Ishai | Israel | 1987 | $\text{Log (CBR)} = 2.20 - 0.71 \times \text{Log} (DCP_I)^{1.5}$ | 56 | Field and Laboratory |
| Livneh | Israel | 1991 | $\text{Log (CBR)} = 2.56 - 1.16 \times \text{Log} (DCP_I)$ | 76 | Granular and Cohesive |
| Livneh & Livneh | Israel | 1992 | $\text{Log (CBR)} = 2.14 - 0.69 \times \text{Log} (DCP_I)^{1.5}$ | 135 | Field and Laboratory |
| Livneh et. al. | Israel | 1995 | $\text{Log (CBR)} = 2.46 - 1.12 \times \text{Log} (DCP_I)$ | Unknown | Granular and Cohesive, Field and Laboratory (collation of data from many previous studies) |
| Ese et. al. | Norway | 1994 | $\text{Log (CBR)} = 2.44 - 1.07 \times \text{Log} (DCP_I)$ | 79 | Aggregate base course, Field and Laboratory |
| Webster et. al. | United States | 1992 | $\text{Log (CBR)} = 2.465 - 1.12 \times \text{Log} (DCP_I)$ | Unknown | Granular and Cohesive |
| | | 1994 | $\text{CBR} = 1 / (0.017 \times DCP_I)^2$ | 102 | For Low Plasticity Clay Soils (CL), for $DCP_I > 10$mm / Blow |
| | | | $\text{CBR} = 1 / (0.0029 \times DCP_I)$ | | For High Plasticity Clay Soils (CH), for $DCP_I > 10$mm / Blow |
| NCDOT | United States | 1998 | $\text{Log (CBR)} = 2.60 - 1.07 \times \text{Log} (DCP_I)$ | Unknown | For 6.31mm / blow $< DCP_I <$ 66.67mm / blow |
| Coonse | United States | 1999 | $\text{Log (CBR)} = 2.53 - 1.14 \times \text{Log} (DCP_I)$ | 15 | Piedmont residual soils, Laboratory |
| Gabr et. al. | United States | 2000 | $\text{Log (CBR)} = 1.55 - 0.55 \times \text{Log (DCPI)}$ | 16 | Aggregate base-course, Laboratory Tests |
| | | | $\text{Log (CBR)} = 1.40 - 0.55 \times \text{Log (DCPI)}$ | | Relationship from Lab Test results (16) reduced by 70% to match Field Tests (4 sites) |
| Nazzal et. al. | United States | 2003 | $\text{CBR} = 2559.44 / [(-7.35 + \times DCP_I^{1.84}) + 1.04]$ | 21 | Gravel and Clay mixtures, including treated soils, Fieldwork and Laboratory |
| Cited in Newcomb et. al. (1999) | Norway | Unknown | $\text{Log (CBR)} = 2.57 - 1.25 \times \text{Log} (DCP_I)$ | Unknown | Noted to be widely adopted in Norway |
| Abu-Farsakh et al. | United States | 2005 | $\text{Log (CBR)} = 2.256 - 0.954 \times \text{Log} (DCP_I)$ | 19 | Laboratory |
| | | | $\text{CBR} = 1.03 + [2{,}600 / (PR^{1.84} - 7.35)]$ | 19 | Fieldwork |
| | | | $\text{CBR} = 1{,}161.1 / PR^{1.52}$ | 38 | Laboratory & Fieldwork |

*Note the equipment used and defined measurement / units of rod penetration ($DCP_I$) may vary between studies

As identified in Table 4-2, the exact relationship between DCP and CBR is dependent on the material / sites being tested, with significantly different relationships being produced for cohesive and granular materials. Both Harison (1987, 1989) and Webster (1992, 1994) identify individual DCP and CBR relationships for granular and clay-dominated soils.

Similarly, a significantly different relationship to correlate DCP and CBR was identified to exist for studies based on fieldwork and in laboratory environments. Both Abu-Farsakh et. al. (2005) and Gabr et. al. (2000) identified separate equations that evaluated the DCP to CBR relationship – and differences between – for both laboratory and fieldwork settings.

Of the relationships presented in Table 4-2, the most widely adopted is arguable that provided by Webster et. al. (1992), which has been extensively utilised by the US Army Corps of Engineers and many subsequent researchers (e.g. Livneh, 1995; Siekmeier et. al., 2000; and Chen et. al. 2001).

### DCP Penetration Rate correlated to Modulus (*E*)

Once an equivalent CBR value is derived from DCP tests an estimate of the stiffness parameter can be made via use of CBR → E correlations. For example, the AASHTO Guide (1993) suggests the use of the formula first derived by Huekelom and Klomp (1962), to calculate the resilient modulus ($M_R$) from a CBR value (Equation 4-2):

$$M_R = 10.34 \times CBR(\%) \qquad \text{(Equation 4-2)}$$

In contrast to the equation included in the updated AASHTO Design Guide (2002), which suggested use of the relationship defined in Equation 4-3.

$$M_R = 17.58 \times CBR(\%)^{0.68} \qquad \text{(Equation 4-3)}$$

However, incorporation of a generic CBR to E relationship adds additional variability to any derived modulus parameter (i.e. DCP → CBR → E), as the specific relationship between CBR and E values is also dependent on, amongst other things, both the moisture content and the plasticity of the material being tested (Brown et al., 1987).

As identified in Section 2.6, for pavement and earthworks projects the modulus parameter should be assessed under comparatively high strain, as it is such conditions at which the material will be subjected to for the life of traffic loading. If shear modulus ($G_0$ or $E_0$) or modulus parameters derived from very low strain testing techniques are provided, then allowance for the reduction of the modulus parameter due to the increased stain condition encountered under subsequent loading is required.

Similar to the various correlations presented for DCP to CBR correlation, a number of researchers have published direct DCP to modulus relationships. However, in addition to the variation based on the equipment utilised, various researchers have correlated the results of DCP testing to various definitions of modulus – for example resilient modulus, *insitu* modulus or the deformation parameter reported by a specific modulus test (e.g. PLT, FWD or LFWD). Additional difficulties in determination of a standardised E parameter with which to correlate the DCP penetration rate also arise due to material testing typically being discontinued before material failure is observed, the stress dependency of the soil material being assessed and the shear failure caused by DCP tests.

Table 4-3 summarises a number of previously published DCP results to various stiffness parameters. Note that Table 4-3 does not include published correlations that relate DCP tests to insitu modulus values determined by Light Falling Weight Deflectometers (LFWD) or static Plate Load Tests (PLTs). Such relationships are identified in the relevant sections that consider LFWD and PLTs as potential QA/QC methods.

**Table 4-3 Examples of historical published relationships between DCP test results and Elastic Modulus (*E*)**

| Author / Study | Study Location | Year of Publication | Relationship* | Comments / Parameters |
|---|---|---|---|---|
| De Beer | South Africa | 1991 | $E_s$ (MN/m$^2$) = 3.05 – 1.07 x Log (DCP$_I$) | $E_S$ = Insitu soil modulus |
| Hassan | United States | 1996 | $M_R$ (psi) = 7,013 – 2,040.8 x Ln (DCPI) | DCPI in inches / blow<br>$M_R$ = Resilient Modulus |
| Pen | Malaysia | 1990 | Log ($E_s$) (MN/m$^2$) = 3.25 – 0.89 x Log (DCP$_I$)<br>Log ($E_s$) (MN/m$^2$) = 3.62 – 1.17 x Log (DCP$_I$) | $E_S$ = Insitu subgrade elastic modulus |
| Chai and Roslie | Malaysia | 1998 | $E_s$ (MN/m$^2$) = 17.6 x (269 / DCP)$^{0.64}$<br>$E_{Back\text{-}calculated}$ = 2,224 x DCP$^{-0.995}$ | $E_S$ = Insitu subgrade modulus<br>DCP = No. Blows to achieve 300mm rod penetration |
| Jianzhau et. al. | United States | 1999 | $E_{FWD}$ = 338 x DCP$^{-0.39}$ | $E_{FWD}$ = Modulus back-calculated from Falling Weight Deflectometer |
| George and Uddin | United States | 2000 | $M_R$ (MPa) = 532.1 x DCP$_I^{-0.492}$ | Fine Grained soil<br>$M_R$ = Resilient Modulus |
| | | | $M_R$ (MPa) = 235.3 x DCP$_I^{-0.48}$ | Coarse grained soil<br>$M_R$ = Resilient Modulus |
| Abu-Farsakh et al. | United States | 2004 | Ln ($E_{FWD}$) = 2.35 + (5.21 / Ln (DCP$_I$)] | $E_{FWD}$ = Modulus back-calculated from Falling Weight Deflectometer |

| Author / Study | Study Location | Year of Publication | Relationship* | Comments / Parameters |
|---|---|---|---|---|
| Chen et. al. | United States | 1999 | $M_R = 78.05 \times DCP^{-0.67}$ | $M_R$ = Resilient Modulus |
| | | 2005 | $E_S = 537.8 \times DCP^{-0.66}$ | $E_S$ = Insitu Young's modulus |
| | | | $E_{FWD} = 338 \times DCP^{-0.39}$ | $E_{FWD}$ = Modulus back-calculated from Falling Weight Deflectometer For 10 mm / blow < $DCP_I$ < 60mm / blow |
| Herath et. al. | United States | 2005 | $M_R$ (MPa) = $16.28 + (928.24 / DCP_I)$ | $M_R$ = Resilient Modulus |
| Mohammed et. al. | United States | 2007 | $M_R$ (MPa) = $151.8 \times DCP^{-1.096}$ | $M_R$ = Resilient Modulus |
| Siekmeier et. al. | United States | 2009 | $E_S$ (MN/m$^2$) = $10^{[3.05-(1.06 \times Log\ (DCP)]}$ | $E_S$ = Insitu soil modulus |
| Von Quintus et. al. | United States | 2009 | $M_R$ (MPa) = $17.6 \times [292 / DCP_I^{1.12}]^{0.64}$ | $M_R$ = Resilient Modulus |

*Note the equipment used and defined measurement / units of rod penetration ($DCP_I$) may vary between studies

It is noted that Table 4-3 only presents direct DCP to stiffness relationships. However, a number of other studies have completed multiple regression analyses and identified that density and moisture content parameters should also be incorporated to further improve the strength of the defined relationships (e.g. George & Uddin, 2000; Rahim & George, 2002; Salgado & Yoon, 2003; Herath et. al., 2005; and Mohammed et. al., 2009).

### DCP Penetration Rate correlated to Relative Density

A number of studies have evaluated the potential for the DCP to be used in place of traditional density QA testing (i.e. DCP blow count threshold is applied instead of an RDD threshold). In general, although considerable scatter is inherently contained within DCP data, the rod penetration rate (per hammer blow) decreases as the dry density increased. Although lower strength than the density influence, increased moisture content also increases the rod penetration rate.

The results of studies that related RDD and DCP penetration rate has been mixed, with many studies identifying that the DCP test results are inherently too variable to practically apply a suitable correlation with RDD (Burnham, 1997; Farrag et. al., 2005). Siekmeier et al. (2000) came to a similar conclusion and identified that the variable response of the DCP based on soil types / composition prevented a good correlation between DCP results and RDD being identifiable.

Edil and Benson (2005) demonstrated that the DCP had potential to be utilised as a compaction QA tool of subgrades if the DCP result was normalised with respect to the moisture content at the time of compaction (assessed parameter was $DCP_I$ averaged over 150mm depth increment divided by moisture ratio). The normalised parameter approached zero and could identify suitably compacted samples, but would not provide a suitable parameter to quantify the RDD actually achieved. Davich et. al. (2006) suggested that the moisture content influence on the DCP test results could be controlled by capping moisture contents at 10%, and identifying suitable thresholds for three (3) different moisture content ranges; (i) less than 5%; (ii) 5 – 7.5%; and (iii) 7.5 – 10%.

Notwithstanding the fact that a DCP test completed in isolation does not provide a quantification of the moisture condition of the tested material – and that variation in moisture may affect the DCP rod penetration rate (for specific materials) – various studies have successfully identified thresholds for DCP application within uniform (likely processed), granular materials used for subgrade / pavement base layers. For such correlations, as summarised in Table 4-4, a limiting penetration rate is applied to represent adequate compaction (although the exact RDD that these rod penetration thresholds represent are unknown).

**Table 4-4 Historical relationships between DCP penetration rate and relative density**

| Author / Study | Study Location | Year of Publication | Limiting Penetration Rate (per blow) that represents "adequate" compaction achieved | Applicability |
|---|---|---|---|---|
| Burnham | Minnesota, United States | 1977 | $DCP_I$ < 19mm per blow (throughout layer) | All freshly compacted base materials |
| | | | Silty Clay: $DCP_I$ < 25mm per blow | Describes suitable compaction of subgrade materials for existing pavements |
| | | | Granular Materials: $DCP_I$ < 7mm per blow | |
| Siekmeier et. al. | Minnesota, United States | 1998 | $DCP_I$ < 15mm per blow for initial 75mm depth | Tests undertaken within 1 day of compaction |
| | | | $DCP_I$ < 10mm per blow for 75mm – 150mm depth | Upper 40mm may be disregarded (seating) |
| | | | $DCP_I$ < 5mm per blow for depths in excess of 150mm | |
| White & Bergeson | Iowa, United States | 1999 | RDD (%) = -0.4 x $DCP_I$ (mm/blow) +92.8 | Granular Materials |
| Abu-Farsakh et al. | Louisiana, United States | 2004 | $DCP_I$ < 5.5mm per blow | Crushed Limestone |
| Wu & Sargand | Ohio, United States | 2007 | $DCP_I$ < 8mm per blow | Base materials |
| | | | $DCP_I$ < 7mm per blow | HMA covered bases |
| Siddiki et. al. | Indiana, United States | 2008 | DCPI < 18.75mm per blow | Coal Ash Fill |

Larsen et. al. (2007) implemented a QA program in Iowa, USA, that related average DCP penetration rate to RDD and also included a maximum allowable variation in DCP results based on the uniformity of the test results in order to ensure the full lift thickness was compacted, rather than having a stiff upper portion and weaker base. The QA thresholds utilised are presented in Table 4-5.

**Table 4-5 Compaction QA via DCP Thresholds – utilised in Iowa, USA (after Larson et. al., 2007)**

| Author / Study | Year of Publication | Material Type | Equivalent RDD (%) | Average $DCP_I$ value over full lift thickness (mm / blow) | Maximum Average Variation in $DCP_I$ index (mm) |
|---|---|---|---|---|---|
| Larson et. al. | 2007 | Cohesive – Select Fill | $\geq$ 100% | $\leq$ 65mm / Blow | $\leq$ 35mm |
| | | Cohesive – General Fill | $\geq$ 95% | $\leq$ 70mm / Blow | $\leq$ 40mm |
| | | Cohesive - Unsuitable | < 95% | > 70mm / Blow | > 40mm |
| | | Cohesion-less – Select Fill | $\geq$ 100% | $\leq$ 35mm / Blow | $\leq$ 35mm |
| | | Cohesion-less / Mixed – General Fill | $\geq$ 95% | $\leq$ 45mm / Blow | $\leq$ 45mm |

### Assessment of DCP for use as QA Tool

From the current available research, it can be observed that the DCP test results are affected by all the factors summarised in Table 4-6. Due to the site specific nature of the combined effect of these factors, any DCP to stiffness parameter relationship is likely to be both site- and material- specific.

**Table 4-6 Summary of influences on DCP test results**

| Factor | Effect on DCP Test Result |
|---|---|
| Moisture Condition | Rate of Rod Penetration increase with increased moisture content |
| Dry Density | Rate of Rod Penetration decreases as dry density increases |
| Gradation / Uniformity | Presence of oversize materials can cause premature rod 'refusal' or artificially decrease rod penetration rate without associated increase in material density or strength |
| Plasticity | Rate of Rod Penetration increases with increased plasticity |
| Confinement | Vertical conferment (e.g. asphaltic layers) may artificially decrease Rate of Rod Penetration, especially in granular layers |
| Side Friction | Friction applied to rods (e.g. in loose / collapsible granular materials) may cause an artificial decrease in Rate of Rod Penetration. This can be exacerbated if DCP is not completely vertical while test is being undertaken |

Although a number of the items detailed in Table 4-6 can be controlled during the filling and compaction processes, if the compacted material undergoing assessment contains a significant portion of ‘oversize’ particles (> 37.5 mm) or is not suitably uniform it would be expected that the resulting variation (CoV) returned by the DCP test will preclude suitable DCP → *E* relationships being derived. The DCP test also appears to be not sensitive enough to appropriately identify a material change across likely compaction thresholds (e.g. based on CoV associated with DCP, there would be little difference in the $DCP_I$ parameter returned from materials compacted at slightly lower than 95% and above 95% RDD).

However, the DCP is identified as being a valuable profiling tool. Rather than reliance on the DCP to provide a reliable correlation with other strength or deformation parameters, it should be utilised to identify the presence of the comparative change in strength with depth (i.e. identify location of comparatively weaker layers within compacted fill), the thickness of a specific layers within the subsurface, or identify locations where significant strength changes are observed (e.g. alluvium overlying rock materials).

### 4.2.2 PANDA Probe

#### Test Equipment and Methodology

The PANDA Probe – or Variable Energy Dynamic Cone Penetrometer – is a test technique in which a cone is manually driven (by repeated hammer blows upon an anvil) through a soil material. For each hammer blow applied the PANDA measures the variable driving energy and the depth of the cone. Thus, using the applied energy and change in cone depth, the soil resistance – reported as ‘cone tip resistance, or $q_d$ – may be calculated.

Figure 4-2 presents the elements of the PANDA Probe test equipment, whilst Table 4-7 details the key dimensions and weights of the PANDA system.

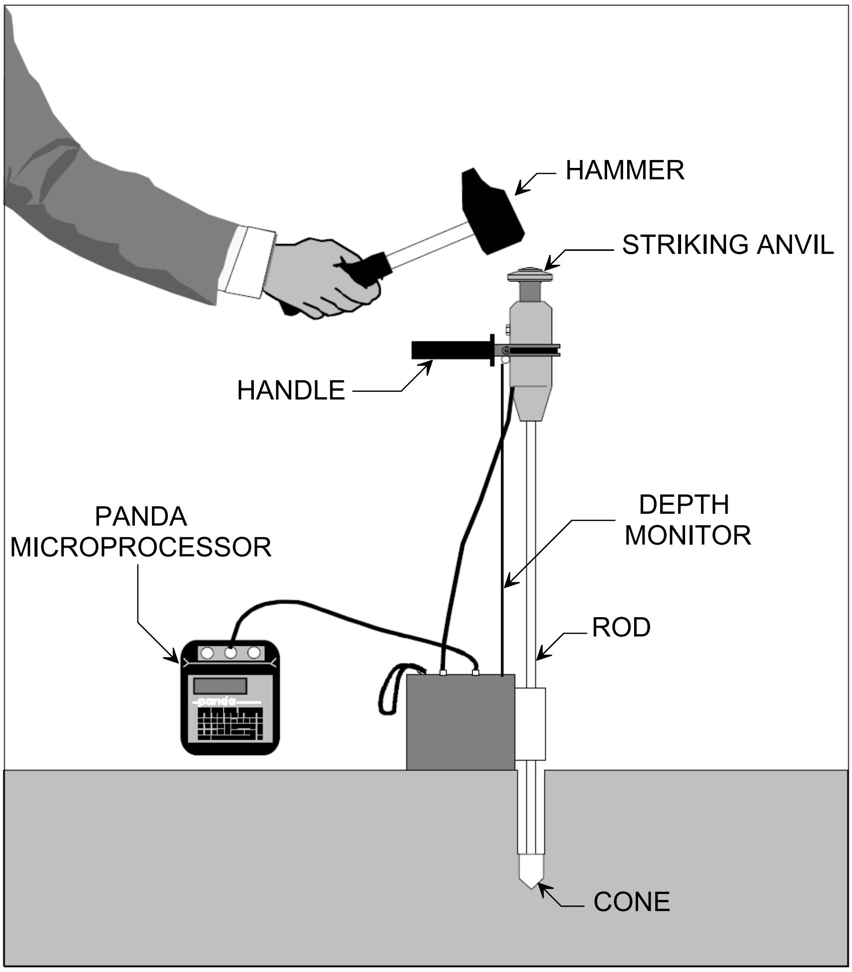



**Figure 4-2 Key components and general equipment setup of PANDA Probe Penetration test**

**Table 4-7 Dimensions and weight of key equipment within PANDA Probe apparatus**

| Element | | PANDA Probe |
|---|---|---|
| Hammer / Weight Drop | Mass | 2 kg |
| | Standard Drop | Variable (each blow measured independently) |
| Cone | Angle | $86^0$ (from centreline of cone) |
| | Diameter | 16 mm |
| | Area | 2 $cm^2$ |

### Applicable Standards

French Standard, 2012 – *NF P 94-105, Soils: Recognition and testing – Control of the quality of compaction –Dynamic penetrometer with variable energy method - Penetrometer calibration, principles and methodology – Interpretation of results* (in French)

No existing Australian Standard, ASTM or Australian Regulatory authority test method currently exists for use of the PANDA Probe. However, regardless of the absence of an approved test method the PANDA probe is already in use in Australia, and has been for a number of years (e.g. by Queensland Department of Main Roads).

### Parameters provided from test

The 'cone tip resistance' ($q_d$) is determined for the length of penetration for each blow via Equation 4-4.

$$q_d = (1 / A) \times [(0.5 \times M \times v^2) / (1 + P / M)] \times (1 / \Delta_{depth}) \qquad \text{(Equation 4-4)}$$

Where, for each blow of the hammer (striking event):

- A = Area of the Cone
- M = Weight of the hammer used for strike (Striking mass)
- P = Weight of the system being driven through material (Struck mass)
- V = Speed of the striking mass
- $\Delta_{depth}$ = penetration due to the striking event

The 'cone tip resistance' ($q_d$) parameter is then used to infer relative density / consistency categories of the subsurface profile or to derive material parameter profiles (e.g. shear strength, *insitu* California Bearing Ratio (CBR) or modulus values).

### Repeatability of Test Technique

Within a uniform material, compacted at a uniform density, the French Standard (NF P 94-105) suggests that characteristic $q_d$ parameter obtained for similar depths within separate tests should be within 10% of each other. This infers the combined equipment and operator error should be in the order of 10% (i.e. +/- 5% about an averaged characteristic $q_d$ value). Furthermore, the same document defines that 'background noise' associated within a PANDA Probe profile (i.e. variability of $q_d$) whilst remaining within any single "material unit" could be up to 20% (i.e. +/-10% about the average $q_d$ of a single layer).

During a trial compacted fill embankment project completed in Queensland, Australia, two (2) side-by-side PANDA Probe tests were completed at a number of locations to assess the repeatability of the test technique and assess the natural variation present in the compacted fill material. The combined PANDA Probe equipment, operator and inherent material variation reported an $80^{th}$ percentile – the threshold at which $80^{th}$ of the dataset was within – of approximately +/- 15% about the average $q_d$ value for each 50mm depth increment. The average variation within corresponding side-by-side $q_d$ parameter pairs was, for individual trial embankments, between +/- 9.0% and +/-17.9%. It was noted the highest of these value (+/-17.9%) was recorded at a site where boulders were recorded to be present within the fill, and the higher variation in PANDA test results associated with this site was likely due to the higher level of inherent material variation rather than the test equipment / technique being applied. For comparative purposes, at all locations the DCP reported 80th percentile repeatability thresholds approximately twice as high as the PANDA Probe (i.e. DCP returned significantly higher – double – variation in side-by-side test results than PANDA Probe).

**Advantages**

- Quick and economical
- Simple, self-contained equipment that requires no calibration
- Requires minimal training
- Hammer imparted energy is measured with each strike, providing a known impact force to which the tip penetration distance (per blow) can be coupled
- Comparative tip movement per blow is consistently monitored, and reported for each hammer blow.
- Can be used to continuously profile comparative material state throughout thickness of layer (as long as refusal does not occur), and confirm thickness of individual layers (as long as comparative density is sufficient to make a distinction of interface depth).
- Has been demonstrated to have high strength correlations with RDD, strength and stiffness properties – e.g. CBR, shear strength and elastic modulus (*E*) – when applied to uniform materials

**Disadvantages / Limitations**

- Not widely used with standard specifications not available for test methodology and interpretation
- Index test that does not directly measure a material parameter
- Site specific correlations with material parameters are required for best results.
- Penetration test may be prone to refusal within gravel / granular materials (i.e. not suitable to assess compaction of granular unbound pavements)

**Existing Literature regarding use as compaction QA test**

Only limited existing published literature was identified to assess the use of the PANDA Probe for QA testing. However, the PANDA Probe is marketed as being suitable for use as a highly effective QA test for compaction to depths of up to 1.5m depth (Solsolution, 1999).

The acceptance criteria for adequate compaction based on PANDA Probe soundings is generally based on correlated laboratory test results for similar material types. The PANDA Probe interpretation software has 18 natural soil types, each with cone tip resistance ($q_d$) parameters equivalent to 95% (for standard proctor compaction effort), 97% (for modified compaction efforts) and 98.5% (for standard proctor compaction effort) and for two (2) different moisture contents (wet or dry of optimum). These results are based on comparative studies undertaken by a French university (Zhou, 1997). For each material, known ranges of $q_d$ values are known to identify inadequate compaction ('region of failure') and adequate compaction ('region of acceptance'). A nominated margin of error – termed the 'area of tolerance' –exists between the compaction threshold ranges that represent the regions of 'failure' and 'acceptance'. The 'area of tolerance' is 2 – 3% RDD. The comparative location of each defined regions are illustrated in Figure 4-3 (after Langton, 1999). The identified regions are constructed for material- and/or site-specific correlations via the comparison of the results of the $q_d$ parameter with the results of *insitu* density tests (sand replacement or nuclear density gauge tests).

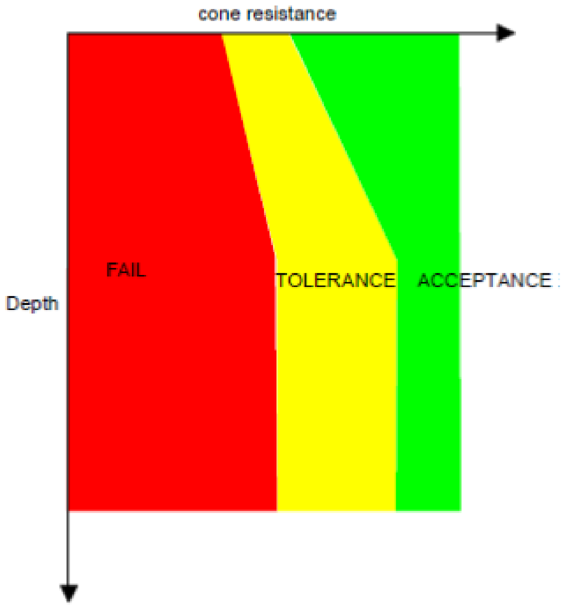



**Figure 4-3 Typical interpretation curve used for interpretation of PANDA results for monitoring the compaction of fill materials (after Langton, 1999)**

Juran et al. (1999) completed a comparative study in which the PANDA Probe's performance as a soil compaction control device was evaluated (alongside the DCP and soil compaction meter). The results of this study identified that although pre-calibration with site materials was required, the PANDA Probe produced a "highly repeatable" test result and had a "low" dependency on operator input or manual recording. Although the PANDA Probe had the advantage in that it could test a full depth profile, it was observed to be sufficiently sensitive to quantify the effect of compaction processes applied to the near-surface. The identified drawback associated with the use of the PANDA Probe was that the operator was required to have a higher level of training for use when compared to DCP. Overall, Juran et. al. (1999) identified that the PANDA Probe appeared to provide the most effective and user-friendly tool for the field assessment of achieved compaction level throughout the full depth of a compacted backfill material.

Langton (1999) reviewed the PANDA Probe in terms of applicability to measuring achieved compaction at the six (6) of the UK's Building Research Establishment's (BRE's) test bed sites (as detailed by Butcher et. al., 1995). His study identified the potential for PANDA Probe testing to be used for 'oversized' layers (deep lift) of materials undergoing compaction, as the PANDA Probe did not have the same limitations in terms of effective test depth as traditional density testing. Langton (1999) concluded that based on the BRE test bed testing, the PANDA Probe provided a good approximation to static cone tip resistance, as measured by CPTs, in stiff clays (i.e. $q_d$ [PANDA] = $q_t$ [CPT]). In soft clays, the relationship between the two (2) cone tip resistance parameters was proposed to be altered to $q_d = (4.17 \times q_t) - 0.58$.

For cohesive materials, various researchers (e.g. Langton, 1999; Butcher et. al, 1995) also proposes relationships between shear strength and the $q_d$ parameter, as defined in Equation 4-5.

$$C_u = \alpha \times q_d \qquad \text{(Equation 4-5)}$$

Where $\alpha$ varies from 0.15 to 0.22

Langton (1999) provided a summary of typical cone tip resistance ($q_d$) values for various materials based on his assessment of six (6) BRE test bed sites. These parameters are reproduced in Table 4-8.

**Table 4-8 Summary of Typical Cone Tip Resistance ($q_d$) Values – From Langton (1999)**

| Material Type / Relative Density | Typical PANDA Cone Tip Resistance ($q_d$) |
|---|---|
| CLAY – Very Soft | 0 – 1 MPa |
| CLAY – Soft to Firm | 1 – 2 MPa |
| CLAY – Firm to Stiff | 2 – 3 MPa |
| SAND and GRAVEL | 4 – 30 MPa |

The Transport Research Board (TRL) also related the PANDA Probe's $q_d$ parameter to CBR (%) for granular sub-base materials. The CBR was estimated from DCP testing, and the penetration rate (in mm/hammer blow) was reported to be equal to approximately (100 / $q_d$). From the utilised DCP to CBR relationship, a $q_d$ to CBR relationship was proposed (Equation 4-6).

$$\text{Log}_{10}(\text{CBR}) = 0.352 + (1.057 \times \text{Log}_{10} q_d) \qquad \text{(Equation 4-6)}$$

Based on this research, Langton (1999) determined that the PANDA Probe was "reliable" and well correlated to both the results of other penetration tests (CPT, DCP, SPT) and to monitor the compaction of soils, earthworks fill and pavement materials.

Fourie et. al. (2013) reported on an assessment of the PANDA Probe as a compaction QA tool for Tailings Storage Facilities (TSF) in Chile, and cautioned that the $q_d$ parameter was not uniquely related to the material's dry density. The authors emphasised the need to complete site-specific correlations between PANDA tip resistance and relative density. It was identified that the sensitivity of the results of this test technique to *insitu* moisture content also required further assessment and that the high moisture contents observed within the TSFs studied were likely to be influencing the magnitude of the resulting $q_d$ parameter. At relatively lower water contents (6 to 10%), the relationship between $q_d$ and dry density has been found to be acceptable. Regardless, due to the PANDA's portability, its ease of use and number of tests that can be completed in a single day, Fourie et. al. (2013) recommends the use of the technique as a comparative profiling and monitoring tool.

Espinance et. al. (2013) summarised the use of PANDA Probe to characterise material stored in Chilean TSFs to profile variation in both spatial and temporal dimensions. This summary paper details the fundamental theory and calibration procedure for developing relationships between dry density and the PANDA Probe's cone tip resistance ($q_d$) parameter. The authors also developed relationships between the $q_d$ parameter and the internal angle of friction and compaction ratio of TSF deposited materials (noting that such materials are less compacted than materials associated with embankments and pavements).

Within Australia, the PANDA has been extensively utilised by the Queensland Department of Main Roads. In a series of recent (2016) trial embankments at the TSRC project the PANDA was evaluated in its ability to provide a cone tip resistance parameter ($q_d$) that could be correlated to field density test measurements (nuclear gauge or sand replacement tests). The $q_d$ parameter returned from the PANDA Probe testing was evaluated to be successful in demonstrating both (i) the difference in *insitu* density between uncompacted and compacted materials; and (ii) locations where the *insitu* compaction threshold (RDD $\geq$ 95%) had (or had not) been achieved. Table 4-9 details the compaction thresholds equated to approximate RDD = 95% for the various materials examined during this trial embankment project.

**Table 4-9 Example of comparison between PANDA Probe cone tip ($q_d$) and *insitu* density ratio (RDD) parameters for site by site testing.**

| Fill Material Origin | Weathering State / Characteristic Strength of Fill | Equivalent PANDA Probe $q_d$ for RDD = 95% |
|---|---|---|
| Sandstone* | Highly to Slightly Weathered / Medium Strength | $q_d$ = 10 MPa* |
| Interbedded Siltstone / Sandstone | Highly to Slightly Weathered / Low Strength | $q_d$ = 11 MPa |
| Basalt | Extremely to Moderately Weathered / Medium Strength | $q_d$ = 13 MPa |

*Note presence of boulders within compacted embankment required further interpretation and filtering of $q_d$ parameter, resulting in removal of higher $q_d$ test results and the reporting of an overall lower $q_d$ parameter than for other trial embankments (i.e. interpretation methodology varied when compared to other trial embankments).

### Assessment of PANDA Probe for use as QA Tool

The PANDA Probe is, due to the nature of it being a rod based penetration test, susceptible to the same issues when adopted as a QA tool as identified for the DCP test, and as previously summarised in Table 4-6.

However, in comparison to the DCP, the PANDA Probe has been found to have significantly greater repeatability and sensitivity to achieved *insitu* density than the DCP. This is due to the PANDA Probe test method including a depth per hammer blow measurement, and the tool's ability to measure the variable stress associated with each hammer blow and account for this when determining the reported cone tip ($q_d$) resistance parameter. Due to these advantages, the PANDA Probe is considered to have greater potential for use as a QA technique than the DCP.

The PANDA Probe is identified as being a valuable profiling tool. Rather than reliance on the DCP to provide a reliable correlation with other strength or deformation parameters, it should be utilised to identify the presence of the comparative change in strength with depth (i.e. identify location of comparatively weaker layers within compacted fill), the thickness of a specific layers within the subsurface, or identify locations where significant strength changes are observed (e.g. alluvium overlying rock materials).

### 4.2.3 Iowa Borehole Shear Tester (BHST)

#### Test Equipment and Methodology

The Iowa Borehole Shear Tester (BHST), developed by Professor Handy at Iowa State University in the 1960's (Handy and Fox, 1967), provides a convenient method to accurately measure the *insitu* drained shear strength of soils (Figure 4-4). It differs from the other QA penetration test techniques considered by this document in that it is not a simple rod penetration test. Instead, the BHST is similar to a laboratory direct shear test in that it involves the application of a normal stress to, and shearing of, the sides of a small diameter borehole that is excavated through the compaction layer to be assessed. The BHST determines the *insitu* drained friction angle and cohesion of the tested material, which can be any soil type (cohesive, mixed or granular).

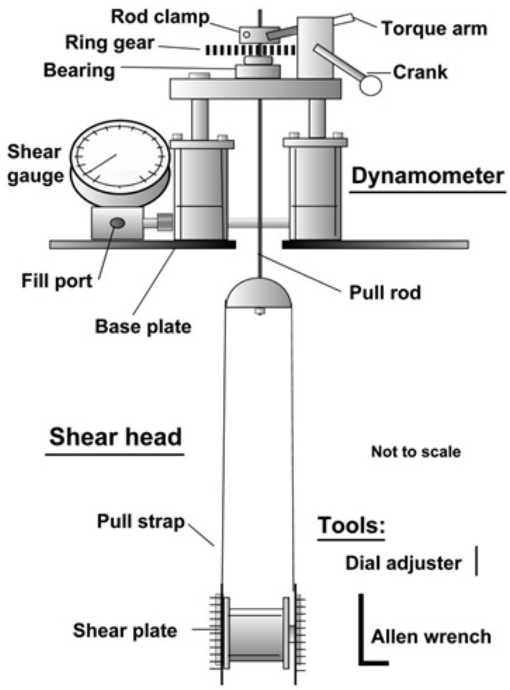



**Figure 4-4 Borehole Shear Test**

To perform the BHST, the operator inserts the shear head into a 75mm diameter borehole to the chosen test depth. The BHST operation is then undertaken in two phases:

1. *Seating / consolidation phase* – A normal pressure is applied to the side wall of a borehole and then left for a duration of time (typically 5 to 15 minutes) to allow surrounding material to consolidate and for any excess pore water pressures to dissipate prior to commencement of the shear phase; and
2. *Shearing phase* – A shear (uplift) force is applied by rotating the crank attached to the base plate at a constant rate (typically 2 revolutions per second). The shear force is transferred along the pull rods to a dynamometer fitted with a hydraulic shear gauge. Shear strength measurements are taken until the soil material is observed to reach its peak shear strength. The peak shear strength is recorded and plotted against its corresponding normal stress.

Both testing phases are repeated for increased normal stresses. The test is typically repeated four to five times with the individual data points used to plot measurements of shear stress against normal stress. Figure 4-5 shows a typical BHST data plot.

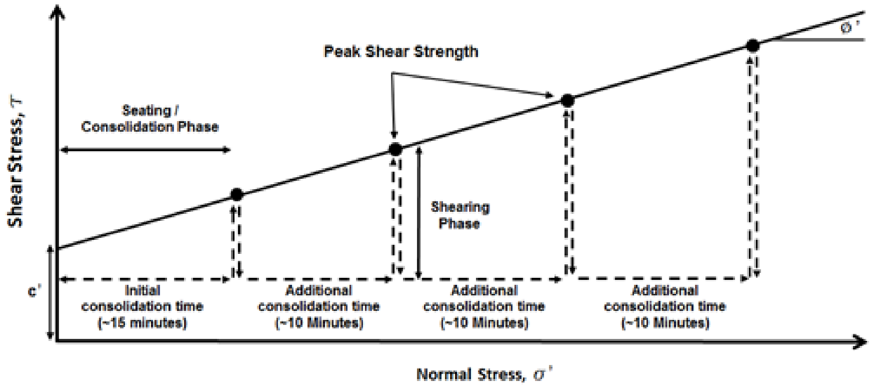



**Figure 4-5 Typical borehole shear testing procedure for clayey silts (Bechtum, 2012)**

With sufficient time allowed for consolidation and drainage, the slope of the line passing through each of the 'peak' shear strength points should be linear and conveniently plot a Mohr Coulomb failure plane. A line of best fit is applied to the data points with the characteristic strength parameters determined from the fitted lines slope and y-axis intercept. Consolidation times vary between material types, with granular soils typically requiring a shorter consolidation durations.

### Applicable Standards

No existing Australian Standard, International Standard or Australian Regulatory authority test method has been identified for use of the BHST.

However, Lutenegger (1987) submitted a suggested test method for performing the Borehole Shear Test to the American Society for Testing and Materials (now ASTM International) Subcommittee on Sampling and Related Field Testing for Soil Investigation (Subcommittee D18.02). This method was published in the Geotechnical Testing Journal (Vol 10 (1), 1987).

### Parameters provided from test

The results of BHST provide *insitu*, drained Mohr Coulomb shear strength parameters – effective cohesion (c') and angle of friction ($\phi'$).

Both peak and residual *insitu* Mohr Coulomb strength parameters can be determined by Borehole Shear Testing.

### Repeatability of Test Technique

Lutenegger and Timian (1987) completed a series of BHST within marine clay and undertook a statistical assessment of the results to assess the repeatability of the test equipment and the effect (if any) that the experience of the equipment operator had upon the test results. The results of these tests indicated that the angle of friction parameter determined by the BHST had a significantly higher repeatability (COV = 7.5 – 11.0%) than the cohesion parameter (COV = 33.7 – 42.1%). The results obtained by each operator were statistically similar, which was interpreted to identify the simplicity of the test. All test results produced a linear relationship with a regression co-efficient ($R^2$) of 0.95 or greater.

Recent studies completed at the TSRC project (2016) in Queensland, Australia similarly evaluated the repeatability of the BHST. Repeated testing ($n$ = 94) was undertaken upon three (3) compacted, granular fill materials of varying origin, and found a high repeatability associated with the effective friction parameter (COV < 10%). Consistent with the findings of Lutenegger and Timian (1987), the drained cohesion parameter had a significantly higher variability (90% < COV < 155%). A summary of the parameters and repeatability of the BHST of the 2016 Australian study is presented in Table 4-10.

**Table 4-10 Repeatability of BHST completed in Queensland, Australia - TSRC trial embankments (2016)**

| | | FILL A – compacted SANDSTONE | Fill B – Compacted Interbedded SILTSTONE / SANDSTONE | Fill C – Compacted BASALT |
|---|---|---|---|---|
| No. of Tests | | 35 | 36 | 23 |
| Effective Friction ($\phi$) | Median (Range) | $43^{\circ}$ ($37^{0}$ – $54^{0}$) | $40^{\circ}$ ($30^{0}$ – $46^{0}$) | $42^{\circ}$ ($30^{0}$ – $45^{0}$) |
| | Coefficient of Variation (COV) | 9.3% | 7.9% | 9.0% |
| Effective Cohesion (c’) | Median (Range) | 1 kPa (0 – 15 kPa) | 1 kPa (0 – 16 kPa) | 5 kPa (0 – 15 kPa) |
| | Coefficient of Variation (COV) | 130% | 154% | 91% |

### Advantages

- Direct measurement of *insitu* parameters, without need to obtain undisturbed samples for laboratory testing or use of empirical correlations.
- Minimises soil disturbance – *insitu* test distorts sample less than laboratory extrusion or remoulding processes.
- Quick and economical (in comparison to traditional laboratory strength evaluation techniques) – a single test can be completed within an hour, allowing multiple tests to be completed within a single day of onsite work.
- Simple, self-contained and portable equipment that requires no calibration.
- Test equipment can potentially be utilised in any orientation, allowing strength measurement of ground anchors or evaluation of material anisotropy.
- Allows assessment of *insitu* parameters at a specific depth, allowing spatial profiling to be undertaken / specific layers of subsurface targeted.
- Can directly assess both peak and residual material parameters.

### Disadvantages / Limitations

- Standard specifications are not available for test methodology and interpretation.
- Requires an associated small diameter borehole to be available for insertion of test equipment into compacted materials.
- Soil pore water pressure condition and matric suction during testing may affect results and accurate determination of material condition can be difficult.
- Gravel (or larger) inclusions can, if present within the loaded zone of the shear plates, produce erroneous (high) shear strength measurement that will require filtering from final dataset.

### Existing Literature regarding use of BHST as QA / Validation test

A number of case studies exist within technical literature that demonstrate the successful and reliable use of the BHST method to evaluate *insitu* Mohr-Coulomb strength parameters – either at the site investigation or Quality Assurance (during construction) phases of a project. These include the direct assessment of strength parameters of cohesive (Lutenegger and Timian, 1987; Simon and Collison, 2002) and granular (e.g. Lohnes and Handy, 1968) materials. Other studies (e.g. Yang et. al., 2006; Handy et. al., 1985) have demonstrated the applicability of the same technique within high weathered (weak) rock materials.

As reported by numerous researchers (e.g. Fairhall, 2016; Bechtum, 2012; Miller et. al., 1998) the BHST derived *insitu* parameters realistically approximate the parameters derived from laboratory triaxial (CU) and shear box testing. Fairhall (2016) reported the friction angle parameter derived from BHST was within 10% of the corresponding laboratory testing. As the BHST results are effective stress parameters, it can also be used as a site-specific conversion technique to correlate undrained penetration test results into effective stress strength parameters (Handy et. al., 1985).

Assessment of either linear or bi-linear behaviour within the constructed graphic that shows the shear response to varied applied normal stress magnitudes can also be used to identify and quantify the normal or over-consolidated nature of tested materials, respectively (Handy, 2002).

Due to the nature of the BHST, in which a distinct depth / location undergoes assessment, Handy (1986) identifies the effectiveness of the technique to profile the strength parameter variation with depth. This is especially applicable to the evaluation of landslide sites, in which strength parameters variation is directly used within the probabilistic analysis of the slope stability of a site. The use of the BHST is noted to be effective due to its sensitivity, small footprint (such that a weakened / shear plane could be identified and material parameters derived) and speed of use. As identified by Yang et. al. (2006), the use of the BHST to characterise *insitu* parameters, and variability thereof, via use of the BHST and the evaluation of such parameters through probabilistic slope stability analyses, can present significant cost savings in comparison to the simple adoption of assumed parameters for design.

Similarly, for earthworks QA programs the BHST can be used to assess the variability of *insitu* strength parameters throughout an entire lift / embankment / zone of influence. As described previously (refer Table 4-10), the spatial variability (vertical and horizontal) of *insitu* strength parameters were effectively assessed via use of the BHST technique during the completion of large scale test embankments in South East Queensland, Australia (2016). To ensure operator variability and error is further minimised in such situations, the BHST can also be automated to a contact rate of shear, as independently demonstrated by Fairhall (2016) and Bechtum (2012).

Evaluation of the BHST measured *insitu* strength parameters within unsaturated soil conditions has been undertaken by a number of researchers (e.g. Miller and Khoury 2012; Ashlock and Lu, 2012; Lu and Likos, 2004). Typical results indicate that although the reported *insitu* parameters are influenced by the matric suction or moisture condition at the time of testing – in which the derived friction angle increases and cohesion parameter decreases as the suction present during a test increases – this effect can be overcome by the adoption of a larger applied normal stress range. This recommendation mirrors that of Handy et. al. (1985) whom identified that the use of higher stress ranges for assessment of stiff soils overcomes any ‘progressive seating’ issues that may occur during the test.

The shear plates associated with the BHST apparatus can be further modified to allow appropriate contact with the medium being tested. This can be in the form of plates suitable for *insitu* assessment of stiff soils or rock materials, or to assess the interface friction angle achieved between pile and soil (e.g. Audibert and Aggarwal, 1982; AbdelSalam et. al., 2012). Lutenegger and Miller (1994) successfully demonstrated the BHST could be used to assess the uplift capacity of drilled pile shafts.

**Assessment of BHST for use as QA Tool**

The BHST appears to be the only field instrument that directly evaluates the Mohr-Coulomb strength parameters in the *insitu* environment. Accordingly, the time, cost and sample disturbance associated with both obtaining and completing laboratory tests upon representative “undisturbed” samples can potentially be removed if the BHST is utilised onsite. Case studies of the BHST have demonstrated its ability to provide realistic cohesion and friction angle parameters when compared to the accepted results of laboratory testing.

Accordingly, for projects where field validation of the design parameters are required – e.g. high embankments where slope stability issues may occur if design Mohr-Coulomb strength parameters are not achieved – the BHST appears to offer the potential of a very valuable QA technique. The ability of the test to complete tests at specific depths and fast turnaround of results once the material is tested offers the ability for *insitu* strength parameters to be quantified.

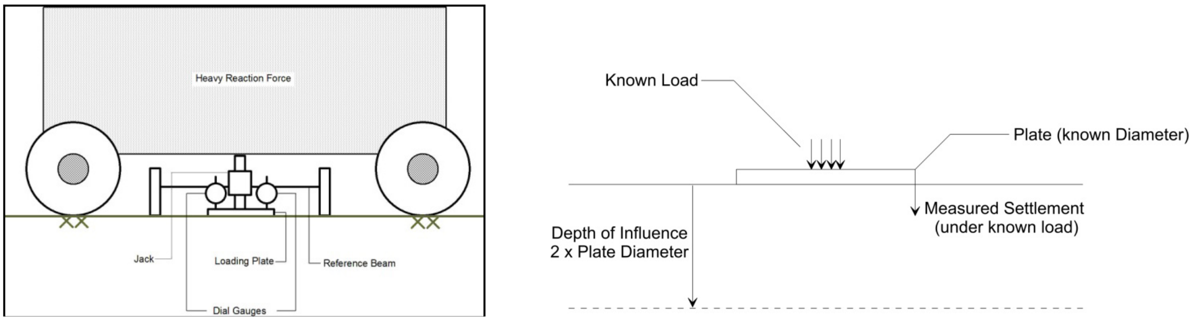
## 4.3 Surface based Loading QA Techniques

### 4.3.1 Static Plate Load Tests (PLTs)

#### Test Equipment and Methodology

The static PLT involves the compression loading of a material via the use of a steel plate, hydraulic jack and reaction load (Figure 4-6). The deflection that arises from the loading of the bearing plate in contact with the surface is monitored by a number – typically three (3) – of dial gauges located around the plate but mounted upon an independent reference beam, the readings of which are combined to produce an average plate settlement for each load increment and an overall load-deformation curve.

The load applied to the bearing plate is generally increased and unloaded over a number of cycles, such that the ground response to initial (i) and reloading (*R*) stresses can be identified. The load and deformation readings are continuously recorded across the loading cycles such that a full stress / deformation curve can be constructed.




(a) (b)

**Figure 4-6 Plate Load Test equipment setup. (a) Equipment and required reaction load; and (b) concepts of PLT, showing load / settlement variables typically recorded during test (after Lacey, 2016).**

Figure 4-7 shows a typical trace of stress / deformation obtained from a PLT test, and provides a conceptual definition of the PLT derived insitu modulus values calculated from the stress / deformation response observed during either the initial ($E_{PLT(i)}$) or reloading cycles ($E_{PLT(R2)}$) of a PLT test.

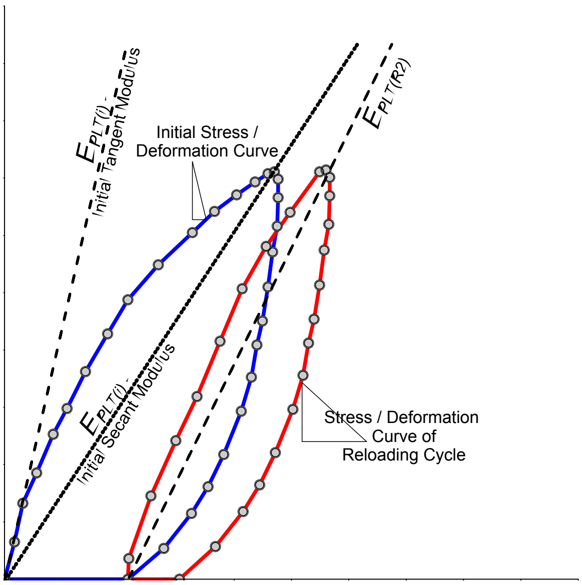



**Figure 4-7 Typical stress / deformation curve from Plate Load Test, annotated with definition of initial (i) and reloading (R2) insitu moduli ($E_{PLT}$) (from Lacey, 2016)**

The 'zone of influence' (i.e. the depth to which the test measures) is typically accepted as twice the rigid plate diameter (2D). The composite elastic modulus ($E_{PLT}$) parameter over the PLT's zone of influence is defined by Boussinesq (elastic) theory and can be derived by use of Equation 4-7.

$$E_{PLT} = K_s \times D \times (1 - \nu^2) \qquad \text{(Equation 4-7)}$$

Where:

$K_s$ = Subgrade Modulus, derived from gradient of stress / settlement curve from loading cycles

$\nu$ = Poisson's Ratio

D = Diameter of Rigid Plate

The German Standard (DIN 18134) applicable to static PLTs provides details regarding the calculation of the strain modulus, $E_v$, as per Equation 4.8.

$$E_v = [(1.5 \times r) / (a_1 + a_2 \times \sigma_{0max})] \qquad \text{(Equation 4-8)}$$

Where:

$E_v$ = Strain modulus (composite of material within zone of influence) for a single load cycle

r = Radius of plate

$a_1$, $a_2$ = the constant of the second degree polynomial fitted to the stress / deformation data for the load cycle being considered

$\sigma_{0max}$ = maximum average normal stress applied to loading plate

**Applicable Standards**

- DIN 18134 – *Soil – Testing Procedures and testing equipment – Plate Load Test, English translation of DIN 18134:2012-04*
- ASTM D1195 – *Standard Test Method for Repetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements.*
- ASTM D1196 – *Standard Test Method for Non-repetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements.*

**Parameters provided from test**

Static PLT testing allows the construction of a full loading stress / deformation curve for the range of stress magnitudes applied. From this the following parameters can be derived (depending on the number of loading cycles applied, and if the loading exceeds the elastic phase loading of the material undergoing testing):

- $E_i$ or $E_{v1}$ = Modulus associated with initial loading cycle
- $E_{(R2,3,4)}$ or $E_{v2,3,4}$ = Modulus associated with reloading cycles
- $K_s$ = Subgrade Modulus / Modulus of Subgrade Reaction
- $q_{ult}$ = Ultimate bearing capacity
- $q_{allow}$ or $q_a$= Allowable bearing capacity

**Repeatability of Test Technique**

As the static plate load test is a direct measurement procedure – whereby the deformation is measured from monitoring of the rigid plate, and the applied load is measured by a load cell – the repeatability of the test technique is largely based on (i) the accuracy of the monitoring equipment used in the PLT system and (ii) the variability associated with the material being tested.

However, as noted by Adam et al. (2009) there are a number of assumptions made during the interpretation of PLT results, including (i) that the material within the zone of influence behaves in a purely linear elastic manner (which is idealised); (ii) the material is not being deformed plastically in the immediate vicinity of the plate; (iii) the test itself does not compact the subsurface being tested, artificially stiffening the material from its pre-test condition; (iv) during unloading cycles the load is not being locally re-distributed.

**Advantages**

- Direct measurement of site- and material-specific deformation response to applied stress / load
- Fully adjustable load magnitude (limited by reaction load) to represent design loading scenario
- Well defined load steps and measured response to each load step are recorded
- Inclination of load plate is evident in test results
- Considered the reference method for field measurement of deformation (modulus) parameter

**Disadvantages / Limitations**

- Comparatively slow test – each test takes in excess of one (1) hour. Exact time taken depends on time required to obtain stabilised readings at each load step, which is in turn determined by time taken to dissipate pore pressures within loaded zone
- Comparatively expensive – due to equipment required and time taken to complete
- Requires a large reaction force (external) to be provided
- Equipment setup requires interaction with external reaction force (plant)
- Cannot be completed in narrow trenches / test pits due to requirement of hydraulic jack connection to reaction force

**Existing Literature regarding use as compaction QA test**

A number of references include details regarding the use of PLTs as standard measures of compaction, and PLTs are frequently used for site-specific studies use PLTs as the reference parameter to which various test methods are correlated to (i.e. PLT results are seen as the “correct” *insitu* modulus parameter).

References that include the PLT as a standard compaction measure include the national road standards of Germany (ZTVA-StB., 1997), Sweden (VVR Väg., 2009), Austria (ISSMGE, 2005) and the UK (IAN73/06, 2009). Such standards generally adopt the modulus measured during the reloading cycle of the PLT ($E_{v2}$) and include a reference to the ratio of modulus parameters determined during both the initial and reloading cycle of the PLT ($E_{v2}$ / $E_{v1}$). The $E_{v2}$ parameter is used in terms of an absolute threshold which must be observed – similar to how RDD measures are typically adopted in Australian earthworks projects – whilst the $E_{v2}$ / $E_{v1}$ ratio is considered a quasi-measure of the compaction level achieved or remaining potential for material settlement to occur.

Table 4-11 through Table 4-13 provides examples of the PLT-derived, *insitu* modulus thresholds used within national specification documents – which can be related to either material type and compaction (density) level (Table 4-11); or the depth from pavement / formation / trafficable surfaces (Table 4-12 and Table 4-13). Note that the standard plate size associated with PLTs in such standards may be either 762mm or 300 / 305mm diameter, and the *insitu* PLT parameter may have to have a scale correction applied to it prior to comparison to the nominated $E_v$ thresholds.

As the reference value for the *insitu* modulus assessment of the near-surface, the results of the static PLT is often correlated to the reference design modulus parameter – resilient modulus ($M_R$). The difference between the two (2) parameters – as compared in Figure 4-8 and defined in Equation 4-9(a) and (b) from elastic half space theory – is the assumption that the static PLT only results in elastic deformation within its limited number of loading cycles. In contrast, the resilient modulus ($M_R$) parameter incorporates the recoverable deformation only (from large number – e.g. 10,000 – loading cycles). In practice, as concluded by Puppala (2008), it appears that the $M_R$ parameter is close to the insitu $E_0$ (very small strain modulus) of stiff materials (i.e $M_R$ ~ $E_0$).

**Table 4-11 Example of PLT derived modulus for QA acceptance criteria for subgrade and earthworks formations, based on material type and compaction threshold (from German Standards – ZTVA-StB., 1997)**

| Material Type | Degree of Compaction ($D_{pr}$) | PLT – Reloading Cycle Modulus ($E_{v2}$) |
|---|---|---|
| Gravel dominated materials – GW / GP (Inc. Aggregate materials) | ≥ 103 % | ≥ 120 MPa |
| | ≥ 100 % | ≥ 100 MPa |
| | ≥ 98 % | ≥ 80 MPa |
| | ≥ 97 % | ≥ 70 MPa |
| Sand dominated materials – SW / SP | ≥ 100 % | ≥ 80 MPa |
| | ≥ 98 % | ≥ 70 MPa |
| | ≥ 97 % | ≥ 60 MPa |
| Mixed and fine grained soils | ≥ 100 % | ≥ 45 MPa |
| | ≥ 97 % | ≥ 30 MPa |
| | ≥ 95 % | ≥ 20 MPa |

**Table 4-12 Examples of PLT derived modulus for QA acceptance criteria for subgrade and earthworks formations, based on depth from top of subgrade (from Austrian and Swedish Standards, ISSMGE, 2005 & VVR Väg., 2009)**

| Location of Test / Formation Level | | PLT – Initial Loading Cycle Modulus ($E_{v1}$) | | PLT – Reloading Cycle Modulus ($E_{v2}$) | |
|---|---|---|---|---|---|
| Austrian Standard (ISSMGE, 2005) | Top of Base Course | ≥ 75 MPa (Rounded) | ≥ 90 MPa (Angular) | | |
| | Top of Sub-base | ≥ 60 MPa (Rounded) | ≥ 72 MPa (Angular) | | |
| | Top of Subgrade | ≥ 25 MPa (Cohesive) | ≥ 35 MPa (Granular) | | |
| | 1000mm below Top of Subgrade | ≥ 15 MPa (Cohesive) | ≥ 20 MPa (Granular) | **Crushed Rock subgrade** | **Sand subgrade** |
| Swedish Standard (VVR Väg., 2009) | 800mm (below top of base course) | | | ≥ 12 MPa | ≥ 16 MPa |
| | 900mm | | | ≥ 9 MPa | ≥ 11 MPa |
| | 1000mm | | | ≥ 6 MPa | ≥ 8 MPa |
| | 1100mm | | | ≥ 4 MPa | ≥ 5 MPa |
| | 1200mm | | | ≥ 3 MPa | ≥ 4 MPa |
| | 1300mm | | | ≥ 2 MPa | ≥ 3 MPa |

**Table 4-13 Example of PLT derived modulus for QA acceptance criteria for pavement formations (from Swedish Standards, VVR Väg., 2009)**

| Pavement Type | Depth Below Base course Surface (mm) | PLT – Reloading Cycle Modulus ($E_{v2}$) | | $E_{v2}$ / $E_{v1}$ Ratio (if $E_{v2}$ threshold not met) |
|---|---|---|---|---|
| | | Minimum (MPa) | Average (MPa) | |
| Asphalt Pavement | 0 – 250 mm | ≥ 125 | ≥ 140 + (0.96 x $\sigma$) | ≤ 2.8 |
| | 251 – 550 mm | ≥ 32 | ≥ 40 + (0.96 x $\sigma$) | ≤ 3.5 |
| | 551 – 650 mm | ≥ 20 | ≥ 30 + (0.96 x $\sigma$) | Not Applicable |
| | 651 – 750 mm | ≥ 15 | ≥ 20 + (0.96 x $\sigma$) | Not Applicable |
| Concrete Pavement | 0 – 250 mm | ≥ 105 | ≥ 120 + (0.96 x $\sigma$) | ≤ 2.8 |
| | 251 – 550 mm | ≥ 45 | ≥ 55 + (0.96 x $\sigma$) | ≤ 3.5 |
| | 551 – 650 mm | ≥ 30 | ≥ 35 + (0.96 x $\sigma$) | Not Applicable |
| | 651 – 750 mm | ≥ 20 | ≥ 25 + (0.96 x $\sigma$) | Not Applicable |

Note: $\sigma$ = Standard Deviation

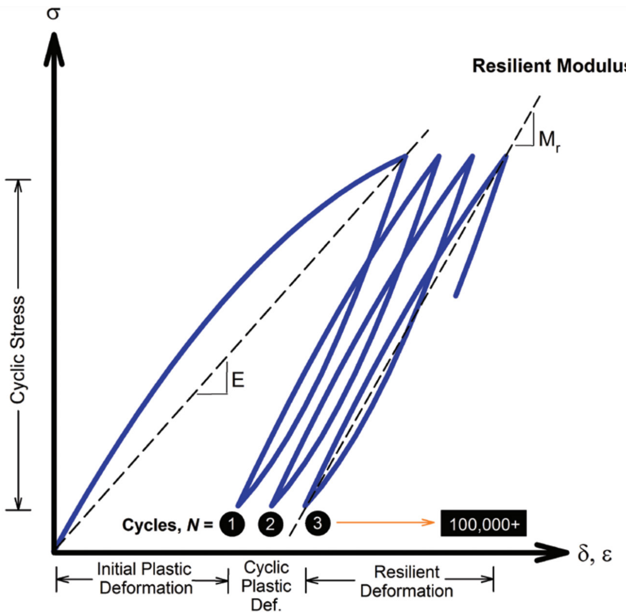



**Figure 4-8 Comparison of location for modulus derived from static tests (PLTs) and resilient modulus testing (from http://www.ingios.com/)**

For *insitu* Modulus (E), typically derived from static Plate Load Tests:

$$E = [f \times \sigma \times (1 - \nu^2) \times r] / d_0 \qquad \text{(Equation 4-9a)}$$

For Resilient Modulus ($M_R$), typically measured by laboratory Tests:

$$M_R = [f \times \sigma \times (1 - \nu^2) \times r] / d_r \qquad \text{(Equation 4-9b)}$$

Where:

$d_0$ = Elastic (instantaneous) Deformation observed under loading stress ($\sigma$)

$d_r$ = Recoverable Deformation observed under loading stress ($\sigma$)

*f* = plate rigidity / shape factor

$\sigma$ = maximum applied stress

r = radius of plate

Within Australia, the use of the PLT derived modulus is generally accepted to be the reference *insitu* modulus parameter. However, due to the required load and duration required to complete such testing, the PLT is very rarely implemented onsite during earthworks or pavement construction projects – with density and FWD testing generally preferred, respectively. Correlations between density and PLT derived modulus parameters are known to be material specific. For example, recent (2016) comparative PLT and density testing completed at the TSRC project (Queensland, Australia) suggested a typical correlation with a RDD = 95% density threshold as detailed in Table 4-14. Within this project, it was observed that 2-cycle (initial and reloading) PLTs were completed at a rate of 2 – 3 tests per day.

**Table 4-14 Example of Comparison between $E_{v2}$ and RDD parameters for site by site testing.**

| Fill Material Origin | Weathering state of fill | Characteristic Strength of Fill | Equivalent $E_{v2}$ for RDD = 95% |
|---|---|---|---|
| Sandstone | Highly to Slightly Weathered | Medium Strength | $E_{V2}$ = 60 MPa |
| Interbedded Siltstone / Sandstone | Highly to Slightly Weathered | Low Strength | $E_{V2}$ = 40 MPa |
| Basalt | Extremely to Moderately Weathered | Medium Strength | $E_{V2}$ = 50 MPa |

## Assessment of PLT for use as QA Tool

Awareness of the PLT throughout the road construction industry is the highest of all the surface based loading QA techniques considered by this review document. It is accepted by most regulatory authorities to be the reference *insitu* modulus measurement technique available, and it is to the $E_{PLT}$ parameter that all other surface based loading QA techniques are compared.

However, the limitations of the PLT test – the duration required to complete a test, requirement for a heavy external load force to be provided for reaction to jack against, and the requirement for operators to interact with (below) the reaction force – prevents its widespread adoption for QA purposes.

### 4.3.2 Light Falling Weight Deflectometer (LFWD)

## Test Equipment and Methodology

Light Falling Weight or Portable Deflectometers – also abbreviated as LFWD, LFD, PFWD, or LWD in technical literature – is a quasi-static plate load test in which a sliding 10kg weight is manually raised upon a guide rod and dropped onto a rigid base plate instrumented with a load cell and velocity transducer. A load pulse is generated when the weight is dropped upon the rubber dampers, which passes through the rigid plate and into the ground as a uniform stress. The load cell and deflectometer respectively measure the imparted force and deflection of the ground below the rigid plate. The key elements of the equipment are shown conceptually in Figure 4-9.

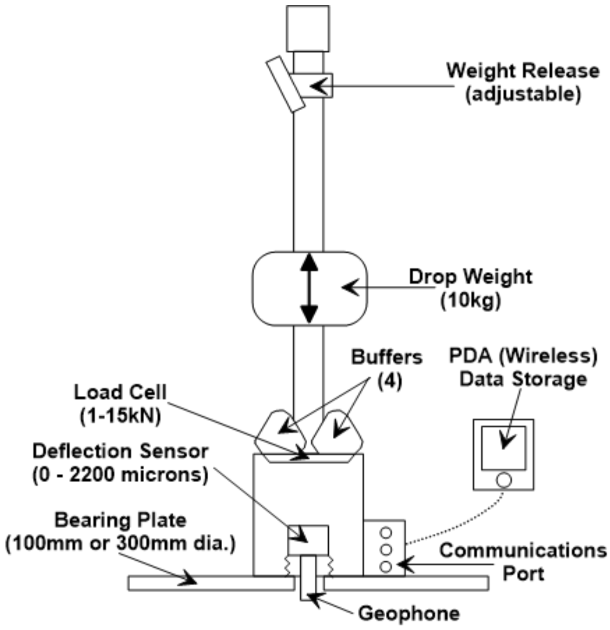



**Figure 4-9 Light Falling Weight Deflectometer (after Fleming et. al., 2007)**

As both force and deflection values are measured over the duration of the load pulse, the composite Young's Modulus ($E_{LFWD}$ or $E_{LWD}$) over the zone of test influence can thus be derived by the classic static elastic theory (Boussinesq elastic half-space) equation, as shown in Equation 4-10.

$$E_{LFWD} = [A \times P \times R \times (1 - \nu^2)] / d_0 \qquad \text{(Equation 4-10)}$$

Where:

A = Plate rigidity factor (π/2 for rigid plate) P = Maximum Contact Pressure

R = Radius of plate $\nu$ = Poisson's Ratio $d_0$ = Peak deflection

A typical test normally consists of a number of repeated weight drops and, depending on the model / configuration of the LFWD (i.e. presence of a load cell), from varying heights to produce a load / deformation response curve, as shown in Figure 4-10 and Figure 4-11 – note this curve may be linear or non-linear depending on if the material is deforming within the elastic or non-elastic phase. Similarly, for cohesive (fines dominated) materials the insitu modulus may decrease as the LFWD test stress (weight height drop) is increased, whilst for granular materials the *insitu* modulus typically increases as the LFWD test stress is increased (unless the composite insitu modulus is being influenced by softer underlying materials).

A single LFWD test (including variation of drop heights) can be completed within approximately 5 minutes, with near-instantaneous reporting of the *insitu* modulus available.

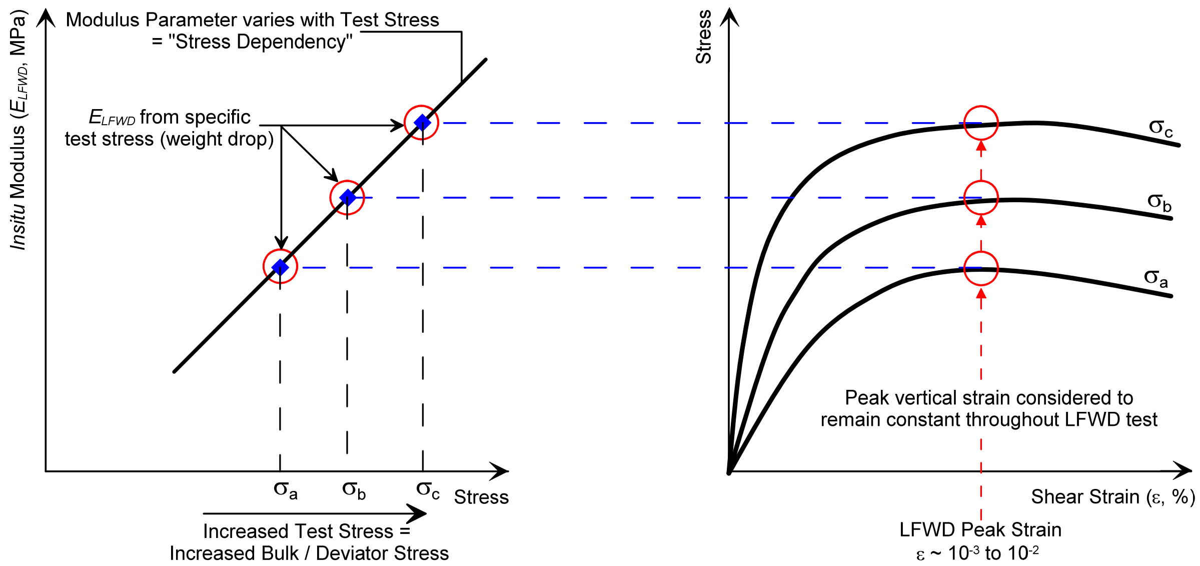



**Figure 4-10 Expected response of LFWD determined insitu modulus parameter for tests completed using a single plate diameter (resulting in approximately consistent strain conditions, ε), and varying weight height drops to vary imparted test stress (σ).**

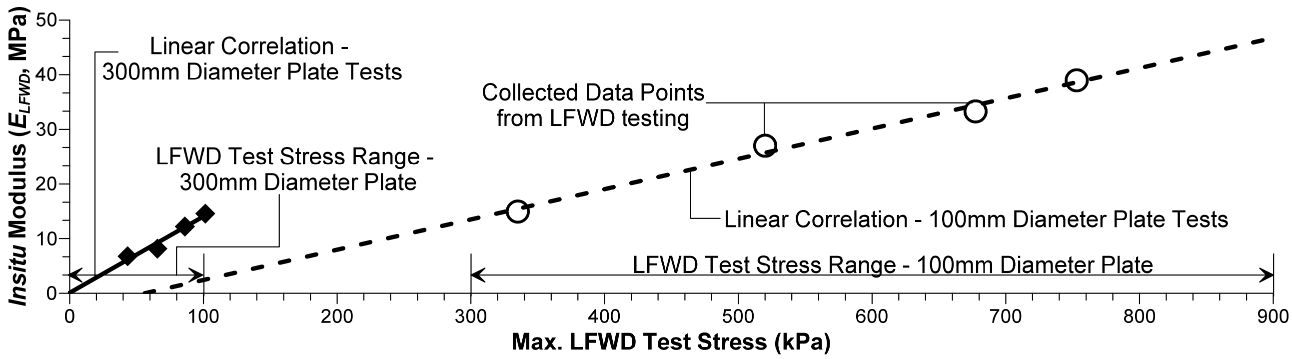



**Figure 4-11 Expected relationship between LFWD test stress ($\sigma_{LFWD}$) and determined *insitu* modulus ($E_{LFWD}$) – displaying stress and strain dependent behaviour based on LFWD test stress (weight drop) and plate diameter respectively**

## Variations within commercially available LFWD Instruments

A number of commercial manufacturers currently produce LFWD instruments which exhibit marked similarities in methodology and adopt the assumption of elastic half space theory to complete the interpretation from the stress / deformation data pair and calculate the *insitu* modulus parameter, $E_{LFWD}$. However, the configuration of each of these LFWDs is manufacturer specific and differences in the derived $E_{LFWD}$ parameter will result based on these individual arrangements.

Variation in the *insitu* modulus measured by specific LFWD equipment can be divided primarily into two classes – with each class of LFWD detailed in a separate ASTM standard – with various LFWD equipment fitted with:

(i) *Deflection Sensor (Mounted location and Type)* – The location and type of deflection sensor installed varies between brands of LFWD. Generally the sensor is either a spring-loaded geophone located in the middle of the plate and in direct contact with the ground surface, or an accelerometer that is built into the plate and measures deflection of the plate (as opposed to deflection of ground surface);

(ii) *Load Transducer (Included or absent)* – LFWD manufacturers either include a load cell for direct measurement of the imparted test stress for each weight drop, or exclude this equipment and simply assume the imparted load is a constant for all tests (i.e. weight height drop cannot be varied; and no test specific record of load is made).

These key differences in commercially available LFWD equipment are shown in Figure 4-12.

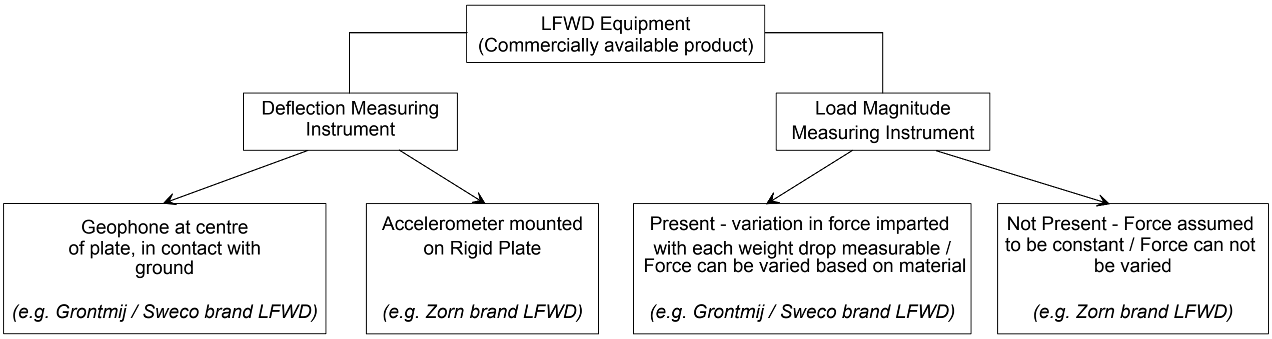



**Figure 4-12 Key variations associated with deflection and pressure monitoring abilities of various LFWD equipment**

The presence of a load cell means that the weight height can be easily varied and the calculated $E_{LFWD}$ of each test is based on the specific load measured to have been imparted during that test (i.e. no assumption of imparted load magnitude based on the height of the weight drop). The inclusion of a central geophone that is in direct contact with the ground results in the response of the ground (and not the plate) being measured for each LFWD test. The presence of both these features – load cell and geophone in contact with ground surface – result in an LFWD instrument that can record a test specific, time-based trace of both load and ground deflection being obtained for each individual 'weight drop' completed (i.e. limits any assumptions of load / deflection).

Additional variation within $E_{LFWD}$ parameters determined via various manufacturer's instruments is due to:

(iii) Size, shape, number and location of buffers (affects rate / efficiency of plate loading)

(iv) Rigid plate thickness and material (affects plate rigidity)

(v) Rigid plate size adopted for specific test (most LFWD equipment can change plate diameter size, which affects test stress mobilised for each weight drop)

Various types of LFWD are shown and compared in Table 4-15 in terms of the presence of the key elements load cell and type of deflection measurement. Additional variation is described in literature that reports of comparative studies (e.g. Fleming et. al., 2000; Vennapussa and White, 2009; Stamp and Mooney, 2013). However, it is important to note that the $E_{LFWD}$ resultant from any individual LFWD instrument still requires material- or site-specific correlation against a reference modulus parameter (e.g. $E_{PLT}$, $E_{FWD}$ or $M_R$) for confident use.

**Table 4-15 Typical details and differences between common LFWD equipment**

| LFWD Manufacturer | Deflection Transducer Header | | | Rigid Plate Thickness (mm) | Load Cell / Max. Load | Buffer |
|---|---|---|---|---|---|---|
| | Type | Location | Accuracy | | | |
| Prima 100 (Sweco / Grontmij A/S) | Geophone | Ground | ±0.002mm | 20 | Yes / 15.0kN | Rubber (Cone) |
| Kerros (Dynatest) | Geophone | Ground | ±0.002mm | 20 | Yes / 15.0kN | Rubber (Flat) |
| Dynatest 3031 (Dynatest) | Geophone | Ground | ±0.002mm | 20 | Yes / 15.0kN | Rubber (Flat) |
| Zorn ZFG (Zorn) | Accelerometer | Plate | ±0.02mm | 20 – 124 | No / 7.07kN | Steel Spring |

### Influence of LFWD Plate Diameter on $E_{LFWD}$ modulus parameter

As per the conclusions of Mooney and Miller (2009), $E_{LFWD}$ values determined from a smaller diameter plate have been consistently reported to be higher than those determined from a plate of large diameter (300mm). This was in agreement with previous research – Mooney and Miller (2009) cite Fleming (2000) and Lin et. al. (2006) – although it was noted that this was contrary to intuition; in which the higher test stress ($\sigma$) and strain ($\varepsilon$) that occurs under testing using a comparatively smaller plate should further yield the material and produce a comparatively lower $E_{LFWD}$ value (i.e. $E_{LFWD}$ parameter produced from smaller plate is representative of higher strain condition and thus deformation parameter is located lower on modulus degradation curve).

The previously reported influence of plate diameter is shown graphically in Figure 3.9 (reproduced from experimental data as summarised by Vennapusa and White, 2009), and demonstrates the comparative increase in the modulus parameter as the plate diameter of both LFWD and PLT testing is reduced.

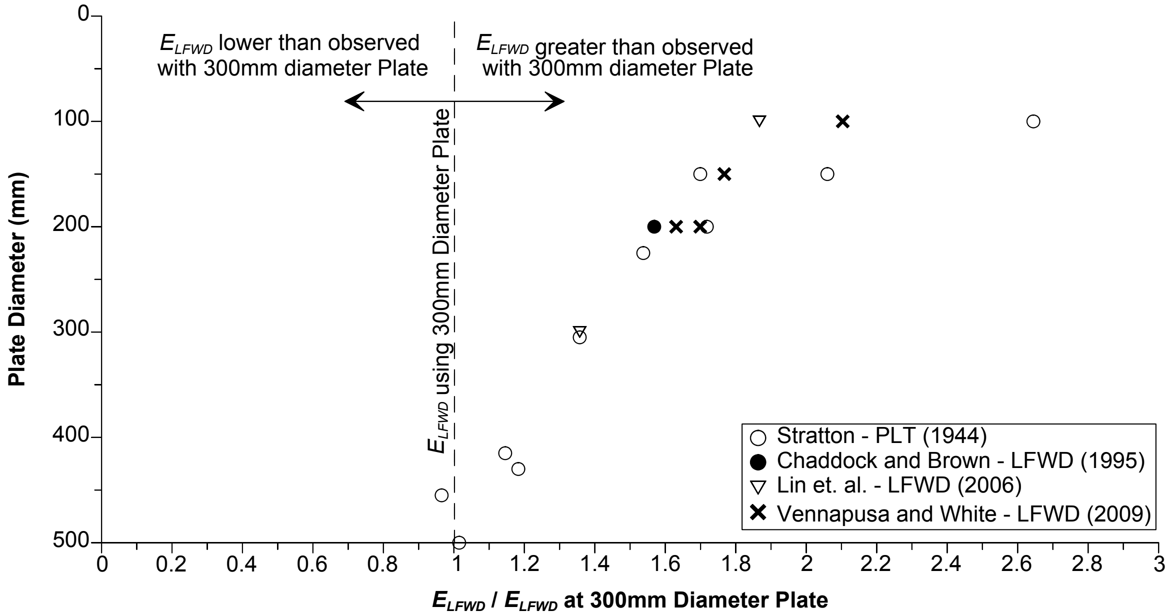



**Figure 4-13 Influence of plate diameter on modulus (E), with results compared to ELFWD determined using 300mm diameter plate (after Vennapusa and White, 2009)**

### Zone of Influence (Test penetration depth)

Lacey (2016) completed both extensive field testing and FEM modelling of the LFWD test and determined for the Queensland, Australia sites considered, the Prima 100 LFWD equipment demonstrated an effective zone of influence of 1.33 to 1.5 times the plate diameter (1.33D to 1.5D). For a 300mm diameter plate, a 'zone of influence' represents the depth to which a stress distribution value of 20% of that the test magnitude imparted at the material surface was observed, which supports Terzaghi's (1936) recommendation to use 20% for estimation of the 'significant depth' of stress influence. This finding is similar to that suggested by Nazzal et. al. (2007) and Mooney and Miller (2009), and consistent with the 1.5D estimated by Fleming (2001) and assumed by Ryden and Mooney (2009).

Nazarian et. al. (2014) completed similar FEM modelling of various LFWD models / brands and assessed the likely zone of influence for the LFWD was both deflection and stress dependent, and thus the achieved zone of influence would be material and device dependent. However, it was noted that the influence depth – adopting a $10^{th}$ percentile of the surface imparted stress as the base of the domain – would be limited to 1.6D, and decrease to 1.33D as the insitu modulus increases. It was also reported that zone of influence of LFWDs fitted with plate mounted accelerometers would be less sensitive to material properties as compared to LFWDs fitted with geophones in contact with the ground surface.

**Applicable Standards**

Two (2) ASTM standards are applicable to the completion of LFWD testing, based on the class of LFWD instrument being utilised (refer Figure 4-12):

For LFWD instruments without a load cell and fitted with a plate mounted accelerometer (e.g. Zorn brand LFWDs) –

> ASTM E2835-11 – *Standard Test Method for Measuring Deflections using a Portable Impulse Plate Load Test Device*

For LFWD instruments fitted with a load cell and fitted with an geophone in contact with the ground (e.g. Sweco / Grontmij brand LFWDs) –

> ASTM E2583-07 – *Standard Test Method for Measuring Deflections with a Light Weight Deflectometer (LWD)*

Although various ASTM standards detail the procedure to complete LFWD testing, there is no standard approach to interpret the LFWD value, which will vary with plate size and the magnitude of the imparted test pressure (drop height) due to stress and strain dependency.

Fleming et. al. (2002, 2007) has previously identified that the calculated $E_{LFWD}$ value is largely meaningless unless also reported with the pressure at which was determined, and stated the need for the LFWD derived modulus to be reported with both the plate diameter and test stress applied. Lacey (2016) also identified this limitation when the results from multiple LFWD tests / sites were required to be compared. To faciliate direct $E_{LFWD}$ parameter comparison, Lacey (2016) proposed and demonstrated the use of a method that standardised the LFWD modulus parameter to suitable test stress standards – either to 100 kPa (suitable for 300mm diameter plate) or 500 kPa test stress values (suitable for 100mm diameter plate).

**Parameters provided from test**

For all LFWD brands, the *insitu* modulus parameter ($E_{LFWD}$) is returned. This parameter is derived from the measured deflection under the load magnitude applied (known or assumed)

Depending on the type (brand) of LFWD (i.e. presence of a load cell), testing may allow the construction of a full loading stress / deformation curve for the range of stress magnitudes applied. From this the following parameters can be derived (depending on the number of loading steps applied, and if the loading exceeds the elastic phase loading of the material undergoing testing):

- $K_s$ = Subgrade Modulus / Modulus of Subgrade Reaction
- $q_{ult}$ = Ultimate bearing capacity
- $q_{allow}$ or $q_a$= Allowable bearing capacity

**Repeatability of Test Technique**

Based on extensive testing by the author throughout Queensland, Australia, and for tests completed upon both natural and processed soil materials, the repeated LFWD test upon a single material (i.e. simply repeated weight drops from a uniform drop height – constant test stress condition – at a single location) produces a Coefficient of Variation (CoV) generally less than 10%, and frequently less than 5%. This is consistent with Nazarian et. al. (2014), whom assessed the accuracy of both classes of LFWD upon laboratory prepared samples and reported a combined equipment and operator variation of 5%.

The repeatability (CoV) determined for the LFWD compares favourably to traditional testing techniques, such as CBR testing (17 to 58%, as reported by Lee et. al, 1983) or field penetration tests such as the Dynamic Cone Penetrometer Test (DCP) and Standard Penetration Test (SPT) (both >50%, as reported by Mellish et. al., 2014 and Phoon and Kulhawy, 1999 respectively).

The precision for the deflection sensors contained within an LFWD instrument is +/- 2 $\mu$m, whilst the accuracy of load cell (where present) and deflection sensors are +/-2 %. However, the variation within the reported (and standardised) LFWD stiffness parameter ($E_{LFWD}$) arising from the repeated field testing of different materials / locations will obviously increase the observed CoV, based on the inherent heterogeneity of the material / site being assessed. For processed granular materials this is generally reported to be up to 15%

(e.g. Fleming et. al., 2009; Umashankar et. al., 2016), while natural materials within a single site have demonstrated a typical CoV of up to 40% (Lacey et. al., 2016; Ashibili, 2005; Nazzal, 2003).

As identified by Nazarian et. al. (2014), the total variation / CoV observed can be considered largely independent of the class of LFWD utilised for testing, based on the similar accuracy of their components. However, note the repeatability / inherent variation in LFWD test results should not be confused with the expected variation in the $E_{LFWD}$ parameter expected to arise due to variation in equipment brand, plate size, weight drop etc. (as discussed in previous sections of this document).

**Advantages**

- Direct measurement of site- and material-specific deformation response to applied stress / load
- Portable equipment, fully self-contained and small test footprint
- Rapid test method (5 minutes per test)
- Repeatable test results
- Repeated tests can be used to observe increase in *insitu* modulus parameter due to changed conditions – either (i) additional compaction effort applied; (ii) geo-reinforcement installed within near-surface; or (iii) additional material being placed above weak subgrade.
- If LFWD instrumented is fitted with load cell:
    - Fully adjustable load magnitude (limited by reaction load) that can be altered to represent design loading scenario or assess modulus variation based on applied stress
    - Well defined load steps and measured response to each load step can be recorded
    - Allowable ($q_{allow}$) and ultimate ($q_{ult}$) bearing capacities of materials can be assessed by repeated LFWD testing using varied test stresses

**Disadvantages / Limitations**

- Quasi-static / dynamic load modulus – needs to be correlated to reference modulus parameter to account for stress / strain variation between test and permanent conditions (generally correlated to reference $E_{PLT}$, $E_{LWD}$ or $M_R$ parameters)
- Modulus parameter is typically "stress dependent" – needs to be calibrated to a standardised test stress
- Various brands / equipment configurations produce varied $E_{LFWD}$ parameters, causing confusion in industry if not properly processed
- Resultant modulus parameter is representative only of material at moisture condition at time of testing
- Surface based test with 'Zone of Influence' limited to 1.33 to 1.5 times plate diameter below surface (i.e. 400mm to 450 mm depth for 300mm diameter plate)

**Existing Literature regarding use as compaction QA test**

Research involving the LFWD as an earthworks and pavement material QA assessment tool has largely involved the assessment and correlation of the $E_{LFWD}$ stiffness parameter to the modulus parameter derived from 'reference' assessment techniques – namely the Plate Load Test ($E_{PLT}$), Falling Weight Deflectometer ($E_{FWD}$) or laboratory derived resilient modulus ($M_R$).

The relationship between the $E_{LFWD}$ and reference modulus test results is known to be material- and site-specific. However, the identification of suitable $E_{LFWD}$ parameters / thresholds for use as QA assessment is further clouded due to the potential variation in LFWD configuration (type / brand, plate size, test stress etc.). Accordingly – as recognised by Fleming et. al. (2002, 2007) and as recommended by both Puppala (2008) and Vennapussa & White (2009) – any use of the $E_{LFWD}$ modulus parameter as a QA tool requires an associated nomination of the LFWD brand, plate diameter and test stress to be applied during field testing.

Table 4-16 presents a number of existing direct relationships for the modulus parameters derived from Plate Load Test (PLT) – either $E_{PLT}$ for the initial ($E_{v1}$) or reloading ($E_{v2}$) test cycles - and the $E_{LFWD}$ parameter reported by various LFWD brands / instrument configurations. Table 4-17 presents the existing relationships between the $E_{LFWD}$ parameter and the stiffness parameter reported by the larger Falling Weight Deflectometers (FWD).

Note that the relationships presented in Table 4-16 and Table 4-17 are from studies that vary significantly in magnitude (i.e. number of paired LFWD and PLT / FWD tests available for analysis, number of sites assessed), test varied materials (e.g. natural, processed or stabilised materials, fine / cohesive and/or granular, and may have been completed in a laboratory or field setting (or a combination of both laboratory and field testing). Accordingly, the derived relationships between the $E_{LFWD}$ and $E_{PLT}$ / $E_{FWD}$ derived modulus parameters varies in quality (i.e. correlation coefficient ($R^2$) varies from 'low quality' (i.e. $R^2 \geq 0.25$) to very high strength ($R^2 \sim 0.95$).

**Table 4-16 Examples of published direct $E_{LFWD}$ to $E_{PLT}$ correlations – LFWD vs. Plate Load Test**

| LFWD Brand | Plate Load Details | | Correlation *(note quantity and strength of correlation varies between studies)* | LFWD Plate Size | Standardised LFWD Test Magnitude | Materials Assessed | Reference |
|---|---|---|---|---|---|---|---|
| | Plate Dia. | $E_{v1}$ or $E_{v2}$ | | | | | |
| Zorn | 300 mm | $E_{v2}$ | $E_{LFWD}$ = 0.45 to 0.56 x $E_{V2}$ | 300mm | NO (Zorn assumes constant load applied, no load cell) | Granular Materials | Weingart (1993) |
| | | $E_{v2}$ | $E_{LFWD}$ = 0.50 to 0.56 x $E_{V2}$ | | | Gravel, sand & mixed soils | ZTVA-Stb (1997) |
| | | $E_{v2}$ | $E_{V2}$ = 600 – [300 / (300 - $E_{LFWD}$)] | | | Unknown | Livneh & Goldberg (2000) |
| | | Unknown | $E_{PLT}$ = 0.41 x $E_{LFWD}$ | | | Very Gravelly Moraine Sand | Hildebrand (2003) |
| Keros | | Unknown | $E_{PLT}$ = 0.81 x $E_{LFWD}$ | 300mm | Unknown / Not Reported | | |
| Zorn | | $E_{v1}$ | $E_{V1}$ = (5/6) x $E_{LFWD}$ | 300mm | NO (Zorn assumes constant load applied, no load cell) | Cohesive | Adam & Kopf (2004) |
| | | | $E_{V1}$ = 150 x ln [180 / (180 – $E_{LFWD}$)] | | | Non-Cohesive | |
| | | $E_{v1}$ | $E_{PLT}$ = (0.43 x $E_{LFWD}$) + 3.25 | | | Silty Sand (SM) | Kim et. al. (2007) |
| | | $E_{v1}$ | $E_{LFWD}$ = 1.58 x $E_{V1}$ | 200mm | 50cm height drop maintained | SP - SM | Vennapusa & White (2009) |
| | | $E_{v2}$ | $E_{LFWD}$ = 0.47 x $E_{V2}$ | | | | |
| | | $E_{v1}$ | $E_{PLT}$ = (1.8 x $E_{LFWD}$) – 10.6 | 200mm | NO (Zorn assumes constant load applied, no load cell) | Well graded Gravel | Zhang (2010) |
| | | | $E_{PLT}$ = (1.5 x $E_{LFWD}$) + 36.5 | 300mm | | | |
| | | | $E_{PLT}$ = (1.2 x $E_{LFWD}$) – 10.6 | 200mm | | | |
| | | $E_{v2}$ | $E_{PLT}$ = (5.1 x $E_{LFWD}$) + 1.2 | 200mm | | | |
| | | | $E_{PLT}$ = (5.1 x $E_{LFWD}$) + 78.1 | 300mm | | | |
| | | | $E_{PLT}$ = (4.6 x $E_{LFWD}$) – 8.3 | 200mm | | | |
| Prima | 300 mm | N/A | Log ($k_{LFWD}$ / $k_{30}$) = [0.0031 x Log ($k_{LFWD}$)] + 1.12 | NA | NA – Deflection Based Assessment | Volcanic soil; silty sand & stabilised crushed stone | Kamiura et. al. (2000) |
| | 200 mm / 250 mm | $E_{v1}$ | $E_{PLT}$ = (0.91 x $E_{LFWD}$) – 1.81 | 200mm | Unknown / Not Reported | Natural, processed & Manufactured materials | Seyman (2001) |
| | | $E_{v2}$ | $E_{PLT}$ = 28.25 x $e^{(0.006 \times ELFWD)}$ | | | | |
| | 300 mm | $E_{v1}$ | $E_{PLT}$ = 22 + (0.70 x $E_{LFWD}$) | 200mm | Unknown / Not Reported | Natural and stabilized clay; crushed limestone; stabilized aggregate | Nazzal (2003) |
| | | $E_{v2}$ | $E_{PLT}$ = 20.9 + 0.69 x $E_{LFWD}$ | | | | |
| | | $E_{v1}$ | $E_{PLT}$ = 1.041 x $E_{LFWD}$ | 200mm | | | Nazzal et. al. (2007) |
| | | $E_{v2}$ | $E_{PLT}$ = 0.875 x $E_{FWD}$ | | | | |

| LFWD Brand | Plate Load Details | | Correlation *(note quantity and strength of correlation varies between studies)* | LFWD Plate Size | Standardised LFWD Test Magnitude | Materials Assessed | Reference |
|---|---|---|---|---|---|---|---|
| | Plate Dia. | $E_{v1}$ or $E_{v2}$ | | | | | |
| Prima *(cont.)* | 300 mm | $E_{v1}$ | $E_{PLT} = 0.63 \times E_{LFWD}$ | 300mm | Unknown / Not Reported | Poorly graded sand | Vennapusa & White (2009) |
| | | $E_{v2}$ | $E_{PLT} = 2.13 \times E_{LFWD}$ | | | | |
| | | $E_{v1}$ | $E_{LFWD} = 13.37 \times e^{(0.059 \times EPLT)}$ | 300mm | YES – LFWD Standardised to 100kPa | Gravel, sand & mixed soils | Lacey (2016) |
| | | $E_{v1}$ | $E_{LFWD} = (1.78 \times E_{PLT}) + 16.70$ | 300mm | YES – LFWD Standardised to 100kPa | Crushed Sandstone | Lacey (2016) |
| | | $E_{v2}$ | $E_{LFWD} = (0.64 \times E_{PLT}) + 9.53$ | 300mm | | | |
| | | $E_{v2}$ | $E_{LFWD} = (1.13 \times E_{PLT}) – 29.84$ | 300mm | YES – LFWD Standardised to 100kPa | Crushed Basalt | Lacey (2016) |

**Table 4-17 Examples of published direct $E_{LFWD}$ to $E_{PLT}$ correlations – LFWD vs. FWD**

| LFWD Brand | Correlated to | Correlation *(note quantity and strength of correlation varies between studies)* | LFWD Plate Size | Standardised LFWD Test Magnitude | Materials Assessed | Reference |
|---|---|---|---|---|---|---|
| Zorn | FWD – 300mm Plate | $E_{LFWD} = 0.50$ to $0.60 \times E_{FWD}$ | 300 mm | NO (Zorn assumes constant load applied, no load cell) | Unknown | Shahid et. al. (1997) |
| Zorn | FWD – 300mm Plate | $E_{LFWD} = 0.53 \times E_{FWD}$ | | | Granular Capping & Clay Subgrade | Fleming et. al. (1998) |
| Zorn | FWD – 300mm Plate | $E_{LFWD} = 0.63 \times E_{FWD}$ | | | Granular Capping | Fleming et. al. (2000) |
| Zorn | FWD – 300mm Plate | $E_{LFWD} = 0.43$ to $1.43 \times E_{FWD}$ | | | Natural / stabilized clay, and granular capping materials | Fleming et. al. (2000) |
| Zorn | FWD – 300mm Plate | $E_{LFWD} = 0.3$ to $0.4 \times E_{FWD}$ | | | Unknown | Livneh & Goldberg (2000) |
| Zorn | FWD – 300mm Plate | $E_{LFWD} = 0.4 \times E_{FWD}$ | | | Very Gravelly Moraine Sand | Hildebrand (2003) |
| Keros | FWD – 300mm Plate | $E_{LFWD} = 0.79 \times E_{FWD}$ | 300 mm | NO - Assumed to simply represent max height weight drop | Very Gravelly Moraine Sand | Hildebrand (2003) |
| Prima | FWD – 300mm Plate | $E_{FWD} = 0.60$ to $1.60\ E_{LFWD}$ | 300 mm | Unknown / Not Reported | Very stiff self-cementing materials | Groenendijk et al. (2000) |
| Prima | FWD – 300mm Plate | $E_{LFWD} = 0.90 \times E_{FWD}$ | 300 mm | NO - Assumed to simply represent max height weight drop | Natural / stabilized clay, and granular capping materials | Fleming et. al. (2000) |
| Prima | FWD – 300mm Plate | $E_{FWD} = 0.97 \times E_{LFWD}$ | 200 mm | Unknown / Not Reported | Natural and stabilized clay; crushed limestone; stabilized aggregate | Nazzal et. al. (2004) |

In addition to the correlation between the $E_{LFWD}$ parameter and the results from side-by-side PLT or FWD, some regulatory agencies have developed and published compaction standards that incorporate the LFWD derived stiffness parameter as an acceptable QA measure (i.e. as an alternative to density testing).

Existing UK standards (IAN73/06, 2009) nominates a minimum 'surface modulus' parameter that are to be demonstrated for each defined material type. The reference measure of modulus for this standard is the

Falling Weight Deflectometer (FWD). LFWD instruments are allowed – as long as the LFWD is equipped with a load cell (e.g. Prima 100) - as long as a project / material specific trial that compares the $E_{FWD}$ and $E_{LFWD}$ parameter at a minimum of 25 test locations is completed, and that the strength of correlation ($R^2$) between the $E_{LFWD}$ and $E_{FWD}$ parameter exceeds 0.45.

Similarly, the Austrian standard (ISSMGE, 2005) allows the use of the LFWD tool for QA assessment of subgrade and pavement materials, as long as the linear regression coefficient ($R^2$) between the LFWD ($E_{LFWD}$) and PLT ($E_{PLT}$) stiffness parameters is 0.5 or higher. Included in this standard are specified minimum $E_{LFWD}$ parameters to be used for QA assessment (reproduced herein in Table 4-18), and based on a comparison between the specified $E_{PLT}$ and $E_{LFWD}$ parameters, the Austrian Standard assumes that for subgrade materials the LFWD measured stiffness value is approximate 0.9 to 1.0 that measured by the initial loading cycle of the plate load test (i.e. $E_{LFWD}$ = 0.9 to 1.0 x $E_{PLT(i)}$). For the stiffer aggregate base and sub-base materials, the adopted relationship trends to $E_{LFWD}$ = 1.2 x $E_{PLT(i)}$.

Current Swedish standards (VVR Väg., 2009) exclude the use of the LFWD derived modulus in the QA assessment of base or sub-base materials, but does allow the LFWD instrument to be used as an acceptable QA measure for subgrade materials. However, the LFWD is only accepted for use if "similar results can be demonstrated" when compared to the PLT derived modulus parameter (i.e. site / material specific trials are required to be undertaken). Table 4-18 identifies the $E_{LFWD}$ parameters specified to be used for QA within the Swedish standards for subgrade materials below a depth of 800mm from the surface of the base course. Back-calculation of the $E_{LFWD}$ and $E_{V2}$ indicates the Swedish standard is based on the assessment that the $E_{LFWD}$ parameter will be between 0.75 and 1.3 times the $E_{PLT(EV2)}$.

**Table 4-18 Examples of LFWD derived modulus for QA acceptance criteria for subgrade and earthworks formations, based on depth from top of subgrade (from Austrian and Swedish Standards, ISSMGE, 2005 & VVR Väg., 2009)**

| Location of Test / Formation Level | | Light Falling Weight Deflectometer ($E_{LWD}$) | | Light Falling Weight Deflectometer ($E_{LWD}$) | |
|---|---|---|---|---|---|
| Austrian Standard (ISSMGE, 2005) | Top of Base Course | ≥ 70 MPa (Rounded) | ≥ 82 MPa (Angular) | | |
| | Top of Sub-base | ≥ 58 MPa (Rounded) | ≥ 68 MPa (Angular) | | |
| | Top of Subgrade | ≥ 30 MPa (Cohesive) | ≥ 38 MPa (Granular) | | |
| | 1000 mm below Top of Subgrade | ≥ 18 MPa (Cohesive) | ≥ 24 MPa (Granular) | **Crushed Rock subgrade** | **Sand subgrade** |
| Swedish Standard (VVR Väg., 2009) | 800 mm (below top of base course) | | | 10 – 15 MPa | 12 – 18 MPa |
| | 900 mm | | | 8 – 12 MPa | 10 – 14 MPa |
| | 1000 mm | | | 5 – 8 MPa | 7 – 11 MPa |
| | 1100 mm | | | 4 – 5 MPa | 5 – 8 MPa |
| | 1200 mm | | | ≥ 3 MPa | 3 – 5 MPa |
| | 1300 mm | | | ≥ 2 MPa | ≥ 3 MPa |

Both the German earthworks / roadworks standard (ZTVA-StB., 1997) and German rail regulatory authorities (DB Netz AG, 1999) have correlated the LFWD stiffness parameter with the reloading PLT modulus for aggregate, processed and natural materials. The same standard correlated the compaction level (RDD) to the $E_{LFWD}$ parameter. These $E_{LFWD}$ thresholds are summarised in Table 4-19. Note that the LFWD stipulated for use by the German Standard are manufactured by Zorn (i.e. single drop height used, no load cell and assumption of 7.07 kN test stress is imparted into subsurface), meaning that the parameters summarised in Table 4-19 would require further correlation prior to adoption or use with other branded LFWD instruments. Across the range of $E_{PLT}$ that the $E_{LFWD}$ is correlated to (20 MPa $\leq E_{PLT(EV2)} \leq$ 180 MPa), the stipulated $E_{LFWD}$ varies between 0.44 to 0.75 of the $E_{PLT}$ parameter, demonstrating the higher stress dependency associated with the LFWD instrument (compared to the PLT).

**Table 4-19 Example of LFWD derived modulus for QA acceptance criteria for subgrade and earthworks formations, based on material type and required compaction threshold (from German Standards – ZTVA-StB., 1997 and Ril 836, 1999)**

| Material Type | Degree of Compaction ($D_{pr}$) | Light Falling Weight Deflectometer ($E_{LWD}$) - Modulus |
|---|---|---|
| Gravel dominated materials – GW / GP (Inc. Aggregate materials) | $\geq$ 103 % | $\geq$ 65 MPa |
| | $\geq$ 100 % | $\geq$ 50 MPa |
| | $\geq$ 98 % | $\geq$ 40 MPa |
| | $\geq$ 97 % | $\geq$ 30 MPa |
| Sand dominated materials – SW / SP | $\geq$ 100 % | $\geq$ 50 MPa |
| | $\geq$ 98 % | $\geq$ 40 MPa |
| | $\geq$ 97 % | $\geq$ 35 MPa |
| Mixed and fine grained soils | $\geq$ 100 % | $\geq$ 25 MPa |
| | $\geq$ 97 % | $\geq$ 20 MPa |
| | $\geq$ 95 % | $\geq$ 15 MPa |

Within Australia, the use of the LFWD as a compaction QA tool has significant potential to be used.

### 4.3.3 Clegg Hammer

#### Test Equipment and Methodology

The Clegg Impact Soil Tester (Clegg Hammer) provides a means for measuring and controlling compaction in both road applications and sports surfaces and to confirm uniform compaction of over wide areas of ground, identifying poorly compacted areas and ineffective rolling of materials. *Insitu* testing can be undertaken with various models of the instrument, which include a drop weight of 2.25 kg, 4.5 kg, 10 kg and 20 kg versions. As per the LFWD tests, the varying weights utilised alter the stress magnitude imparted during a test and thus the 'zone of influence' over which the composite test results are representative.

As shown in Figure 4-14, the Clegg tester consists of a compaction hammer operating within a vertical guide tube. When the hammer is released from a fixed height it falls vertically through a guide tube and strikes the surface under test, decelerating at a rate determined by the stiffness of the material within the region of impact. A precision accelerometer mounted on the hammer feeds its output to a hand-held digital readout unit which registers the hammer deceleration. The resultant readings are presented in "gravities" or Impact Values (IV). The IV indicates soil strength and has been presented to demonstrate good correlation with *insitu* Californian Bearing Ratio (CBR) test results.

**Figure 4-14 Clegg Impact Soil Tester**

Sports surface testing – which requires only shallow surface investigation - utilises the light (2.25kg) version of the Clegg hammer. However, the most popular (and 'standard') Clegg instrument is the tool that is fitted with a 4.5 kg drop hammer. Heavier duty – 10 kg and 20 kg hammer weights – versions are manufactured for testing harder materials.

The 10 kg version is also available with a variable drop height (5 drop heights) such that the change in the response of a material to test stress variation can be assessed – similar to the evaluation of the "stress dependency" identified in Section 2.5 (however, it is noted that the Clegg hammer test is an index test and the tool does not directly measure either material deflection or imparted load). The variable drop height hammer is considered more versatile and, like an LFWD instrument fitted with a load cell, would potentially allow variation in behaviour of the compacted material under a number of test stress magnitudes to be assessed, such that the allowable and utilimate bearing capacity of the material could me estimated, or the consistency / variation of a material's stiffness throughout the lift thickness could be assessmesed.

Based on the location of its development – Western Australia – the use of Clegg Hammers has been widespread across Western Australia since the 1970's. The Queensland Department of Main Roads has also extensively utilised the Clegg Hammer for the past 2 decades, predominantly for assessment of pavements.

#### Applicable Standards

- AS 1289.6.9.1: *Methods for testing soils for engineering purposes- Method 6.9.1: Soil Strength and Consolidation test - Determination of stiffness of soil - Clegg Impact Value (CIV).*
- ASTM D 5874 – *Standard Test Method for Determination of the Impact Value (IV) of a Soil*.

#### Parameters provided from test

The Clegg impact soil tester (CIST) is suitable for identifying weak spots in pavement applications (top 1500mm only). Weakness may be due to poor material, low compaction or high moisture contents. Some typical values are provided in Table 4-20.

**Table 4-20 Typical Clegg Impact Values (IV) for assessment**

| Material Type | Minimum IV | Typical Field IV | Header |
|---|---|---|---|
| Base pavement | 60 | 30<br>60<br>70 | poor; soft underfoot, damp to wet data<br>satisfactory; hard, dense very<br>hard, very dense data |
| Sub - base pavement | 50 | 30<br>50<br>60 | poor; soft underfoot, damp to wet<br>satisfactory; Very stiff, dense<br>Hard, dense |
| CBR 10 material – working platform or laterite gravels | 25 | 20<br>25 | poor<br>satisfactory |

#### Advantages

- CIH simple to operate
- Correlations with CBR values are available.

#### Disadvantages / Limitations

- Not strictly a modulus measuring device

### 4.3.4 Briaud Compaction Device (BCD)

The Briaud Compaction Device (BCD) was developed by researchers at Texas A&M University from 2003 onwards. The current iteration of the BCD is manufactured by Roctest Ltd., based in Quebec, Canada.

#### Test Equipment and Methodology

The BCD utilises a surface based plate in contact with the ground surface that, during operation, measures the bending strain of the plate and records the associated bending strain. Stiffer (harder) soils result in low plate bending / low bending strains due to the applied force, while softer (weaker) soils result in significant plate bending / high bending strains being developed. The BCD incorporates a 1.85 mm thick, 150 mm diameter stainless steel plate with eight (8) strain gauges mounted on the upper plate surface, and an internal load cell to monitor the load vs. stain response of the soil during a test. Each instrumented plate is individually calibrated by the manufacturer. The key aspects of the BCD are conceptually shown in Figure 4-15.

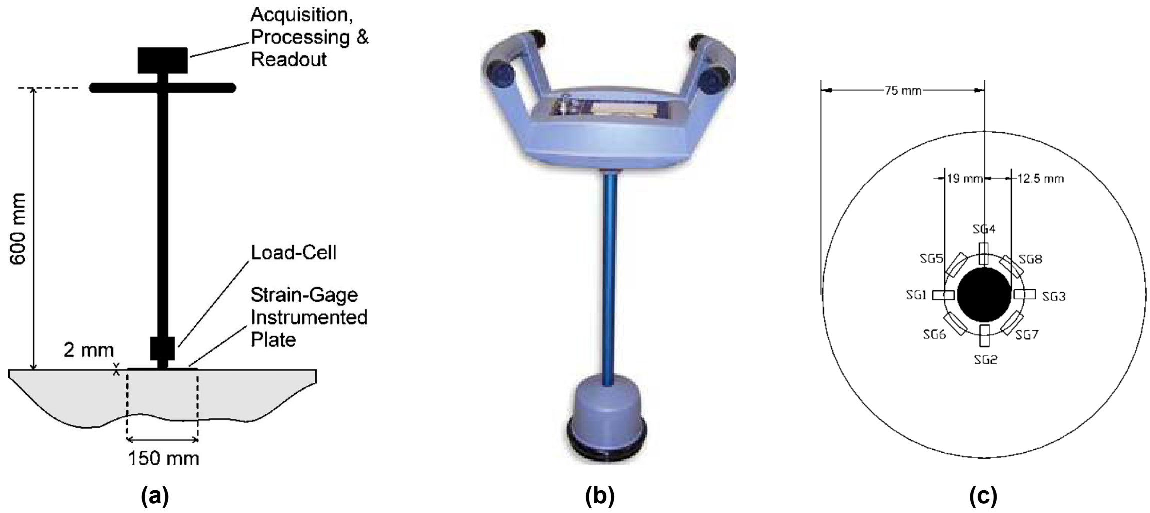



**Figure 4-15 Briaud Compaction Device (BCD)– (a) Conceptual of instrument (from Briaud et. al., 2006); (b) photo of equipment; and (c) plan view of BCD plate showing mounted strain gauges (from Briaud et. al., 2009)**

The radial and axial strains measured during the BCD test procedure are internally compared with numerical simulation results to estimate the low strain modulus parameter. The strain level associated with the BCD is approximately $10^{-3}$ % (Weidinger and Ge, 2009).

As a punching bearing capacity failure occurs when the BCD plate is pushed into very soft materials and no bending would be observed for very hard soils, there is both a lower and upper stiffness limit to the range of materials that can be successfully assessed by the BCD. Materials with moduli between 5 and 150 MPa can be assessed by this tool.

The height of the equipment is 855 mm, weighs a total of 9.6 kg and can operate on either external power or rechargeable batteries. Accordingly, the equipment is extremely portable

A single BCD field test involves the placement of the plate perpendicular to the ground surface and load the BCD instrument with the body weight of the operator. The test load is standardised to 223N and the application of the load should be gradually increased to this level over a duration of at least 5 seconds to prevent inaccurate measurement (i.e. load cannot be instantaneously applied). An initial loading cycle is discarded and at least 4 additional loading cycles are completed, with the BCD instrument rotated 90 degrees between each cycle. The recommended methodology is to initially prepare the surface with a wet, fine, uniform sand material in a 4 to 5 mm thick layer to ensure good seating / contact between the BCD and ground surface.

As identified by Glagola et. al. (2015), the requirement for the downward force to be applied by using the self-weight of the operator is “somewhat difficult” and produces variation in the BCD provided modulus parameter, especially between operators of the BCD.

BCD testing can also be undertaken within a laboratory setting by placement of the plate within the proctor mould. By repeating BCD testing in parallel with the determination of dry density tests, the modulus versus moisture content curve can be derived and compared to the dry density versus water content curve. The results of laboratory testing of this type is shown in Figure 4-16.

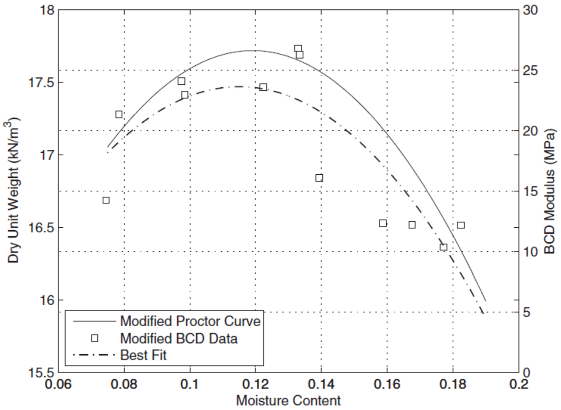



**Figure 4-16 Mirrored BCD modulus and dry unit weight versus moisture content curves, derived from tests completed upon samples subjected to modified compactive effort (from Weidinger and Ge, 2009)**

### Zone of Influence (Test penetration depth)

Briaud et. al. (2009) defined the technical specification of the BCD and nominate the zone of influence of the instrument varies based on the modulus of the material being tested. This is based on the work of Li (2004), whom reported that the zone of influence reduces from 311 mm for a material with a modulus of 3 MPa, to 121 mm for a material possessing a modulus of 300 MPa. Briaud et. al. (2009) further refined this to:

- 240 mm depth for materials with a modulus of 10 MPa
- 150 mm for materials with a modulus of 100 MPa
- 121 mm for materials with a modulus of 300 MPa (however the nominated BCD range of applicability is capped at 150 MPa).

Numerical simulation confirms the zone of influence of the BCD – defined as the depth to which 10% of the surface imparted stress is observed – is at least 150 mm.

As identified by Weidinger and Ge (2009), the limited zone of influence of the BCD in comparison to other QA/QC tools, may limit the applications to which the BCD is applicable. In terms of earthworks and pavements, the BCD would need to be assessed in terms of its ability to provide QA across the full layer of compacted earthworks / pavement materials. In comparison to existing density QA tests (sand replacement or nuclear density) the BCD does not offer any improvement in terms of test depth / zone of influence.

### Applicable Standards

No existing Australian Standard, International Standard or Australian Regulatory authority test method has been identified for use of the Briaud Compaction Device.

### Parameters provided from test

The fully self-contained equipment internally processes the recorded load / strain data and reports a single, standardised, modulus parameter – described herein as $E_{BCD}$.

The $E_{BCD}$ parameter is the modulus determined from a test load of 223 N spread over a 150mm diameter circular plate, and is thus a modulus parameter reported at a imparted test stress of 12.6 kPa.

### Repeatability of Test Technique

Briaud et. al. (2009) assessed the repeatability of the BCD test upon a rubber test block. A Coefficient of Variation (CoV) of 0.5% was determined for tests completed under a standardised test stress. Varying the pressure applied to the uniform test block produced a linear relationship with the measured strain with a correlation coefficient ($R^2$) of 0.98.

For field testing, Briaud et. al. (2009) indicates that a CoV of up to 15% was observed. This CoV could be reduced to less than 5% if a 2 to 4 mm thick sand cushion was installed prior to testing.

However, other researchers have found a significantly higher CoV when the BCD has been evaluated. Galgola et. al. (2015) reported CoVs of 29.67% (stabilised subgrade), 54.77% (clayey / silty granular soil) and 61.64% (fine sand) for testing completed in a laboratory test box setting. As per the findings of Siekmeier et al. (2014), this variability was attributed to the amount of human interaction / influence included with the test procedure.

For BCD testing within a Proctor mould (laboratory based testing), Weidinger and Ge (2009) reported the testing of a number of moulds of compacted, single source, low plasticity silt produced a variation of 4% about the mean value.

**Advantages**

- Rapid test method (5 seconds per test) – quickest of all QA tools assessed herein.
- Can be used in both laboratory and field settings – target modulus threshold, and modulus vs. moisture content curve, determined by laboratory testing of compacted samples can be directly transferred to field QA testing.
- Direct measurement of site- and material-specific deformation response to applied stress / load.
- Portable equipment and small test footprint.
- Provides a modulus parameter standardised to a single test stress.
- Highly repeatable test results (as reported by developer's research).

**Disadvantages / Limitations**

- Stress and strain level of test (very small) is not representative of foundations / traffic loading, producing a modulus parameter that is incompatible for direct use in design.
- Limited depth of penetration / zone of influence compared to other surface based *insitu* modulus QA test measurement tools.
- Limited range of applicable materials (5 MPa $\leq$ E $\leq$ 150 MPa), which eliminates most unbound pavement / aggregate / processed materials from being assessed by BCD.
- Variable test results (high CoV of results reported by some validation test programs), and reported parameter is dependent on operator interaction with equipment.

**Existing Literature regarding use as compaction QA test**

Initial correlation between the BCD returned modulus (defined herein as $E_{BCD}$) parameter and reference field modulus measurement via 150 mm diameter plate load tests ($E_{PLT}$) were completed by Briaud. et al. (2009). Ten tests were completed upon 10 material units – sands, compacted road base and crushed lime compacted ground materials – to compare the reported parameters over a large stiffness range (18 MPa < $E_{PLT}$ < 96 MPa). Combined, the modulus parameters were found to be related as per Equation 4-11 with a high strength correlation coefficient ($R^2$) of 0.93.

$$E_{PLT} = 0.90 \times E_{BCD} \qquad \text{(Equation 4-11)}$$

Both Briaud et. al. (2006) and Lenke et. al. (2003) have demonstrated that the BCD derived modulus parameter is moisture sensitive, and a relationship for compacted material can be constructed in the laboratory via use of the same samples / moulds as dry density testing. As the BCD derived modulus was demonstrated to be more sensitive to the sample's moisture condition than the relative dry density (RDD) parameter, the authors argued that the $E_{BCD}$ was a better parameter for use in QA of achieved compaction. For a Proctor (modified) compactive effort the shape of the $E_{BCD}$ parameter versus moisture content mirrors that of the dry density versus moisture content curve (i.e. peak $E_{BCD}$ observed at OMC). However, Weidinger and Ge (2009) reported that if a standard compactive effort was applied, the $E_{BCD}$ has a peak at a lower moisture content than the OMC value observed in the dry density relationship.

Briaud et. al. (2009) also attempted to correlate the BCD derived modulus parameter with the resilient modulus ($M_R$) parameter for samples obtained from two (2) locations in Texas. Although a very strong linear relationship was defined between $E_{BCD}$ and $M_R$ parameters for each location ($R^2$ > 0.95), the ratio between the materials was significantly different ($M_R$ = 0.71 and 1.255 times $E_{BCD}$). This illustrated the material and site specific nature of the relationship, with recommendations that if local correlations were developed then the BCD could be used extensively to estimate $M_R$ parameter in the field.

Weidinger and Ge (2009) evaluated the BCD for soil compaction control in Illinois, USA upon low plasticity silt materials. The BCD testing was undertaken in laboratory moulds after the material had been subjected to a Proctor (modified) compaction effort. Via comparison with the results of identical ultrasonic pulse velocity testing, a relationship between the $E_0$ and $E_{BCD}$ could be defined as per Equation 4-12 (likely specific for the material specified).

$$E_0 = 5.24 \times E_{BCD} + 26.642 \qquad (R^2 = 0.82) \qquad \text{(Equation 4-12)}$$

No Australian based QA testing or research associated with the BCD instrument was identified to exist with published literature.

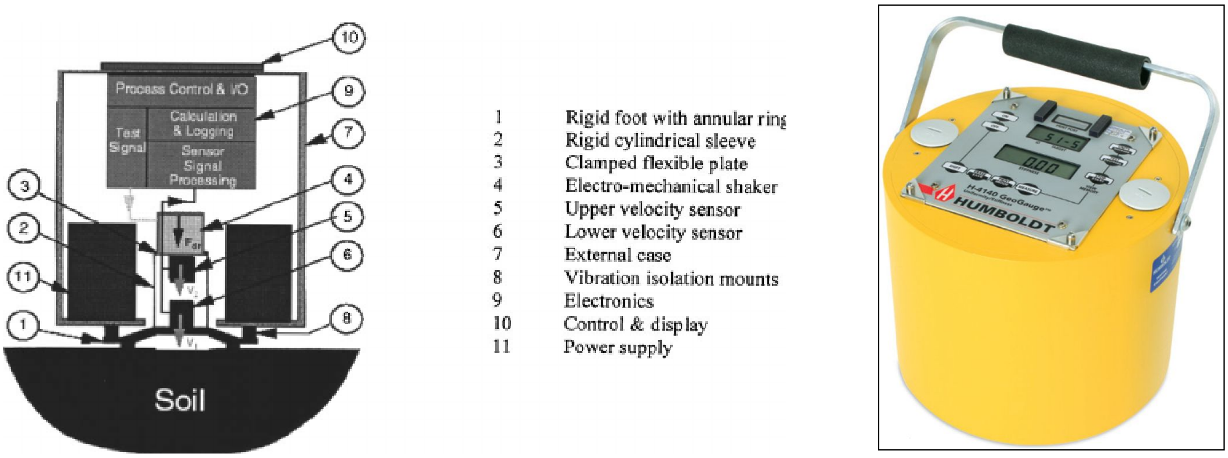
### 4.3.5 Geogauge (Soil Stiffness Gauge)

The GeoGauge – alternatively referred to as the Soil Stiffness Gauge (SSG) – is hand portable equipment manufactured by Humboldt Mfg. Co. (Illinois, U.S.A.). The origin of the instrument was by defence industries for the field detection of land mines.

**Test Equipment and Methodology**

The Geogauge is a surface based plate stress test that measures the impedance of near-surface materials under known loads. The gauge imparts very small displacements of the soil ($\leq$ 1.27 x $10^{-6}$ m) under 25 programmed steady-state frequencies between 100 and 196 Hz. The stiffness (modulus) parameter returned by the gauge – the Geogauge Stiffness ($H_{SG}$) parameter – is the average stiffness observed across all 25 frequencies.

Figure 4-17 shows a schematic representation of the Geogauge. The electro-mechanical shaker generates the small dynamic force (~9 N) that is imparted to the soil via the contact annular ring. A geophone within the instrument's body measures the resulting deflection to produce the load / deflection (stiffness) response of the material and a Geogauge Stiffness ($H_{SG}$) parameter is reported. A typical single test of the geoguage takes approximately 1 minute.




(a) (b)

**Figure 4-17 Geogauge (Soil Stiffness Gauge) Equipment – (a) Schematic cross section of instrument (from Alshibli et. al., 2005); and (b) Instrument with external case housing (www.humboltmfg.com)**

The geogauge weighs about 10 kg, has a diameter of 280 mm and a height of 254 mm. The rigid annular ring that is fixed to the base of the external housing and is required to be in effective contact with the ground surface during the test has outside and inside diameters of 114 mm and 89mm respectively, and has a thickness of 13 mm. The instrument is powered by 6 D-Cell disposable batteries, and the memory has the capacity to record approximately 500 measurements.

Sawangsuriya et al. (2002) identified that the geogauge mobilises very small soil displacements, with the resulting geogauge reported ($H_{SG}$) parameter representing the *insitu* modulus at a $10^{-3}$% to $10^{-2}$% (very low) strain condition. The geogauge reported stiffness parameter $H_{SG}$ is then – via user input of the material's Poisson's ratio – transformed into an elastic (Young's) modulus parameter, as per Equation 4-13.

$$E_{SG} = H_{SG} [(1 - v^2) / (1.77 \times R)] \qquad \text{(Equation 4-13)}$$

Where:

$E_{SG}$ = Elastic (Young's) Modulus as measured by the Geogauge (very low strain)

$H_{SG}$ = Geoguage stiffness reading

R = Radius of geogauge footing / annulus (57.15mm)

$\nu$ = Poisson's Ratio

**Zone of Influence (Test penetration depth)**

Due to its lightweight nature and small footprint, the geogauge has a limited zone of influence. The manufacturer reports the tool was specifically developed to provide composite stiffness parameters over a depth of between 220 to 300 mm in most soils (and that its impedance function was "matched" for such a depth interval).

Meher et. al. (2002) evaluated the geogage and reported a typical measurement depth of between 150 and 250mm. Softer materials have a lower zone of influence while stiffer soils could expect a greater penetration depth. From their laboratory testing evaluation program, Abu Farasakh et. al. (2004) reported an influence depth of between 190 and 200mm.

Based on these measurements, the geogauge is promoted as an effective method of assessing the composite stiffness parameter throughout a single compacted layer (i.e. 300mm loose layer thickness compacted to 200mm). However, the geogauge does not offer any improvement in terms of test depth / zone of influence in comparison with the effective testing depth of existing density QA tests (sand replacement or nuclear density).

**Applicable Standards**

- ASTM D6758-08 – *Standard Test Method for Measuring Stiffness and Apparent Modulus of Soil and Soil-Aggregate In-Place by Electro-Mechanical Method.*

**Parameters provided from test**

The use of the geogauge provides only a single (standardised) Geogauge stiffness ($H_{SG}$) parameter, which is then converted to *insitu* modulus ($E_{SG}$) via use of a user defined Poisson's Ratio.

This is similar to any other *insitu* modulus test detailed herein, that allows the construction of load and deformation response datasets / relationships, but require the adoption of a suitable Poisson's Ratio for conversion into Young's modulus parameter. As per other instruments, the resultant $E_{SG}$ parameter also requires further assessment based on the comparable strain between test technique and proposed site loading / development.

The $H_{SG}$ stiffness parameter is a composite value across the test's 'zone of influence', and is an average stiffness observed across all 25 frequencies that are assessed during a single test.

**Repeatability of Test Technique**

In all cases, the repeatability of the Geoguage test technique has been reported to be superior to traditional laboratory tests.

The manufacturer (Humbolt Mfg. Co., 2000) reports that the Coefficient of Variation (CoV) associated with the Geogauge instrument is significantly less than 10% – for fine grained soils the CoV is published to be less than 2% and for coarse-grained soils the CoV is below 5%.

In field trials, Von Quintas et. al. (2009) reported a CoV of 15%, whilst Abu-Farsakh et. al. (2004) reported CoV values of between 0.4 and 11.4%. Both of these researchers reported a material dependent CoV and that the CoV was highest when the modulus parameter was lowest (e.g. CoV > 10% when $E_{SG}$ < 50MPa).

More recently, Nazarian et. al. (2014) assessed the accuracy of the Geogauge by repeated testing upon laboratory prepared samples. For samples that had an average reference modulus (E) of 42.8 MPa, and range of 62 MPa (i.e. +/- 31 MPa), the CoV associated with equipment and operator variation was reported to be 11% to 14%. Combined with inherent material variation, the total CoV observed within Geogauge determined moduli was 24%.

**Advantages**

- Direct measurement of site- and material-specific deformation response to applied stress / load
- Portable equipment and small test footprint
- Rapid test (1 – 2 minutes per test)
- Repeated tests can be used to observe increase in modulus parameter due to additional compaction effort applied

**Disadvantages / Limitations**

- Stress and strain level of test (very small) is not representative of foundations / traffic loading, producing a modulus parameter that is incompatible for direct use in design
- Limited depth of penetration / zone of influence compared to other surface based *insitu* modulus QA test measurement tools.
- Appropriate / effective contact between annulus of Geogauge in practice frequently requires additional preparation of test footprint, which in turn may affect the resultant stiffness parameter
- Resultant stiffness parameter is sensitive to water content
- Test technique is sensitive to presence of shrinkage cracks and voids within materials – not applicable to aggregate or granular materials containing less than 20% fines, and thus excluding may processed granular aggregate / pavement construction materials.
- Results can be affected by construction traffic operating immediately adjacent to test location

**Existing Literature regarding use as compaction QA test**

As the manufacturer promotes the use of the Geogauge as an alternative tool for compaction QA, a number of studies have evaluated the performance of the geogauge within compaction projects. Frequently, the geogauge has been trialled alongside a number of other innovative assessment techniques (e.g. DCP, LFWD or SPSA).

Humbolt (2000) initially issued a report that assessed the suitability of the Geogauge for assessing density of compacted earthworks materials, by the application of a theoretical relationship between shear modulus and density. Table 4-21 details the proximity of these side-by-side nuclear density gauge and geogauge determined density values.

**Table 4-21 Variation between side-by-side testing of geogauge and nuclear density gauge (Humbolt, 2000)**

| Difference between Nuclear Density Gauge and Geogauge measured density | Frequency |
|---|---|
| Less than 5% variation | 88% |
| Between 5% and 10% variation | 10% |
| Between 10% and 15% variation | 2% |

As described by Humbolt (1999) and Meher et. al. (2002), a number of regulatory and research agencies attempted to replicate such a correlation between Geogauge determined stiffness and density parameters, and thus use the Geogauge for compaction control. They found mixed results were being reported, as summarised in Table 4-22. This is considered to demonstrate that the relationship between stiffness and density is not universal, can be influenced by many factors, and any derived relationship is largely material- and site-specific.

**Table 4-22 Pre-2002 research assessing the potential for use of geogauge for compaction control**

| Strength of relationship between insitu density as measured by Geogauge and traditional testing | Regulatory Authority / Organisation |
|---|---|
| Good | North Carolina Department of Transport (NCDOT)<br>H.C Nutting (now Terracon Consultants)<br>Federal Highway Administration (FHWA) Laboratory testing |
| Fair | California Polytechnic State University<br>Michigan Department of Transport (MDOT) |
| Poor | City of San Jose<br>Florida Department of Transport (FDOT)<br>New York State Department of Transport (NYSDOT) |

The use of the geogauge to demonstrate the effect of the addition of compaction effort (i.e. increased modulus parameter due to increased number of roller passes) was evaluated by Lenke et. al. (2003) and Meehan et. al. (2012). Mixed results have been observed, with Lenke et. al. (2003) and Edil & Sawangsuriya (2005) reporting observable increase in $E_{SG}$ with additional roller passes whilst both Meehan et. al. (2012) and Rose (2013) have found the $E_{SG}$ parameter insensitive to roller effort / *insitu* density. These variable findings are interpreted to be likely due to the varied materials and compaction state / change assessed by each study.

The focus of a large number of researchers has been to validate the use of the geogauge against industry accepted assessment of modulus parameters – either via static (Plate Load Test, $E_{PLT}$), dynamic (trailer mounted Falling Weight Deflectometer, $E_{FWD}$) or laboratory (resilient modulus, $M_R$) measurements. The relationships previously reported include those detailed in Table 4-23, although Puppala (2008) identifies that more research is required to develop appropriate factors to determine the design resilient modulus parameter from *insitu* modulus values.

**Table 4-23 Defined relationships between Geogauge stiffness ($H_{SG}$) or equivalent Young's Modulus ($E_{SG}$) and reference deformation test techniques**

| Correlation of Geogauge with - | Defined Relationship** | Correlation Coefficient ($R^2$) | Researcher |
|---|---|---|---|
| Resilient Modulus ($M_R$) | $M_R = (37.654 \times H_{SG}) - 261.96$ | $R^2 = 0.82$ | Chen et. al. (1999) |
| | $M_R = 22.69e^{(0.12 \times HSG)}$ | Unknown | Wu et. al. (1998) |
| | Cohesive Soils:<br>$M_R = [86.7 \times (E_{SG}^{0.3}) / w(\%)] + (2.2 \times \gamma_d)$ | $R^2 = 0.60$ | Gudishala (2004) |
| | Granular Soils:<br>$M_R = 20.3 \times E_{SG}^{0.54}$ | $R^2 = 0.83$ | Gudishala (2004) |
| Plate Load Test ($E_{PLT}$)* | $E_{PLT(i)} = (0.3388 \times E_{SG}) + 84.7$<br>(initial PLT loading cycle) | $R^2 = 0.66$ | Nelson & Sondag (1999) |
| | $E_{PLT(i)} = (1.62 \times E_{SG}) - 75.58$<br>(initial PLT loading cycle) | $R^2 = 0.87$ | Abu-Farsakh et. al. (2004) |
| | $E_{PLT(R2)} = (0.8962 \times E_{SG}) + 25.9$<br>(PLT re-loading cycle) | $R^2 = 0.23$ | Nelson & Sondag (1999) |
| | $E_{PLT(R2)} = (1.50 \times E_{SG}) - 65.37$<br>(PLT re-loading cycle) | $R^2 = 0.90$ | Abu-Farsakh et. al. (2004) |
| | $E_{PLT} = (0.66\ E_{SG}) + 11.65$ | $R^2 = 0.83$ | Baus and Li (2006) |
| Falling Weight Deflectometer (FWD) | $E_{FWD} = (47.53 \times H_{SG}) + 79.05$ | Unknown | Wu et. al. (1998) |
| | $E_{FWD} = (1.17 \times E_{SG}) - 20.07$ | $R^2 = 0.81$ | Abu-Farsakh et. al. (2004) |
| California Bearing Ratio (CBR) | $CBR = 0.00392 \times (E_{SG})^2 - 5.75$ | $R^2 = 0.84$ | Abu-Farsakh et. al. (2004) |

* Note that PLT plate diameters and stress level for $E_{PLT}$ parameter calculation may vary between researchers.
**Note that $E_{SG}$ parameter may be required to be input in MPa or ksi depending on relationship (refer to researcher specific study).

Based on the developed relationships, specifications that define geogauge modulus parameter thresholds to material QA assessment have been developed, such as that presented in Table 4-23.

**Table 4-24  Example of QA specification incorporating geogauge measured stiffness parameter**

| Base Material Type / Quality | Geogauge Modulus ($E_{SG}$) | Falling Weight Deflectometer ($E_{FWD}$) | Researcher |
|---|---|---|---|
| Weak | < 87 MPa | < 140 MPa | Chen et. al. (1999) |
| Good | 156 – 209 MPa | 310 – 450 MPa | |
| Excellent | > 261 MPa | > 700 MPa | |
| Sand @ 90% RDD | 205 MPa | N/A | Farrag et. al. (2005) |
| Silty Clay @ 90% RDD | 310 MPa | | |
| Granular / Stone Base @ 90% RDD | No Correlation (due to equipment seating issues) | | |

A large number of researchers – e.g. Swensen et. al. (2006); Abu-Farsakh et. al. (2004); Baus and Li (2006) – note that the moisture content of the material at the time of testing influences the reported *insitu* stiffness parameter, especially for cohesive materials. Von Quintas et. al (2009) indicated fine grained (clay) materials did not directly correlate with laboratory measured modulus parameters, whilst granular materials did replicate laboratory measurements. Gudishala (2004) further demonstrated this dependency for cohesive soils, and incorporated both the density and water content of the tested material into a relationship between laboratory measured resilient modulus and the geogauge determined modulus parameter (reproduced in Table 4-23).

Other issues with the implementation of the geogauge for field based QA assessment have been reported by various researchers. This includes issues with seating of the instrument such that suitable contact with the ground is achieved, or that the placement of bedding sand prior to testing may alter the measured stiffness parameter (e.g. Simmons, 2000; Miller and Mallick, 2003). Granular materials were reported that the geoguage reported poorer correlations with density test results when sand and stone materials are tested (Farrag et. al., 2005), due to equipment seating problems. Similarly, although the manufacture identifies that equipment operating in the vicinity of geogauge testing does not interfere with the steady-state vibratory measurements, Simmons (2000) and Miller and Mallick (2003) identify that the vibration caused by these plant (and / or trains) can alter the reported modulus parameter.

Within Australia, only limited technical assessment of the Geoguage as a QA tool is available. As detailed by Drechsler and Parken (2010), the geoguage was utilised to assess *insitu* stiffness and Young's modulus parameter of rail ballast materials. As per other researchers, this study found the geogauge derived parameter was susceptible to seating conditions within the coarse ballast material. Although it is understood that the geogauge has successfully been utilised for verification of achieved stiffness at other QA test results within major construction projects in Queensland, Australia, limited published works regarding this use is currently available.

### Assessment of GeoGauge for use as QA Tool

The geogauge is assessed to offer a high potential for use as a earthworks QA tool, as long as the QA procedure is based on the verification / acceptance of insitu modulus parameter rather than attempting to relate the geogauge reported stiffness parameter to achieved insitu density. The speed of testing, ease of operation, portability and low CoV associated with test results make it an ideal QA tool. It would be expected that if suitable trial / comparison testing was completed at the outset of a project, the GeoGauge could be accepted as a valid QA method during production earthworks and pavement installation.

However, within any specification developed for the GeoGauge case must be taken to ensure that the following items are accounted for: (i) site / material specific relationship to reference modulus parameter (e.g. PLT, FWD or resilient modulus verification) noting that relationships would be expected to be significantly different for cohesive and granular materials; (ii) modulus parameter variation based on moisture content variation (i.e. material sensitivity); and (ii) modulus reduction required based on comparative stress and strain magnitudes induced by GeoGauge test and during lifetime of development.

Due to the limited zone of influence associated with the GeoGauge (200mm), the use of the tool is limited to conventional earthworks lift thicknesses (300mm loose layer compacted to 200mm).

# 5. Assessment / Ranking of Reviewed QA Tests

This section presents a comparison of some of the testing equipment currently available and are either not considered or under-utilised by Road authorities. This may be either due to a lack of familiarity of the capabilities of the testing instruments, or the presence of existing Australian Standards that detail a standardised test methodology. Each sub-section considers and ranks the 'innovative' equipment detailed in Section 4 of this document in terms of specific aspects – e.g. repeatability, cost, test duration and effective depth of testing ('zone of influence'). Section 5.3 then provides an overall summary of the individual rankings applicable to each of techniques considered, and weights each of the indivudal rankings to produce an overall comparative ranking of the suitability / attractiveness of each instrument for use as an earthworks / pavement materials QA tool.

Note that these assessments and rankings do not encompass all equipment currently commercially available on the market, but focuses on "easy-to-accept" testing. Most of these have been around for over 2 decades and "new / recent" equipment developed over the past 10 years have not been adequately examined (likely due to ARRB funding limitations that prevents the thorough examination of all potential equipment).

## 5.1 Comparative of available QA Techniques

Table 5-1 details the comparative assessment of the reviewed QA equipment / techniques in their suitability to directly measure the stiffness of placed and compacted subgrade and pavement materials. Table 5.2 details the comparative assessment of the reviewed QA equipment / techniques in terms of the costs – presented as principal and per test costs - and the duration to complete a field test.

**Table 5-1 Comparison of QA Test Techniques – Measured Parameter, Repeatability and Applicability**

| QA Technique | Measured Parameter | Required Correlation to Modulus | Can Asses Stress Dependency | Repeatability of Field Test (uniform material) | Reported Strength of Correlation to Modulus / Suitability to test materials | | | Noted Issues |
|---|---|---|---|---|---|---|---|---|
| | | | | | Cohesive | Sand | Gravel | |
| DCP | Rod Penetration Rate | Yes (DCP → E) | No | < 60% | Poor | High | Medium | Results heavily affected by:<br>particle size;<br>moisture content |
| PANDA Probe | Cone Tip Resistance | Yes ($q_d$ → E) | No | < 30% | Medium | High | High | Results affected by particle size |
| Plate Load Test | Modulus (Stress-Deformation) | No | Yes | < 10% | High | High | High | Reference modulus test |
| LFWD (Prima 100) | Modulus (Stress-Deformation) | No | Yes | < 15% | Medium | High | High | Load Cell allows range of test stress responses to be considered |
| LFWD (Zorn) | Modulus (Deformation only) | No | No | < 15% | Medium | Medium | Medium | Absence of load cell limits functionality |
| Clegg Hammer | Clegg Impact Value (CIV) | Yes (CIV → E) | Yes | < 15% | Poor | Medium | Medium | No direct measure of stress - deflection |
| Geogauge (Soil Stiffness Gauge) | Modulus (Harmonic Stress-Deformation) | No | No | 0 – 30% | Poor | Medium | Medium | Issues with testing:<br>fine grained soils with high moisture contents;<br>dry sands<br>Uses small strain Stiffness |
| Borehole Shear Tester | Insitu Mohr-Coulomb Strength Parameters | N/A (Strength Test) | N/A | N/A | High | Medium | Poor | Requires borehole to complete.<br>Testing of dry non-cohesive materials difficult |

**Table 5-2 Comparison of QA Test Techniques – Cost and Test Duration**

| QA Technique | Approx. Principal Cost of Equipment ($AUD) | Yearly Calibration / Consumable Costs | Duration (per test) | | | | Typical Turnaround of Test Results | No. of Tests Per Day | Comments |
|---|---|---|---|---|---|---|---|---|---|
| | | | Setup | Field Test | Laboratory | Interpretation / Reporting | | | |
| DCP | $2,000 | < $500 | 5 Mins | 10 Mins | N/A | 10 Mins | < 1 Day | ≤ 30 | External recording generally required |
| PANDA Probe | $27,500 | < $500 | 5 Mins | 10 Mins | N/A | 10 Mins | < 1 Day | ≤ 30 | PDA provides details parameter for each blow |
| Plate Load Test | $30,000 (electronic) | NIL | 15 Mins | ≥ 1 Hour | N/A | 30 Mins | 1 Day | 2 - 4 | Requires External Reaction Force to be provided |
| LFWD (Prima 100) | $17,000 | $2,000 | 5 Mins | ≤ 5 Mins | N/A | 5 Mins | < 1 Day | ≤ 100 | PDA records full data history |
| LFWD (Zorn) | $9,000 | $2,000 | 5 Mins | ≤ 5 Mins | N/A | 5 Mins | < 1 Day | ≤ 100 | Limited Information record |
| Clegg Hammer | $12,000 (9.1kg Hammer) | < $500 | 5 Mins | ≤ 5 Mins | N/A | 15 Mins | < 1 Day | ≤ 100 | External information record required |
| Geogauge (Soil Stiffness Gauge) | $15,000 | < $500 | 5 Mins | 75 secs | N/A | 5 Mins | < 1 Day | ≤ 100 | Limited Information record |
| Borehole Shear Tester | $22,000 | < $500 | 30 Mins | 20 Mins | N/A | 15 Mins | 1 Day | ≤ 10 | Requires auger borehole to complete |
| Nuclear Density Gauge | $10,000 | $2,000 | 5 Mins | 60 Secs | 24 Hours | 15 Mins | ≥ 3 Days | ≤ 30 | Laboratory Testing for Oversize / MDD required<br>Density Measurement ≠ Modulus |
| Sand Replacement Test | $750 | < $500 | 5 Mins | ≥ 30 Mins | 24 Hours | 15 Mins | ≥ 3 Days | ≤ 10 | Laboratory Testing for Oversize / MDD required<br>Density Measurement ≠ Modulus |

## 5.2 Comparative of available QA Techniques – Zone of Influence of Test

Table 5-3 details the comparative assessment of the reviewed QA equipment / techniques in terms of the depth achieved by the technique.

**Table 5-3 Comparison of QA Test Techniques – Depth (Zone of Influence) of Test**

| QA Technique | Measured Parameter | Zone of Influence | |
|---|---|---|---|
| | | Description | Example |
| DCP | Rod Penetration Rate | Rod Penetration Length | N/A |
| PANDA Probe | Cone Tip Resistance | Rod Penetration Length | N/A |
| Plate Load Test | Modulus (Stress-Deformation) | 2.0 x Plate Diameter | 300mm Plate = 600mm |
| LFWD (Prima 100) | Modulus (Stress-Deformation) | 1.0 to 1.5 x Plate Diameter | **300mm Plate** = 300 – 450mm |
| LFWD (Zorn) | Modulus (Deformation only) | 0 to 1.5 x Plate Diameter | **300mm Plate** = 300 – 450mm |
| Clegg Hammer | Clegg Impact Value (CIV) | Varies based on hammer weight and drop height | 10kg Hammer = 200 – 300mm |
| Geogauge | Modulus (Stress-Deformation) | 150 – 250mm | 150 – 250mm |
| Borehole Shear Tester | Insitu Mohr-Coulomb Strength Parameters | Immediate Location of Test | N/A |
| Sand Replacement Test | Relative Dry Density (RDD) | Depth of Excavation (Max. 300mm) | Typically 200 – 250mm |
| Nuclear Density Gauge | Relative Dry Density (RDD) | Depth of Probe (Max. 300mm) | Typically 200 – 250mm |

## 5.3 Comparative of available QA Techniques – Summary Assessment

Table 5-4 consolidates the findings presented in Table 5-1 through Table 5-3, and summarises the suitability of each test in terms of cost, speed and total turn-around time in the provision of results. In all cases presented in Table 5-4, 1 star is least desirable whilst 5 stars represents the best suited test.

**Table 5-4 Summary Assessment of QA Test Techniques – Cost, Speed of test and duration between testing and provision of results**

| QA Technique | Measured Parameter | Ratings: 1 Star = Least Advantageous (Highest Cost / Lowest Productivity; 5 Star = Most Advantageous (Lowest Cost / Highest Productivity) | | | |
|---|---|---|---|---|---|
| | | Principal Cost | Operating Cost Per Test | No. of Tests / Day | Total Test Result Turnaround Time |
| DCP | Rod Penetration Rate | ★★★★☆ | ★★★★☆ | ★★★★☆ | ★★★★☆ |
| PANDA Probe | Cone Tip Resistance / Blow | ★☆☆☆☆ | ★★★★☆ | ★★★★☆ | ★★★★☆ |
| Plate Load Test | Insitu Modulus (Stress-Deformation) | ★☆☆☆☆ | ★☆☆☆☆ | ★☆☆☆☆ | ★★★☆☆ |
| LFWD (Prima 100) | Insitu Modulus (Stress-Deformation) | ★★★☆☆ | ★★★★☆ | ★★★★★ | ★★★★☆ |
| LFWD (Zorn) | Insitu Modulus (Deformation only) | ★★★★☆ | ★★★★☆ | ★★★★★ | ★★★★☆ |
| Clegg Hammer | Clegg Impact Value (CIV) | ★★★☆☆ | ★★★★☆ | ★★★★★ | ★★★★☆ |
| Geogauge (Soil Stiffness Gauge) | Insitu Modulus (Harmonic Stress-Deformation) | ★★★☆☆ | ★★★★★ | ★★★★★ | ★★★★☆ |
| Borehole Shear Tester | Insitu Mohr-Coulomb Strength Parameters | ★★☆☆☆ | ★★★☆☆ | ★★☆☆☆ | ★★★☆☆ |
| Nuclear Density Gauge | Relative Dry Density (RDD) | ★★★☆☆ | ★★☆☆☆ | ★★★★☆ | ★★☆☆☆ |
| Sand Replacement Test | Relative Dry Density (RDD) | ★★★★★ | ★☆☆☆☆ | ★★☆☆☆ | ★★☆☆☆ |

Of the reviewed penetration and surface based direct modulus measurement devices, the potential for each technique to be effectively utilised as a QA tool for earthworks compacted received a weighted rating – converted to a percentage – based on the following weighted criteria (in order):

- Accuracy, repeatability and reliability of equipment (30%)
- Requirement / Duration / Ease of results processing to report measured parameter (25%)
- Duration of field completion of test (20%)
- Operating Cost (15%)
- Principal Cost (10%)

The traditional density test techniques were also evaluated as reference values in this assessment.

The results of this assessment are summarised inTable 5-5. This ranking demonstrates that the completed study concluded that all of the considered 'innovative' test techniques had the potential to provide more appropriate *insitu* parameters (i.e. modulus or index relatabile to modulus) than the traditional density test techniques, and largely within a quicker timeframe. Based on these advantages the ‘innovative’ test techniques were considered to be: (i) more attractive and appropriate for use in the field than the density tests associated with current Earthworks Specifications; and (ii) overcome the limitations associated with the ‘accepted’ test procedures that are currently used commonly within the geotechnical industry.

**Table 5-5 Overall comparative assessment of available QA Test Techniques – traditional / existing vs innovative**

| QA Technique | Test Status | Measured Parameter | Total Ranking (%) | Overall Comparative Assessment |
|---|---|---|---|---|
| Sand Replacement Test | Traditional Density Test – Reference Value | Relative Dry Density (RDD) | 71 | ★★★☆☆ |
| Nuclear Density Gauge | Traditional Density Test – Reference Value | Relative Dry Density (RDD) | 66 | ★★☆☆☆ |
| DCP | Traditional / Accepted – Penetration Test | Rod Penetration Rate | 58 | ★☆☆☆☆ |
| PANDA Probe | Innovative – Penetration Test | Cone Tip Resistance / Blow | 74 | ★★★☆☆ |
| Plate Load Test | Traditional / Accepted – Modulus Test | Insitu Modulus (Stress-Deformation) | 52 | ★☆☆☆☆ |
| LFWD (Prima 100 Model) | Innovative – Modulus Test | Insitu Modulus (Stress-Deformation) | 82 | ★★★★★ |
| LFWD (Zorn Models) | Innovative – Modulus Test | Insitu Modulus (Deformation only) | 78 | ★★★★☆ |
| Clegg Hammer | Innovative – Modulus Test | Clegg Impact Value (CIV) | 78 | ★★★★☆ |
| Geogauge (Soil Stiffness Gauge) | Innovative – Modulus Test | Insitu Modulus (Harmonic Stress-Deformation) | 79 | ★★★★☆ |
| Borehole Shear Tester | Innovative – Insitu Strength Parameter Test | Insitu Mohr-Coulomb Strength Parameters | 66 | ★★☆☆☆ |

Of the ‘innovative’ test techniques, the ‘Surface Based Impact Devices’ that allow direct measurement of the *insitu* modulus parameter (load vs. settlement) was considered to be more attractive than penetration test devices. Furthermore, the LFWDs fitted with both a load-cell and geophones in contact with the ground (e.g Prima 100 model, Grontmij / Sweco brand equipment) were evaluated to offer greater flexibility in terms of usage, and thus higher attractiveness for use as QA tools, than those that were not fitted with load-cells and with plate mounded accelerometers (e.g. Zorn brand LFWD models).

# 6. Conclusions

Year 1 of this project has completed the literature review (this document).

The literature review concluded that there is a high potential for the new QA methods to be utilised in construction sites. These new methods not only will provide a direct measure of the *insitu* modulus value, but also can lead to reducing time delay caused by traditional density measurement methods. Year 1 also prepared a summary discussion paper with the intent to generate and to help identify upcoming TMR projects for remainder of this project.

Independently, the Toowoomba Second Range Crossing (TSRC) full scale trial embankments occurred during the period of this literature review (2016 / 17). TSRC had project specific objectives, but highlighted the lag between ongoing projects and the benefits of alternative *insitu* Q.C. testing. Rather than repeatedly assessing each proposed QA / QC tool via project specific trials, a more “universal” approach is here advocated to assess the potential for use of each tool within road project earthworks / pavement QA/QC framework. Note that limited material specific calibrations would still be required, similar to the requirement of calibration of nuclear density gauges for use of individual projects.

The relative industry ‘attractiveness’ of different equipment are summarised in the Figure 6-1, plotted within a matrix against the assessed current usage / ‘acceptance’ of each tool within industry. The ‘attractiveness’ to industry of each considered instrument is based on the derived weighted ranking detailed in Section 5.3 of this report.

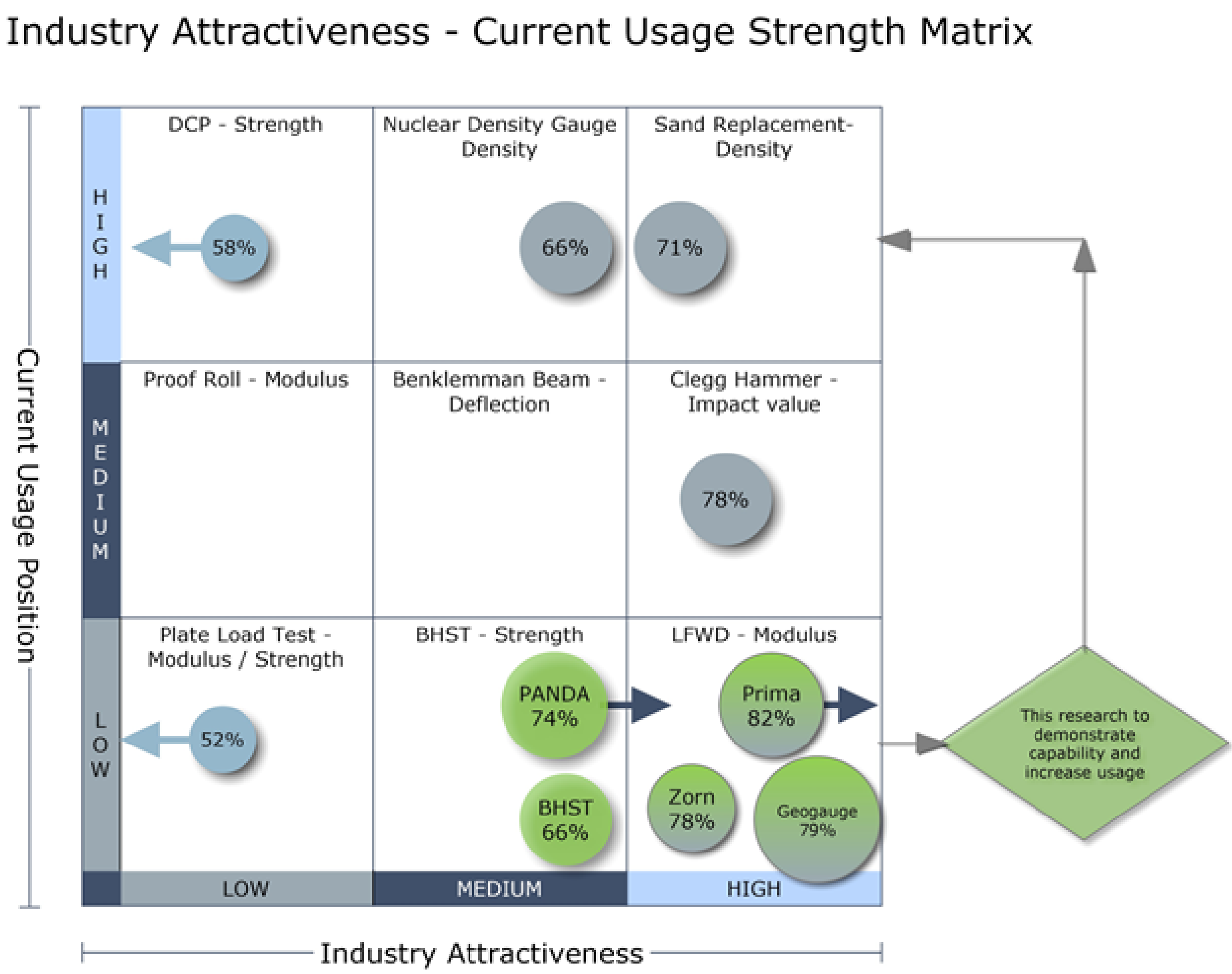



**Figure 6-1 Current industry acceptance / usage of innovative QA/QC tools compared to potential for use (industry attractiveness)**

Figure 6-1 identifies that available ‘innovative’ equipment potentially suitable for use as QA / QC tools (i.e. industry ‘attractiveness’) lags significantly behind the current usage of such tools. Similarly, as demonstrated throughout this report, the use of ‘innovative’ equipment has the potential to provide results that are superior to the traditional measures of density (i.e. high current usage tools) – i.e. by the direct measure of *insitu* stiffness parameters rather than the reliance on the assumed (and potentially incorrect) generic correlation between density and material stiffness / strength properties.

The next phases of study (Year 2 and beyond) that requires funding has three (3) broad objectives:

1. Demonstrate the reliability, advantages and limitations of nominated ‘innovative’ equipment that is not currently widely used, by applying to “live” projects
2. Knowledge Transfer of available ‘innovative’ QA / QC tools to road authorities and industry, and the potential advantages offered by their adoption (e.g. cost, time, deeper lifts, better field verification of design parameters).
3. Documentation of standard method of testing, and the assessment of material specific properties arising from use of nominated ‘innovative’ equipment, for potential use as QA / QC acceptance thresholds for project specifications.

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Please contact the undersigned if any further information or clarification is required.

Regards,

**Dr Dr. David Lacey**
Senior Geotechnical Engineer
Foundation Specialists Group

**Dr. Burt Look**
Principal Geotechnical Engineer