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Guidelines for the Structural Evaluation of Road Pavements

DN-PAV-03061

November 2024

DN Design

Technical

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Table of Abbreviations

Heading	Definition
AC	Asphalt Concrete
AMD	Absolute Mean Deviation
DCP	Dynamic Cone Penetrometer
DMI	Distance-Measuring Instrument
DPT	Dynamic Plate Test
FWD	Falling Weight Deflectometer
GPS	Global Positioning System
GNSS	Global Navigation Satellite System
GPR	Ground Penetrating Radar
HBM	Hydraulically Bound Material
HRA	Hot Rolled Asphalt
HWD	Heavy Weight Deflectometer
IA	Implementation Authority
IAPDM	Irish Analytic Pavement Design Method
INAB	Irish National Accreditation Board
IRT	Infra-red Thermometer
ITM	Irish Transverse Mercator
LA	Local Authority
LTE	Load Transfer Efficiency
LWD	Light Weight Deflectometer
ME	Mechanistic-Empirical
NM	Network Management
PAMS	Pavement Asset Management System
PARR	Pavement Asset Repair and Renewal
PQC	Pavement Quality Concrete
QA	Quality Assurance

Heading	Definition
SCI	Surface Curvature Index
QMS	Quality Management System
RMS	Root Mean Squared
SMA	Stone Mastic Asphalt
TAF	Temperature Adjustment Factor
TII	Transport Infrastructure Ireland
TRL	Transport Research Laboratory
UGM	Unbound Granular Material
VI	Void Intercept
WMAPT	Weighted Mean Annual Pavement Temperature

1. Introduction

This publication provides guidance on the use of the Falling Weight Deflectometer (FWD) deflection data for the assessment of National Road pavements in Ireland. It should be read in conjunction with TII publication, DN-PAV-03060 Test Method for the Structural Evaluation of Road Pavements, which specifies the requirements for FWD equipment and how measurements are to be collected, analysed and reported.

The focus of this publication, when used in conjunction with DN-PAV-03060, is to accurately characterise the structural condition of existing pavements. The resulting test data is required for use in the mechanistic-empirical thickness design procedures in accordance with TII Publication DN-PAV-03021 Analytic Pavement & Foundation Design, and the associated Irish Analytic Pavement Design Method (IAPDM) software. Deflection data collected by the FWD can be used to characterise the parameters of the pavement layers through backcalculation, in which the engineering material parameters of the pavement layers are estimated based on the measured surface deflections, the magnitude of the load, and information on the pavement layer thicknesses.

This publication provides additional best practice guidance on deflection testing of existing and new pavement structures, as well as guidance on backcalculation techniques and data interpretation procedures to analyse and present those results. The primary focus relates to the type of materials used in flexible and flexible/composite pavement structures, including FWD testing on Unbound Granular Mixtures. FWD data should be interpreted in association with other pavement condition indicators and pavement layer thickness information including visual inspections, Ground Penetrating Radar (GPR), coring, trial pits and DCP Tests. This document also describes the methods recommended for assessing pavement layer thickness, for use in the analysis of FWD deflection data.

2. Apparatus

2.1 FWD Equipment

TII publication DN-PAV-03060 Test Method for the Structural Evaluation of Road Pavements defines the Falling Weight Deflectometer (FWD) as a dynamic plate test (DPT) device which records the deflection response of road pavements. It can be a trailer mounted device which is towed behind a vehicle, or a vehicle mounted device. Additional information and guidance on the FWD equipment are provided in the following clauses.

2.1.1 General Description of FWD

FWD is used for the non-destructive deflection testing of National Road pavements in Ireland. The FWD records pavement surface deflection bowls at discrete test points on the pavement surface, by measuring surface deflections at radial distances from the centre of an impulse test load. The primary purpose of deflection testing is to determine the structural adequacy of an existing pavement and to assess its capability of supporting future traffic loadings. FWD devices are used to measure pavement deflections in response to a stationary dynamic load, similar in scale and duration to a passing wheel load.

A load cell measures the peak impact load applied to a load plate. The vertical deflections produced by the impact are measured both at the centre of the plate and at a series of radial positions from the centre. The measured deflections at these radial positions are a function of the applied load and can be used to determine the structural strength of the pavement under test. The data obtained is used to evaluate the structural capacity of pavements for research, design, rehabilitation, and pavement management purposes.

2.1.2 Load Cell

The load cell is placed centrally above the load plate. It measures the peak impact load produced when the “falling weight” is dropped on to the rubber buffers positioned above the load cell.

2.1.3 Load Pulse

The shape of the load pulse is intended to be similar to that produced by a moving wheel load. Figure 2.1 shows a typical longitudinal strain profile for a wheel moving at 100km/h on a rolled asphalt road base.

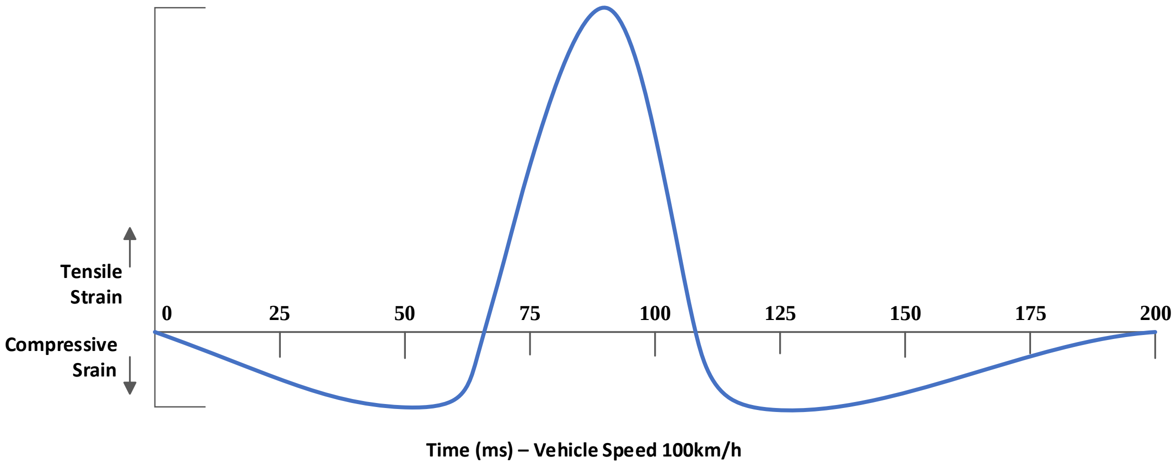

Figure 2.1 - Typical Longitudinal Strain Profile for Moving Wheel (100 km/h)

2.1.4 Load Plate

The rubber mat affixed to the underside of the loading plate may feature a grooved or similar pattern on the contact surface. The plate may be segmented to ensure good contact with the road surface. An example of a segmented load plate, with D1 central deflection sensor, is shown in Figure 2.2.

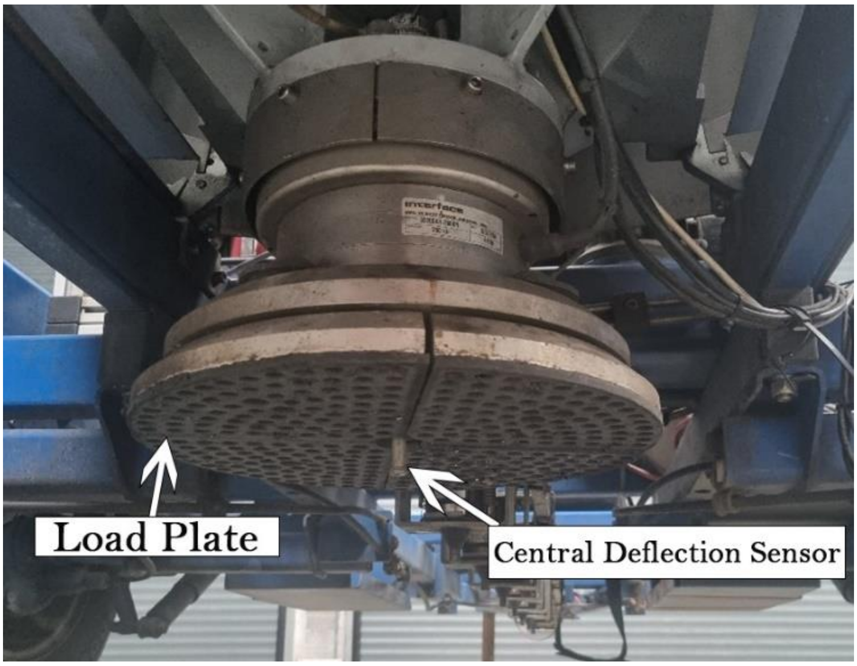

Figure 2.2 - Typical Segmented FWD Load Plate and Sensor

2.1.5 Deflection Sensors

When an FWD applies the impulse load to the pavement surface, the resulting vertical deflection of the pavement is recorded by several deflection sensors (geophone or seismometer). The deflection sensors are located at the centre of the load plate and on a radial axis from the loading plate. This results in a basin shaped deflection profile, with the deflections typically decreasing as their spacing moves away from the centre of the load application. Figure 2.3 shows a typical pavement deflection basin.

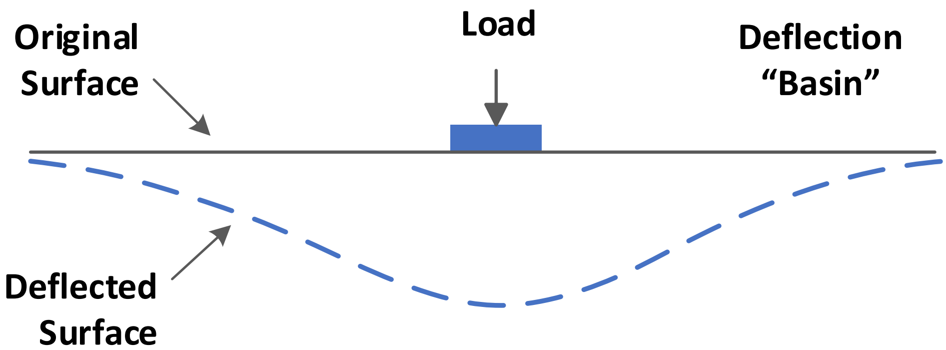

Figure 2.3 - Typical Pavement Deflection Basin

3. Accreditation and Calibration Procedures

The minimum accreditation and calibration requirements for equipment, testing and structural evaluation of road pavements are set out in TII Publication DN-PAV-03060 Test Method for the Structural Evaluation of Road Pavements.

4. FWD Measurement Procedures

4.1 Survey Method and Test Setup - General

FWD testing in live traffic requires appropriate traffic management to be in place during the survey. The appropriate traffic management in accordance with the Traffic Signs Manual (Chapter 8) should be arranged well in advance of the FWD survey being carried out. This is usually done in consultation with the relevant Implementation Authority (IA). The type of traffic management required will depend on a risk assessment of the site to comply with Section 19 of the Safety, Health and Welfare at Work Act and in accordance with the Traffic Signs Manual (Chapter 8), taking into account the traffic and geometric characteristics of the particular site.

The FWD survey can commence as soon as the traffic management has been set up.

DN-PAV-03060 specifies that the temperature range for testing flexible or flexible/composite pavements shall be between 5°C and 30°C. If testing outside of this temperature is unavoidable, for any reason, this shall be noted and reported.

4.2 Test Method for Performance Assessment of UGM

Station spacing for performance verification testing of UGM is specified as 25m intervals in DN-PAV-03060. FWD test intervals may need to be reduced on short test sections or for more detailed pavement investigation, e.g. when attempting to identify “weak” locations. A minimum of 10 test points per test section should ideally be taken to ensure a good representation of the UGM condition for the test section.

4.3 Test Method for Structural Evaluation of Existing and New Pavement Structures

The test method defined in DN-PAV-03060 applies to the use of FWD on existing or newly constructed pavements. This includes the performance assessment of new construction where some or all bound pavement layers have been laid.

4.3.1 Preparation for Measurements

As specified in DN-PAV-03060, the choice of survey alignment shall be defined for each test section to be tested prior to commencing any surveys, in agreement with the implementation authority and their designer. The location of the FWD tests will usually be governed by the information which is required from the FWD survey. The usual practice is to locate the loading plate in one of the wheel paths of the lane to be surveyed. When testing a single wheel path, FWD testing is typically carried out in the nearside wheel path to assess the line of greatest deterioration. This is often the first location to show distress signs on a road pavement. Tests can also be carried out between the wheel tracks, if required, for comparison purposes and to ascertain the residual life of the relatively untracked pavement.

For multi-lane dual carriageways, FWD tests should be carried out, at a minimum, in the heaviest loaded lane. Other lanes can be tested to satisfy particular requirements. The comparison of the heaviest loaded lane with other less loaded lanes can give additional information.

The FWD test spacing intervals are also agreed in advance of commencing surveys with the implementation authority and their designer. For testing at intervals greater than 25m on multiple lanes of a test section, test locations should be staggered across adjacent lanes.

When testing both lanes of a single carriageway two-way road with staggered test locations, the staggering of test points should be implemented as illustrated in Figure 4.1, which shows an example for testing at 50m intervals.

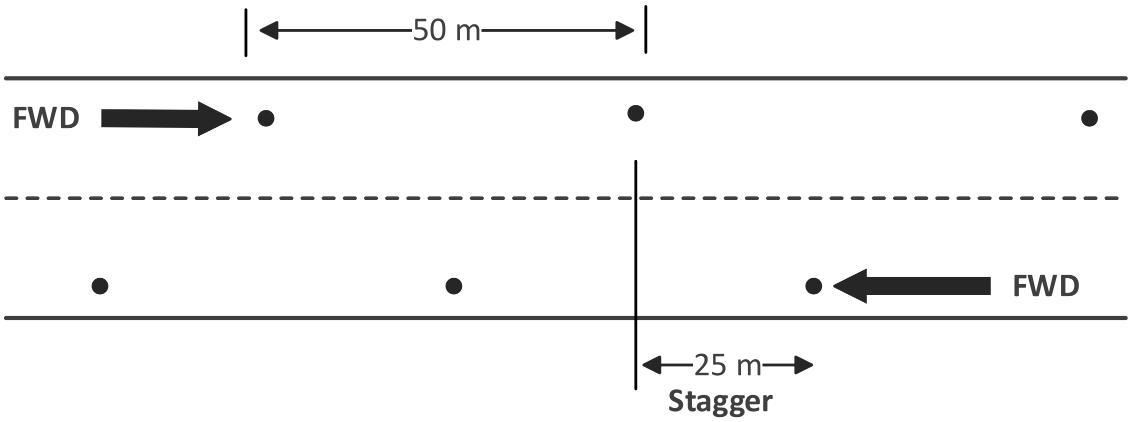

Figure 4.1 - Staggered FWD Test Points on Two Lane Road

For testing on multiple lanes of motorways or dual carriageways with staggered test locations, staggering of FWD test points across adjacent lanes should be implemented as illustrated in Figure 4.2, using an example of 50m test intervals along all lanes of a four-lane road.

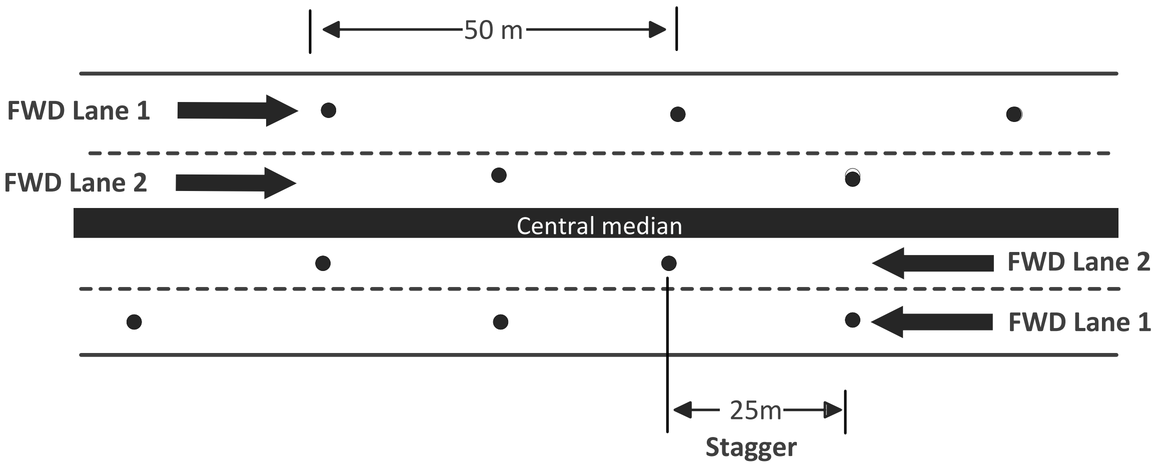

Figure 4.2 - Staggered FWD Test Points on Motorways and Dual Carriageways

FWD test intervals may need to be reduced on short test sections such as slip roads and roundabouts or for more detailed pavement investigation, e.g., when attempting to identify “weak” locations. On flexible/composite pavements with hydraulically bound material (HBM) base, some close spaced testing may be carried out to assess crack frequency and severity in the HBM. In general, a minimum of 10 test points per test section should be taken to ensure a good representation of the pavement condition.

4.3.2 FWD Setup

4.3.2.1 Load Plate and Target Load

The target load for analysis of National Roads sections is specified as 40kN (to simulate a wheel load on a standard 80kN axle) in DN-PAV-03060. The contact pressure equivalent of the target load (40kN) on a 300mm diameter plate is 566kPa. FWD deflections are normalised to the defined target load of 40kN using the equivalent target peak stress (i.e. pressure) of 566kPa by linear extrapolation or interpolation, as defined in Clause 5.2.1 of DN-PAV-03060 and described in Clause 5.2.1 of this publication. The standard axle is characterised within the IAPDM as a single wheel with 40kN half axle load and a tyre pressure of 559kPa as defined in DN-PAV-03021 (Analytic Pavement & Foundation Design). There is a slight difference possibly resulting from a simplification of the FWD load plate diameter of 300mm vs ~301.8mm. It does not affect the applicability of the FWD backcalculated stiffnesses to the IAPDM models due to possible stress dependency of the pavement materials under the FWD loading.

In certain situations, adjustments to the target load may be necessary for special purposes, which require careful interpretation. For instance, a higher load of 50kN may be applied when greater pavement deflection is needed, such as when testing heavy-duty or rigid pavement structures, or for calibration and correlation purposes.

Most FWD devices have an automatic load targeting feature so that at every measuring location the load is adjusted in such a way that the target load can be reached. This is normally targeted on the final drop in the sequence, with the two preceding drops usually at a loading below and above the target respectively, thus allowing the software's “learning” features to target the desired load more accurately.

4.3.2.2 Deflection Sensors Positions

The minimum number of sensors available is specified as seven. This is to ensure that the full influence of the load pulse on the pavement is recorded. The D1, central deflection, is at the centre of the load plate and the remaining sensors should be adjustable to be located in radial positions from 200mm to 2100mm at a minimal spacing of 100mm. The standard set-up of the sensors for testing of new and existing pavements is at 300mm intervals as specified in Table 4.3 of DN-PAV-03060.

4.3.3 Pavement Temperature

Pavement temperatures are required for correcting the measurement to reference conditions and normalising back-calculated moduli for design purposes after the deflections have been measured. They are also required before surveys commence to ensure that the conditions are suitable for measurement.

The temperature range for testing flexible or flexible/composite pavements is specified as between 5°C and 30°C in DN-PAV-03060. In general, FWD measurements can be carried out over a wide range of surface temperatures. However, at very low temperatures (<5°C) unbound materials may be frozen or partially frozen which can significantly affect the results. In addition, at high temperatures (>30°C) the response of asphalt becomes increasingly viscous, and it is more difficult to distinguish between sound and unsound materials. If testing outside of the temperature range between 5°C and 30°C is unavoidable, for any reason, this shall be noted and reported.

Given that the response of bituminous layers is temperature dependent, the temperature must therefore be measured at the time of test. The method used to determine the temperature at a depth of 100mm is specified in DN-PAV-03060.

4.3.4 Works Performance Assessment using FWD

FWD testing can be used as a quality assurance method for new pavement construction, helping to identify whether the pavement meets or deviates from the design performance requirements. The Irish Analytic Pavement Design Method (IAPDM) specifies the layers that make up a pavement structure and details the materials used in the construction of each layer. Guidance on the IAPDM is provided in DN-PAV-03021 Analytic Pavement and Foundation Design. A typical Quality Assurance program using FWD testing would involve evaluating any or all of the pavement layers listed in Table 4.1 at relevant stages throughout the construction process. The testing programme detailing which layers are to be tested may be site specific and specified by the implementation authority and their designer. The designer must determine if any additional testing requirements beyond the standard are necessary, or any supplementary testing should be recommended in a pavement investigation report.

The FWD test method for Unbound Granular Mixtures (UGM) is defined in DN-PAV-03060, with further guidance available in Clause 4.2 above.

FWD assessment of bound pavement layers should be carried out using the FWD test method specified for testing of existing and new pavements in DN-PAV-03060 with further guidance provided in Clause 4.3 of this publication. The temperature of bituminous bound layers at the time of testing should be recorded in accordance with the procedures defined in DN-PAV-03060.

Table 4.1 - Potential Pavement Layers for FWD Testing (New Construction)

Layer	Pavement Material
Capping	Unbound Granular Mixtures (4.2)
Subbase	Unbound Granular Mixtures (4.2)
Base Course	Unbound Granular Mixtures (4.2)
	Hydraulically Bound (4.3)
	Bituminous (4.3)
Binder Course	Bituminous (4.3)
Surface Course	Bituminous (4.3)

When repeat testing is carried out on the same layer or after the construction of additional layers, it is important to accurately position the FWD tests each time. This ensures valid comparisons over time and enables monitoring of performance improvements due to the added layers. The test points should be located as closely as possible to the previous ones, both longitudinally and transversely within the lane, using measurements such as offset from the lane edge, chainage, and GPS coordinates.

5. Data Analysis and Reporting of FWD Deflection Measurements

5.1 Performance Assessment of UGM

The performance categories for UGM layers are detailed in Table 2.9 of CC-SPW-0800 Road Pavements – Unbound and Hydraulically Bound Mixtures.

5.2 Existing and New Pavement Structures

5.2.1 Normalisation of Deflections

The normalisation of deflections to reference conditions makes the comparison of deflections more straightforward. The normalised deflections can also be used to calculate the mean deflection value of a subsection and to calculate the standard deviation of the deflection values of a subsection.

The actual peak load achieved during an FWD test will depend on the reaction of the pavement to the load application. The deflections are normalised to the defined target load of 40kN by linear extrapolation or interpolation. This means that the measured deflections are multiplied by the ratio, target load over measured load. The normalising of deflections to standard load makes the comparison of deflections possible. The contact pressure equivalent of the target load (40kN) on a 300mm diameter plate is 566kPa. For example, if the deflections of a specific drop are due to a loading pressure of 570kPa, then the measured deflections are multiplied by 566/570 = 0.993 to give normalised deflections.

5.2.2 FWD Deflection Parameters

There are a number of different ways of presenting FWD deflection data including tabulation of results by segment or plotting the various output parameters (D1, SCI and D7) against distance. The deflections can be analysed by plotting more than one deflection parameter against distance on the same graph. An example of such a plot is shown in Figure 5.1.

The first plot shows the central deflection (D1). D1 provides an indication of the overall pavement structural condition of both bound and unbound layers. Lower D1 results are more desirable from a structural viewpoint, with higher D1 results indicating a poor structural condition.

The second plot is the Surface Curvature Index (SCI) which indicates the condition of the upper pavement (bound) layers. High SCI readings would generally indicate poor load spreading ability in the upper pavement layers. In cases where this plot takes the same shape as the D1 plot then the upper layers have a large influence on the pavement structural condition. This is usually the case with flexible pavements. SCI values in excess of 250 microns (normalised to 40kN) indicate poor load-spreading ability in the upper pavement layers and are not suitable for bituminous only overlays, as there is a higher risk of premature cracking.

The third plot of D7 sensor deflections, measured at 1800mm from the centre of the load plate, relates to the subgrade strength. Low values here indicate a stiff subgrade. In cases where this plot takes the same shape as the D1 plot then the subgrade layer has a large influence on the pavement structural condition.

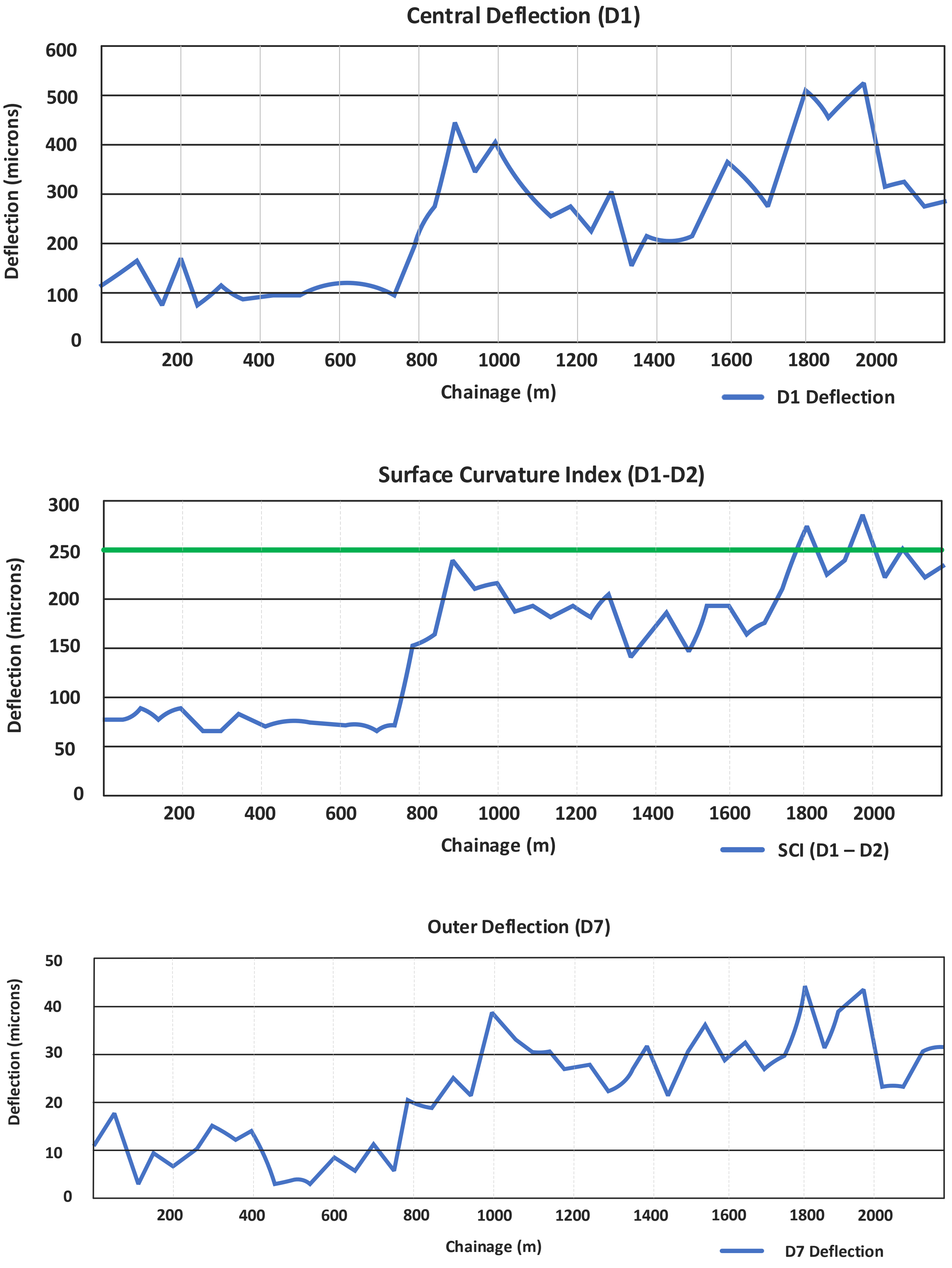

Figure 5.1 - Typical FWD Deflection Plot

DN-PAV-03060 specifies that test sections shall be split into homogenous segments and categorised, based on the criteria for D1, SCI and D7 results on National roads, as outlined in Tables 5.2 to 5.4 in Clause 5.2.3 of that standard. These tables provide guidance on the relevance of recorded deflection values for National Roads, based on typical deflection results observed in Ireland. The criteria can be used to assess the structural condition of existing and new pavement structures in terms of D1, SCI and D7, respectively. In assessing existing pavement structures on National roads based on FWD survey data, and possible interventions that may be necessary, the D1 and SCI criteria may be considered in unison as shown in Table 5.1 which contains guidance on the relevance of combined D1 and SCI parameter values.

Table 5.1 - Summary of FWD Deflection Data for National Roads (Upper Pavement Levels)

D1 Criteria	SCI Criteria	Comment
<100	<40	Very Strong Pavement
100 – 200	40 - 80 Microns	Strong Pavement
200 – 350	80 - 130 Microns	Reasonably Strong - May require overlay depending on traffic volume
350 – 500	130 - 180 Microns	Moderate Pavement - Probably requires overlay depending on traffic volume
500 – 700	180 - 250 Microns	Moderate to weak pavement requiring overlay
>700	> 250 Microns	Poor Pavement (Strengthening or reconstruction required)

5.2.3 Subdivision into Homogeneous Subsections

A homogeneous subsection is a stretch of the test section where the measured deflection bowls are generally similar. Subdivision of a test section may be carried out for a number of reasons and using a variety of techniques. Within a given road section, the measured deflections on one part of the section are often significantly higher or lower than those measured on another part. In this case, it is desirable to divide the main section into subsections, each with a significantly different load response. Along with visual assessment of deflection plots, there are several statistical techniques available to divide a series of data into homogeneous parts.

When continuous information on layer thickness is available, this information can be used for the subdivision of the test section into homogeneous subsections. This data can be obtained using a variety of techniques such as GPR, coring, trials pits, as constructed data etc. (see chapter 6). Where the construction information shows that the pavement is not uniform, the pavement should be divided into sub-sections of homogeneous construction.

5.2.4 Reporting

5.2.4.1 FWD Deflection Parameters in Strip Maps

Strip maps are an important tool to visually interrogate large amounts of data from different sources. TII PAMS creates TII PAMS Strip Maps from the Network Survey data. However, for scheme level pavement design more detailed Extended Scheme Design strip maps shall be developed to identify design sections and to assist in determining the root cause of existing pavement distress. The Extended Scheme Design Strip Maps shall include scheme survey data collected and detailed in Clause 3.2 of the Irish Analytic Pavement Design Method (IAPDM) to the extent agreed with TII Network Management. IAPDM specifies the minimum data to be included in preparing the extended scheme design strip map. This includes scheme survey data collected from an FWD survey.

Guidance on the IAPDM is provided in DN-PAV-03021 (Analytic Pavement and Foundation Design). The Pavement Assessment, Repair and Renewal Principles (PARR) also detail the requirements for strip maps, including FWD survey data. Guidance on PARR Principles is provided in AM-PAV-06050.

Colour coding for the assessment of deflection bowl parameters for use in strip maps are presented in Table 5.2.

Table 5.2 - Summary of FWD Deflection Data

Stiffness Category	Traffic Light Assessment	D1	SCI	D7
Very Low	RED	>700	>250	>50
Low		500 to 700	180 to 250	40 to 50
Reasonable	YELLOW	350 to 500	130 to 180	30 to 40
Moderate		200 to 350	80 to 130	20 to 30
Very High	GREEN	100 to 200	40 to 80	10 to 20
High		<100	<40	< 10

Further guidance showing the use of backcalculated stiffnesses from FWD in the preparation of strip maps is provided in Chapter 7, Clause 7.1.6 and Table 7.1.

5.3 Temperature Correction of Deflections

Since the stiffness moduli of bituminous bound layers are significantly affected by temperature, consequently the measured deflections are affected too. The standard practice has been to account for this temperature effect, by normalising FWD back-calculated bituminous bound layers moduli to a standard temperature of 20°C. In some cases, the temperature correction of deflections to reference conditions is required to allow uniform comparison of deflections from the same site, measured at different temperatures. Ideally the full temperature gradient of the bituminous layers is required but in practice it has been found that a more restricted range of measurements is adequate. The pavement temperature determined at a depth of 100mm below the road surface at the time of testing is used to correct deflections. The temperature of bituminous bound layers at the time of testing should be recorded in accordance with the procedures defined in DN-PAV-03060.

An empirical method was developed for the temperature correction of D1 deflections and Surface Curvature Index (SCI) based on an exponentially decaying depth function. The coefficients for the formula were determined by an iterative least-squares method, derived from measured pavement temperatures. A linear temperature scaling factor is used within the formula, with the linear coefficient called the temperature adjustment factor (TAF), selected with an exponential decay function.

The use of this linear temperature scaling factor should be limited to between temperatures of 5°C and 30°C. While the relationship can be applied to measurements taken at temperatures outside this range, the absolute values of the adjusted deflections should be treated with caution.

When considering temperature corrected deflections, a site may be characterised by a weighted mean annual pavement temperature (WMAPT).

Information available from Met Éireann indicates that ground temperatures at a depth of 100mm in Ireland typically average about 10°C over the course of a given year. Taking this into account and the fact that FWD testing is not typically carried out below 5°C, a WMAPT of 13°C is considered appropriate for average Irish FWD testing conditions.

Temperature correction of deflections shall be limited to pavements with a minimum bituminous material thickness of 75mm. The thickness on thin bituminous pavements (less than 75mm thickness) is such that temperature would not have any significant effect on the overall pavement structure.

When temperature correction of deflections is required, the correction of D1 and SCI to a reference temperature shall be carried out according to the following methods:

5.3.1 Temperature Correction of Central Deflection (D1)

The temperature adjustment factor for D1 deflection shall be calculated as:

$$TAF_{D1} = -0.041 + 0.0775e^{-0.0243h}$$

Where:

TAF_{D1} Is the temperature adjustment factor for D1 deflection

h Depth of the bituminous bound layers (mm)

The temperature corrected D1 deflection shall be calculated as:

$$D1_{temp\_norm} = D1_{meas} + \Delta T * TAF_{D1} * D1_{meas}$$

Where:

$D1_{temp\_norm}$ Is the temperature corrected D1 deflection in microns

$D1_{meas}$ Is the measured D1 deflection in microns

ΔT Is the reference temperature minus the measured temperature in °C

5.3.2 Temperature Correction of Surface Curvature Index (SCI)

The temperature adjustment factor for SCI shall be calculated as:

$$TAF_{SCI} = -0.0595 + 0.0792e^{-0.0152h}$$

Where:

TAF_{SCI} Is the temperature adjustment factor for SCI

H Depth of the bituminous bound layers (mm)

The temperature corrected SCI shall be calculated as:

$$SCI_{temp\_norm} = SCI_{meas} + \Delta T * TAF_{SCI} * SCI_{meas}$$

Where:

SCI_{temp_norm} Is the temperature corrected SCI in microns

SCI_{meas}	Is the measured SCI in microns
ΔT	Is the reference temperature minus the measured temperature in °C

6. Pavement Construction Information

An attempt should be made to establish as full a picture as possible of the pavement construction and maintenance history of the section under investigation. Layer thicknesses shall be established, based on actual data from a site investigation process or as-built pavement structure data. On recently constructed pavements, as-built information is usually readily available from the implementation authority or maintenance contractors. However, in the case of older less formal pavements, this type of information may be more difficult to ascertain. Information relating to layer thickness, materials used, ground conditions, date of opening, etc. are of particular interest.

It is essential that accurate and reliable thickness information be obtained for FWD data analysis and the backcalculation process (Clause 7.1). To determine the pavement structure composition, a site investigation process may be required, which could involve techniques such as Ground Penetrating Radar (GPR) surveys, core sampling or opening trials pits. Detailed guidance and information on these methods are provided in the following clauses and elaborated in Appendices A to C.

6.1 Ground Penetrating Radar

Ground Penetrating Radar (GPR) may be used to determine the pavement structure at traffic speed. Cores are typically required to identify or confirm each type of construction and determine layer thicknesses at specific locations within the survey site in order to calibrate or confirm the calibration of the GPR system that will be used in the survey. For pavement structural evaluation, the following three key parameters are derived from GPR measurements:

- Location of changes in construction
- Material thickness
- Material type.

GPR measurements are collected along the line of the nearside wheel path. Appropriate antennae are used to achieve a minimum layer thickness resolution of 20mm within the top 500mm of the pavement. Appropriate antennae are used to achieve penetration of 1500mm for lower layers. A combination of different frequency antennae is needed to achieve both minimum layer resolution and minimum penetration requirements. The antennae should include one high-frequency antenna in the range 1500 to 2000 MHz yielding high resolution at shallow depths (upper layer pavement thickness measurement) and one low-frequency antenna in the range 450 to 600 MHz providing better resolution at deeper depths. Raw data collected is saved in digital format on the collection device. Longitudinal cross-sections of the site must be displayed in a report detailing location and depth. Graphs must show all individual layers and colour-code them with a legend display. The material types collected from GPR data are reported in terms of the generic classifications of pavement engineering materials (e.g. asphalt, concrete or granular materials) rather than the more detailed descriptions such associated with asphalt mix (e.g. AC, HRA or SMA). However, for core samples, the thickness of each material layer is reported.

A detailed description of GPR surveys and the required analysis is provided in Appendix A.

6.2 Coring

Coring may also be used to verify material thickness, and to determine the material type of each layer. Core samples may also provide information as to the extent of pavement deterioration when taken at locations where such deterioration had occurred. Cores may show defects within the bound layer such as poor compaction, delamination between layers and ‘stripping’ of the asphalt binder away from the aggregate. These defects are noted in the core log.

If the sole purpose of a coring program is to supplement the information generated by GPR or determine material layer thicknesses for use in FWD analysis and backcalculation, then photographing of cores and preparation of a core log as prescribed in Appendix B, is not necessary. In such scenarios, the total depth of bound layers should be measured and recorded. Where there are distinct differences in the bound materials identified in the core, e.g. bituminous, concrete, hydraulically bound, etc., these should be recorded separately. If the requirement to produce a core log is specified, the coring procedure followed, and core log preparation shall be in accordance with the guidance outlined in Appendix B.

Core locations are targeted based on a review of the GPR and FWD surveys. Appendix B describes the Core sampling process as carried out in accordance with EN 12697-27 (Bituminous mixtures – Test methods for hot mix asphalt – Part 27: Sampling). The number, interval, and target locations of cores to be taken is to be agreed with Implementation Authority, however, care should be taken to:

- Avoid junctions or other locations where it would be difficult to arrange a lane closure
- Avoid bridge decks and other non-pavement features
- Target locations from the GPR and/or FWD data which are representative of the section to be interpreted
- Choose locations where the material boundaries in the GPR data are relatively flat to minimise the effects of errors in the location referencing of the GPR and core data.

The procedure in determining the core sample thickness is to be in accordance with section 4 of EN 12697-36 (Bituminous mixtures – Test methods for hot mix asphalt – Part 36: Determination of the thickness of a bituminous pavement. Core holes are to be reinstated in accordance with the CC-SPW-00900 (Specification for Road Works Series 900 – Road Pavements – Bituminous Materials) Clause 10.1.14. The location of all changes in construction (evident either on or below the surface) are reported where there is a change of material type. The thickness of each material layer recorded is reported for core samples.

6.3 Trial Pits

Trial pits can be excavated in some cases to determine the type and thickness of the various pavement layers. This is generally used for large site investigation projects as it has the added benefit that laboratory testing can be carried out on the pavement materials and subgrade soil extracted from the trial pit in order to assess existing ground conditions. Information obtained on the pavement constituents is useful in the later analysis of the FWD deflection data. The process for trial pit excavation and reporting is described in Appendix C.

6.4 Dynamic Core Penetrometer

Dynamic Cone Penetrometer (DCP) as described in Appendix D is a method for use in the bottom of trial pits to measure the strength and thickness of the foundation layers. The DCP uses an 8kg hammer dropping through a height of 575mm and a 60° cone having a maximum diameter of 20mm. The strength of the material is assessed on the rate of penetration per drop or “blow”. Normally two people are needed to complete the test. One holds the apparatus by the handle while the second person lifts the drop weight. The second observes the readings and records them in the appropriate form. The strength of the foundation layers is expressed in equivalent California Bearing Ratio (CBR) values. If required, the Subgrade Surface Modulus estimated from the CBR value can also be reported.

7. Backcalculation, Layer Stiffness Evaluation and Input to IAPDM

The test data collected is required for use in the mechanistic-empirical thickness design procedures in accordance with TII Publication DN-PAV-03021 Analytic Pavement & Foundation Design, and the associated Irish Analytic Pavement Design Method (IAPDM) software. Pavement designs are carried out using the IAPDM web-based software, which allows for the consideration of actual material performance characteristics within the design method. Deflection data collected by the FWD can be used to characterise the parameters of the pavement layers through backcalculation, a process that estimates the engineering material parameters of the pavement layers based on the measured surface deflections, the magnitude of the load, and information on the pavement layer thicknesses.

Various techniques exist for estimating pavement layer stiffness moduli from FWD deflections. An estimate of the stiffness of the pavement layer can be obtained directly by surface modulus equations but more accurate estimates need a more complex approach that is generally referred to as 'backcalculation' or 'back-analysis'. The FWD backcalculation process involves iteratively adjusting existing pavement layer stiffness values within a theoretical linear elastic pavement model until the modelled pavement deflection bowl matches the measured deflection bowl from FWD testing. These derived pavement layer stiffness values are then utilised in pavement strengthening or overlay design within the framework of the IAPDM.

7.1 Backcalculation of Layer Moduli

7.1.1 Backcalculation Overview

The FWD backcalculation process is an iterative process used to determine existing pavement layer stiffnesses by varying the existing pavement layer stiffnesses within a theoretical pavement linear elastic model until the modelled pavement deflection bowl replicates the FWD measured deflection bowl. It is essential that accurate and reliable thickness information be obtained prior to back-analysis. The existing pavement layer thickness and back-calculated stiffness is used in the IAPDM as part of an existing pavement strengthening/overlay design.

Most methods operate under the assumption that a unique set of layer moduli can be found to make the theoretically predicted deflection bowl equivalent with the measured one. Initially, an assumed pavement structure allows for the prediction of a theoretical deflection bowl using preset (seed) moduli values. These moduli values are then adjusted iteratively until the predicted deflection bowl matches the one measured by the FWD. Some approaches utilise a database of deflection bowls, from which layer moduli are derived using regression equations. A wide range of software programs are available for conducting this analysis. This section primarily focuses on the straightforward linear elastic method to be used for backcalculation on National roads.

7.1.2 Pavement Modelling and Backcalculation Guidelines

The results of back-analysis are significantly influenced by both the type of program used and the methodology employed for pavement modelling. This includes considerations such as the number of layers incorporated into the model and the assumed properties assigned to each layer. Adhering to standard rules ensures that the pavement modelling approach adequately facilitates the back-analysis of FWD data, leading to reliable estimates of pavement layer stiffnesses and performance characteristics.

There can be occasions where the standard back-analysis procedure does not produce representative or realistic estimates of stiffness. The following lists some example scenarios where alternative analysis procedures can sometimes produce more realistic estimates of layer stiffness.

a) Non-linear Subgrade or Bedrock Presence:
   - Consider using a layered subgrade model or a stiff layer at depth.

b) Unrealistically High/Low Stiffness of Asphalt or HBM/Concrete Layers:
   - Consider using a two-layer model combining all bound materials and conduct forward-analysis to establish revised guide limits of the materials (supplementary testing is required to identify low stiffness layers)
   - Alternatively, fix the stiffness of the asphalt layer based on indirect tensile tests adjusted for FWD loading time and temperature.

c) Presence of Poor-Quality Bound Materials:
   - Consider subdividing bound layers into intact and poor layers or, in severe cases, combine poor materials with foundation layers to accurately represent the pavement condition.

If an alternative back-analysis procedure is used, it must supplement the standard procedure. The reasons for using an alternative back-analysis procedure shall be clearly stated.

7.1.3 Temperature Normalisation of Layer Moduli

Stiffnesses derived from back-analysis represent estimates of the in-situ values at the time of testing. The stiffness of bituminous bound layers is very dependent on temperature. Therefore, in order to compare the stiffnesses of bituminous bound layers obtained with those expected from standard materials, it is necessary to first adjust them to the standard reference temperature of 20°C.

The impact of temperature on the stiffness of bituminous bound layers can vary significantly and is influenced by various material properties. This relationship specified in DN-PAV-03060 allows for the adjustment of bituminous bound layer stiffness within a recommended temperature range (for flexible pavements with a bituminous bound base) of 10 to 25°C. The relationship can also be used where measurements are taken at other temperatures. However, caution is advised when extrapolating stiffness values from measurements taken at temperatures outside this range.

In cases where bituminous bound layers are severely cracked, the temperature dependency of stiffness is typically lower compared to intact materials. Consequently, temperature adjustment should generally be avoided for severely cracked layers throughout its depth.

7.1.4 Goodness of Fit

When performing backcalculation, the goodness of fit, also referred to as the measure of convergence, is evaluated by comparing calculated and measured deflections bowls. There are two goodness of fit parameters that are commonly used for indicating how well the program has matched the data. These are the Absolute Mean Deviation (AMD) and the Root Mean Squared Deviation (RMS).

The AMD indicates whether or not there is an overall bias to the calculated deflection bowl relative to the measured bowl. The RMS indicates how well, on average, the calculated bowl matches the measured bowl. Although a good fit does not in itself indicate that a correct solution has been obtained, a poor fit does indicate that the solution found is suspect.

Back-analysis programs vary in their ability to match calculated to measured deflections. Poor fits can also be obtained where cracks or other discontinuities are present in the pavement, where incorrect assumptions about layer thicknesses or material types are made, or where layer de-bonding is present. Increasing the number of layers normally improves the level of fit but does not necessarily lead to more realistic estimates of layer stiffness.

7.1.5 Interpretation of Backcalculation Results

The back-calculated stiffness moduli derived from FWD measurements can be used:

- To assess the relative contribution of bound and unbound materials to the pavement strength
- To indicate any weak areas that need replacing or special consideration
- To identify the structural quality of a critical layer (or interface)
- To calculate stresses and strains in pavement layers due to the load imposed
- To calculate the estimated (total) pavement life, using the calculated stresses and strains in combination with a fatigue curve or deformation criterion and the traffic history
- To determine the residual pavement life, using the calculated total pavement life and the predicted traffic in the near future
- To calculate the overlay thickness if the residual pavement life is shorter than the required pavement design life.

The remaining life and rehabilitation/overlay design procedures are carried out within the IAPDM software as detailed in DN-PAV-03021.

Conclusions regarding layer weaknesses must be supported by more than one type of observation or measurement. Layer stiffness must always be checked for correlation with data from other site surveys and investigations. These can include pavement visual condition, the layer condition evident in cores and any laboratory test results. It can also include scheme survey data collected and detailed in Clause 3.2 of DN-PAV-03021 (IAPDM) and included on Extended Scheme Design Strip Maps. The requirements for strip maps, including FWD survey data, is also specified in AM-PAV-06050 (PARR). The use of strip maps as a tool to visually interrogate large amounts of data from different sources was also described previously in Chapter 5, Clause 5.2.4.1 with guidance on the use of deflection bowl parameters in strip maps presented in Table 5.5. Table 7.1 provides guidance on the interpretation of back-calculated stiffnesses from FWD for various materials, with colour coding for the assessment of back-calculated stiffnesses for use in the strip maps presented.

Table 7.1 - FWD Back-calculated Layer Stiffness (MPa) Categories

Stiffness Category	Traffic Light Assessment	Bituminous	HBM	UGM
Poor	RED	<3000	<3000	<200
Fair	YELLOW	3000 to 5000	3000 to 9000	200 to 500
Good	GREEN	>5000	>9000	>500

Backcalculation for stiffness moduli requires significant engineering judgment, as does interpreting the results. If the layers are too numerous or too thin, it is difficult if not impossible to back-calculate stiffness moduli accurately. This is also the case when stiff layers form part of the pavement structure. Materials categorised as “fair” and having some deterioration in stiffness are not automatically deemed unserviceable. Depending on other factors, such as overall pavement condition and performance indicators, these materials may still be suitable for use in the pavement structure, either with or without additional strengthening measures.

Some of the factors that influence the layer stiffness of various materials are given in Table 7.2.

Table 7.2 - Factors Affecting Layer Stiffness

Material	Stiffness Decreases	Stiffness Increases
Asphalt	High voids Cracking Layer debonding Stripping	Low voids Binder-hardening
Cracking	High moisture Clay contamination	Low moisture Natural-cementing
Layer debonding	High moisture	Low moisture

7.2 Reporting of Backcalculation Results and Use in the IAPDM

The design process detailed in DN-PAV-03021 (Analytic Pavement and Foundation Design) is implemented through the web-based Irish Analytic Pavement Design Method (IAPDM) software. The IAPDM is a Mechanistic-Empirical (ME) pavement design method developed specifically for Irish conditions.

The IAPDM uses a Multi-Layer Linear Elastic (MLLE) model to calculate the pavement response to traffic loads. This pavement response is then correlated to an expected long-term performance using empirical models to assess the capacity of the pavement structure to carry predicted future traffic loads.

Design models incorporated within the IAPDM work on the basis of an 85th percentile level of reliability. Design reliability relates to the probability of the actual occurrence of the modelled pavement performance under the specified design traffic loading. This level of design reliability is suitable for the National Road network.

The existing pavement layer stiffness derived from the FWD backcalculation process is used in the IAPDM as part of an existing pavement strengthening/overlay design. Details on the layer thicknesses and moduli are required which allows the existing pavement structure to be modelled in IAPDM as part of this process.

In order to carry out an existing pavement remaining life and rehabilitation design in the IAPDM, the designer requires the following information per design section:

1. Pavement structure layer thicknesses and material types – determined from GPR, coring and/or trial hole investigations, as outlined in Chapter 6.
2. Pavement layer stiffnesses backcalculated from the homogenous section 85th percentile deflection bowl.

The 85th percentile deflection bowl shall be based on the location where the D1 value is closest to the 85th percentile for that segment. Back-calculated moduli for bituminous materials require temperature correction to 20°C from the temperature of the bituminous material at the time of FWD testing.

The remaining life and rehabilitation design procedures within the IAPDM software are detailed in DN-PAV-03021.

For the works performance assessment of bound pavement layers using FWD, the determination of the layer thicknesses of the pavement structure being tested can typically be based on "as-built" information, which should be readily available from the designer, contractor, or implementation authority. If the "as-built" data is unavailable or requires verification, a site investigation program involving Ground Penetrating Radar (GPR) and/or coring may be necessary. Once the pavement layer thicknesses are established, the deflection data from the FWD survey can be used to characterise the pavement layers' parameters through backcalculation. The back-calculated layer moduli for bound layers, constructed at the time of FWD testing, can be evaluated against the design requirements. Additionally, the stiffness of existing pavement layers derived from the FWD backcalculation process can be applied in the Irish Analytic Pavement Design Method (IAPDM). This enables the pavement structure, including any additional designed layers, to be modelled and evaluated for life expectancy in line with design specifications.

7.3 Surface Modulus Plot

In order to obtain an impression of the stiffness of the pavement layers a face modulus plot can be constructed. Such a plot gives an indication of the stiffness at different equivalent depths.

The surface modulus at a point, distance ‘r’ from the centre of the loaded area, is roughly equal to the “weighted mean elastic stiffness” below a depth ‘R’ on the load centre line. Note that the depth ‘R’ is based on the “equivalent pavement thickness” where the thickness of the pavement layers is converted to an equivalent thickness of a material with an elastic stiffness equal to the subgrade stiffness. At a point sufficiently far from the loaded area, the deflection is not influenced by the upper pavement layers. Therefore, the surface modulus calculated at the outer points on the deflection bowl is approximately equal to the subgrade modulus. Such plots indicate the stiffness of the pavement at different equivalent depths and can be used as guidance for the selection of further investigation and analysis methods. Further details of this method are given in FEHRL Report 1996/1.

The surface modulus at the top of the pavement (equivalent depth = 0mm) shall be calculated as:

$$E_o = 2(1 - v^2)a\frac{\sigma_o}{d_1}$$

The surface modulus at the equivalent depth R (valid for r > 2a) shall be calculated as:

$$E_o(r) = (1 - v^2)a^2\frac{\sigma_o}{r \cdot d_r}$$

Where:

E_o	Is the surface modulus at the centre of the loading plate (MPa)
$E_o(r)$	Is the surface modulus at the distance r (MPa)
v	Is Poisson’s ratio
σ_o	Is the contact pressure under the loading plate (kPa)
a	Is the radius of the loading plate (mm)
d_1	Is the central deflection, D1 (microns)
r	Is the distance from the sensor to the loading centre (mm)
d_r	Is the deflection at a distance r (microns).

Poisson's ratios for surface modulus analysis shall be selected for the surface material from the values provided in Figure 7.1 (Clause 7.1.2) of DN-PAV-03060.

Figure 7.1 shows two examples of a 'surface modulus' plot. The surface modulus at the equivalent depths (R) were calculated using the relationship described above and for both examples, the surface modulus is plotted against equivalent depth.

The left plot shows an increasing surface modulus with decreasing equivalent depth. This means that the stiffness modulus of the lower layers is less than that of the upper layers. The stiffness of the subgrade will be around 100 MPa in this case, as indicated by the broken red line.

The right plot shows a pavement that has a 'soft' interlayer between the upper layers and the subgrade. The stiffness of the subgrade is about 300 MPa in this example as indicated by the red broken line. The stiffness of the 'soft' interlayer will be approximately 150 MPa as indicated by the green broken line.

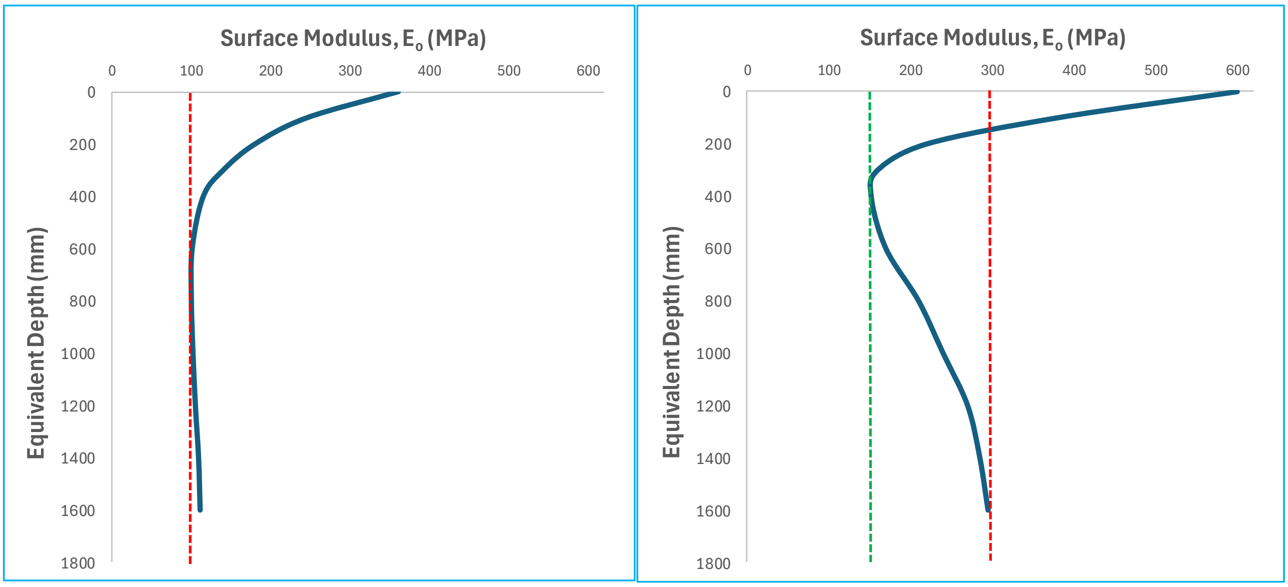

Figure 7.1 - Examples of Surface Moduli Plots

8. References

8.1 TII Publications

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b) Transport Infrastructure Ireland, AM-PAV-06050: Pavement Assessment, Repair and Renewal Principles, TII Publications.

c) Transport Infrastructure Ireland, CC-SPW-00900: Specification for Road Works Series 900 - Road Pavements - Bituminous Materials, TII Publications2.

d) Transport Infrastructure Ireland, CC-SPW-00800: Road Pavements- Unbound and Hydraulically Bound Mixtures, TII Publications.

e) Transport Infrastructure Ireland, DN-PAV-03061: Guidelines for the Use of the Falling Weight Deflectometer in Ireland, TII Publications.

f) Transport Infrastructure Ireland, CC-GSW-04009: Falling Weight Deflectometer Setup and Quality Assurance for Works Performance Assessment of Unbound Granular Mixtures, TII Publications.

g) Transport Infrastructure Ireland, CC-PAV-04007: Requirements for the Reinstatement of Openings in National Roads, TII Publications.

h) Transport Infrastructure Ireland, DN-PAV-03074: Design of Bituminous Mixtures, Surface Treatments, and Miscellaneous Products and Processes, TII Publications.

8.2 European Standards

a) Transport Research Laboratory, Accreditation and Quality Assurance of Dynamic Plate Test Survey Device.

b) Highways England, Design Manual for Roads and Bridges - CS 229 Data for pavement assessment.

c) Highways England, Design Manual for Roads and Bridges - CD 227 Design for pavement maintenance.

d) Transport Research Laboratory, LR 1132 The Structural Design of Bituminous Roads, 1984.

8.3 Others

a) EU Commission, COST 336 Use of Falling Weight Deflectometers in Pavement Evaluation, 2000.

b) Department of Transport, Traffic Signs Manual Chapter 8 – Temporary Traffic Measures and Signs for Road Works.

c) Department of Transport, Tourism and Sport, Guidelines on the Depth of Overlay to be used on Rural Regional and Local Roads.

d) American Institute of Physics, Journal of Applied Physics, Vol 23, pp 126-128, D.M. Burmister, The General Theory of Stresses and Displacements in Layered Soil Systems III.

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h) Austroads, Pavement Design: A Guide to the structural design of road pavements, 2004.

i) Austroads, Technical Basis of Austroads Design Procedures for Flexible Overlays on Flexible Pavements, 2008.

j) Austroads, Pavement Deflection and Curvature Measurement: Falling Weight Deflectometer (FWD), 2017.

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l) U.S. Department of Transportation Federal Highway Administration, FHWA-HRT-16-011: Using Falling Weight Deflectometer Data with Mechanistic-Empirical Design and Analysis, Volume III: Guidelines for Deflection Testing, Analysis, and Interpretation, 2017.

m) U.S. Department of Transportation Federal Highway Administration, FHWA-RD-98-085: Temperature Predictions and Adjustment Factors for Asphalt Pavement, 2000.

n) U.S. Department of Transportation Federal Highway Administration, FHWRT-HRT-16-009: Using Falling Weight Deflectometer Data with Mechanistic-Empirical Design and Analysis, Volume I: Final Report, 2017.

Appendix A – Ground Penetrating Radar

A.1 Introduction

GPR is a non-destructive tool that can be used to obtain information about the construction of a pavement and its internal features. This information can be used to enhance pavement condition information obtained from visual condition, deflection surveys, coring, and trial pits. A photograph of typical GPR Equipment is presented on Figure A.1.

Typically, GPR can provide information about changes in pavement construction, layer thicknesses and defects/features within the pavement. The quality of the information obtained from ground radar is largely a function of three factors:

a) The electrical properties (dielectric constant and the conductivity) of the materials forming the pavement

b) The type of GPR equipment employed

c) The processing software and analysis methodology including calibration procedures employed.

Figure A.1 - Typical Ground Penetrating Radar (GPR) Equipment

A.2 Scope

This part gives guidance on the appropriate use of GPR on paved roads only and does not cover the monitoring of services such as subsurface drains, buried pipes and any other non-pavement-related features. The advice sets out the requirements for successful operation of a GPR survey, for quality control of the survey, and for the presentation of results.

A.3 GPR Operational Legislation & Standards

GPR describes a device that uses radio waves for the purpose of detecting or obtaining images of buried objects or determining the physical properties beneath the ground. The emissions from the radar are intentionally directed down into the ground for this purpose.

The Service Provider is required to carry out all GPR surveying in accordance with The European Telecommunications Standards Institute ETSI EG 202 730 V1.1.1 (2009-09).

A.4 Operating Principles

GPR operates by transmitting a pulse of electromagnetic radiation from an antenna into a pavement. The electromagnetic radiation penetrates down into the pavement as an energy wave, with an envelope in the shape of a cone. A typical output from a GPR survey is shown in Figure A.2.

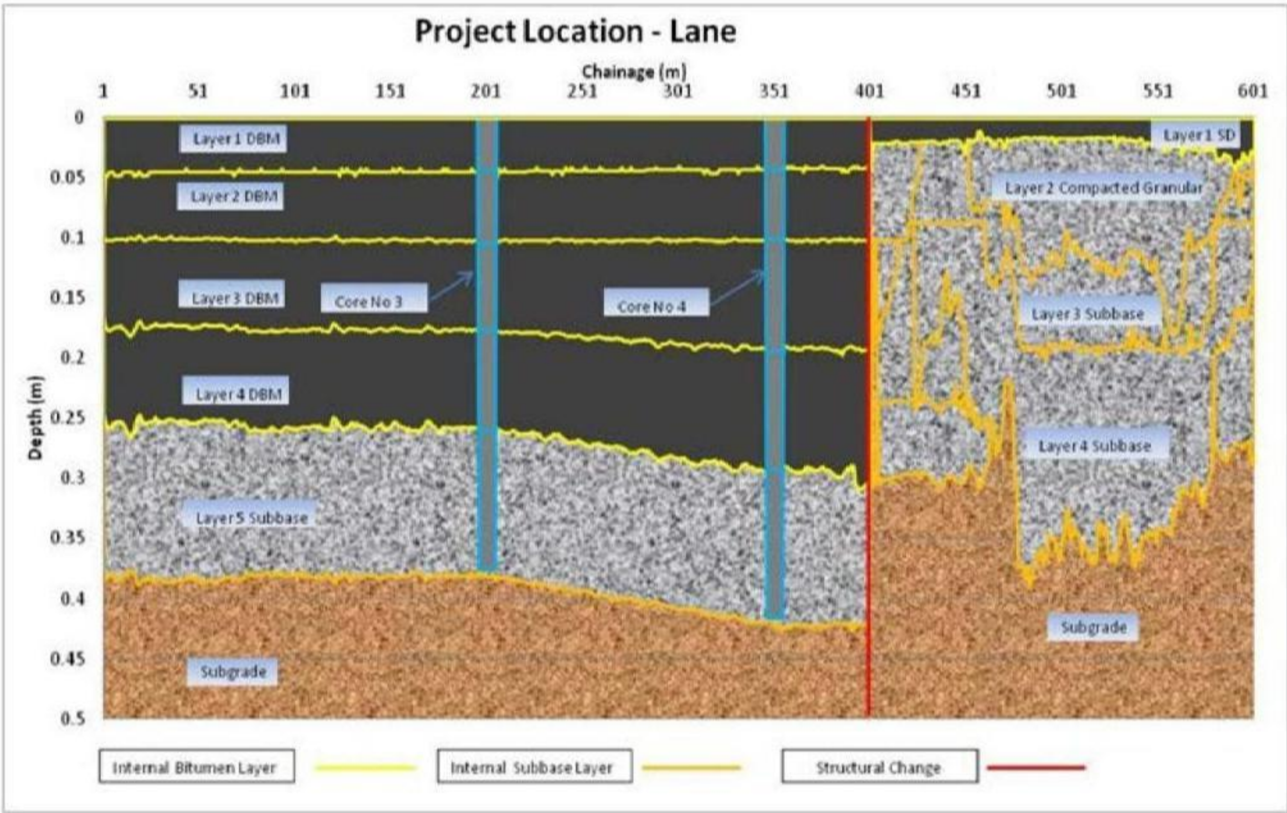


Figure A.2 - Output from a Ground Penetrating Radar Survey

As the wave travels through the various pavement layers, its velocity is changed, and its strength is attenuated. Part of the signal will be reflected back at buried discontinuities or interfaces between different materials such as different pavement layers. These reflected signals and the two-way travel time contain the information about the interior of the pavement. The strength of the reflected wave depends mainly on the difference in dielectric constant of the adjacent materials in the pavement, the greater the difference the stronger the reflection.

A.5 Radar Types

GPR measuring systems are generally defined by the:

a) Single or multi-channel system
b) Measured points per waveform
c) Sampling rate
d) Method of sampling (per fixed interval of time or distance)
e) Antenna operating frequency
f) Antenna signal coupling
g) Antenna type.

A digital system shall be used that will allow storage of data directly to hard disk with the capability of using various filtering processes to remove noise and digitising data. The equipment shall have 512 samples/scan capability, thus producing a high resolution of data.

The sampling rate in the direction of travel depends on the speed of travel, the firing rate of the radar pulses, the number of points per waveform, and the number of channels available. Multi-channel systems allow a wide range of data collection options such as one measuring line being scanned with antennas operating at different frequencies in one run. This option is useful for network-level surveys where data is only gathered from one line (e.g., nearside wheel path). There are two types of antenna design, dipoles, and horns. Dipoles operate most effectively when they are ground coupled, however, they can be air coupled provided the gap between the antenna surface is small (<200mm). Horn antennas are air-coupled and operate with a large air gap (≈ 400mm). Both systems combined can be easily adapted to surveys at traffic speed (50-80km/hr). The antennae should include one high-frequency antenna in the range 1500 to 2000 MHz yielding high resolution at shallow depths (upper layer pavement thickness measurement) and one low-frequency antenna in the range 450 to 600 MHz providing better resolution at deeper depths.

A.6 Location Referencing

Accurate location referencing is fundamental to the collection of good quality ground-penetrating radar data particularly as thicknesses will have to be calibrated or checked against pavement cores.

Radar systems that take samples of road at fixed time intervals will cause some location errors when the speed of travel of the system changes therefore it is a requirement that the system uses an electronic distance measuring system that controls the radar system to take samples at fixed distance intervals. Data export can then be easily referenced to chainage. It should also incorporate a GPS system, with all data geo-referenced so that all GPR data is located precisely, and distance measurements can be verified. The survey vehicles must be equipped with a global navigation satellite system (GNSS) so that all recorded data is referenced to 3-dimensional spatial coordinates. The GNSS equipment is typically integrated with an inertial measurement system in order that ITM co-ordinates can be derived from the GNSS data irrespective of the quality of the satellite coverage.

The GNSS equipment must have the capability to compensate for signal loss (e.g., encountered in urban and forested areas) such that the coordinates are provided under all survey conditions to the required level of accuracy.

Measurements shall include the 3-D spatial coordinates of the equipment position during the survey at points separated by no more than 5m of longitudinal distance travelled. The coordinates must be reported in meters. Reports shall include the 3-dimensional spatial coordinates using the ITM Grid Coordinate system.

A.7 Calibration and Layer Calculations

The thickness of a layer is related to the signal travel time in the layer by the equation:

$$D = V \cdot \left(\frac{T}{2}\right)$$

D = Layer thickness (mm)

V = Velocity of radar signal in layer (mm/ns)

T = Two-way travel time of signal in layer (ns)

The velocity of radar signal within a layer is related to the layer material's dielectric constant ε by the equation:

$$V = \frac{299}{\sqrt{\varepsilon}}$$

Where $V =$ velocity of radar signal in layer (mm/ns)

The GPR Service Provider must be able to display the capability to use all three of the following methods for calculating layer thicknesses:

a) Using published data and typical velocities for different pavement materials (See Table A.1). These have been calculated from the material's dielectric constant and hence affect the signal velocity greatly

Table A.1 - Range of Velocities and Dielectric Constants for Various Pavement Materials

Pavement Material	Velocity (mm/ns)	Dielectric Constant
Air	299	1
Asphalt	90 - 160	4 - 10
Concrete	100 - 130	5 - 9
Hydraulically Bound Mixture	100 - 120	6 - 9
Granular	70 - 120	6 - 18
Water	33	81

b) This method involves using calibration cores which enable velocities to be accurately calculated if done carefully by:
   - Ensuring core position is accurately located within the GPR data
   - Correctly estimating the layer thicknesses from a core

  - Ensuring that the complete core is extracted from the pavement.

c) This is based on calculating the velocity using the reflection coefficient and requires a horn antenna. Before making measurements, a metal plate is placed on the pavement surface to determine the amplitude of the signal returned from a perfect reflector. This amplitude of the signal returned from the pavement surface and the other layer interfaces in the construction to obtain the velocity at the top of each layer. This method is most accurate at shallow depths for determining pavement layers. Therefore, it must combine a dipole antenna at lower frequency to compensate for estimating layers deeper than horn antenna. It is important to avoid standing water as this may affect the calibration.

A.8 Reporting of Data

GPR data should be reported in both graphical and tabular format. Longitudinal cross-sections of the site must be displayed in a report detailing Chainage Vs Depth. Graphs must show all individual layers and colour-code them with a legend display. Where core data is available, locations must be identified along with any changes in construction.

Additional details must also be given such as:

a) Date of survey
b) Road number
c) Road type and direction
d) Construction
e) Surface conditions
f) Survey length.

Tables detailing depths of bound and granular layers should be reported at average lengths, agreed in consultation with TII Network Management. The following are typical examples of changes of pavement constructions which GPR can detect:

a) Changes from hydraulically bound to asphalt base and vice versa
b) Hidden trenches covered by bituminous surfacing.

However, if the construction change is outside the line of the survey, such as haunch construction, or of a short length then the survey may not be able to detect the change.

Table A.2 also gives the constraints and other requirements when utilising GPR to ascertain a specific pavement feature. Ongoing developments in GPR systems make it likely that more features will be detected accurately and reliably as this technology develops into the future.

A.9 Calibration of GPR for Determination of Depth of Features

Cores will be required to identify or confirm each type of construction and determine layer thicknesses at specific locations within the survey site in order to calibrate or confirm the calibration of the GPR system that will be used in the survey.

Guidance on coring operations is set out in Appendix B.

Table A.2 - Accuracy and Reliability of Pavement Features by GPR

Pavement Features	Classification (see below)		Constraints and Requirements
	Slow speed <30km/hr	Traffic speed >80km/hr	
Construction changes	A	A	If the construction changes are outside the line of the survey or of a short length they may not be detected.
Bound and unbound layer thicknesses and profiles	A	A	Low-speed surveys are needed for reinforced layers. Caution is required for interpreting disintegrated lean concrete layers. The best depth resolution for concrete is 20mm (2.5GHz antenna).
Variation of sub-base moisture content (duplicate surveys required)	B	C	Signal velocity changes with material and with moisture content. To eliminate the uncertainty of data interpretation, one survey must be carried out in a 'dry' season when the sub-base is likely to be in an equilibrium moisture condition and an identical survey carried out when the sub-¬base is deemed to be wet (i.e., in the 'wet' season). Note that the interface between the sub-base and the subgrade must be visible in the signals for the technique to work.
Depths of surface cracks in fully flexible pavements	C	D	Specialised GPR equipment, a specifically trained operator and slow survey speed are required. Sample cores are required for calibration of the crack depth measurements. No equipment available for traffic speed survey.
Broad types of pavement materials	C	C	Some idea of material type can be obtained by examining the signal attenuation, amplitude of reflections at material boundaries, continuity of response from within the material, and automatically determined or self-calibrated signal velocity within the material. However, the only certain way to identify materials is to use core data.
Debonding of pavement materials	D	D	This feature might be visible in the bound material at a slow speed with a high-frequency antenna, higher chance to be detected with the presence of water in the debonded area.
Debonding of joint sealant	D	D	Might be detected at a slow speed with special GPR systems. Not proven to date.

Pavement Features	**Classification (see below)**		**Constraints and Requirements**
	Slow speed <30km/hr	**Traffic speed >80km/hr**	
A - Sufficient accuracy and reliability to be used for pavement assessment B - Use to confirm assessment of pavement condition based on other data C - Use with caution and as a guide, along with other construction/condition of pavement D - Unproven and candidates for future research			

Appendix B – Coring

B.1 Introduction

Coring may be required to supplement the information generated by GPR, to verify material thickness, and to determine the material type of each layer. Core samples may also provide information as to the extent of pavement deterioration when taken at locations where such deterioration had occurred. Cores may also show defects within the bound layer such as poor compaction, delamination between layers and ‘stripping’ of the asphalt binder away from the aggregate. These defects are noted in the core log.

The Core sampling process shall be carried out in accordance with EN 12697-27 (Bituminous mixtures – Test methods for hot mix asphalt – Part 27: Sampling). The procedure in determining the core sample thickness is to be in accordance with section 4 of EN 12697-36 (Bituminous mixtures – Test methods for hot mix asphalt – Part 36: Determination of the thickness of a bituminous pavement. Core holes are to be reinstated in accordance with the CC-SPW-00900 (Specification for Road Works Series 900 – Road Pavements – Bituminous Materials) Clause 10.1.14.

B.2 Location Referencing

Accurate location referencing of all pavement condition data is essential to allow reliable comparison between each type of data. The locations of all cores, trial pits, in situ tests and material samples must be referenced against network sections to an accuracy of ± 1m longitudinally and ± 0.1m transversely from the nearside lane edge. All locations must be referenced using ITM grid co-ordinates.

B.3 Coring

Cores shall be drilled in bound material pavement layers at selected locations along the pavement under investigation. The core samples will be used to assess the following aspects of the bound pavement layers condition:

a) Layer thickness
b) Stripping of bitumen in bituminous materials
c) Degree of binding in hydraulically bound materials
d) Bond between layers
e) Assessment of the depth of cracking into the pavement layer.

For time and cost reasons, it will never be possible to carry out all the coring necessary to fully explain all defects and determine the proper remedial treatment at all locations. A limited number of core locations will have to be selected which represent all the defective or weak areas, but biased to the worst areas where remedial works are likely to be more substantial. The strategy for deciding the locations for coring or trial pitting will vary depending on the specifics of each site. Factors to be considered are:

a) The extent of existing information
b) Consistency of construction throughout the site
c) GPR layer thickness profile (if available)
d) Types and locations of defects
e) Consistency of defects and deflections
f) Whether or not defects and deflections at a given location are consistent with each other

g) Locations of high and low FWD deflection (assuming that the FWD survey has already been carried out)

h) Proximity of live traffic lanes and the safety of operatives and road users.

Ideally, the coring of the pavement should be carried out after visual, FWD and GPR surveys have been completed in order that the most effective coring locations can be selected. However, if traffic management or other operational considerations require that the coring is carried out concurrently with the other two surveys, it is essential that a reconnaissance or simplified visual inspection is carried out from the verge or hard shoulder to define, as a minimum, the principal areas of deterioration so that cores can then be positioned so as to provide the maximum amount of information as to why the pavement is deteriorating.

Figure B.1 - Trailer Mounted Coring Rig and Towing Vehicle

When selecting locations for coring, locations at representative cracks should be considered. This can provide additional information on the crack depths and whether or not any adjacent material has disintegrated. Extreme locations such as intersecting cracks or cracks where the adjacent asphalt or concrete is disintegrating should be avoided as successful core recovery is unlikely. It is recommended that, where core location is critical, the intended core positions are paint marked on the road surface to avoid confusion.

Where coring forms part of an investigation into rutted or deformed pavement areas, straddling the ruts or deformation by a set of three cores can provide additional information such as which of the bound layers have reduced thickness. Where intensive rutting is present, the opening of a trial pit can assist in determining which layers are deformed.

Although defects and pavement construction may be similar over a whole scheme, cores taken at defects over the whole length of interest can assist in determining if the causes and depth of defect are similar. The spacing of cores should be defined in agreement with the implementation authority and their designer.

Cores should normally extend the full thickness of bound layers to determine the total thickness and to ensure that the full depth of cracking is recorded. The cores should also indicate any loss of integrity of materials, such as stripping of the binder. It is important that the core logs are recorded by a competent person and include an accurate location relative to the network sections.

Coring on road pavements must always be carried out in accordance with Health and Safety Legislation and with appropriate traffic management in accordance with the Traffic Signs Manual Chapter 8.

150mm diameter cores are preferable to 100mm diameter as they will normally provide sufficient material for any laboratory testing and, if sited on cracks, are more likely to be successfully extracted. However, if only layer thicknesses are required from sound pavement, 100mm diameter or smaller cores could be satisfactory.

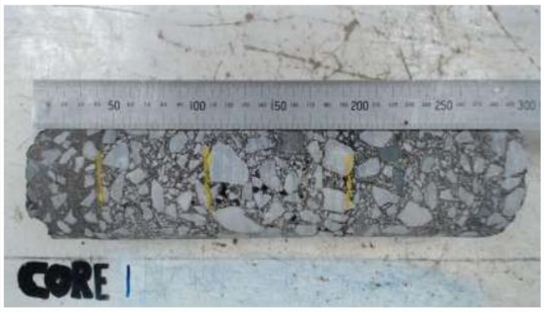
The asphalt layers responsible for rutting may be identified by extracting a set of three cores sited across a rut at spacings of approximately 0.5m and comparing the layer thicknesses. The distances of the top surfaces of the cores to a straight edge should also be recorded to assist in matching the layer thinning deduced from the cores to the actual rut depth. Alternatively, a large transverse slab may be sawn from the pavement if the necessary equipment is available.

For each core, a full record of the core details must be made in the form of a Core Log, an example of which is shown in Figure B.2. The log must include a good quality digital colour photograph with a scale strip and the core reference clearly visible. Natural lighting usually produces the best detail in the photographs. Flash photography, particularly when the core surface is wet, should be avoided as it produces strong highlights which obscure the details.

CORE LOG

Client:	Transport Infrastructure Ireland (TII)	Project No:	
		Date Cored:	
Road No:		Direction:	WB
Section:		Procedure Used:	EN12697-36: 2003 - Clause 4.1

Layer No.	Top (mm)	Bottom (mm)	Thickness (mm)	Material	Layer Condition	
1	0	47	47	HRA		
2	47	111	64	AC		
3	111	198	87	AC		
4	198	307	109	AC		



Additional Comments:

Key: HRA= Hot Rolled Asphalt; AC= Asphaltic Concrete; SMA= Stone Mastic Asphalt; SD= Surface Dressing; HBM= Hydraulically Bound Material; WCG= Well Compacted Granular; DBM= Dense Bitumen Macadam; LMC= Lean Mix Concrete; PQC= Pavement Quality Concrete; MS= Micro-Surfacing; HDM= Heavy Duty Macadam; HFS=High Friction Surface. LEBM= Low Energy Bound Mixtures
Key: LHWP= Left Hand Wheel Path; Centre = Centre of the Lane; RHWP= Right Hand Wheel path; HS=Hard Shoulder
HTSF504, Rev4, 111220

Core No:	1	Chainage (m):	1250
Core Diameter (mm):	100	Core Depth (mm):	307
Easting:	656166	Lane Location:	LHWP
Northing:	747023		
Operator:	DC	Date Measured:	11/04/2023

Figure B.2 - Sample Core Log

The following reference information must be stated on the log sheet for each core:

a) Core reference
b) Section reference, chainage and ITM co-ordinates
c) Traffic direction
d) Lane and offset
e) Coring date
f) Pavement condition at core location including presence of cracks and their orientation.

The following details must be stated on the log sheet for each core:

a) Thickness of each bound layer
b) Type of material present for each layer
c) Condition of the material in each layer, e.g. sound, cracked, friable, etc
d) Layer voiding and segregation (if present)
e) The depth of cracking, providing the cores are suitably located in relation to the pattern of cracking; this may require additional cores in some cases
f) The nature of the material at the bottom of the core hole, e.g. crushed stone, gravel or further bound material.

Where pavement material has disintegrated during coring and there is only partial recovery of material, the layer thicknesses should be determined from the core hole if this is possible.

Appendix C – Trial Pits

C.1 Trial Pits

Excavation of trial pits on road pavements must always be carried out in accordance with the Health and Safety Regulations applicable to the area and with appropriate traffic management in accordance with the Traffic Signs Manual Chapter 8. The necessary risk assessments must be carried out before work commences. In any locations where there may be buried services, the public utility organisations must be contacted for details of the locations of their plant. Cable location devices must be available and be used by a competent operator. In situations of doubt, excavation must proceed with caution, by hand methods.

The lateral location of the pit will depend on the nature of the distress being investigated but it is common for the pit to include the nearside wheel track of lane 1 and part of the hard shoulder, where present. The plan dimensions of the pit are to some extent controlled by the excavation method and the required final depth. Usually, the trial pit boundary is cut to a depth of at least 50mm with a rotary diamond saw, to ensure a neat reinstatement. Excavation is then carried out using a combination of pneumatic tools, hand labour and sometimes a mini excavator. A typical plan size would be 0.6m wide x 1.0m long for a pit of 0.6m depth. If a greater pit depth is required or the excavator bucket width is greater than 0.5m, larger plan dimensions will be needed.

If large samples of asphalt are required for compositional analysis or tests on a recovered binder, or if hidden cracks in lower layers are being sought, the surfacing and base must be removed layer by layer and a large piece of material from each layer, approximately 300mm square, retained for analysis as required.

In flexible pavements, where a trial pit is being opened to investigate which layers have deformed and caused rutting, a rotary diamond saw of at least 150mm cutting depth, must be used to obtain a clean-cut face within the asphalt layers. A steel straight edge across the width of the pit can be used as a datum line.

The surface of each layer should be closely examined before excavation is continued and particular care taken in removing the lower layer of base, to avoid damaging the surface of the sub-base. The general appearance of each layer must be noted. Subject to a satisfactory Health and Safety Risk Assessment an air lance may be useful in clearing away detritus and allowing observation of any cracking.

The pavement layer details revealed in the pit sides must be recorded on a trial pit log similar to the example shown in Figure C.1. If the construction variation across the pit is complex, a diagram must also be provided.

Photographs of the pit faces are desirable but the confined space within the pit and indistinct material boundaries and characteristics sometimes make it difficult to produce useful images.

The collection and testing of samples from the granular layers will depend on the purpose for the trial pit and the materials discovered during excavation. However, generally it would be prudent to take samples of all distinct foundation layers for possible testing. Samples of sub-base material should be retained for grading, classification and the determination of moisture content, and the final layer again carefully removed to reveal the sub-grade or capping.

Density testing of foundation layers is only recommended where the stiffness or strength is unexpectedly low and low compaction or high voids are suspected, as it is a laborious and slow process. The sand replacement method is preferred to the nuclear test, as the latter will require calibration against the former unless the results are only to be used on a comparative basis. In order that the density value can be interpreted, it will also be necessary to carry out a Proctor test of the material in the laboratory to determine the Maximum Dry Density (MDD) and hence determine the relative compaction.

Capping, if present, may also be investigated in the same manner as for sub-base if this is possible in a safe manner. The record of the distribution of any moisture is particularly important. If the foundation layers in roads containing statutory undertaker’s equipment are holding water, samples may be taken for analysis, e.g. the presence of chlorine suggests that a water main may be leaking. Note should be made of any contamination between the subgrade and the sub-base or capping layers.

Backfilling of core holes and reinstatement of trial pits area must be carried out in accordance with TII Specification for Road Works Series 900 and the relevant requirements of CC-PAV-04007 (Requirements for the Reinstatement of Openings in National Roads), where applicable.

TEST PIT LOG

Test Pit: 17 **Date:** 26 July 2006

Location: M1 CH. 113 + 45 Northbound Slow Lane **Rut Depth (mm):** Near Side = 3mm
Off Side = 0mm

Material	Depth From Surface (mm)		Thickness of Layer (mm)		Test Sample No.	Required Tests	Comments
	N/S	O/S	N/S	O/S			
14mm slag HRA Surface course	35	45	35	45	5678	1. Bitumen content 2. Bitumen properties 3. Grading	Surface condition sound
20mm limestone HRA Overlaid Surface course	99	95	64	50	5679	1. Bitumen content 2. Bitumen properties 3. Grading	
28mm limestone HRA Binder course	159	151	60	56	5680	1. Bitumen content 2. Grading	
40mm limestone DBM Base	219	191	60	40	-		
40mm limestone DBM Base	329	306	110	115	-		
Granular sub-base material Type 1 crushed limestone	850	870	521	564	5681	1. Grading	Slight water seepage @ 0.80m at centre of N/S lane. Terram between sub-base and subgrade
Firm mottled grey & olive brown silty CLAY with traces of fine to medium sand and medium gravel	1100	1100	250	230	NS 5682 OS 5683	1. Atterberg limits 2. Field moisture content	
Firm dark grey silty CLAY with traces of sand with some medium gravel size ironstone cobbles	1300	1400	200	300	NS 5684 OS 5685	1. Atterberg limits 2. Field moisture content	Trial hole terminated @ 1.30 - 1.40m with water seepage @ 0.80m as above

Figure C.1 - Sample Trial Pit Log

Appendix D – Dynamic Cone Penetrometer

D.1 Introduction

The Dynamic Cone Penetrometer (DCP) is a method for use on subgrades or in the bottom of trial pits to indicate the strength and thickness of the foundation layers. The equipment is simple and low cost, although it is labour intensive, typically requiring at least two operators. On existing pavement structures, it is necessary to core or remove the bound layers before DCP testing can be undertaken. Both the coring and DCP testing require that appropriate traffic management be installed in accordance with the Traffic Signs Manual Chapter 8.

D.2 Description of DCP Equipment

The DCP uses an 8kg hammer dropping through a height of 575mm and a 60° cone having a maximum diameter of 20mm as shown on Figure D.1. The strength of the material is assessed on the rate of penetration per drop or “blow”. In practice the depth of penetration is recorded at increments of about 10mm, together with the number of blows to achieve this. The number of blows between readings will vary depending on the strength of the layer being penetrated.

D.3 DCP Procedure

Normally two people are needed to complete the test. One person stands on the stool and holds the apparatus by the handle while the same person lifts the drop weight. The second observes the readings and records them on the appropriate form. The DCP shall always be used in accordance with the Health and Safety Regulations and with appropriate traffic management in accordance with the Traffic Signs Manual (Chapter 8). The necessary risk assessments must be carried out before work commences. In any locations where there may be buried services the public utility organisations must be contacted for details of the locations of their plant. Cable location devices must be available and be used by a competent operator. In order to avoid any potential damage to the underground services, it is essential to ensure that there are no services beneath the test location before the test starts. In situations of doubt, the location of the test must be moved or the test abandoned.

Readings every 5 to 10 blows are normally satisfactory for good quality granular layers, but for weaker sub-base layers and subgrade every 1 to 2 blows may be more appropriate. Using the device, the boundaries of soils layers of different strength can be identified by the change in the rate of penetration and the thickness of the layers approximately determined. The DCP test is terminated at a depth of 2 metres or when the cone will not penetrate a material.

The DCP may penetrate granular or lightly stabilised materials in foundation layers reasonably easily. However, in strongly stabilised layers, very dense, high quality crushed stone and granular materials with large particles progress will be much slower or negligible. If there is no measurable penetration after 20 consecutive blows it can be assumed that the DCP will not penetrate the material. The cone must be replaced when its diameter is reduced by 10 per cent.

Figure D.2 gives an example of a field sheet which may be used to record the general reference information for each test together with typical data. The data is normally plotted as the cumulative number of blows (+ve x axis) against depth of penetration (-ve y axis). A change in slope of the plotted data indicates a change of strength and/ or material type. The thicknesses of different strength layers are usually determined by inspection and the average penetration rate, in mm per blow, calculated for each. The penetration rate can be converted to a nominal CBR value using the following relationship developed by the TRL.

$$CBR = 10^{(2.48 - 1.057 \times Log_{10} P)}$$

Where P = The penetration rate in mm per blow.

(NB. The accuracy of this relationship reduces for CBR values below 10 per cent.)

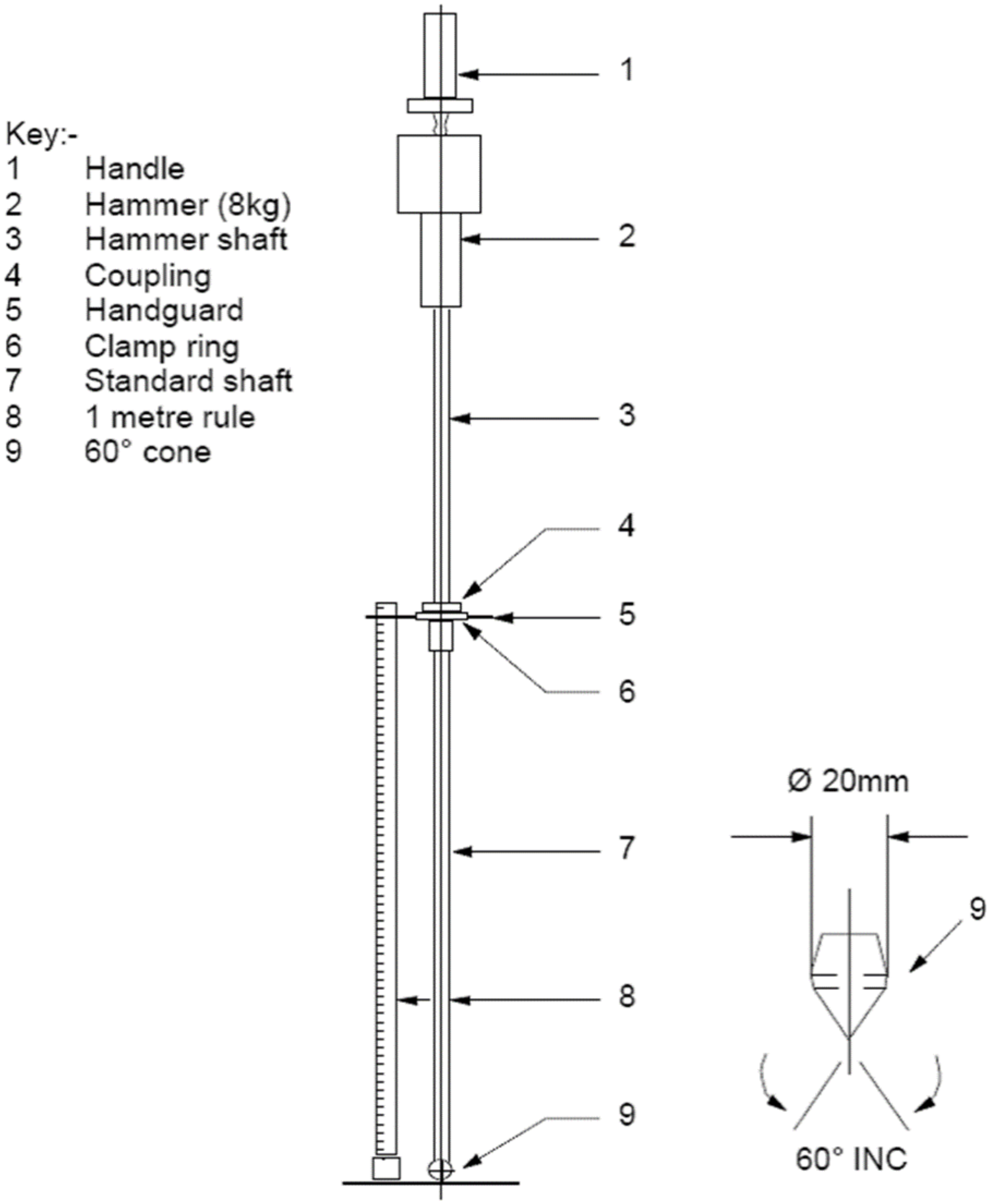


Figure D.1 - Dynamic Cone Penetrometer

DCP FIELD SHEET								
Project: M1 J10/J11			Job No: 12345			Operator: KG		
Date 04/06/14			Time 22.30					
Depth from road surface to start of test: 375mm								
Notes: Drizzle								
Test ID: C	Lane:1		Section 50/8		Chainage: 31m		Direction: NB	
Offset: 0.7m								
No of blows	Σ blows	Penetration (mm)	No of blows	Σ blows	Penetration (mm)	No of blows	Σ blows	Penetration (mm)
0	0	393	2	21	593	2	69	742
1	1	431	2	23	605	2	71	751
1	2	451	2	25	611	2	73	765
1	3	466	2	27	614	1	74	778
1	4	475	4	31	621	1	75	805
1	5	486	4	35	627	1	76	840
1	6	496	4	39	635	1	77	850
1	7	504	4	43	642	1	78	863
1	8	511	4	47	650	1	79	870
1	9	516	4	51	665	1	80	878
2	11	525	4	55	676	1	81	888
2	13	534	4	59	694	1	82	906
2	15	545	4	63	710	1	83	920
2	17	561	2	65	721	1	84	935
2	19	581	2	67	732			

Figure D.2 - Sample Dynamic Cone Penetrometer Field Sheet

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