WARRIP

WESTERN AUSTRALIAN ROAD RESEARCH
AND INNOVATION PROGRAM

Development of a Main Roads rehabilitation supplement to Austroads Part 5 – Stage 2

Technical basis report

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AN INITIATIVE BY:

SUMMARY

Main Roads Engineering Roads Note 16 (ERN16) provides advice on pavement evaluation and rehabilitation treatment design. It is a supplement to the Austroads *Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design*.

ERN16 was developed under WARRIP Project 2018-11: *WA rrehabilitation manual or supplement to Austroads part 5 – stage 2.*

This report describes the development of ERN16, namely:

- Section 2 – consultation with Main Roads Western Australia staff and industry to identify current practices in evaluation and treatment design, past learnings and the need for the provision of advice and guidance in the appropriate use of ERN16.
- Section 3 – the development of design charts to simplify the thickness design of asphalt overlay and inlays.
- Section 4 – the research undertaken to provide a process to enable Traffic Speed Deflectometer maximum deflections to be used in the thickness design of granular overlays.

ACKNOWLEDGEMENTS

ARRB would like to acknowledge the various Main Roads staff and industry personnel who provided input into this project through the distributed questionnaire.

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1 INTRODUCTION

The Austroads *Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design* (Austroads 2019a) provides advice for the investigation of existing sealed pavements and the selection of pavement strategies and treatments. This new edition of Part 5 has been recently published (AGPT05-19). As part of WARRIP Project 2017-006 (*Towards best practice in management of road pavement assets*), ARRB interviewed staff in the Main Roads Western Australia (Main Roads) regions to assess their asset management practices. One of the main findings was that rehabilitation treatments are usually designed using empirical and local methods, as there is no guidance on the use of AGPT05-19.

Main Roads do not mandate the use of AGPT05-19, nor does it have a WA-specific guideline available like other road agencies. It was identified that there was a need for such a document in WA to capture the state-specific learnings from rehabilitation practices used in the different regions. Such a document would provide Main Roads with specific direction to regional managers regarding the investigation of existing sealed pavements and the selection of pavement strategies best suited in their region.

1.1 OBJECTIVE

The objective of this WARRIP project is to prepare a Main Roads rehabilitation document which acts as a supplement to AGPT05-19. This supplement will accumulate all existing Main Roads documentation related to rehabilitation practices into a single document, including input from Main Roads staff involved in asset and network management from all regions.

This report presents the outcomes of a targeted questionnaire undertaken at the start of the project to capture specific regional practices and to gain a better understanding of the gaps in knowledge to inform the direction and emphasis of the supplement. Also presented is the technical basis for the development of overlay and inlay charts, including the development of a TSD (Traffic Speed Deflectometer) adjustment factor tailored to Western Australian conditions.

2 MAIN ROADS PRACTICES, LEARNINGS AND NEEDS

To ensure a complete and relevant rehabilitation supplement to AGPT05, targeted feedback was sought from Main Roads regional and metropolitan staff and also the Western Australian Pavements Group (WAPG) which represents industry, including consultants and contractors. The following sections summarise the responses from both Main Roads and industry through the WAPG.

2.1 MAIN ROADS FEEDBACK

To understand current practice, historical learnings and the potential needs of the different regions, a questionnaire was sent to key Main Roads staff throughout the regions. A list of staff who provided a response to the questionnaire is shown in Table 2.1.

The questionnaire enabled feedback, either through email or by arranging a follow-up phone or video interview to discuss answers in further detail. The questionnaire covered the following areas of interest:

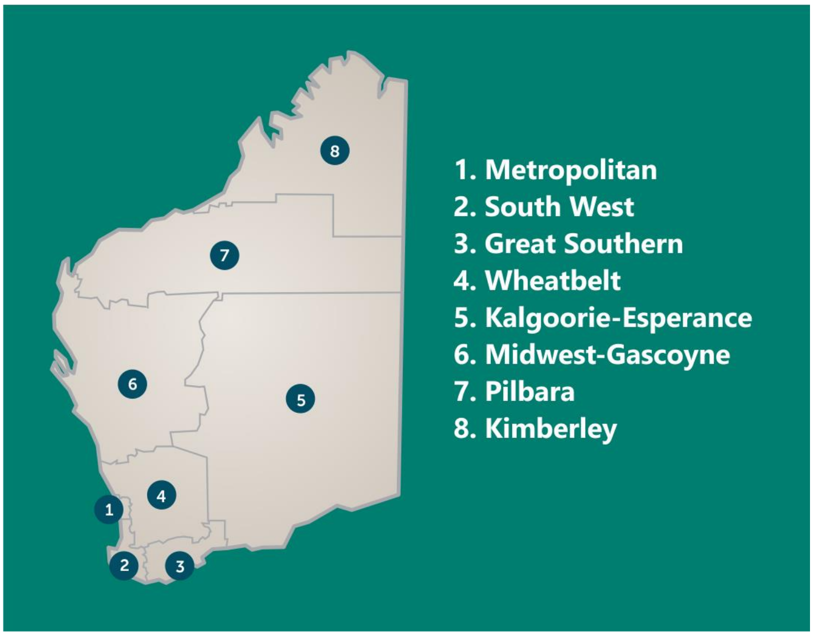
- common treatments for various defects
- trigger criteria for the initiation of rehabilitation works
- rehabilitation project scope and budget responsibility
- design responsibility
- minimum data requirements for design and available data
- adopted design process and design life
- areas of potential improvement in relation to region-specific rehabilitation works
- desirable inclusions for the pavement rehabilitation supplement
- reference to available regional rehabilitation design reports or other region-specific rehabilitation documentation.

A full copy of the distributed questionnaire is included in Appendix A.

Table 2.1: Questionnaire response record – Main Roads staff name, position and region

Name	Position	Region
Mark Russell	Delivery Manager	Goldfields-Esperance
Sardar Khan	Network Manager	
Peter Kernutt	Materials Manager	Great Southern
Kom Siripun	Asset Manager	
Maria Drysdale	Network Manager	Kimberley
Ammar Mohammed	Manager Network Manager	Metropolitan
Chris Skantzos	Maintenance Planning Officer	
Brad Pearce	Asset Manager	Mid-west Gascoyne
Nick Durie	Asset Management Officer	
Scott Buckingham	Network Manager	Pilbara
Andrew Pyke	Regional Manager	
Trevor Spivey	Materials Manager	South-west
Bruce Hancock	Maintenance Planning Manager	
Garnet Gregory	Regional Materials Manager	Wheatbelt
Janet Hartley-West	Network Manager	
Yogesh Shinde	Asset Manager	
Louise Adamson	Network Manager	

Figure 2.1 Main Roads Western Australia – region boundaries



Source: Main Roads (2020)

2.1.1 GOLDFIELDS-ESPERANCE REGION

Initiation of rehabilitation works

Rehabilitation works are initiated primarily through consultation with the Road Maintenance Intervention Parameters (RMIPs) instruction document (Main Roads 2016). This technical instruction provides guidance on the condition at which defects should be scheduled for repair. Various inputs are also used including high speed deflection data (TSD), information from the maintenance management information system (MMIS) and the results of visual inspections.

Project Scoping and design

Project and budget scoping, in addition to the design of rehabilitation works, is undertaken by the local Network and Asset Managers in consultation with key stakeholders.

The severity of a defect or condition of a pavement chosen for rehabilitation will determine the required data for the design. The design methodology used depends on the works to be undertaken.

Table 2.2 details the design lives for different rehabilitation designs.

Table 2.2: Goldfields-Esperance Region: Typical adopted design life for different rehabilitation design scenarios.

Design scenario	Typical adopted design life (years)
Granular overlay	20
Asphalt overlay	12
Pavement repairs	6–10
New pavements and widenings	40

Typical treatments

Table 2.3 presents typical treatments used in the Goldfields-Esperance region for the treatment of common rehabilitation scenarios.

Table 2.3: Goldfields-Esperance Region: Typical treatments for common rehabilitation scenarios.

Rehabilitation scenario	Common treatment
General shape loss	Microsurfacing
Rutting in the wheelpath	Microsurfacing/pavement stabilisation/reconstruction
Cracking	Crack patching/reseal
Cracking with only minor shape loss	Crack patching/cold mix reshape
Proposed increase in traffic	Overlay

Areas for improvement

The following key areas for improvement were noted in the questionnaire:

- Improved method for determining strength of materials due to unreasonable CBR results in the past.
- Non-destructive method for checking the density of pavement repair works.
- Lower-quality material allowance for isolated pavement repairs and unsealed road re-sheeting.

Supplement inclusions

The following items were identified as important inclusions in the new supplement:

- Importance of drainage and climate variability in different regions.
- Guidance on stabilisation selection and dosage design depending on in situ material properties

2.1.2 GREAT SOUTHERN REGION

Initiation of rehabilitation works

Specific trigger values for roughness and rutting are used to initiation rehabilitation works and also to determine the type of treatment for sealed roads. A rut depth above 30 mm and a roughness (IRI) above 4 is typically used in conjunction with data from the MMIS system and visual inspections.

Project scoping and design

Project and budget scoping, in addition to the design of rehabilitation works, is undertaken by the local Asset Manager and Asset Management officers and the local Materials Manager in consultation with key stakeholders.

The severity of a defect or condition of a pavement chosen for rehabilitation will determine the required data for the design. The following data types are often available:

- traffic counts
- Falling Weight Deflectometer (FWD) or TSD
- pavement dippings
- material characterisation data, e.g. particle size distribution (PSD), plasticity, California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS).

The design methodology used depends on the works to be undertaken.

Table 2.4 details the design lives for different rehabilitation designs.

Table 2.4: Great Southern Region: Typical adopted design life for different rehabilitation design scenarios

Design scenario	Typical adopted design life (years)
Granular overlay	40
Asphalt overlay	–
Pavement repairs	10–20
New pavements and widenings	40

Typical treatments

Table 2.5 presents typical treatments used in the Goldfields-Esperance region for the treatment of common rehabilitation scenarios.

Table 2.5: Great Southern Region: Typical treatments for common rehabilitation scenarios

Rehabilitation scenario	Common treatment
General shape loss	Asphalt overlay/premix treatment/grader-laid asphalt
Rutting in the wheelpath	Seal in wheelpaths/microsurfacing/asphalt overlay/stabilisation/gravel top-up and stabilisation
Cracking	Crack patching/reseal/geotextile reinforced seal (GRS)
Cracking with only minor shape loss	Crack patching/cold mix reshape/stabilisation
Proposed increase in traffic	Pavement and or seal widening in conjunction with asphalt overlay if budget permits

Areas for improvement

The following key areas for improvement were noted in the questionnaire:

- Improved method for determining strength of materials due to unreasonable CBR results in the past.
- Non-destructive method for checking the density of pavement repair works.
- Lower-quality material allowance for isolated pavement repairs and unsealed road re-sheeting.

Supplement inclusions

The following items were identified as important inclusions in the new supplement:

- Importance of drainage and climate variability in different regions.
- Guidance on stabilisation selection and dosage design depending on in situ material properties.

2.1.3 KIMBERLEY REGION

Initiation of rehabilitation works

Rehabilitation is generally initiated based on shape loss, followed by a visual inspection to determine the susceptibility to further failure.

Project scoping and design

Project and budget scoping, in addition to the design of rehabilitation works, is undertaken by the local Asset Manager; this is then submitted for funding consideration. The proposed treatment takes into account the quantity and quality of available materials to carry out the rehabilitation. The overall budget for all rehabilitation works is assessed by the local Asset Manager, Network Manager and, if necessary due to assessed risks, the Regional Manager.

The data used to carry out the design depends on the severity or condition of the pavement, with pavement dippings carried out as a minimum. This information is often supplemented with:

- FWD information if available or can be arranged
- as-constructed data if available.

The FWD data is generally used to prioritise the sections for rehabilitation and to assist with allocating budgets.

The design methodology used in the Kimberley Region typically depends on the available data and works to be undertaken.

Table 2.6 details the design lives for different rehabilitation designs.

Table 2.6: Kimberley Region: Typical adopted design life for different rehabilitation design scenarios

Design scenario	Typical adopted design life (years)
Granular overlay	–
Asphalt overlay	–
Pavement repairs	–
New pavements and widenings	40

Typical treatments

Table 2.7 presents typical treatments used in the Kimberley region for the treatment of common rehabilitation scenarios.

Table 2.7: Kimberley Region: Typical treatments for common rehabilitation scenarios

Rehabilitation scenario	Common treatment
General shape loss	Overlay
Rutting in the wheelpath	Surface correction/localised repair/rehabilitation
Cracking	Reseal/rehabilitation
Cracking with only minor shape loss	Reseal/rehabilitation
Proposed increase in traffic	Varies depending on existing pavement, proposed increase and composition of traffic

Areas for improvement

No areas for improvement were noted in the questionnaire.

Supplement inclusions

The following item was identified as an important inclusion in the new supplement:

- Comparative data such as photos or FWD results from across the state that can be used as benchmarking to determine if treatment is warranted. This should also include commentary about common distress mechanisms/features such as water ponding adjacent to the pavement, significant changes in design traffic, etc.

2.1.4 METROPOLITAN REGION

Initiation of rehabilitation works

The Metropolitan Region uses dTIMS to carry out analysis of the current and future condition of the road network. This model is currently being refined and investigation of new thresholds for critical routes is being carried out. However, visual investigation remains as the main trigger of rehabilitation work. Other triggers include the need for resurfacing treatments spanning less than 10 years (except for open-graded asphalt (OGA)), as well as a combination of factors such as deflection, curvature, rutting, roughness and cracking.

Project Scoping and design

Project and budget scoping for rehabilitation works is carried out as a collaboration between the Asset Manager and Maintenance Planning Manager with consultation with Downer Mouchel (DM) Roads. The Main Roads staff determine the scope, budget, treatment type and priority. Discussions with DM Roads is carried out to confirm site conditions, as well as any historical considerations.

The Metropolitan Region utilises ERN9 in the design process, with most of the pavement designs undertaken by the Material Engineering Branch (MEB). The typical targeted design life of all new pavements is 40 years and at least 20 years for maintenance work (excluding holding treatments which may only achieve a couple of years life).

The information that is typically input into a design is dependent on the treatment; however, the following data is typically used:

- traffic counts
- FWD or TSD
- pavement dippings
- texture data for reseal
- material characterisation data such as PSD, plasticity, CBR and UCS.

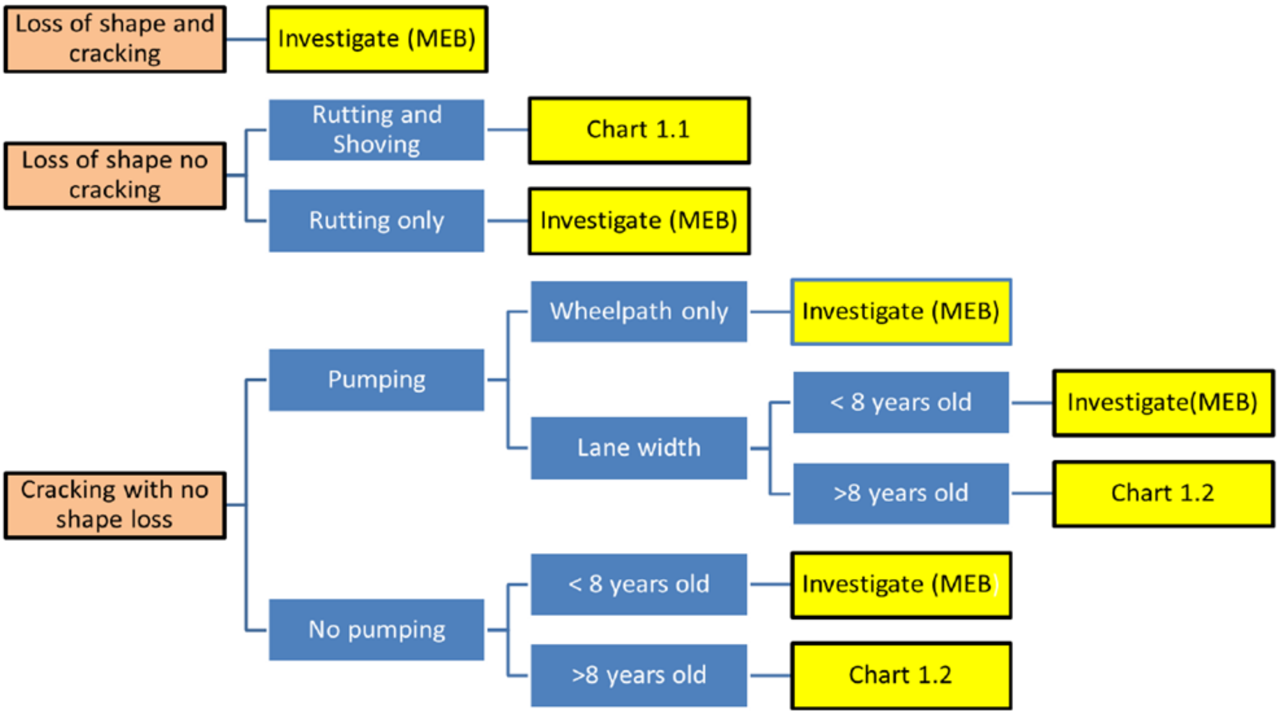
The Metropolitan Region has a guide to assist with the selection of resurfacing treatments (Main Roads 2018). Eight types of resurfacing treatments are included in the guide, as per Table 2.8. The guide includes three charts that either point to the adequate resurfacing treatment for a given pavement condition and age or indicate that the MEB should be consulted. These charts are shown in Figure 2.2, Figure 2.3 and Figure 2.4.

Table 2.8: Metropolitan Region: Surfacing types

Treatment type	Treatment description
Type 1	GRS + overlay
Type 2	SAMI seal + overlay
Type 3	C170 seal + overlay
Type 4	Plane + replace
Type 5	Place + SAMI seal + replace
Type 6	Place + GRS + replace
Type 7	Plane + C170 seal + replace
Type 8	Plane + asphalt + seal + asphalt

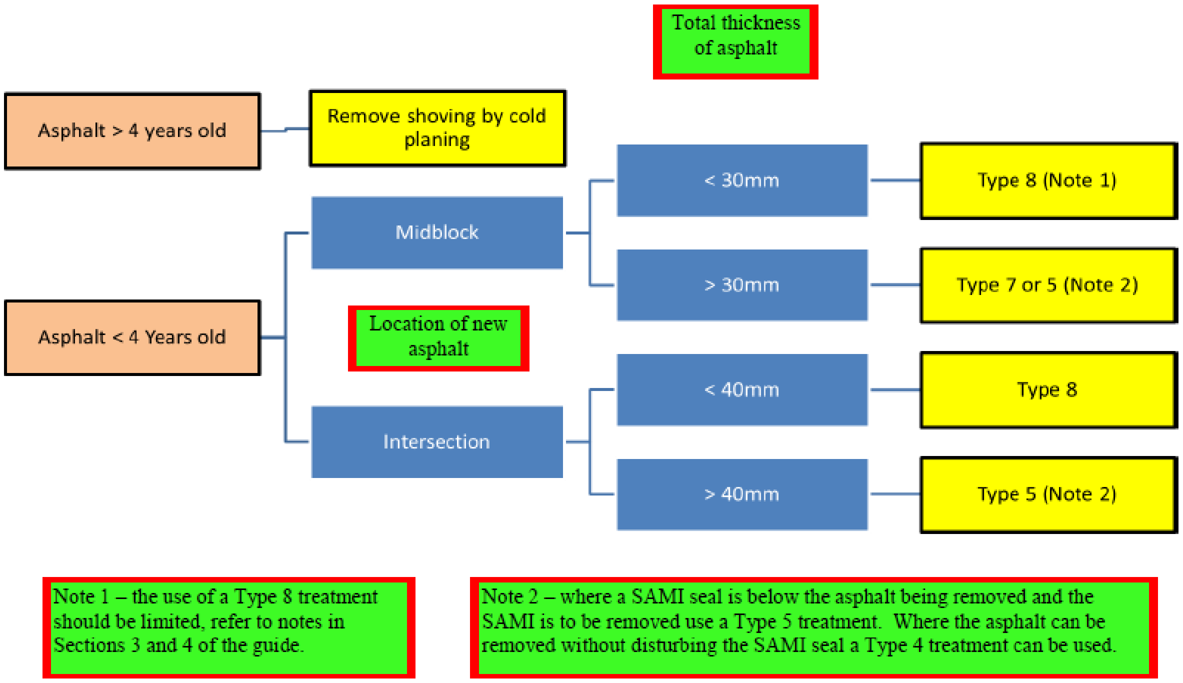
Source: Main Roads (2018).

Figure 2.2 Chart 1: Resurfacing existing DGA with new DGA



Source: Main Roads (2018).

Figure 2.3 Chart 1.1: Repair of asphalt deformation

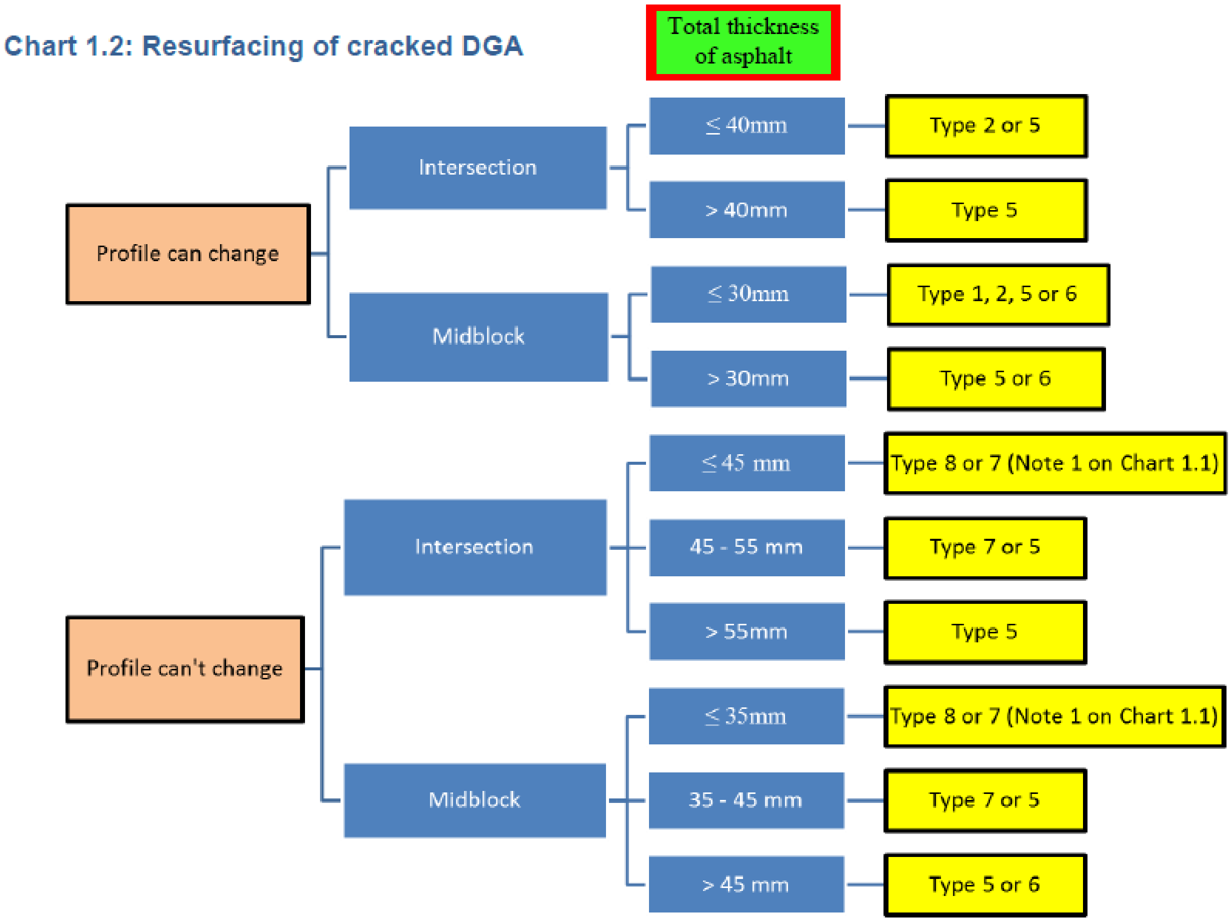


Note 1 – the use of a Type 8 treatment should be limited, refer to notes in Sections 3 and 4 of the guide.

Note 2 – where a SAMI seal is below the asphalt being removed and the SAMI is to be removed use a Type 5 treatment. Where the asphalt can be removed without disturbing the SAMI seal a Type 4 treatment can be used.

Source: Main Roads (2018).

Figure 2.4 Chart 1.2: Resurfacing of cracked DGA



Source: Main Roads (2018).

Table 2.9 details the design lives for different rehabilitation designs.

Table 2.9: Metropolitan Region: Typical adopted design life for different rehabilitation design scenarios

Design scenario	Typical adopted design life (years)
Granular overlay	–
Asphalt overlay	–
Pavement repairs	–
Holding Treatments	1–2
Maintenance works	20
New pavements and widenings	40

It should be noted that if an upgrade is proposed for the road section the design life may be reduced.

Typical treatments

Table 2.10 presents typical treatments used in the Metropolitan Region for the treatment of common rehabilitation scenarios.

Table 2.10: Metropolitan Region: Typical treatments for common rehabilitation scenarios

Rehabilitation scenario	Holding treatment	Common treatment
General shape loss	Profile proud asphalt (knock off tops). This works well for non-cracked or flushed asphalt.	Asphalt – repair, profile SAMI and replace Seal – repair and SAM (14/7 S45R) or asphalt inlay or overlay
Rutting in the wheelpath	FloCon laid asphalt. In the past the region used microsurfacing and/or seal strips in the wheelpaths.	Asphalt – profile SAMI and replace Seal – FDA repair or patches and reseal or 7 mm DGA rut fill and reseal in same season; some microsurfacing in the past
Cracking	Crack patching (wand and/or 5 mm chip seal)	Asphalt – profile SAMI and replace Seal – SAM seal (14/7 S45R)
Cracking with only minor shape loss	Crack patching (wand and/or 5 mm chip seal)	Asphalt – repair, profile SAMI and replace Seal – FDA repair or patches and reseal
Proposed increase in traffic	No action	Asphalt – Nil – project upgrades Seal – SMA overlay (if pavement cannot cope)

Where asphalt inlays/FDA treatments are carried out these typically entail:

- Replacing about 120 mm of the existing pavement with two or three layers of asphalt placed in a one night shift. In small jobs only one asphalt mix is used (usually 14 mm intersection mix). Bigger jobs can use two mixes: a 20 mm intermediate mix (one or two layers 90 mm thick) and a 14 mm intersection mix (usually 40 mm thickness).

Areas for improvement

The following key areas for improvement were noted in the questionnaire:

- Occasionally MEB will provide a design that is difficult to construct. These are sometimes amended by Metro Region, e.g. a 300 mm FDA pavement treatment in five lifts has constructability issues. Metro Region will replace this with 2 x 20 mm Intermediate (180 mm) lifts + wearing course.
- Issues in the past with foam bitumen have made the region cautious.
- Lack of published specifications for foam bitumen and EME2 and a lack of industry knowledge of the products.
- Rehabilitation options that take into account constructability and quick opening works.
- Consideration of short-term (holding) and long-term (final) treatments.

Supplement inclusions

The following items were identified as important inclusions in the new supplement:

- Rehabilitation options that take into account constructability and quick opening works.
- Seal application options/flow chart similar to asphalt charts.
- Acceptable life of each treatment and performance measures (rutting, roughness or FWD for a treated section).

2.1.5 MID-WEST GASCOYNE REGION

Initiation of rehabilitation works

The Mid-west Gascoyne Region uses dTIMS modelling to identify roads required for rehabilitation, the triggers for each road are therefore dependent on Road Category. The region also uses analysis tools such as Tableau and recently Power BI to sort and visualise this data. Following the desktop analysis, the requirement for rehabilitation is validated in the field.

Project Scoping and design

Project scoping of rehabilitation works is undertaken by the local Asset Management section, this includes initial treatment selection, scoping and budget submission. The Asset Management and Project Development sections then carry out further scoping and development of the project. The rehabilitation design is carried out by the regional Materials section and the design methodology follows Engineering Road Note 9 (Main Roads 2013). For larger projects a consultant is sometimes engaged. An example is Onslow Road: this road had a significant increase in anticipated traffic and the consultant provided upgrade requirements.

Generally, FWD data is used to initially identify the section requiring treatment and is used for project justification. The design is then carried out using information from a pavement investigation and laboratory testing. This information typically includes:

- pavement dippings
- material characterisation data such as particle size distribution (PSD), plasticity, Modified Maximum Dry Density (MMDD) and CBR
- as-constructed data if available.

Table 2.11 details the design lives for the different rehabilitation designs.

Table 2.11: Mid-west Gascoyne Region: Typical adopted design life for different rehabilitation design scenarios

Design scenario	Typical adopted design life (years)
Granular overlay	–
Asphalt overlay	–
Pavement repairs	–
New pavements and widenings	–
Rehabilitation	40

Typical treatments

Table 2.12 presents typical treatments used in the Mid-west Gascoyne Region for the treatment of common rehabilitation scenarios.

Table 2.12: Mid-west Gascoyne Region: Typical treatments for common rehabilitation scenarios

Rehabilitation scenario	Common treatment
General shape loss	Microsurfacing/single lift stabilisation
Rutting in the wheelpath	Dry racking (where very minor patching and reseal proposed)/ microsurfacing/stabilisation/rehabilitation
Cracking	Crack patching/SAM seal/stabilisation Cracking due to high cement content in floodways due to re-stabilisation is treated as follows: Traditional treatment is to box out the base and sub base and replace with new compliant stabilised material (nominal 1.5% cement stabilisation). Given the diminishing reserves of naturally-occurring materials, the region is seeking alternative treatments where possible. The region is trialling double- and single-lift stabilisation utilising existing material re-stabilised with foam bitumen. The intent is to reduce the amount of new base material that needs to be sourced.
Cracking with only minor shape loss	Crack patching/SAM seal
Proposed increase in traffic	Design based on empirical design from ERN9-13

Areas for improvement

The following key areas for improvement were noted in the questionnaire:

- Better reporting of investigation works to ensure staff movements do not affect this. It is proposed to introduce a design report template to address this.
- Training of new regional staff that are a part of the pavements and resurfacing program.

Supplement inclusions

No items were identified as important inclusions in the new supplement.

2.1.6 PILBARA REGION

Initiation of rehabilitation works

Roughness and rutting are the main structural criteria used to consider pavements for rehabilitation. However, the majority of rehabilitation is initiated by visual inspections. The MMIS system and TSD data is also now analysed to assist with identifying roads requiring inspections. The Pilbara Region also uses a Power BI report from the Asset Management Modelling and Analytics Manager to obtain the pavement rating summary.

Project Scoping and design

The regional staff carry out the scoping and design for the rehabilitation works. The scoping typically entails regional staff carrying out site inspections to assess the condition of the pavement along with the any immediately available data. The section is then further investigated to determine the treatment and to enable design to be carried out. The Pilbara Region uses the Department of Planning, Transport and Infrastructure (DPTI) South Australia method (DPTI 2014) of analysing deflection data to assist with determining which layer is inadequate (See section 3.2.1). The treatment is decided after evaluation of the costs and risks and agreement between the Network, Maintenance and Asset Managers. The project is then approved by the Network Manager; smaller works are carried out by the Maintenance Manager.

The design is carried out using the following information:

- pavement dippings
- material characterisation data such as PSD, plasticity, mmDD, CBR and UCS

- investigation of materials blending
- FWD if required on larger sections
- historical data, i.e. what has worked in the past/what information is available in the area.

The Pilbara Region uses the ERN9-13 empirical design charts, noting that, on occasions, the ESAs are greater than what are allowed in ERN9-13. Despite this, the charts are still utilised and the pavements are preforming satisfactorily. The overlay design charts in AGPT Part 5 (Austroads 2019a) are also used.

Table 2.13 details the design lives for the different rehabilitation designs.

Table 2.13: Pilbara Region: Typical adopted design life for different rehabilitation design scenarios

Design scenario	Typical adopted design life (years)
Granular overlay	30+
Asphalt overlay	–
Pavement repairs	–
New pavements and widenings	30+
In-situ stabilisation	10–15

Typical treatments

Table 2.14 presents typical treatments used in the Kimberley region for common rehabilitation scenarios.

Table 2.14: Pilbara Region: Typical treatments for common rehabilitation scenarios

Rehabilitation scenario	Common treatment
General shape loss	Cold mix (if minor and temporary)/asphalt overlay/granular overlay/stabilisation (0.8–0.9% cement)
Rutting in the wheelpath	Cold mix (if minor and temporary)/granular overlay/stabilisation
Cracking	Crack patching/GRS/C170 or PMB reseal
Cracking with only minor shape loss	Crack patching/GRS/C170 or PMB reseal/stabilisation
Proposed increase in traffic	Granular overlay/granular stabilisation

Areas for improvement

The following key areas for improvement were noted in the questionnaire:

- Requirements for the investigation of granular pavements for high traffic loadings: experience in the Pilbara indicates that the in-service life of pavements with good quality material is greater than that predicted using the empirical design method.
- Improvement in awareness of thin granular overlays as opposed to granular stabilisation as consolidation of the existing pavement is lost when blending.
- material breakdown is a key issue that is often overlooked and can result in pavement failure.
- Consideration needs to be given to the application of Atterberg data in construction – how important is the PI of a material that is over 10m above the ground and fully encapsulated in a seal for example.
- The importance of the resilience of materials and the need to have minimum limits for the resilient modulus at various traffic levels.

Supplement inclusions

The following items were identified as important inclusions in the new supplement:

- Pavement options for roads with high design ESAs (DESAs) in regional areas, i.e. where the availability of materials is limited.

- A table/chart which provides indicative treatments for different types of failures, and what is required prior to implementation of the treatment.

2.1.7 SOUTH-WEST REGION

Initiation of rehabilitation works

The main rehabilitation triggers in the South-west Region are rutting exceeding the maintenance intervention levels, regular pavement failures and patching index (ratio of patches to area). Rehabilitation is also triggered from the findings of geotechnical investigations, e.g. thin pavements, low strength materials. MMIS is typically not used due to road name and alignment changes that have not been captured or updated in the MMIS.

Project scoping and design

The Materials Manager carries out the design and provides treatment options based on the findings of the investigation. On larger projects consultants have been engaged to carry out the investigation and design. In some instances where unusual conditions are encountered, the MEB is consulted; however, this is not common.

The region relies heavily on pavement dippings and laboratory testing, along with visual assessments, to carry out the designs. The aim of the assessments is to determine the mode of failure and to collect traffic information. Layer thickness and in situ moisture content are key inputs into the designs.

The empirical design method outlined in ERN9-13 is used. This method applies for DESAs below 1×10^8; if CBR is unknown then a conservative value is chosen. The Region has moved away from cement/lime stabilisation as it has been found that the benefits are relatively small compared to the other issues encountered such as cracking. The focus is now placed on good-quality material and good construction practices; mechanical stabilisation is also carried out.

Table 2.15 details the typical adopted design life for different rehabilitation design scenarios.

Table 2.15: South-west Region: Typical adopted design life for different rehabilitation design scenarios

Design scenario	Typical adopted design life (years)
Granular overlay	40
Asphalt overlay	–
Pavement repairs	–
New pavements and widenings	40
In-situ stabilisation	20

Typical treatments

Table 2.16 presents typical treatments used in the South-west region for the treatment of common rehabilitation scenarios.

Table 2.16: South-west Region: Typical treatments for common rehabilitation scenarios

Rehabilitation scenario	Common treatment
General shape loss	Microsurfacing/asphalt overlay/granular overlay
Rutting in the wheelpath	Microsurfacing/asphalt overlay/granular overlay/stabilisation
Cracking	Crack patching/GRS/reseal
Cracking with only minor shape loss	Surface correction and reseal/GRS
Proposed increase in traffic	Dependent on funding includes reduce reseal life cycle/upgrade depending on failure mechanism

Areas for improvement

The following key areas for improvement were noted in the questionnaire:

- An archiving system that allows for improved record keeping of historical pavement information such as test reports, past filed worksheets, geotechnical reports, etc. There are difficulties in maintaining the current MMIS.
- Knowledge transfer between MEB and the regions would be beneficial.
- The region has found that 98% dry density ratio (DDR) with 75% dryback ensures good performance of seals; lesser requirements for density and dryback have caused issues in the past.
- Investigation of treatments for pavements that have been cement stabilised and are now cracking.

Supplement inclusions

The following items were identified as important inclusions in the new supplement:

- Guidance on the use of fabrics placed between subbase and basecourse layers and their application/benefits. Also guidance on the incorporation of geogrids for thin pavements susceptible to cracking.
- Guidance on low-cost treatments to enable the budget to extend further.

2.1.8 WHEATBELT REGION

Initiation of rehabilitation works

The main triggers for pavement rehabilitation in the Wheatbelt Region are excessive pavement repairs outside of maintenance intervention levels, large areas of pavement failure in old pavements, pavements where routine maintenance cannot address the issues, and pavements that have been stabilised a number of times and are showing signs of failure due unsound material. dTIMS and MMIS data is also examined along with visual inspections, with shape loss being another key indicator that rehabilitation is likely required. Rutting is also examined and if it is increasing by 5 mm/year this is also a trigger; however, simply the fact that the pavement is rutting is not a trigger.

Project scoping and design

The Materials Manger and Asset Manager scope the projects that require rehabilitation. Information from geotechnical investigations is also used to assist with the scoping and the determination of treatments.

The information used to carry out the designs includes:

- traffic data
- FWD or TSD data
- MMIS information
- pavement dippings

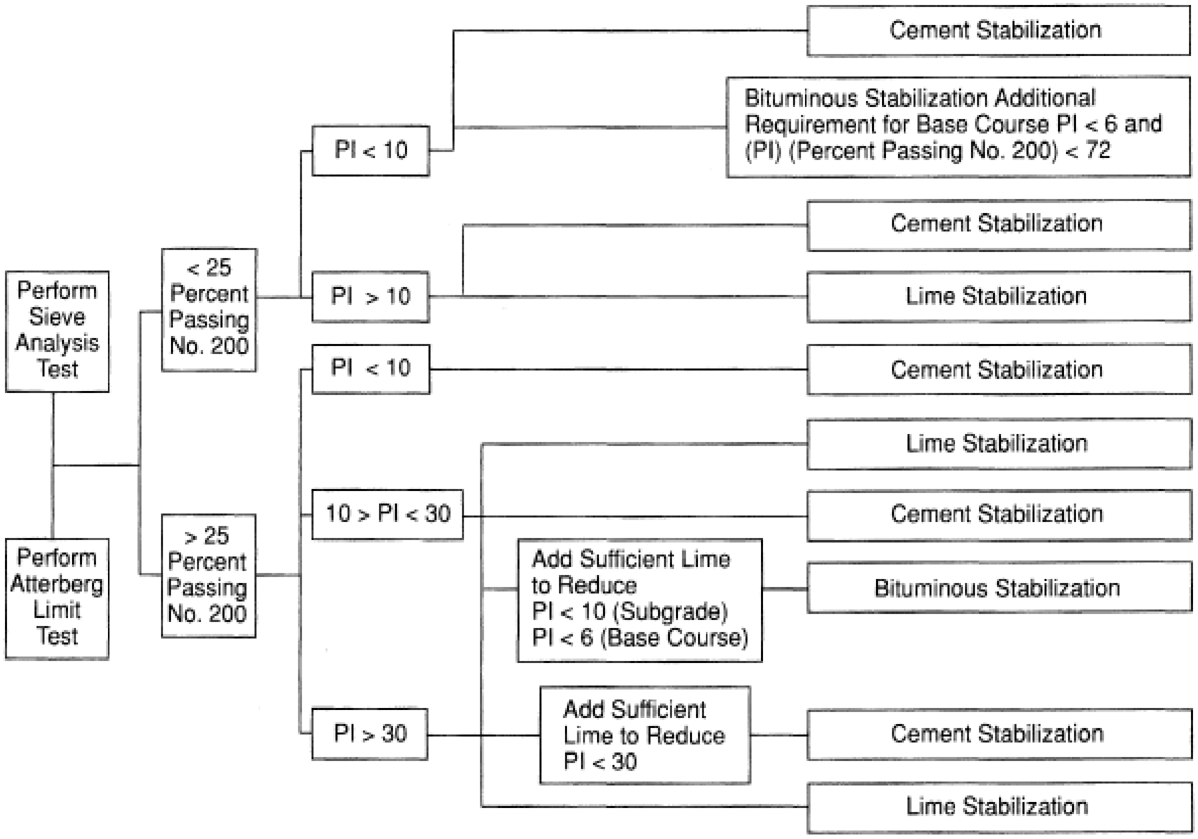
- local experience
- material characterisation data such CBR, moisture content, Atterberg Limits, etc.

The methodologies used for rehabilitation design are ERN9-13, CBR design charts and deflection-based designs using the methods in Austroads (2019a). Stabilisation is also carried out: the method predominantly used is modified rather than bound, i.e. using lower amounts of stabilising binders based on research carried out in South Africa. Typically foam bitumen and/or lime stabilisation is used with a recipe approach (based on experience), rather than formal design. The design team approaches treatment decision-making guided by the following:

1. For major arterials such as the Great Northern Highway (GNH), Great Eastern Highway (GEH) and the Albany Hwy and other heavily-trafficked roads: foam bitumen stabilisation (FBS) with lime and a double seal.
2. For more lightly-trafficked roads with lower risk, alternatives to FBS can be considered, e.g. lime only for plastic soils or low-level cement (0.8% low heat cement) for low plasticity soils.
3. For major arterial roads such as GNH, GEH, Albany Hwy and other heavily-trafficked roads, moderate cement (2% low heat) can be considered as long as a geotextile reinforced seal (GRS) is applied full width using PF2 grade cloth with a double seal at the time of construction.
4. emulsion-only stabilisation can be used with adequate controls for shoulder reconstruction and minor widening.

Scenarios outside of the above are discussed with the regional representatives. Figure 2.5 is used as a pointer to the recommended stabilisation treatment choices.

Figure 2.5 Stabilisation decision chart used in the Wheatbelt Region



Source: Email correspondence with Yogesh Shinde, Main Roads.

Table 2.17 details the design lives for different rehabilitation designs.

Table 2.17: Wheatbelt Region: Typical adopted design life for different rehabilitation design scenarios

Design scenario	Typical adopted design life (years)
Granular overlay	25
Asphalt overlay	–
Pavement repairs	–
New pavements and widenings	40
In-situ stabilisation	20–25

Typical treatments

Table 2.18 presents typical treatments used in the Wheatbelt Region for the treatment of common rehabilitation scenarios.

Table 2.18: Wheatbelt Region: Typical treatments for common rehabilitation scenarios

Rehabilitation scenario	Common treatment
General shape loss	Microsurfacing/thin overlay/stabilisation with bitumen and or lime/granular overlay/reconstruction
Rutting in the wheelpath	Dry racking/microsurfacing/stabilisation with bitumen and or lime/granular overlay/reconstruction
Cracking	Crack patching/GRS/two coat seal/rubber reseal
Cracking with only minor shape loss	Crack patching/thin overlay/GRS/microsurfacing and reseal
Proposed increase in traffic	Granular overlay/seek additional funding upon failure
Other	Enrichment seals

Areas for improvement

The following key areas for improvement were noted in the questionnaire:

- Dissemination of learnings within the region as there is significant experience with knowledge of treatments and how/where they should be applied.
- Ability for the IRIS system to detail the historic rehabilitation: knowledge of previous treatments would enable better decision making when it comes to rehabilitation requirements.
- Additional guidance on brownfields projects as standard specifications have an emphasis on Greenfields/new construction.
- Guidance on the design of overlay treatments, etc.

Supplement inclusions

The following items were identified as important inclusions in the new supplement:

- Additional emphasis of the requirements of surveillance of rehabilitation by Main Roads supervisors and Project Managers to ensure that what is built is as per the designs.
- Flow chart to demonstrate the processes and issues that need to be considered when carrying out design or determining the treatment. These items could include environment/field conditions, moisture, traffic, drainage, landscaping, etc.
- Detail of who in Main Roads can be contacted for particular issues relating to pavements.
- A table/chart which provides indicative treatments for different types of failures, and what is required prior to implementation of the treatment.

2.2 INDUSTRY FEEDBACK

2.2.1 WESTERN AUSTRALIAN CONSULTANTS

Feedback from the consultants that are members of the WAPG in relation to Main Roads rehabilitation projects was requested. However, some consultants predominately consult for Local Government. It is recognised that rehabilitation measures for local government roads can be different to that of state roads; however, for inclusiveness, this feedback is also detailed.

The questionnaire covered the following areas of interest:

- common treatments for various defects
- information typically provided to carry out rehabilitation designs
- other information that would assist with carrying out designs
- adopted design process and design life
- areas of potential improvement in relation to region-specific rehabilitation works
- desirable inclusions for the pavement rehabilitation supplement
- any additional comments or information that may be relevant in the development of the guide.

A full copy of the distributed questionnaire is included in Appendix A.

Design information provided by Main Roads

The design information that is provided to consultants to carry out designs varies depending on the project. However, the typical design information provided or obtained from publicly-available information is:

- design traffic
- existing pavement configuration
- Weigh-in-motion (WIM) data
- IRIS information
- FWD data
- roughness and rutting data
- pavement material properties.

The design information that the consultants would also find beneficial in carrying out the designs, but which are typically not provided, include:

- historical maintenance information
- future changes in traffic loading and details on the possibility of changes to concessional loading
- site-specific ground water monitoring
- historical details of local gravel pits and any material tests on the gravel
- age of the wearing course
- back-calculated modulus.

Design methodologies

The design methodologies that are used by consultants in Western Australia are:

- empirical design
- mechanistic empirical design
- UCS to determine the stabilising binder content
- determination of the optimum binder content when using foam bitumen stabilisation

- Lime Demand Test when carrying out lime stabilisation
- deflection and curvature charts
- development of site-specific models such as roughness and deflection/time.

Table 2.19 details the design lives for different rehabilitation designs. It should be noted that the following Table is a combination of the feedback received from industry: where two values have been provided by differing consultants, both have been listed.

Table 2.19: Consultants: Typical adopted design life for different rehabilitation design scenarios

Design scenario	Typical adopted design life (years)
Granular overlay	40
Asphalt overlay	OGA – 7 DGA – 15 Reseal – 15 Asphalt treatments – 20
Pavement repairs	–
New pavements and widenings	40
In-situ stabilisation	40

Typical treatments

Table 2.20 presents typical treatments recommended by the consultants for the treatment of common rehabilitation scenarios.

Table 2.20: Consultants: Typical treatments for common rehabilitation scenarios

Rehabilitation scenario	Common treatment
General shape loss	Asphalt overlay/asphalt corrector/stabilisation/granular overlay/reconstruction
Rutting in the wheelpath	Dry racking/seal in wheelpaths/microsurfacing/granular overlay/ stabilisation/reconstruction
Cracking	Crack patching/GRS/reseal/profile, apply SAMI or GRS and asphalt/SAMI and asphalt overlay/SMA/stabilisation/reconstruction
Cracking with only minor shape loss	Surface correction and reseal/GRS
Proposed increase in traffic	Dependent on funding includes reduce reseal life cycle/upgrade depending on failure mechanism

Areas for improvement

The following key areas for improvement were noted in the questionnaire:

- Higher shift factors for asphalt should be researched.
- New technologies such as Polycom and stabilisation using CarbonCor should be considered.
- Embrace New technologies that can assist with the limitations that are sometimes experienced with FWD/back-calculation should be considered.

Supplement inclusions

The following items were identified as important inclusions in the new supplement:

- Ability to carry out, or guidance on, methods to carry out life cycle cost analysis for each treatment.
- Guidance on bitumen stabilisation and PMB asphalts.
- Guidance on the performance testing that any new rehabilitation material should be subjected to.
- Blending spreadsheets to assist with the mechanical stabilisation of materials.

- Guidance in terms of pavements that have already been modified, particularly bitumen-stabilised limestone.
- Further guidance on the selection of the modulus of existing pavements.
- Correlation between FWD and subgrade CBR.

3 ASPHALT OVERLAY AND INLAY DESIGN CHARTS

The Austroads *Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design* provides advice for the investigation of existing sealed pavements and the selection of pavement strategies and treatments. A new edition of Part 5 has been recently published (AGPT05-19) (Austroads 2019a).

WARRIP project *WA Rehabilitation Manual or Supplement to Austroads Part 5 – Stage 2* has the objective of preparing a supplement to Part 5, which will be published as *Main Roads Engineering Road Note 16* (ERN16). One of the tasks included in the development of ERN16 is the development of asphalt overlay and inlay design charts. Such charts provide a simplified method of design as an alternative to the more rigorous method the Austroads mechanistic-empirical procedure (MEP) provides in AGPT05-19. The following sections provide the technical basis for the development of these charts.

3.1 DESIGN CHARTS FOR FATIGUE

The report *Technical Basis of Austroads Design Procedures for Flexible Overlays on Flexible Pavements* (Austroads 2008) describes the development of Austroads asphalt overlay design charts that were included in the 2004 and 2011 editions of AGPT05. These design charts were deleted from the 2019 edition of AGPT05.

The method used to determine the Main Roads design charts is similar to that used to derive the Austroads overlay design charts except that:

- Three design charts are provided, one for thin asphalt overlay/inlays thicknesses (40 mm to 70 mm) on pavement surfaces without existing asphalt, another for thin asphalt overlays on asphalt-surfaced pavements, and a chart for the design of thick overlays/inlays (80 mm to 200 mm).
- The allowable traffic loadings to asphalt overlay fatigue were determined using the axle-strain method used in the design of new pavements in the *Guide to Pavement Technology Part 2: Pavement Structural Design* (Austroads 2018), abbreviated herein as AGPT02-17.
- The maximum asphalt overlay thickness was increased from 150 mm to 200 mm.
- The design charts were developed for use with FWD curvatures (D_0 – D_{200}) under a 700 kPa plate contact stress.
- The design charts were only developed for a single in-service temperature, this being the weighted mean annual pavement temperature (WMAPT) for Perth, 29 °C, and a single heavy vehicle design speed of 60 km/h.

The method includes the following steps:

1. A wide range of existing flexible pavement configurations were selected for the derivation of the design charts. These pavements excluded cemented material pavement layers. Linear elastic pavement modelling was used to predict the 700 kPa FWD curvatures prior to overlay/inlay.
2. For each overlay/inlay thickness from 40 mm to 200 mm on each existing pavement configuration, linear elastic pavement modelling was used to determine the tensile strain at the bottom of the asphalt overlay/inlay under both an 80 kN load on a single axle with dual tyres and a 53 kN load on a single axle with single tyres as defined in AGPT02-17.
3. Using the strains calculated in step 2 and the traffic load distribution, the strains under each axle load of each axle group type were calculated as described in AGPT02-17.
4. Using the strains calculated in step 3 and the asphalt fatigue relationship, for each overlay/inlay thickness and each existing pavement configuration, the total allowable traffic loading to asphalt overlay/inlay fatigue was determined considering the damage due to the number of repetitions of each axle load of each axle group type.

5. For each overlay thickness from 40 mm to 200 mm on each existing pavement configuration, linear elastic pavement modelling was used to determine the relationship between the curvature prior to overlay/inlay (step 1) and the allowable traffic loading to asphalt overlay/inlay fatigue.

3.1.1 EXISTING PAVEMENT CONFIGURATIONS

The modelling was undertaken for a wide range of pavements (Table 3.1,

Table 3.2) including existing sprayed seal surfaced and thin (35 mm) asphalt-surfaced unbound granular pavements. The 35 mm asphalt thickness used was based on Main Roads advice. Note that the effect of this existing asphalt in supporting the overlay/inlay was limited to thicknesses up to 70 mm. For projects that require treatments more than 70 mm thick, Main Roads advised it was unlikely that such projects would include existing asphalt layers.

Consequently, the following three design charts were developed:

- chart for thin overlay/inlay thicknesses (40 mm to 70 mm) on pavement surfaces without existing asphalt
- chart for thin overlay/inlay thicknesses on asphalt-surfaced pavements
- chart for the design of thick overlays/inlays (80 mm to 200 mm), assumed to be placed on a surface without existing asphalt.

Table 3.1: Pavement configurations modelled to derive design charts

Layer number	Material	Thicknesses	Moduli (MPa)
1	Asphalt overlay/inlay	40 mm to 200 mm in 5 mm and 10 mm steps	1930 (Section 3.1.3)
2	Existing surface: sprayed seal asphalt (DG14C320)	0 mm 35 mm	N/A (Section 3.1.2)
3	Unbound granular	Table 2.2	Maximum modulus of 350, 500, 700 and 900 MPa with AGPT02-17 design rules
4	Subgrade	Semi-infinite	30, 50, 70, 100, 120 and 150

Table 3.2: Granular thicknesses used to derive design charts

Subgrade modulus (MPa)	Granular thicknesses (mm)
30 and 50	300, 400, 500, 600, 700
70	200, 300, 400, 500, 600
100, 120 and 150	100, 200, 300, 400, 500

3.1.2 MODULUS OF EXISTING ASPHALT

One of the overlay/inlay design charts was developed assuming the overlay/inlay was placed on a 35 mm thick existing asphalt layer (Figure 3.3). The modulus of this existing asphalt needed to be selected. The existing asphalt was assumed to be size 14 mm dense-graded asphalt made using Class 320 binder (DG14C320).

In relation to the prediction of the FWD curvatures prior to overlay, at the time of deflection testing the existing asphalt was assumed to have a modulus of 2190 MPa. The existing asphalt was assumed to be a DG14C320 mix. At a WMAPT of 29 °C, a heavy vehicle design speed of 75 km/h (equivalent speed assumed for FWD) and an air voids content of 8.8%, a 35 mm thick DG14C320 mix would have a design modulus of 2790 MPa for new design. Consistent with the derivation of the Austroads overlay charts (Austroads 2008) it was assumed that the existing asphalt at the time of deflection testing had the beginnings of fatigue cracking (i.e. 3 m/m^2). With such cracking intensity the modulus is 0.78 times the value for new asphalt (Austroads 2008). Accordingly, a trafficked asphalt modulus of 2190 MPa (0.78 x 2790 MPa) was assumed at the time of deflection testing.

The design charts also required prediction of the strains at the bottom of the overlay/inlay under each heavy vehicle axle load of each axle group type. During the service life of an overlay treatment, further fatigue cracking of the existing asphalt is assumed to occur due to the additional traffic loading applied. During this period, it is assumed the existing asphalt is crocodile cracked (i.e. 15 m/m^2) consistent with the Austroads (2008) overlay charts. At a WMAPT of 29 °C, a heavy vehicle design speed of 60 km/h and air voids of 8.8%, a 35 mm thick DG14C320 mix has a design modulus of 2570 MPa when used in the design of new pavements. However, in a crocodile-cracked state the modulus is 0.32 times the value for new asphalt (Austroads 2008). Accordingly, a trafficked asphalt of 820 MPa (0.32 x 2570 MPa) was assumed during the overlay design period.

3.1.3 PROPERTIES OF ASPHALT OVERLAYS/INLAYS

Main Roads advised that the overlay charts should assume the asphalt mix used is a size 14 mm dense-graded asphalt with A15E polymer modified binder (DG14A15E). In addition, the modulus of the overlay/inlay was calculated using the in-service pavement temperature for Perth (WMAPT = 29 °C) and a heavy vehicle design speed of 60 km/h.

For the thin (40 mm to 70 mm) overlay/inlay charts, the following asphalt characteristics were assumed:

- An allowance for construction tolerances was made by using predicted overlay strains for an overlay thickness 10 mm above the specified thickness.
- A design modulus for the DG14A15E mix of 1930 MPa (29 °C, 60 km/h, 8.8% air voids) was calculated for a Main Roads presumptive indirect tensile modulus of 5500 MPa for DG14C320 (25 °C, 40 milliseconds rise time and 5% air voids) and a factor of 0.75 (Austroads 2018) to convert the modulus from the value for a DG14C320 mix to the DG14A15E mix.
- A volume of binder of 10.3% was used for the DG14A15E mix.

For the thick (80 mm to 200 mm) overlay/inlay chart, the following asphalt characteristics were assumed:

- Allowance for construction tolerances made by using predicted overlay strains for an overlay thickness 10 mm less than the specified thickness.
- A design modulus for the DG14A15E mix of 2020 MPa (29 °C, 60 km/h, 7% air voids) was calculated for a Main Roads presumptive indirect tensile modulus of 5000 MPa for DG14C320 (25 °C, 40 milliseconds rise time and 5% air voids) and a factor of 0.75 (Austroads 2018) to convert the modulus from the value for a DG14C320 mix to the DG14A15E mix.
- A volume of binder of 11% was used for the DG14A15E mix.

3.1.4 PREDICTION OF CURVATURES AND ASPHALT STRAINS

For each existing pavement configuration before overlay/inlay, the before overlay/inlay curvature under 50 kN FWD loading was predicted using the linear elastic model CIRCLY. The FWD load was modelled using a plate contact stress of 700 kPa and a plate radius of 150 mm.

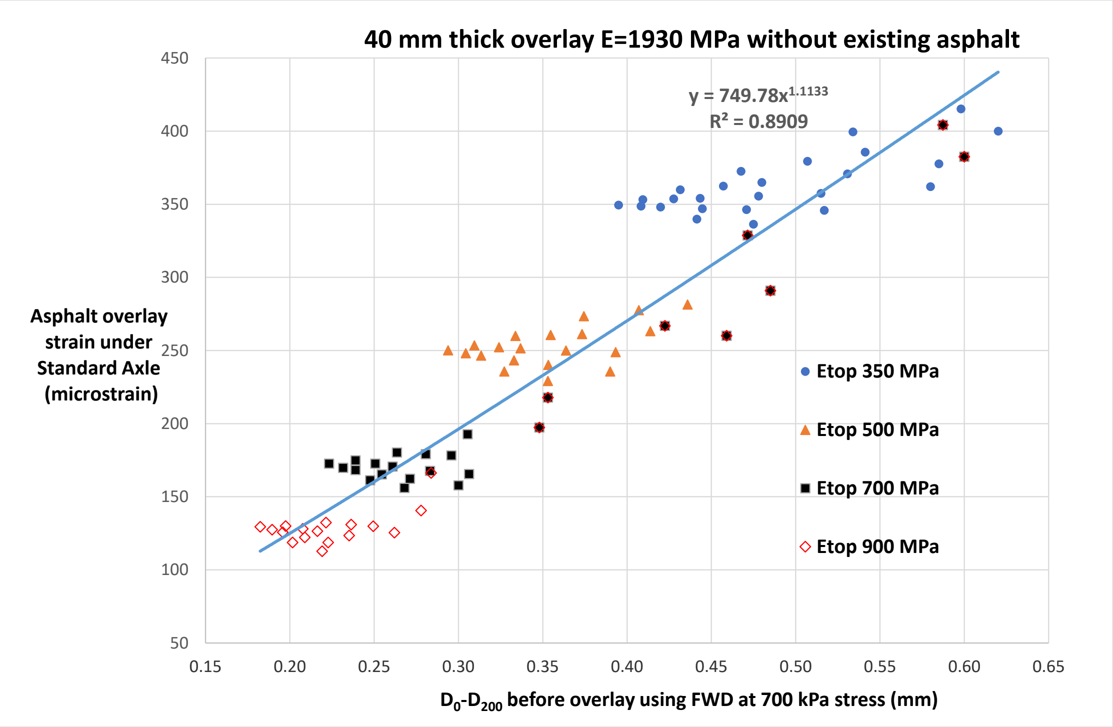
Similarly, for each existing pavement configuration and for overlay/inlay thickness, tensile strains at the bottom of the overlay/inlay were predicted under the following two loads:

- a single axle with dual tyres under an 80 kN axle load
- a single axle with single tyres under a 53 kN axle load.

This is consistent with the method for the design of new pavements (Austroads 2018).

Figure 3.1 is an example of the correlation between the predicted FWD curvature prior to the overlay/inlay and the tensile strains at the bottom of a 40 mm thick asphalt overlay due to a single axle with dual tyres under an 80 kN axle load. As expected, the higher the top modulus of the granular material the lower the curvatures and strains.

Figure 3.1 Example of correlation between curvature before overlay and overlay strain



3.1.5 RELATIONSHIPS TO PREDICT ASPHALT OVERLAY FATIGUE LIFE FROM CURVATURE

To calculate the fatigue life, the maximum strains at the bottom of the overlay/inlay needed to be calculated for each axle load of each axle group type of the traffic load distribution (TLD). Main Roads advised that the generic urban TLD (Appendix B) should be used to develop the design charts.

Using the method described in AGPT02-17, the strains for each load in the TLD were calculated under the following two loads:

- an 80 kN single axle fitted with dual tyres
- a 53 kN single axle fitted with single tyres.

For each axle load, the allowable load repetitions to fatigue cracking were calculated using Equation 25 of AGPT02-17, the parameters in Table 3.3 and the predicted asphalt tensile strains. As it is common to express the design traffic in units of ESA, this unit was used in the allowable traffic loading calculations rather than the number of heavy vehicle axle groups.

Table 3.3: Design inputs for fatigue design charts

Design chart	Overlay modulus (MPa)	Volume of binder (%)	Fatigue factor K	SF/RF
Thin overlay/inlay charts	1930	10.3	4494	1
Thick overlay/inlay chart	2020	11.0	4689	1

Plots of the before treatment curvature against allowable traffic loading of the overlay/inlay are provided in Appendix C. In each graph only the allowable traffic loadings of 10^5 ESA or more are plotted as asphalt fatigue is not considered a distress mechanism for lightly-trafficked roads (Austroads 2017).

As mentioned in Section 3.1.1, for overlay/inlay thicknesses up to 70 mm, design charts were developed with and without allowance for support provided by existing asphalt layers. As shown in the plots in Appendix C, where there is existing asphalt to support the overlay/inlay, the fatigue life is higher for a given curvature. Consequently, the data was analysed separately to develop the following three Main Roads design charts:

- chart for thin overlay/inlay thicknesses (40 mm to 70 mm) on pavement surfaces without existing asphalt

- chart for thin overlay/inlay thicknesses on asphalt-surfaced pavements
- chart for the design of thick overlays/inlays (80 mm to 200 mm).

Equations of best fit to the data were obtained by regression analysis. A power relationship (Equation 1) generally provided the best fit to the data.

$$N = \left(\frac{a}{D_0 - D_{200}}\right)^b \tag{1}$$

where

N = allowable number of load repetitions (ESA)
$D_0 – D_{200}$ = FWD curvature measured under a 700 kPa contact stress (mm)
a, b = regression coefficients (Table 3.4).

Table 3.4: Regression coefficients

Overlay/inlay thickness (mm)	Regression coefficient		Design traffic range (ESA)
	a	b	
Thin overlay/inlay on pavements without existing asphalt			
40	3.6481	0.1610	1.2×10^5 to 7.7×10^7
45	4.2465	0.1745	1.2×10^5 to 4.5×10^7
50	5.0035	0.1879	1.2×10^5 to 3.1×10^7
55	6.1894	0.2043	1.2×10^5 to 2.3×10^7
60	7.7412	0.2212	1.4×10^5 to 1.7×10^7
65	8.9946	0.2325	1.5×10^5 to 1.4×10^7
70	11.333	0.2486	1.7×10^5 to 1.2×10^7
Thin overlay/inlay on pavements with existing asphalt			
40	84.336	0.3615	5.0×10^5 to 4.6×10^7
45	99.169	0.3798	3.7×10^5 to 3.0×10^7
50	125.02	0.4012	3.3×10^5 to 2.2×10^7
55	162.26	0.4238	2.9×10^5 to 1.7×10^7
60	221.86	0.4494	2.8×10^5 to 1.3×10^7
65	296.59	0.4719	2.8×10^5 to 1.1×10^7
70	394.83	0.4924	2.8×10^5 to 1.0×10^7
Thick overlay/inlay			
80	8.9274	0.2271	1.0×10^5 to 2.1×10^7
85	10.898	0.2415	1.2×10^5 to 1.7×10^7
90	13.894	0.2583	1.0×10^5 to 1.6×10^7
95	17.575	0.2746	1.0×10^5 to 1.3×10^7
100	22.824	0.2919	1.0×10^5 to 1.3×10^7
110	39.018	0.3256	1.0×10^5 to 1.3×10^7
120	60.665	0.3507	2.0×10^5 to 1.4×10^7
130	95.962	0.3755	2.8×10^5 to 1.7×10^7
140	152.95	0.3994	4.0×10^5 to 2.0×10^7
150	243.54	0.4225	7.0×10^5 to 2.5×10^7
160	401.74	0.4472	1.0×10^6 to 3.0×10^7
170	674.57	0.4724	1.3×10^6 to 3.7×10^7
180	1094.2	0.4952	1.7×10^6 to 4.3×10^7
190	1640.6	0.5144	2.0×10^6 to 4.8×10^7
200	2530.8	0.5356	2.5×10^6 to 5.3×10^7

3.1.6 DESIGN CHARTS TO PREDICT ALLOWABLE TRAFFIC LOADING IN TERMS OF OVERLAY FATIGUE

Using Equation 1 and the coefficients in Table 3.4, Figure 3.2, Figure 3.3 and Figure 3.4 are the design charts to predict the allowable traffic loading to fatigue cracking of an overlay/inlay.

Figure 3.2 Design chart to predict fatigue life of thin asphalt overlays/inlays on pavements without existing asphalt

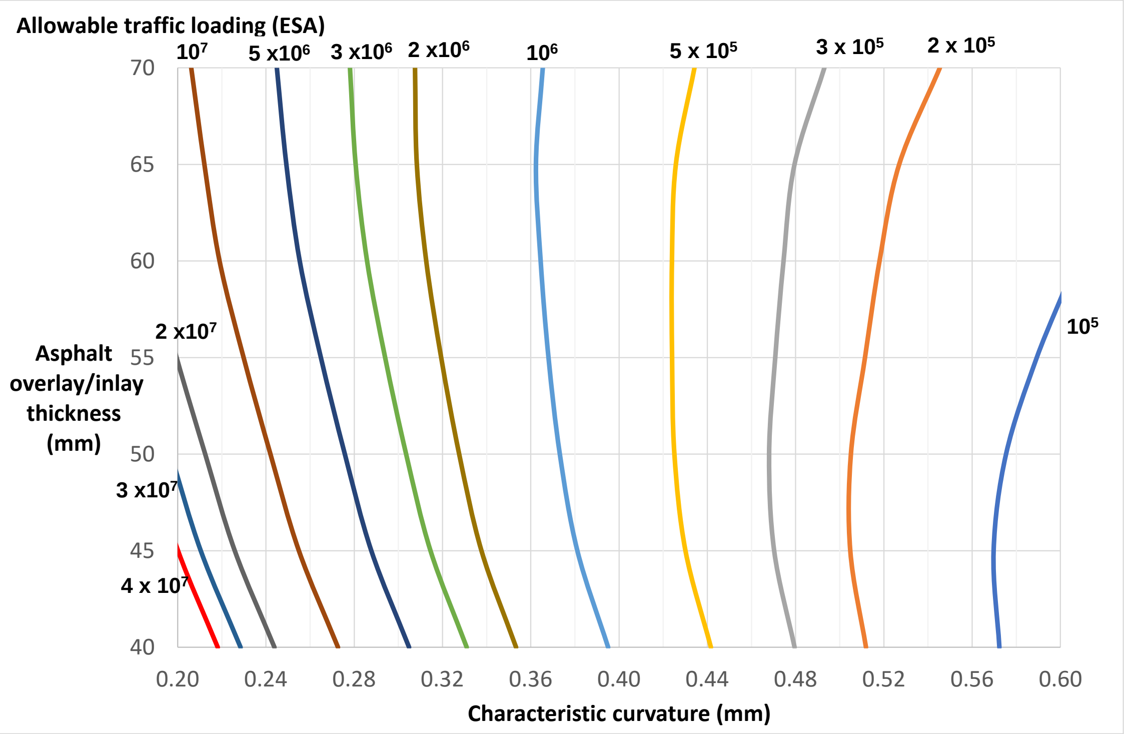


Figure 3.3 Design chart to predict fatigue life of thin asphalt overlays/inlays on pavements with existing asphalt

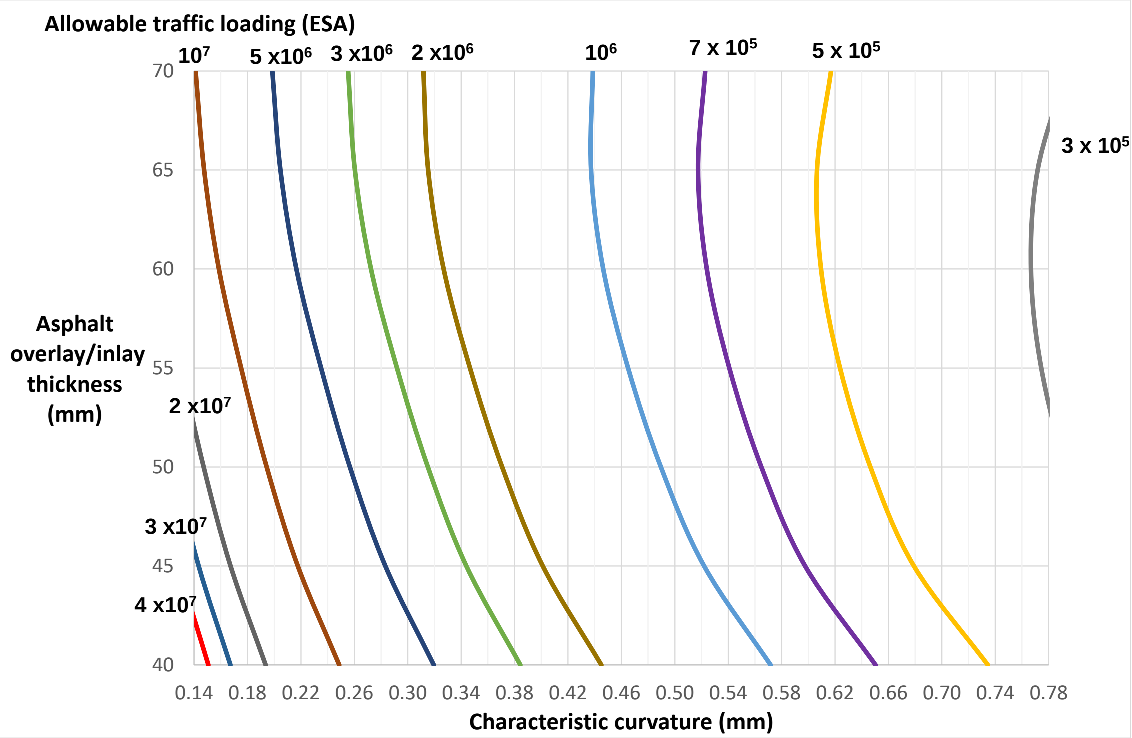
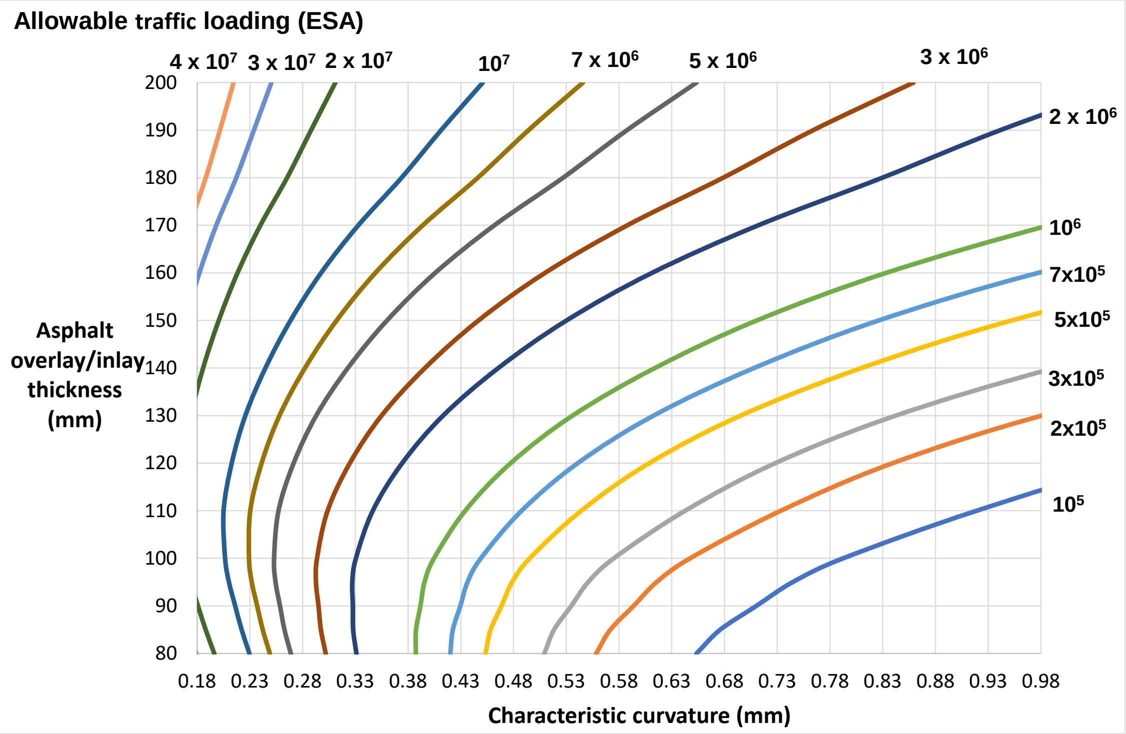


Figure 3.4 Design chart to predict fatigue life of asphalt overlay/inlay thicknesses more than 70 mm



3.2 DESIGN CHARTS FOR PERMANENT DEFORMATION

The development of the asphalt overlay design charts that were included in the 2004 and 2011 editions of AGPT05 are described in Austroads (2008). These design charts were deleted from the 2019 edition of AGPT05.

In relation to the design of asphalt overlays to inhibit permanent deformation, the 2004 and 2011 Guides included:

- Benkelman beam design deflections (Figure 6.5 of Austroads 2011)
- deflection standardisation factors to convert 566 kPa FWD maximum deflections to equivalent Benkelman beam values (Figure 6.3 of Austroads 2011)
- a design chart that related the Benkelman beam characteristic maximum deflection to the required asphalt overlay thickness (40 mm to 150 mm) of an asphalt mix with C320 binder at a temperature of 25 °C (Figure 6.9 of Austroads 2011)
- overlay thickness adjustment factors to convert the thickness to the project WMAPT and the binder type of the overlay mix (Figure 6.10 of Austroads 2011).

In 2005, plane and reinstatement (P&R) design charts were developed (Jameson 2005) for Transport SA, now the Department of Planning, Transport and Infrastructure (DPTI). This research extended the Austroads (2011) Figure 6.9 by:

- providing for overlay/inlay thicknesses of up to 200 mm
- extending the maximum design traffic loading to 2 x 10^7 ESA
- considering a wider range of pavement configurations including those after planning.

As discussed Section 3.2.1, this DPTI P&R design chart for permanent deformation was modified for use by Main Roads in the design of overlays and inlays.

3.2.1 DPTI DESIGN CHART

Jameson (2005) describes the development of the DPTI P&R design chart to inhibit permanent deformation. The P&R design chart (DPTI 2014) is shown in Figure 3.5. From this chart, the asphalt overlay thickness at a WMAPT of 25 °C may be determined. The process used to develop the DPTI chart was similar to the development of the chart for the design of overlays (

Figure 3.6) described in Austroads (2008), but extended as described at the start of Section 3.

The thicknesses derived from Figure 3.5 and

Figure 3.6 are similar, but as the DPTI chart covers a wider range of deflections and asphalt treatment thicknesses, it can be used for both overlays and inlays.

Figure 3.5 DPTI P&R design chart for permanent deformation

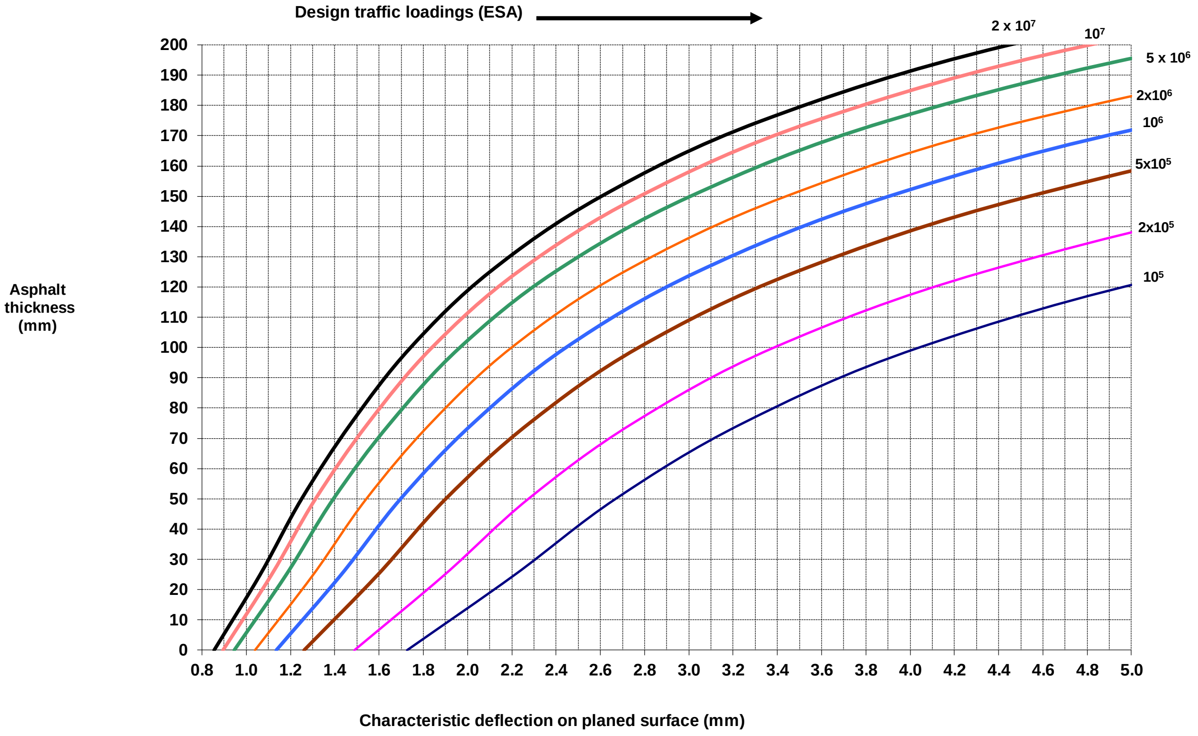
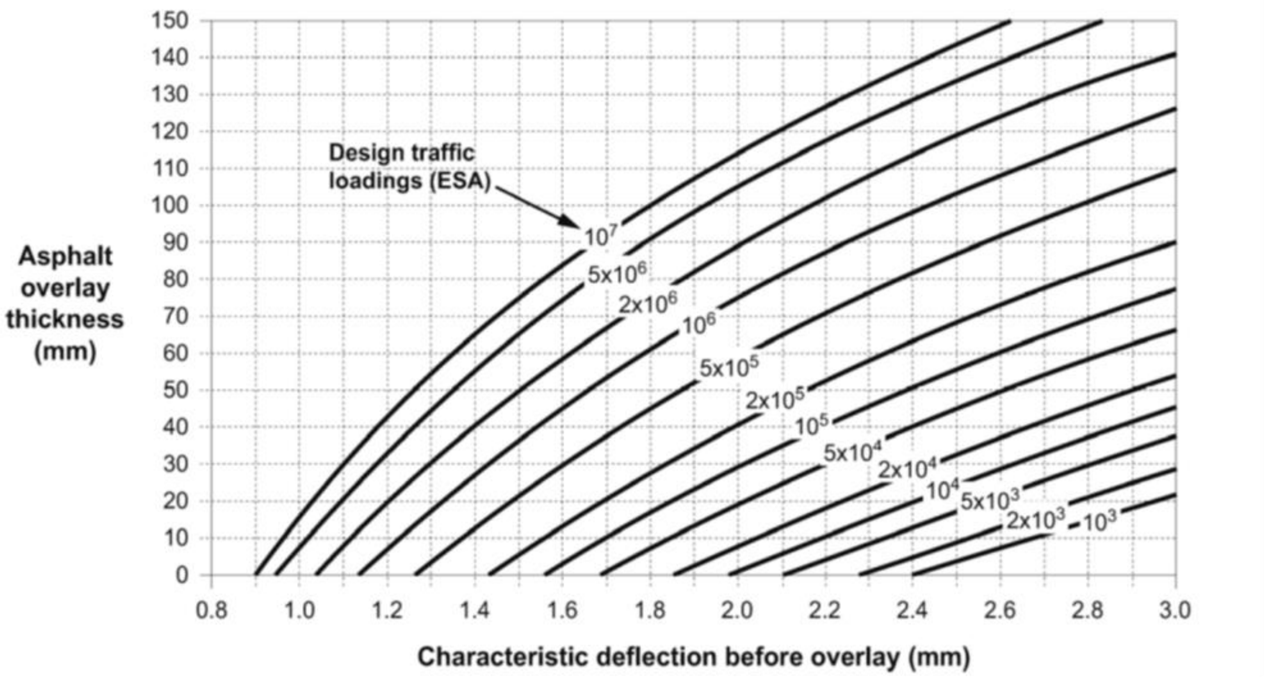


Figure 3.6 Austroads (2011) design chart for permanent deformation



3.2.2 MAIN ROADS DESIGN CHART FOR PERMANENT DEFORMATION

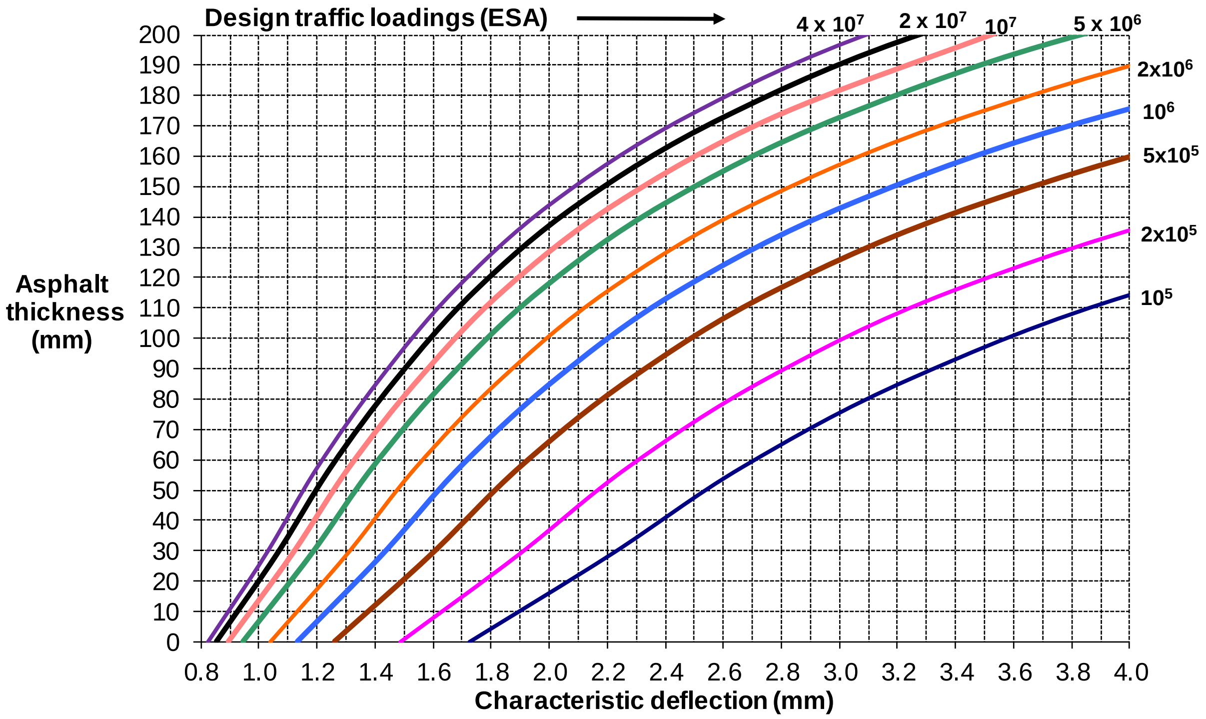
It is proposed that the Main Roads chart for overlays and inlays be developed based on the DPTI chart (Figure 3.5), but extended to a maximum design traffic loading of 4 x 10^7 ESA, consistent with the maximum loading in the curvature design charts (Figure 3.2, Figure 3.3 and Figure 3.4).

The Main Roads design chart method is applicable to a WMAPT of 29 °C (Perth) and to a size 14 mm asphalt mix with A15E binder. From Figure 6.10 of Austroads (2011), the overlay thickness at a temperature of 29 °C using such mixes is about 15% greater than that provided in Figure 3.5 and

Figure 3.6.

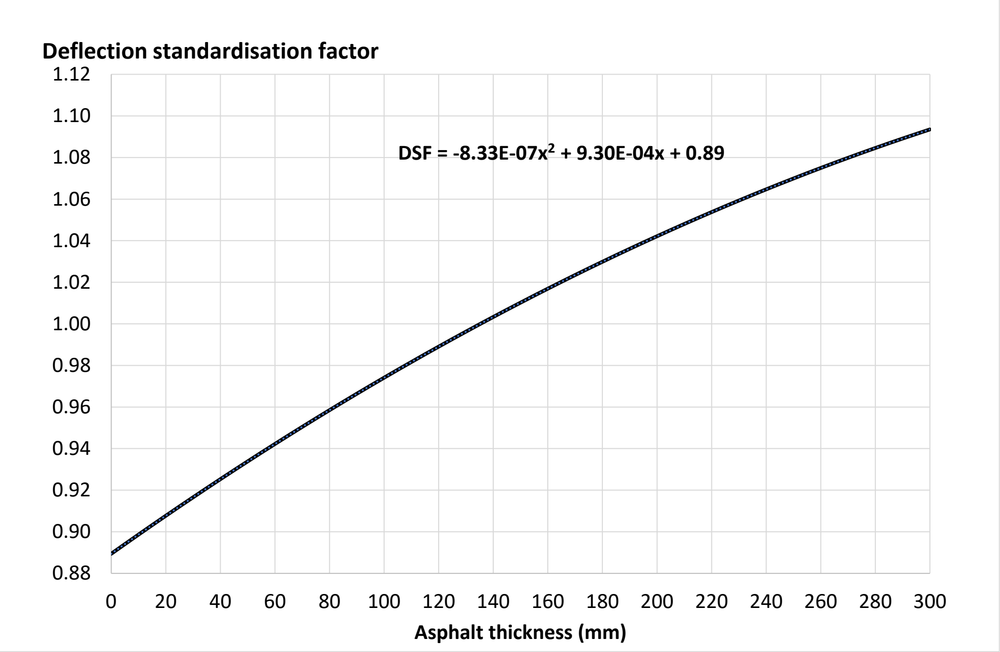
By increasing the Figure 3.5 overlay thicknesses and extrapolating the chart from 2 x 10^7 to 4 x 10^7 ESA, Figure 3.7 shows the proposed Main Roads design chart to determine the overlay/inlay thickness to inhibit permanent deformation.

Figure 3.7 Asphalt overlay design chart for permanent deformation at WMAPT of 29 °C



Note that characteristic deflections in Figure 3.7 relate to Benkelman beam rebound deflections. Consequently, measured FWD maximum deflections using a plate contact stress of 700 kPa need to be converted to estimated rebound maximum deflections under an 80 kN single axle with dual tyres (tyre inflation pressure 550 kPa) measured using a Benkelman beam. Austroads (2011) provided the deflection standardisation factors (DSFs) for use with FWD maximum deflections obtained using a 566 kPa contact stress. Assuming maximum deflections increase linearly with FWD plate load, the DSFs for use with 700 kPa deflections were obtained by multiplying the 566 kPa FWD DSF values by 0.81 (= 566/700). Figure 3.8 shows the resulting DSF values for use with FWD maximum deflections measured using 700 kPa contact stress.

Figure 3.8 Deflection standardisation factors for use with FWD maximum deflections measured using 700 kPa contact stress



3.3 CHARTS TO PREDICT DEFLECTION AND CURVATURE INCREASES DUE TO COLD PLANING

A chart-based asphalt inlay design procedure requires estimates of the increases in characteristic deflections (CD) and characteristic curvatures (CC) due to the cold planing of pavement materials.

This section details how charts were developed to allow DPTI to estimate the CD and CC on the planed surface from the values on the existing pavement prior to planing. These charts are recommended for use by Main Roads with some modifications.

3.3.1 DEFLECTION INCREASES

Development of the DPTI Charts

The DPTI overlay and P&R design procedures for permanent deformation (DPTI 2014) are based on Benkelman beam rebound deflections plus deflection standardisation factors to enable the use of maximum deflections measured with the FWD and Deflectograph. Consequently, the DPTI design procedure required a method to estimate the increases in Benkelman beam rebound deflections due to material excavation.

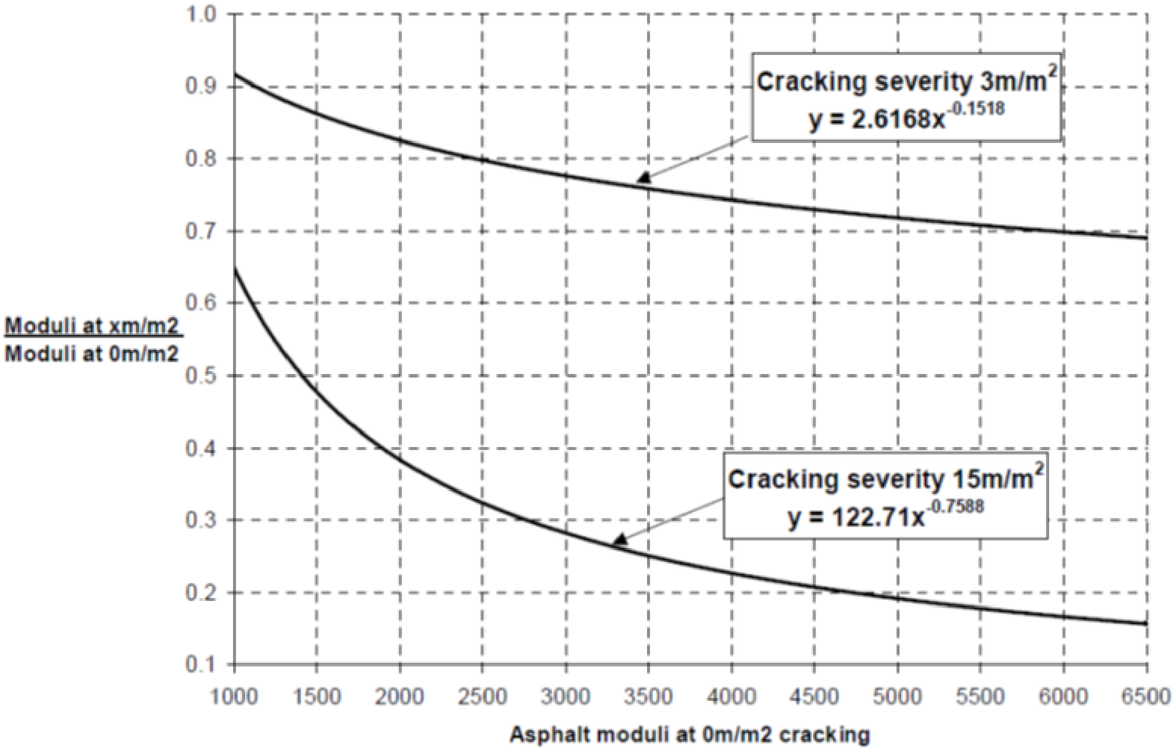
In the development of the DPTI design charts (Jameson 2005), it was decided to use mechanistic procedures to predict the increases in Deflectograph, rather than Benkelman beam, deflections as the Deflectograph is commonly used by DPTI. As asphalt and granular moduli are assumed to be the same during Deflectograph and Benkelman beam testing, the effect of excavating pavement materials was assumed to be the same also.

As discussed in Austroads (2008), due to the complexity of Deflectograph load geometry, linear elastic modelling was not sufficiently reliable to predict absolute deflection values. Nevertheless, the modelling was considered suitable for examining the effects of pavement material excavation as it utilised the ratio of two predicted maximum deflections rather than the absolute values.

The relationship between the increases in Deflectograph maximum deflections with the thickness of asphalt planed from the surface were predicted for pavements at a WMAPT of 27.5 °C, similar to the WMAPT for Adelaide. The pavement configurations modelled are summarised in Table 3.5. For each pavement configuration the deflections were predicted with and without selected asphalt thicknesses removed.

The increases in deflection depend on the modulus of the asphalt removed. As usually the modulus of the existing asphalt is not tested, it was decided to generate the design charts assuming the area to be planed is crocodile cracked (15 m/m^2). Austroads (2008) describes how the asphalt moduli used for the design of new pavements is reduced due to the severity of cracking (Figure 3.9). At a WMAPT of 27.5 ºC, the new asphalt modulus under Deflectograph loading (2 km/h) was assumed to be 1350 MPa. The modulus of crocodile cracked asphalt was obtained by multiplying this modulus by about 0.52 (Figure 3.9). Hence, in predicting the effect of asphalt excavation, the modulus of the existing asphalt at 27.5 °C under Deflectograph loading was assumed to be 700 MPa.

Figure 3.9 Relationships used to adjust asphalt modulus for severity of cracking



Source: Austroads (2008).

The deflections before and after asphalt planing were plotted for each asphalt thickness (Figure 3.10) and a regression line fitted to the data.

Table 3.5: Pavement configurations modelled for planing effects

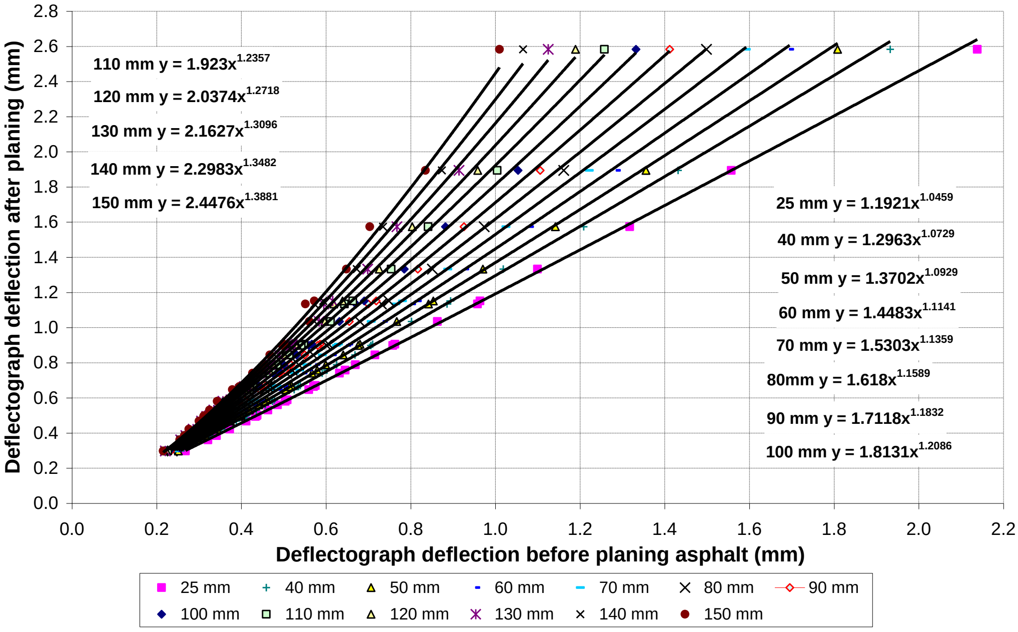
Layer number	Material	Thicknesses	Moduli
1	Asphalt to be removed	25, 40 to 150 mm in 10 mm steps	For deflection charts: 700 MPa for cracked under Deflectograph loading at 27.5 °C For curvature charts: 850 MPa for cracked asphalt under FWD loading at 27.5 °C
2	Granular	200 to 600 mm in 100 mm steps	Maximum moduli of 350 MPa, 500 MPa and 700 MPa, Austroads (2004) Guide sub-layering rules
3	Subgrade	Semi-infinite	30 MPa, 50 MPa, 70 MPa and 150 MPa

Source: Jameson (2005).

The increases in Deflectograph maximum deflections due to the planing of granular materials were also predicted as P&R often involves the removal of granular materials. The pavement configurations modelled were the same as those shown in Table 3.5 except that the pavements had zero thickness of asphalt and the effects on deflections of removing 25 mm to 250 mm of granular material were predicted.

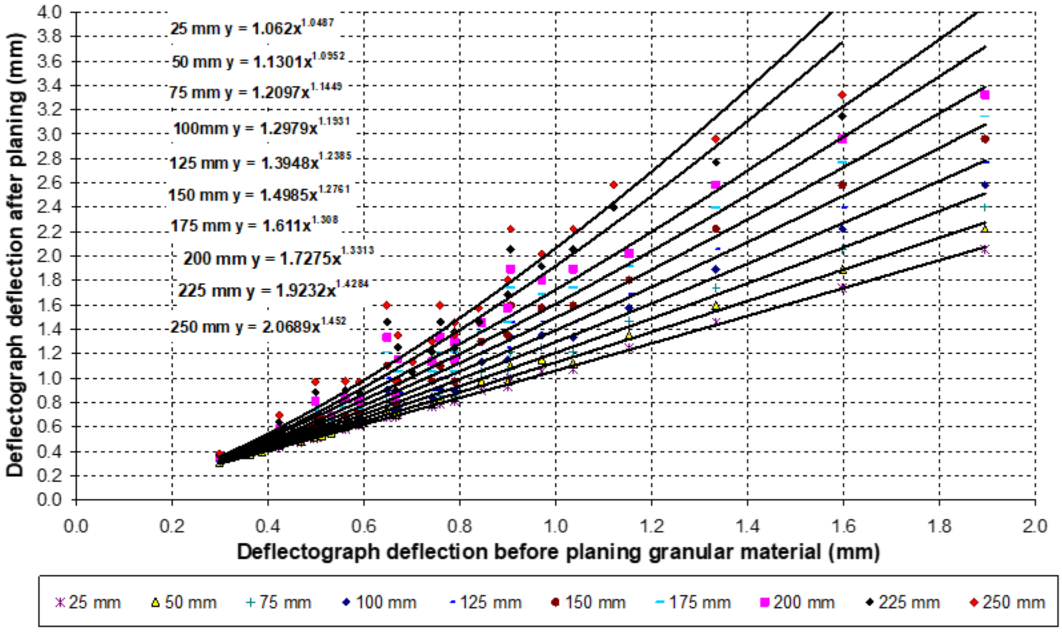
The deflections before and after planing of the granular materials were plotted for each thickness planed and a regression line fitted to the data (Figure 3.11).

Figure 3.10 Deflectograph maximum deflection increases due to planing of asphalt



Source: Jameson (2005).

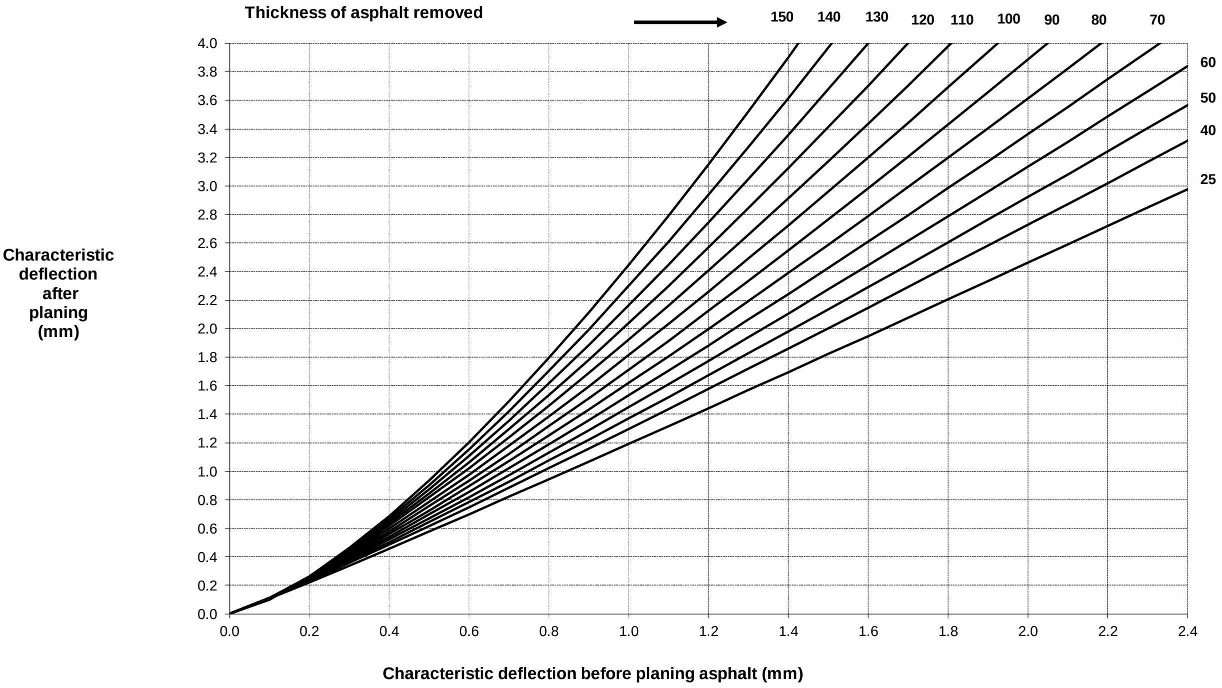
Figure 3.11 Deflectograph maximum deflection increases due to planing of granular material



Source: Jameson (2005).

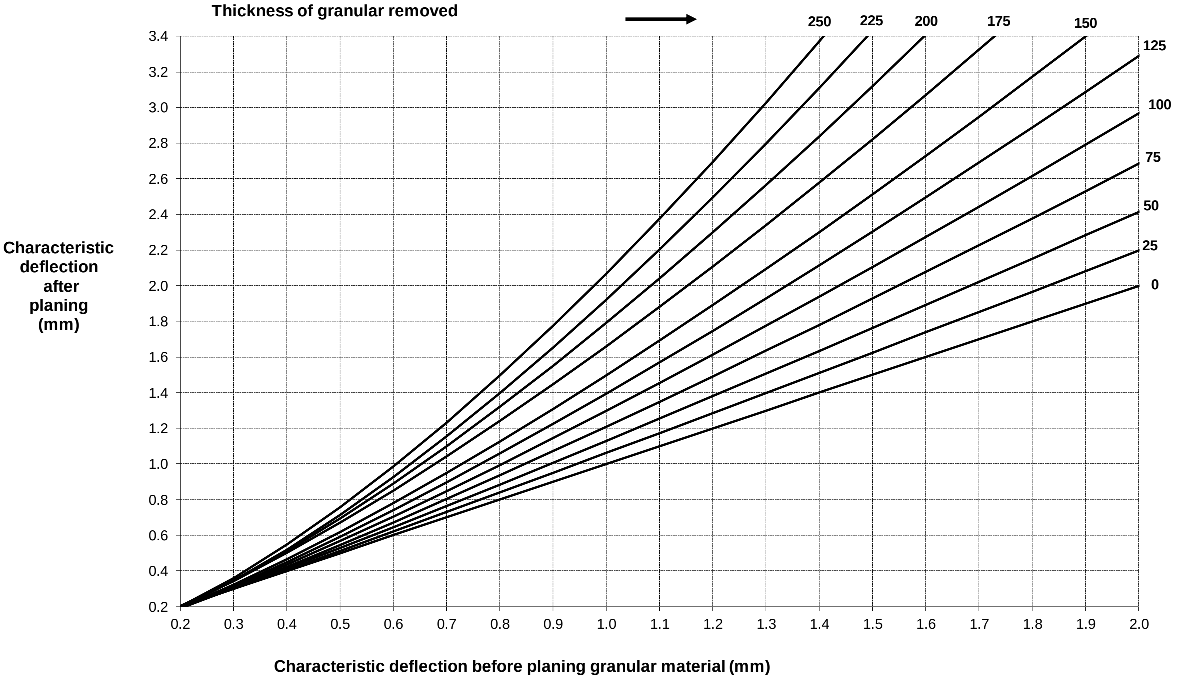
Figure 3.12 and Figure 3.13 show the resulting design charts adopted by DPTI (2014).

Figure 3.12 DPTI design chart for maximum deflection increases due to planing of asphalt



Source: DPTI (2014).

Figure 3.13 DPTI design chart for maximum deflection increases due to planing of granular material



Source: DPTI (2014).

Main Roads design charts

It is proposed that Main Roads adopt the DPTI charts for the effect of material excavations on deflections, modified as described below.

It is proposed to reduce the maximum deflections after planing plotted in the design charts to better reflect the maximum values predicted in the chart development. For example, in relation to the removal of asphalt the predicted maximum deflection measured using the Deflectograph was about 2.6 mm (Figure 3.10), which is equivalent to a Benkelman beam measured deflection of 2.6 x 1.2 = 3.1 mm. Accordingly, it is proposed that the Main Roads design chart be limited to a Benkelman beam maximum deflection of 3 mm as shown in Figure 3.14.

Similarly, with the removal of granular material the predicted Deflectograph maximum deflection was 3.4 mm. Accordingly, it is proposed that the Main Roads design chart for granular material removal be limited to 4 mm as shown in Figure 3.15.

Figure 3.14 Proposed Main Roads design chart to predict maximum deflection after excavation of asphalt

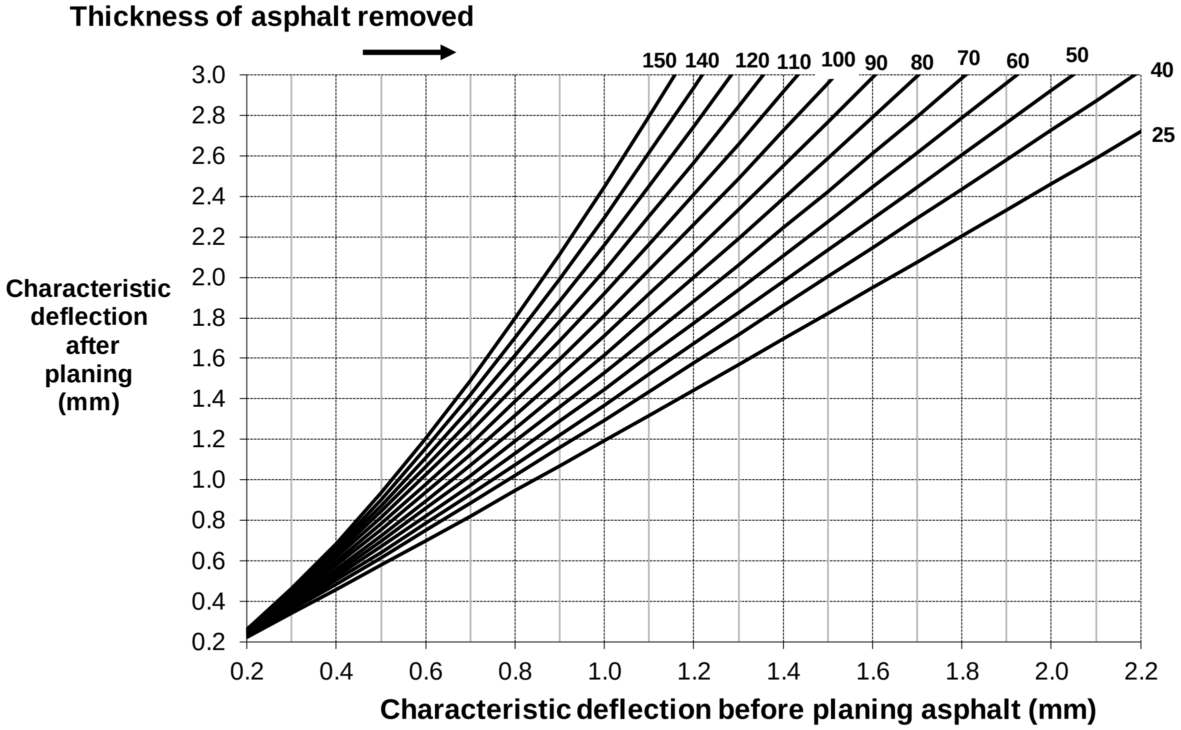
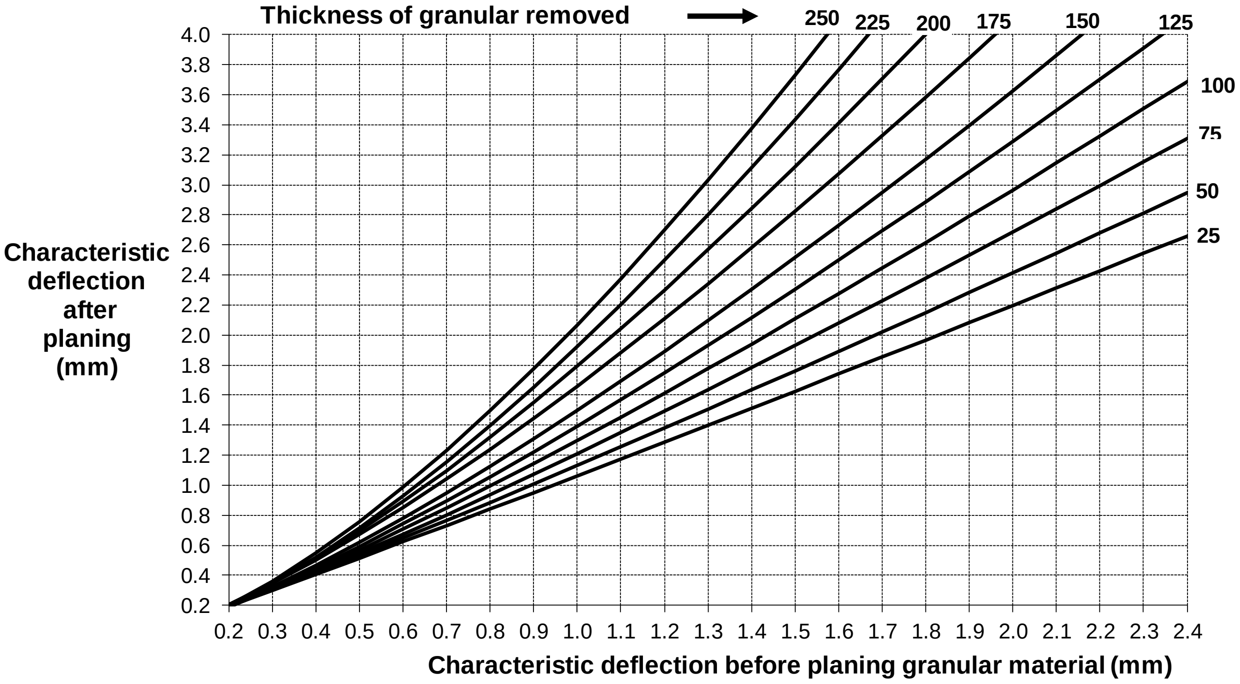


Figure 3.15 Proposed Main Roads design chart to predict maximum deflection after excavation of granular material



3.3.2 CURVATURE INCREASES

DPTI Design Charts

The DPTI design charts (Jameson 2005) were developed using mechanistic modelling to predict the effect of excavating asphalt and granular materials on FWD curvatures assuming a plate contact stress of 566 kPa. The development of these charts for the effect of the excavation of asphalt and granular materials is described below.

The increases in FWD curvatures with the thickness of asphalt removed were predicted at a WMAPT of 27.5 ˚C, this temperature being in the middle of the 25–30 °C range of interest to DPTI. The pavement configurations modelled are summarised in Table 3.5. The design chart for the effect of asphalt excavation was were prepared assuming the existing asphalt is crocodile cracked (15 m/m^2). Austroads (2008) describes how asphalt used for the design of new pavements is reduced due to the severity of cracking (Figure 3.9). At a WMAPT of 27.5 ºC, the new asphalt modulus under FWD loading was assumed to be 3000 MPa. The modulus of crocodile cracked asphalt was obtained by multiplying this modulus by about 0.28 (Figure 3.9). Hence, in predicting the effect of asphalt excavation, the existing asphalt modulus at a temperature of 27.5 °C under FWD loading was assumed to be 850 MPa.

For each pavement configuration the curvatures were predicted before and after selected thicknesses of asphalt planing. The curvatures were plotted, and a regression line fitted to the data; Figure 3.16 is an example. Using the regression lines, the design chart shown in Figure 3.17 was obtained.

Figure 3.16 Example of the changes in predicted curvatures due to removal of 100 mm and 150 mm thicknesses of asphalt

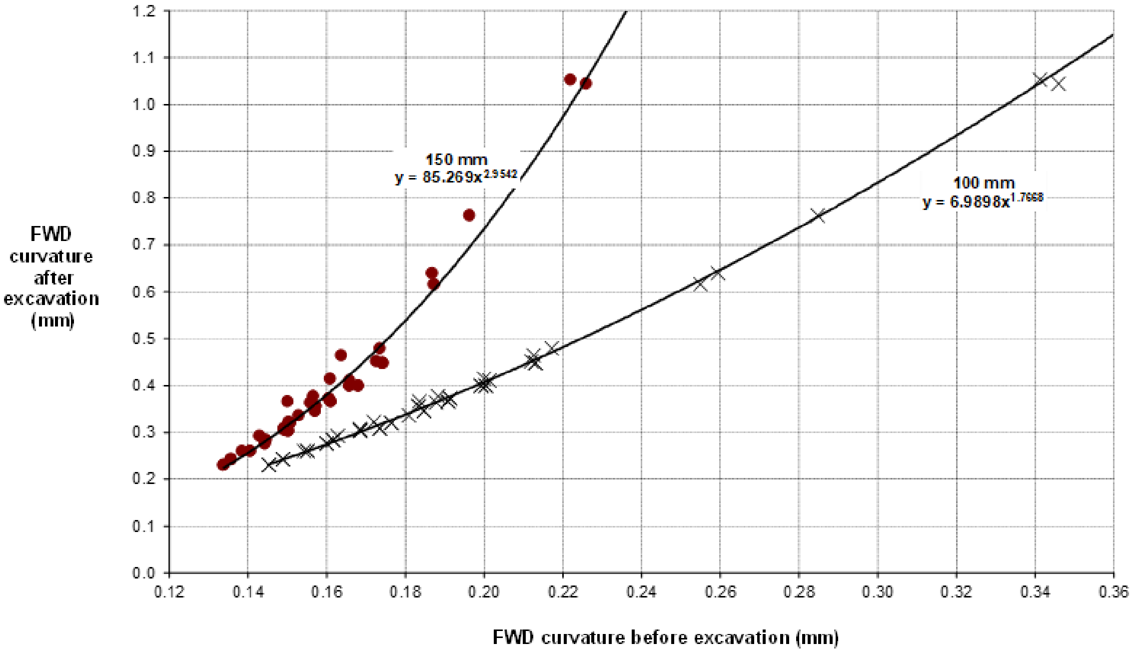
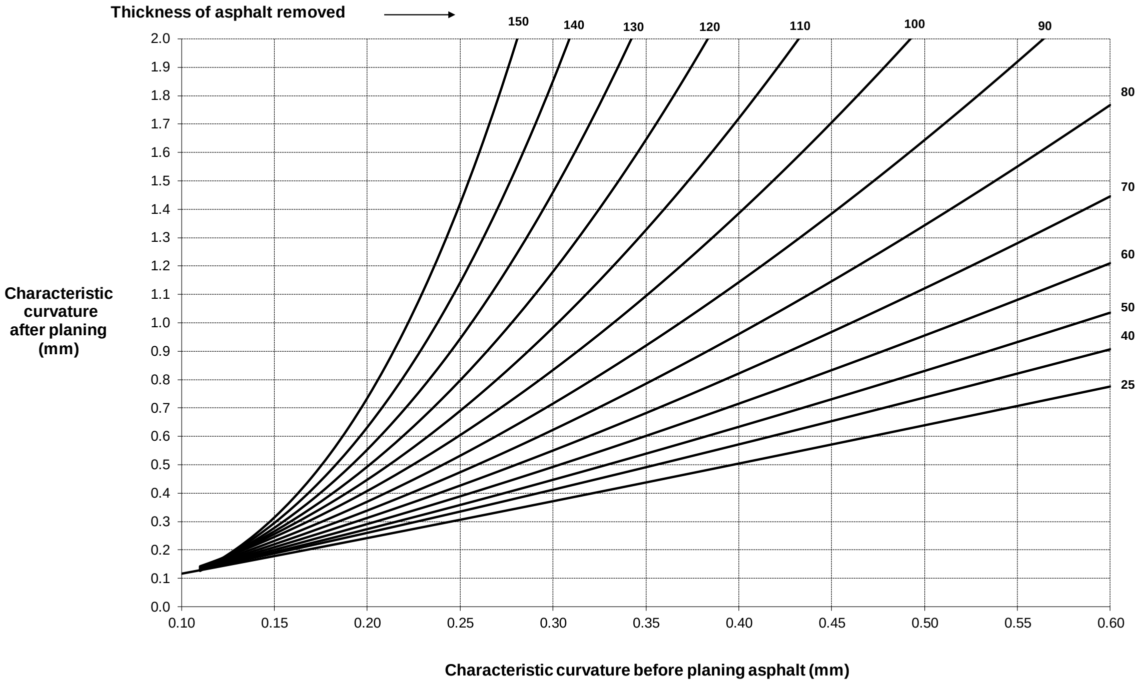


Figure 3.17 DPTI curvature increases due to planing of asphalt



Source: DPTI (2014).

The increases in FWD curvature due to the excavation of granular materials were also predicted as inlays may involve the removal of granular materials. The pavement configurations modelled were the same as shown in Table 3.5, except that the pavements had zero thickness of asphalt and the effects on curvature of removing 25 mm to 250 mm of granular material were predicted. The curvatures before and after planing of granular materials were plotted for each thickness excavated and regression lines fitted to the data; Figure 3.18 is an example. Using the regression lines, the design chart shown in Figure 3.19 was obtained.

Figure 3.18 Example of changes in predicted curvatures due to removal of 100 mm and 150 mm thicknesses of granular material

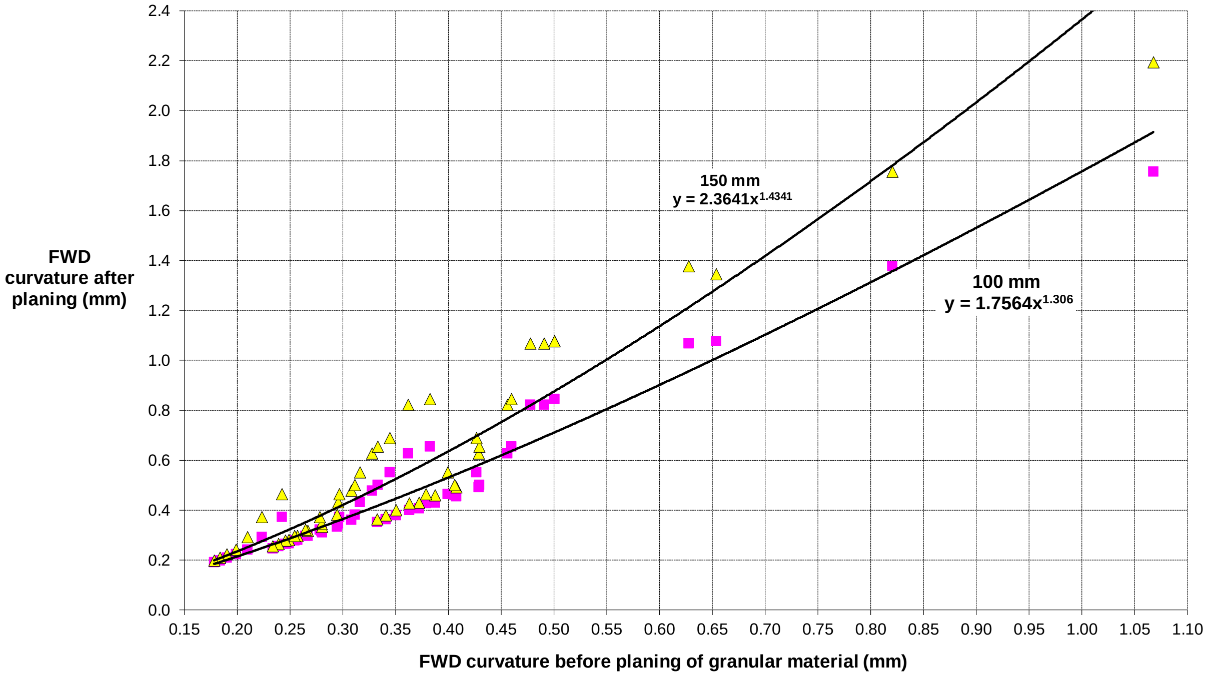
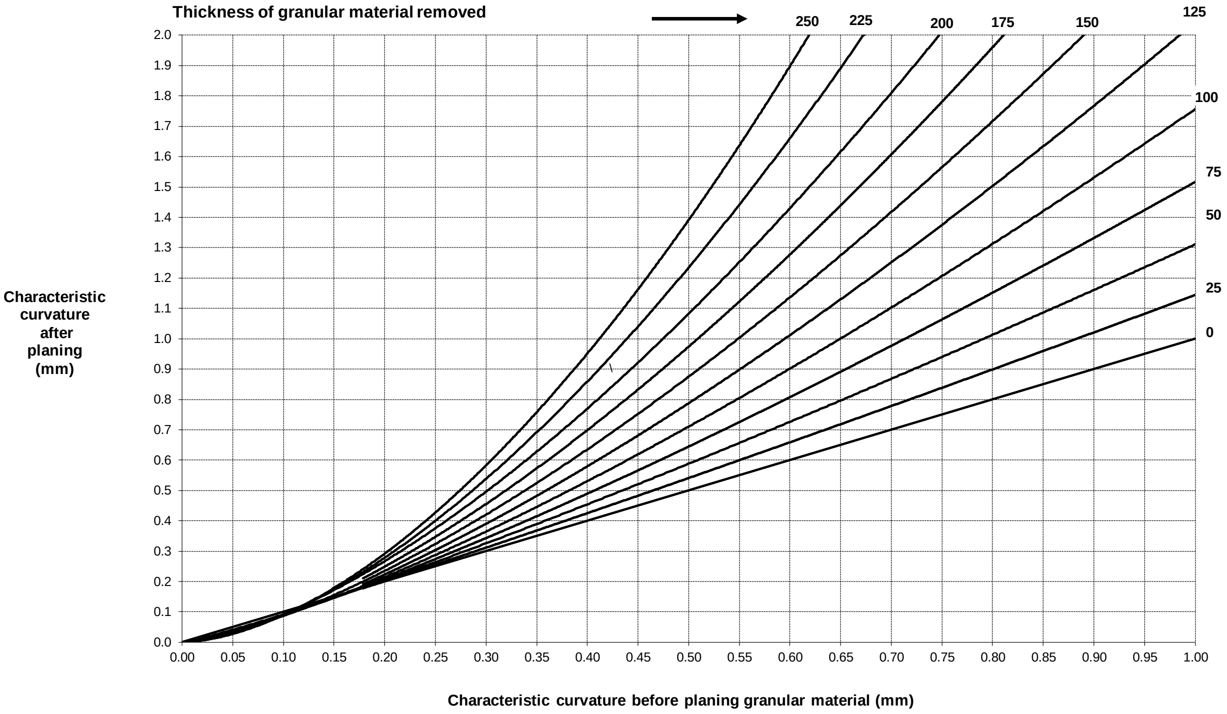


Figure 3.19 FWD curvature increases due to planing of granular material



Source: DPTI (2014).

Main Roads Design Charts

The Main Roads design procedure for asphalt fatigue of overlays and inlays is based on FWD curvatures ($D_0 – D_{200}$) under a 700 kPa plate contact stress. Consequently, the design procedure required a method to estimate the increase in FWD 700 kPa curvatures due to material excavation.

The modelling process used to derive the DPTI curvature increase charts assumes pavement curvatures before and after excavation increase linearly with applied load. Therefore, for a given pavement configuration both the curvatures before and after excavation under a 700 kPa stress are a factor of 1.24 (700/566) higher than under a contact stress of 566 kPa. As both curvatures are increased by the same factor, the DPTI curvature increase charts may be used by Main Roads to estimate increases in 700 kPa curvatures due asphalt and granular materials.

It is noted that the Main Roads design curvature charts were developed assuming the existing asphalt had a modulus of 2190 MPa at the time of deflection testing (Section 3.1.2) and assuming the existing asphalt had the beginnings of fatigue cracking (3 m/m^2). By comparison, the DPTI curvature increase charts assume that the asphalt that needs to be planed is more severely cracked (15 m/m^2) and has an associated modulus of 850 MPa. This was discussed with Main Roads and it was accepted as it is likely that areas to be planed and reinstated are more severely cracked than areas to be overlaid.

The DPTI design charts allowed the predictions of curvatures up to 2.0 mm after excavation. By comparison, the maximum before overlay/inlay 50 kN curvature in the proposed Main Roads design chart (Figure 3.4) is half this value. Consequently, it is proposed the DPTI design charts be limited to a maximum curvature of 1.0 mm. The proposed Main Roads charts are shown in Figure 3.20 and Figure 3.21.

Figure 3.20 Proposed Main Roads design chart to predict curvature after excavation of asphalt

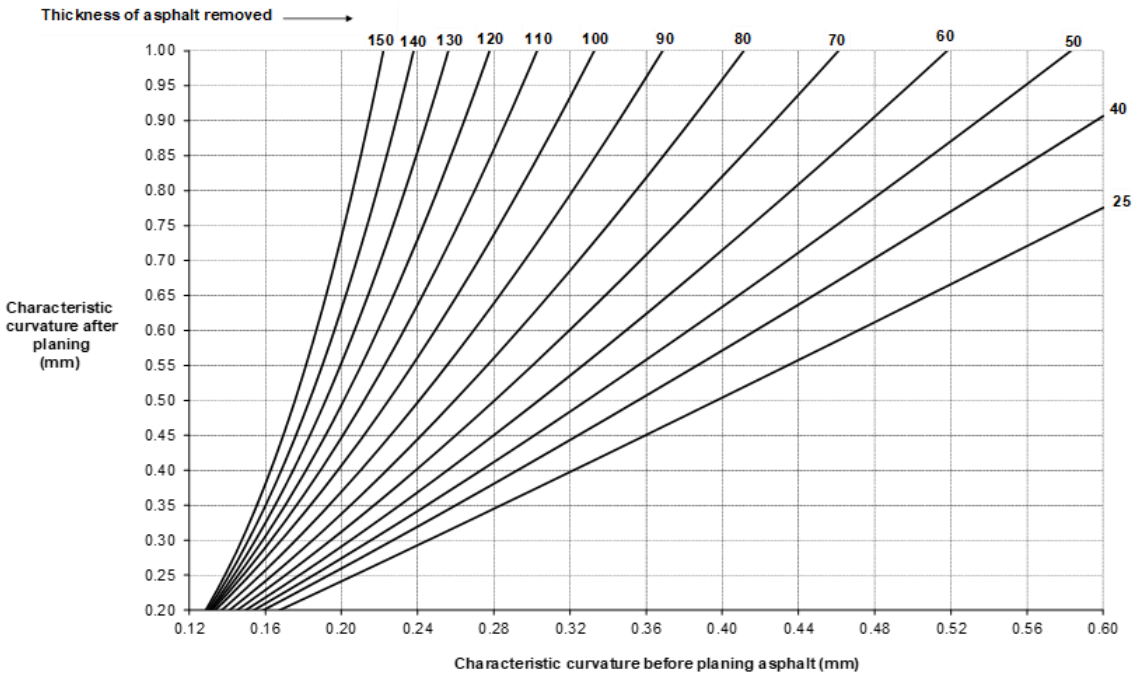
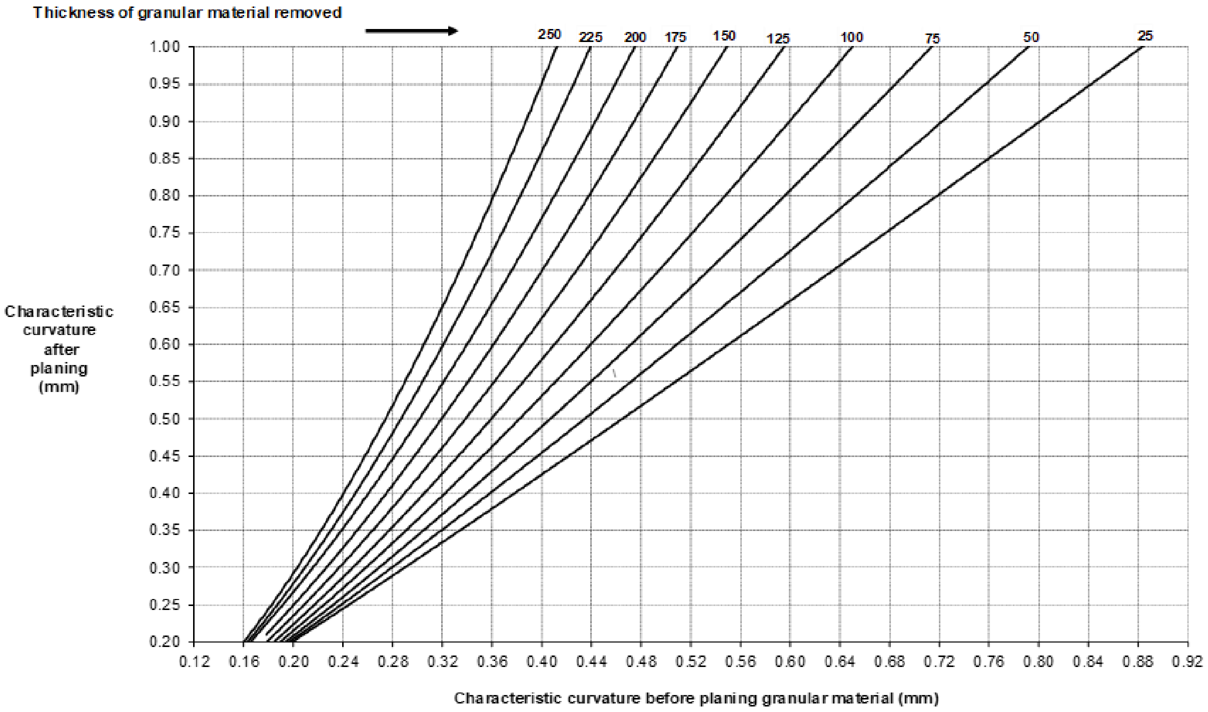


Figure 3.21 Proposed Main Roads design chart to predict curvature after excavation of granular material



3.4 SUMMARY

This section has describes the development of the Main Roads asphalt overlay and inlay design charts. They are used to predict the following:

- The allowable traffic loading in relation to asphalt fatigue life of an overlay/inlay from the measured FWD curvatures ($D_0 – D_{200}$) under a 700 kPa plate contact stress before asphalt overlay/inlay. For overlays/inlays up to 70 mm thick, two design charts have been developed: one for use when the pavement on which the overlay is placed includes existing asphalt, the other for overlays on surfaces without existing asphalt. In relation to overlays/inlays of 80 mm to 200 mm, a single design chart is recommended based on the assumption that such thick treatments will only almost always be applied to surfaces with existing asphalt.
- The allowable traffic loading in relation to pavement permanent deformation from the measured FWD maximum deflection (D_0) due to 700 kPa plate contact stress before asphalt overlay/inlay.
- The increase in maximum deflections and curvatures due to cold planing/excavation of existing asphalt and granular materials. These design charts were based on the DPTI (2014) design charts.

These charts have been included in the Main Roads rehabilitation supplement, ERN16.

4 TRAFFIC SPEED DEFLECTOMETER DEFLECTION STANDARDISATION FACTORS

4.1 INTRODUCTION

The Traffic Speed Deflectometer (TSD) is now used by Main Roads for road network strength evaluation. Procedures are provided in AGPT05-19 (Austroads 2019a) for the project-level design of granular overlays from TSD maximum deflections. These procedures require the development of a deflection standardisation factors (DSF) to enable Benkelman beam maximum deflections to be estimated from measured TSD values. Equivalent Benkelman beam deflections were required as the AGPT05-19 design deflections were empirically derived from measured Benkelman beam deflections.

Austroads (2019b) describes the method to derive the AGPT05-19 DSF based on correlating TSD and FWD deflections measured on Queensland and New Zealand roads.

To evaluate the validity of using the AGPT05-19 DSF on the Western Australian road network, the Western Australian data was analysed and compared to the data used to derive the AGPT05-19 DSF.

Procedures for the design of asphalt overlays using curvature (D_0 – D_{200}) are not provided in AGPT05-19. Hence there was no need for Austroads to develop curvature standardisation factors (CSF). However, given the proposed use by Main Roads of the asphalt overlay and inlay design charts (Section 3), CSFs were derived for consideration by Main Roads in ERN16.

4.2 OVERVIEW OF WA DEFLECTION DATASETS

To develop DSF and CSF for the TSD, deflection data for both the TSD and FWD needed to be available, with the following attributes:

- same locations (i.e. road, lane, and approximate chainage/coordinates)
- similar climate conditions at the conduction of the deflection surveys (or a pavement structure/subgrade that would allow for reliable climate correction).

The available deflection data found for WA with correlating TSD and FWD locations is listed in Table 4.1.

Table 4.1: Available Western Australian TSD-FWD location-paired deflection data

Id	Road	Pavement type	Lane	FWD – date	FWD – avg. Temperature (°C)	TSD – date	TSD – avg. Temperature (°C)	Number of paired points
11	Bussell Hwy	Sealed granular	Left	16–17/05/2019	7.14	03/12/2018	46.33	185
12	Bussell Hwy	Sealed granular	Right	16–17/05/2019	7.63	04/12/2018	34.18	187
21	Forrest Hwy	Sealed granular	Left	17/08/2018	21.76	07/12/2018	39.25	52
22	Forrest Hwy	Sealed granular	Right	17/08/2018	24.92	07/12/2018	38.53	20
31	Kwinana Fwy	Asphalt surfaced granular (60 mm thick)	N/A	26/10/2018	17.58	26/10/2018	20.73	6
41	Leach Hwy	Asphalt surfaced granular (30 mm thick)	N/A	27/10/2018	17.83	27/10/2018	20.39	6
51	Trial Mile Road	Asphalt surfaced granular (60 mm thick)	Left	12/04/2018	27.38	14/04/2018	26.23	150
52	Trial Mile Road	Asphalt surfaced granular (60 mm thick)	Right	12/04/2018	28.32	14/04/2018	33.70	150
61	Great Eastern Hwy	Sealed granular	Left	07–22/11/2018	41.12	08–09/11/2018	41.80	927
62	Great Eastern Hwy	Sealed granular	Right	08–23/11/2018	44.70	15/11/2018	42.17	939
71	Tonkin Hwy Extension.	Sealed granular	Left (NB)	30/01–01/02/2020	29.54	02/05/2020	32.47	385
72			Left (SB)	25–26/01/2020	40.51	02/05/2020	29.55	407
73	Tonkin Hwy Extension.	Sealed granular	Right (NB)	29–30/01/2020	34.29	02/05/2020	33.73	390
74			Right (SB)	26–29/01/2020	42.37	02/05/2020	32.40	400

Where: Hwy = highway, Fwy = freeway, NB = Northbound, SB = Southbound, and Trial Mile = section of Kwinana freeway/Perth-Bunbury highway.

4.3 DATA CONSIDERATIONS AND POTENTIAL ISSUES

For the collated and paired data, the potential issue of seasonal variation in temperature and moisture conditions during deflection testing was considered. For the Kwinana Freeway, Leach Highway, Trial Mile Road and Great Eastern Highway data, there was no seasonal variation due to the FWD and TSD surveys collecting data within a maximum of two weeks of each other (if not within two days, or on the same day).

However, it is evident in Table 4.1 that the FWD and TSD surveys of the Tonkin Highway Extension, Bussell Highway and Forrest Highway occurred at separated points in time. In addition, the TSD and FWD surveys of the Bussell and Forrest Highways were conducted under significantly different temperature conditions.

To further assess the variation in conditions, and potential moisture effects on the subgrades, between each of the deflection surveys, rainfall and temperature information (sourced from the Bureau of Meteorology (2020)) for the six months prior to data collection was considered (Figure 4.1 to Figure 4.5).

Figure 4.1 Weather prior to FWD collection: Bussell Highway

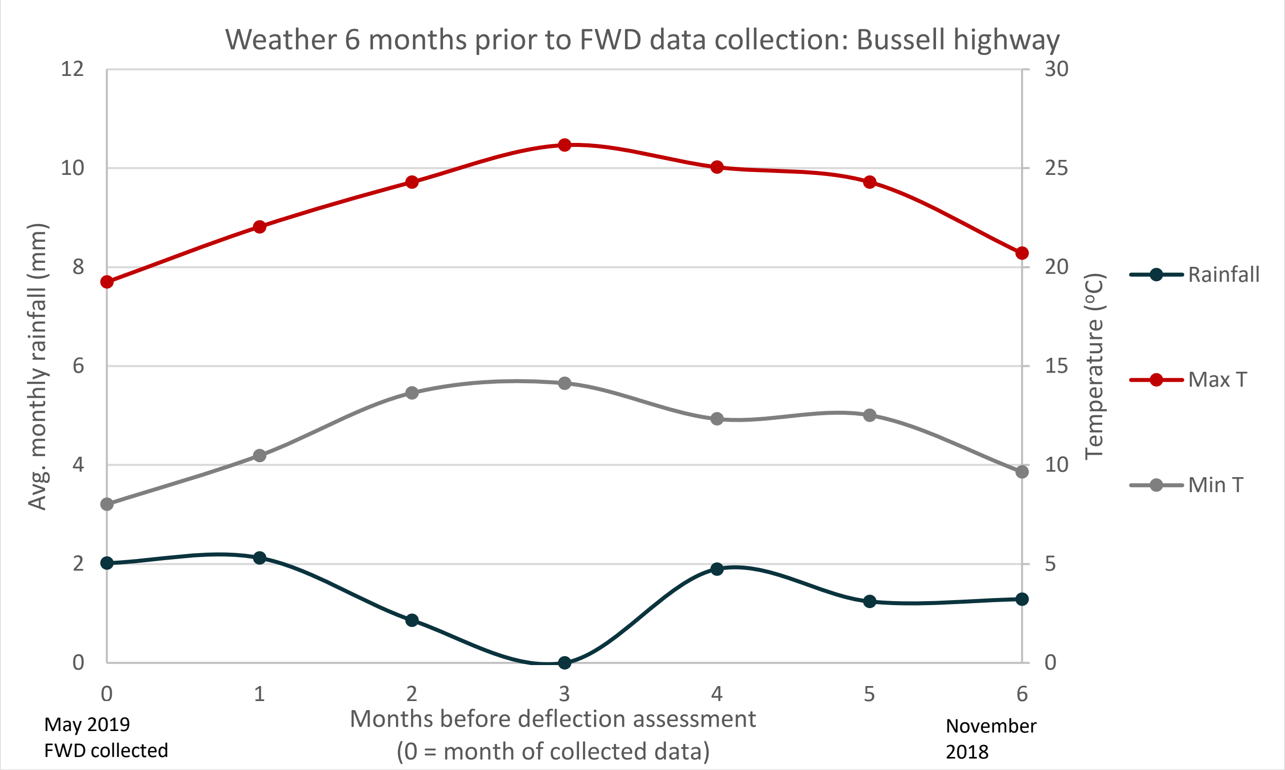


Figure 4.2 Weather prior to FWD collection: Forrest Highway

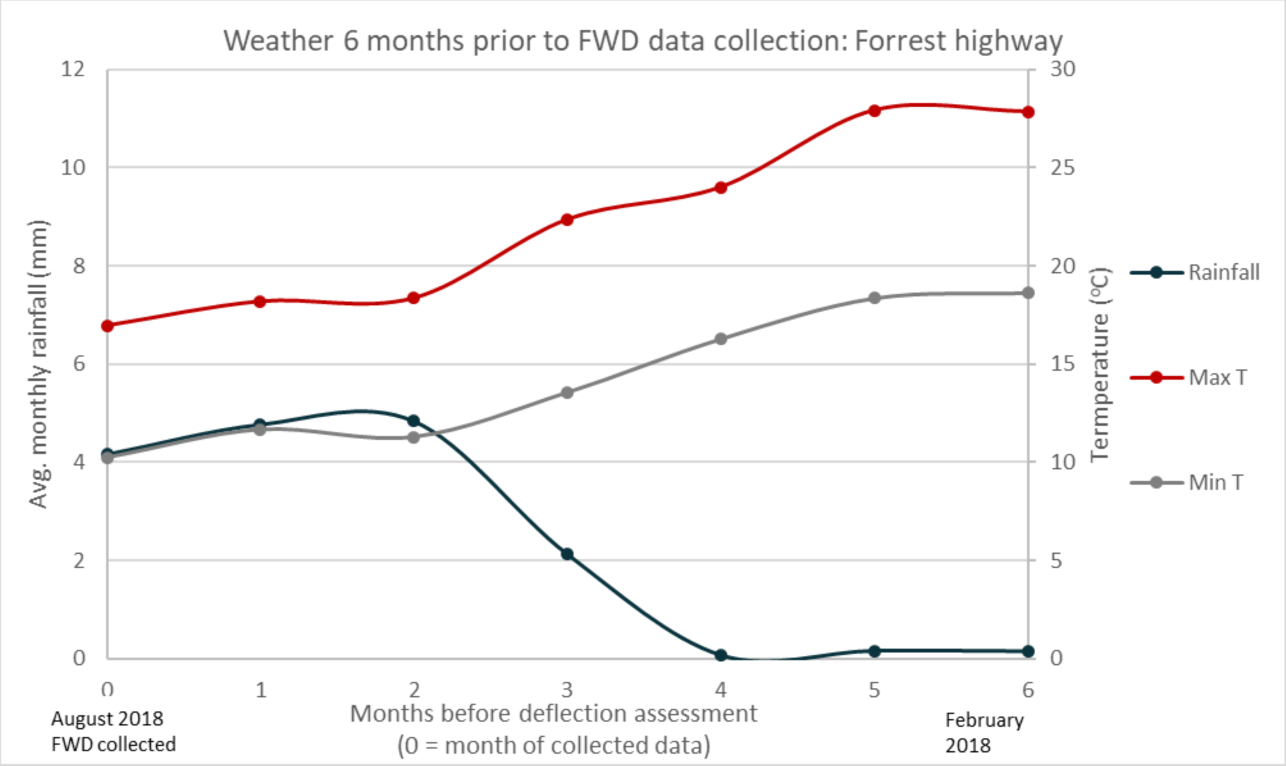


Figure 4.3 Weather prior to TSD collection: Bussell highway and Forrest Highway

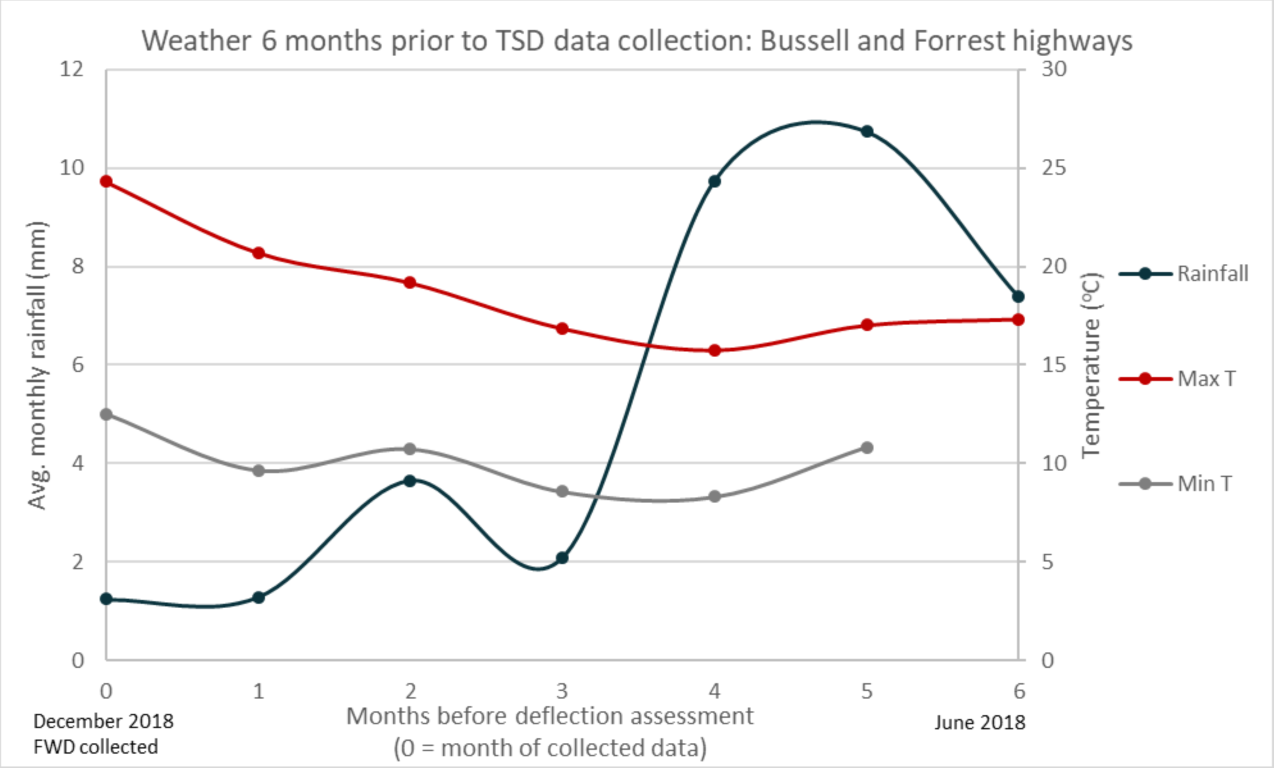


Figure 4.4 Weather prior to FWD collection: Tonkin Highway Extension

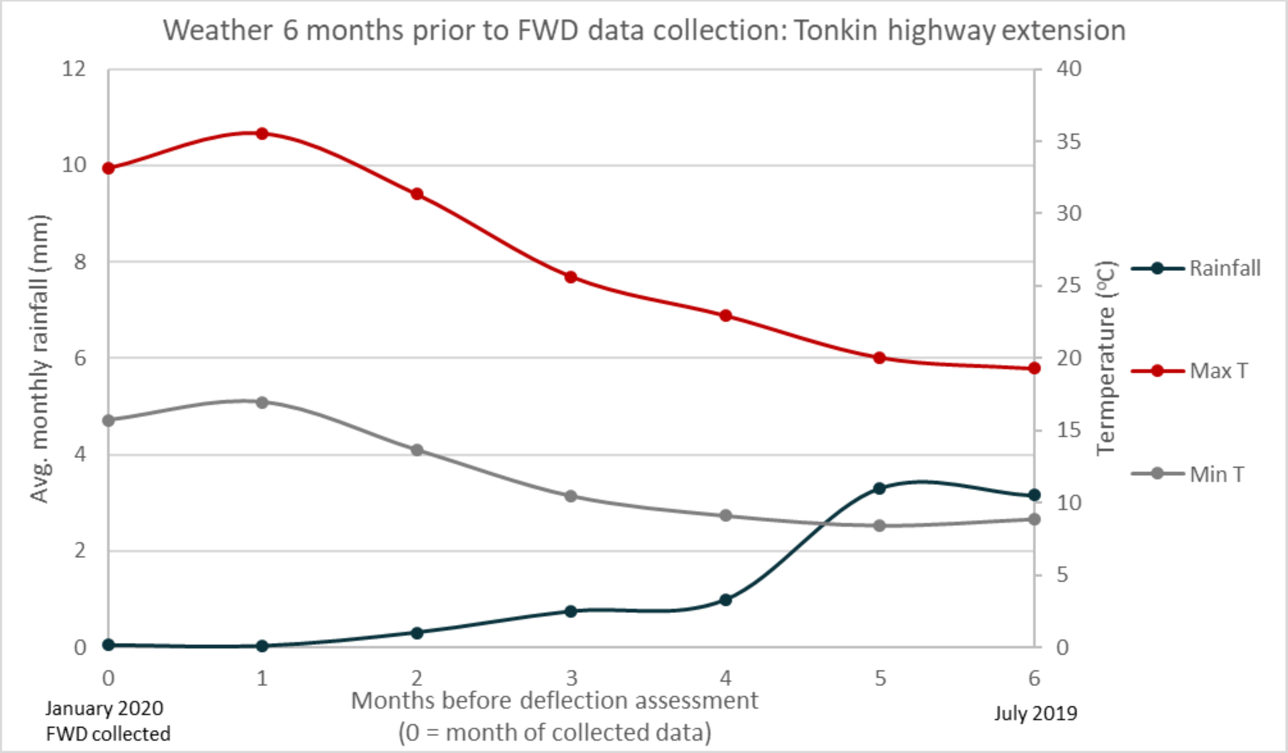
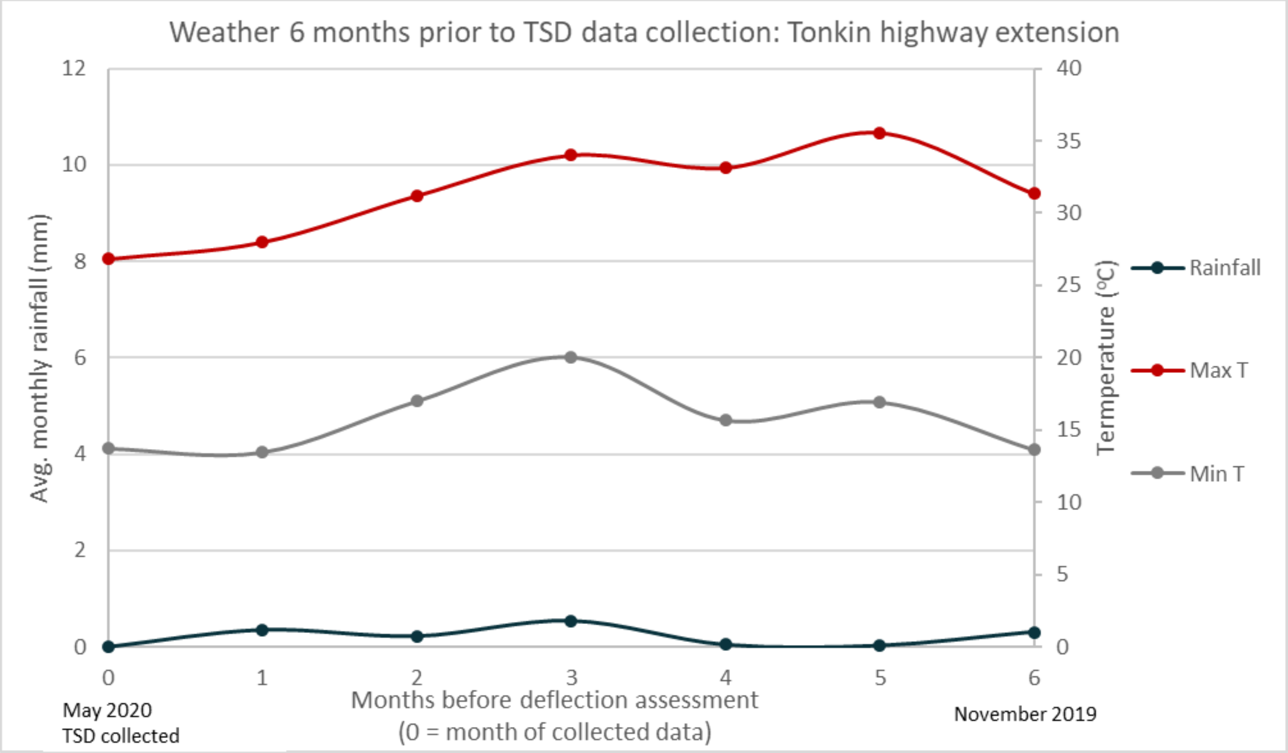


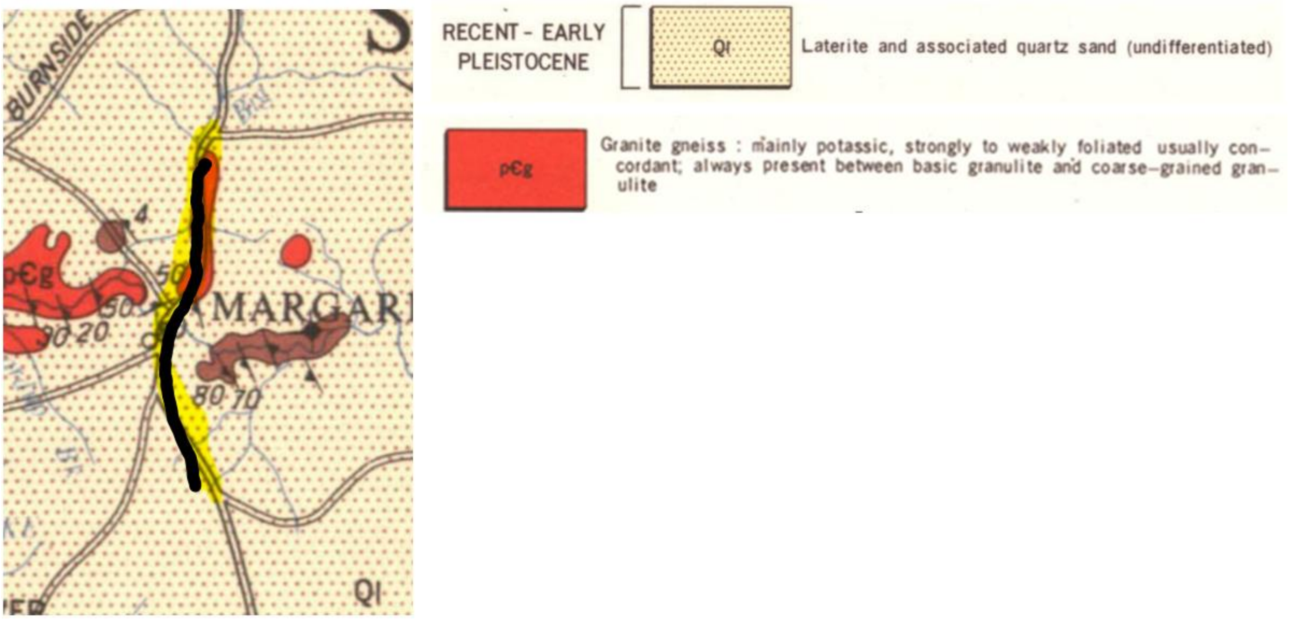
Figure 4.5 Weather prior to TSD collection: Tonkin Highway Extension



4.3.1 BUSSELL HIGHWAY

The meteorological data for the Bussell Highway (Figure 4.1 and Figure 4.3), indicated that the months preceding the TSD data collection were comparatively cooler and wetter than those preceding the FWD assessment. The surveyed area of the Bussell Highway was also found to have a subgrade composed mostly of laterite (clayey-gravel sand mix), with a portion of the road lying over a granite or slightly weathered rock subgrade (Figure 4.6). The concern with this subgrade is that the clayey areas, when subject to wet periods, are prone to some swelling (i.e. sensitive to moisture). With the noted difference in moisture conditions preceding the different deflection surveys, if the subgrade had insufficient time to dry prior to the TSD data collection, then the response under load would be notably different between the TSD and FWD surveys and, unlike a temperature effect, this would be very difficult to correct.

Figure 4.6 Bussell highway geology



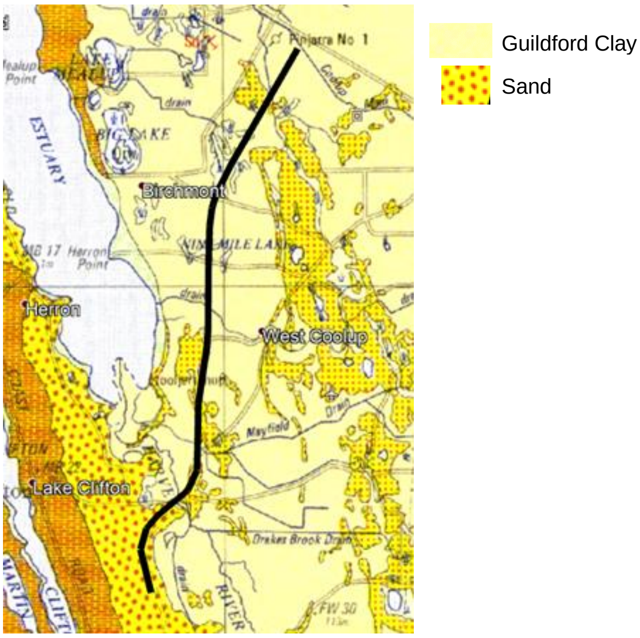
Source: Geological Survey of Western Australia (1967).

4.3.2 FORREST HIGHWAY

For the Forrest Highway, it was noted from the meteorological data (Figure 4.2 and Figure 4.3) that the months preceding the TSD survey became slightly drier than those preceding the FWD survey. However, the temperatures leading up to each deflection survey were, on average, fairly similar.

The geology for the surveyed area of the Forrest Highway was also considered (Figure 4.7) and it was noted that most of the pavement was constructed over a Guildford clay subgrade. This material tends to swell and shrink with moisture/drying. However, the pavement formation for the Forrest Highway appears to be located on an embankment which will act as a buffer against movement in the subgrade, and thus mitigate the shrink-swell moisture effects of the clay.

Figure 4.7 Forrest highway geology



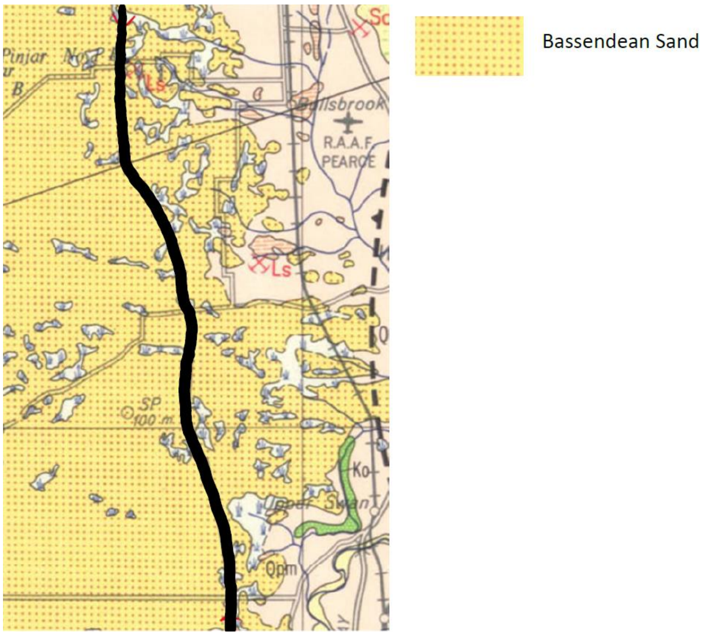
Source: Geological Survey of Western Australia (1980).

4.3.3 TONKIN HIGHWAY EXTENSION

The meteorological information (Figure 4.4 and Figure 4.5) for the Tonkin Highway Extension showed that the conditions in the months leading up to both the TSD and FWD surveys were fairly similar. While the lead up to the FWD survey was slightly wetter, the overall rainfall was fairly low (especially in the three months prior to data collection). The temperatures were also rising, and higher average temperatures in the lead up to the FWD survey would result in the pavement sufficiently drying out to a comparable state as that experienced during the TSD survey.

Further, the geology of the subgrade (Figure 4.8) was found to be almost exclusively Bassendean sand, which is not a moisture-sensitive material. As such, if there were any notable moisture condition differences between the TSD and FWD survey periods, the subgrade would be unlikely to impact on the pavement behaviour to any significant extent during the deflection testing.

Figure 4.8 Tonkin Highway Extension geology



Source: Geological Survey of Western Australia (1978).

4.4 INITIAL ASSESSMENT OF DATA SUITABILITY

In terms of collection conditions, the available datasets, except for the Tonkin Highway Extension, Bussell Highway and Forrest Highway, were identified as suitable for inclusion in the collated dataset to derive DSF and CSF for use with the TSD data.

In the case of the Tonkin Highway Extension, while both the TSD and FWD surveys occurred during different seasons, the weather conditions, alongside a stable subgrade, leading up to both surveys were deemed to be sufficiently similar such that any potential effects would be insignificant. As such, the Tonkin Highway Extension data was also deemed suitable for inclusion in the collated dataset.

For the Bussell and Forrest Highways, the weather conditions experienced in the time leading up to the TSD survey compared to the FWD survey were notably different. This difference, considered in conjunction with the identified pavement formations, suggested that the moisture sensitivity effects of the natural subgrade under the Forrest Highway would be mitigated by the embankment. This enabled the pavement response to load to be considered as mostly unaffected by the differing moisture conditions for both deflection surveys on the Forrest Highway, deeming the data suitable for inclusion in the collated dataset.

However, for the Bussell Highway, the subgrade moisture sensitivity, and the differing environmental conditions, was likely to have a significant impact on the pavement response to load, and thus on the FWD-to-TSD correlation. As such, the deflection data for Bussell Highway was flagged for exclusion from the collated dataset for the next step of the project in deriving the TSD adjustment factor.

4.5 PRELIMINARY DATA ASSESSMENT

Prior to deriving the DSF and CSF values through regression analysis on the TSD and FWD datasets listed in Table 4.1, the quality of the data was assessed. While the above discussion outlines the general availability of TSD and FWD datasets with suitably similar collection conditions, the appropriateness of the deflection datapoints must also be considered.

The following outlines the preliminary processing and assessment of the obtained datasets, in regard to deflection value normalisation, data pairing, and the apparent TSD vs. FWD behaviour at a surface level.

4.5.1 METHODOLOGY

The preliminary analysis methodology applied to the deflection data is modelled on Section 2 of Austroads (2019b). A general overview of the applied process, and any deviations from that reported in Austroads (2019b), is presented as follows:

- Location pairing[1]:

TSD and FWD datapoints are paired based on their location by matching the GPS coordinates of the two sets of deflection data (if provided). If GPS coordinates are unavailable, the designated road, lane, and chainage/Straight Line Kilometre (SLK) are used.

- Temperature correction[2]:

Where applicable (i.e. for asphalt pavements) FWD deflection datapoints (D_0 and curvature only) are temperature corrected to the recorded temperature of their paired TSD deflections using equation 39 from Austroads (2008), with regression coefficients applicable to an asphalt thickness of either 25 mm or 50 mm.

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[1] Unlike in Austroads (2019b), the recorded locations of the collated data points for these datasets were well aligned. As such, there was no need to average multiple recorded data points together over a selected interval to ensure that both the TSD and FWD values represented the same section of road.

[2] The thicknesses of asphalt within the collated dataset were 30 mm (Leach Highway), and 60 mm (Kwinana Freeway and Trial Mile Road). As Austroads (2008) only lists regression coefficients in 25 mm thickness increments, the coefficients for the nearest asphalt thicknesses (25 mm and 50 mm, respectively) were selected.

- Load normalisation:

All deflections are normalised to a load of:

a. 50 kN for TSD maximum deflections and curvatures, or
b. 40 kN for FWD maximum deflections and curvatures.

- Preliminary processing of the data:

The FWD and TSD deflection data is compared in XY coordinates in Figure 4.9 (D_0) and Figure 4.10 (curvature). Any datapoints that appeared to be highly scattered, or subsets of data that demonstrated unusual behaviour, were flagged for closer investigation and potential removal, subject to the initial regression analysis described in Section 4.6.

4.5.2 OVERVIEW OF PROCESSED DEFLECTION DATA

The deflection data for each road is provided in Appendix D.

A review of all the deflection datasets listed in Table 4.1 was undertaken using Figure 4.9 and Figure 4.10. It was identified that a majority of the data was well-aligned with consistent, and fairly linear correlation between the FWD and the TSD deflections. However, while the presence of scatter is an expected reality in these deflection datasets, it was noted that there was more scatter in the Bussell Highway and Great Eastern Highway datasets. The scatter in these datasets also showed a notable deviation from the trend in the main body of data (most notably in Figure 4.10).

As noted in Section 4.4, the Bussell Highway dataset was flagged for possible exclusion due to differences in environmental conditions that were expected to impact the pavement's response to load. Considering this, and the preliminary assessment of the data, the Bussell Highway data was excluded from the regression analysis to derive DSF and CSF values (Section 4.6) to avoid introducing a bias effect from potential environmental factors.

For the Great Eastern Highway, there was no immediately obvious reason for the scatter in the FWD vs TSD data as noted above. However, as Figure 4.9 and Figure 4.10 indicate a significant amount of scatter, the deflection data for the Great Eastern Highway was flagged for potential exclusion from the DSF and CSF derivations, subject to further investigation through preliminary regression analysis (Section 4.6.2).

Similarly, the right lane of Trial Mile Road showed unexpected behaviour compared to the other datasets. The maximum deflections for Trial Mile Road (Figure D.7) clearly showed two distinct populations between the right and left lanes for the same TSD range. Additionally, the curvatures for the right lane (Figure D.8) were distinctly different to the left lane of Trial Mile Road, and to all the other datasets. As such, this dataset was excluded from the analysis.

Figure 4.9 Comparison of FWD vs TSD maximum deflection data (D_0) – all roads

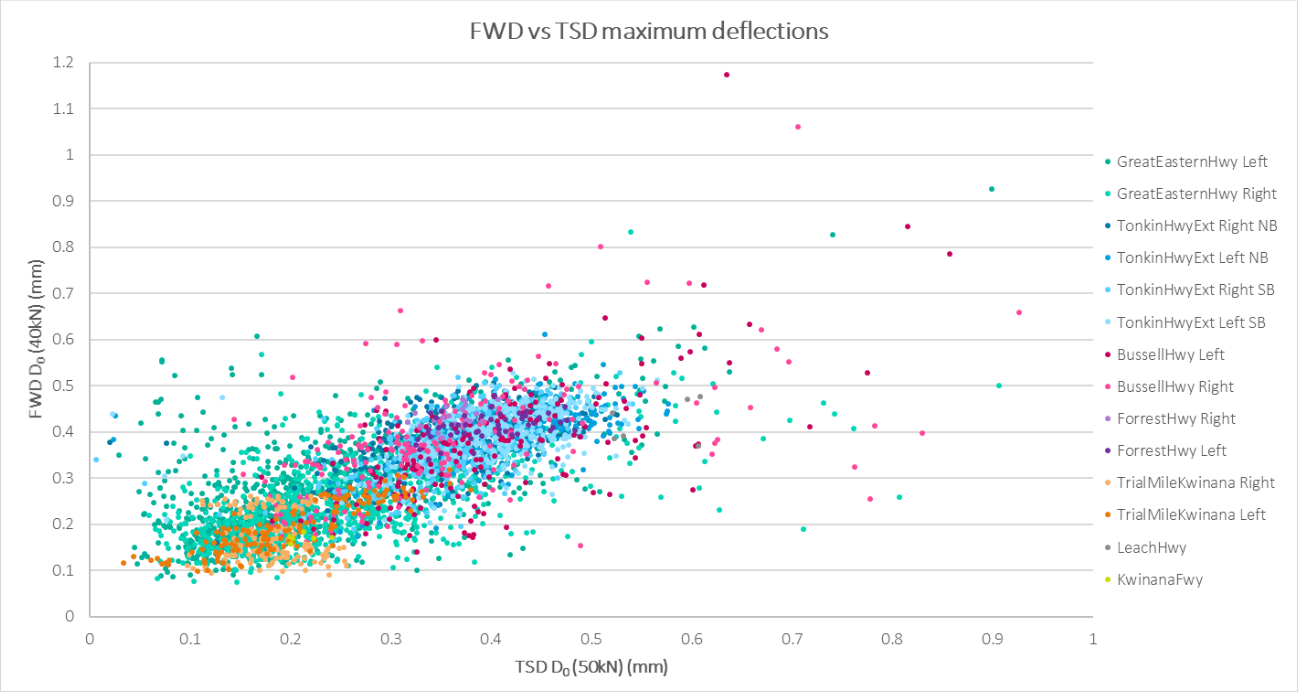
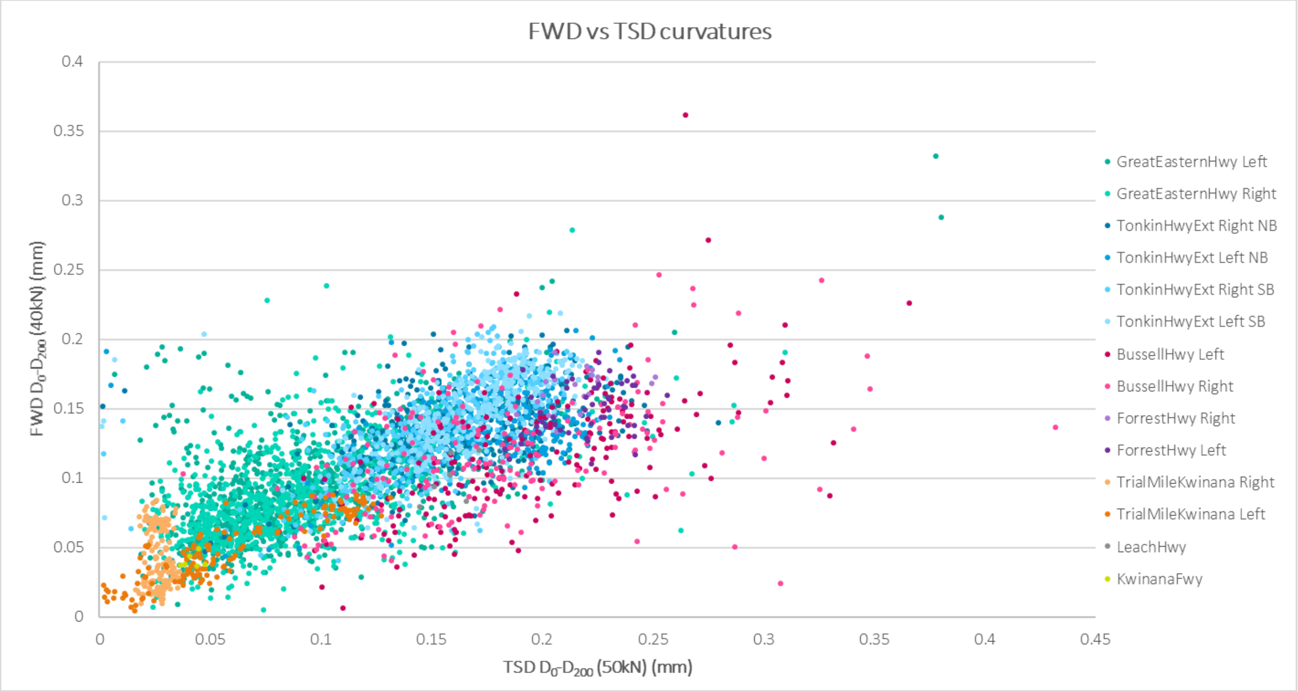


Figure 4.10 Comparison of FWD and TSD curvature (D_0 – D_{200}) data – all roads



Additionally, it is worth noting that the deflection data covers a low and limited range of magnitudes. A majority of the D_0 data for both the FWD and TSD are below 0.6 mm. As intervention levels for rehabilitation are typically based on FWD values higher than this (e.g. D_0 > ~0.7 mm for granular overlays as per Austroads (2019b)), the DSF factor derived using this data may not be able to appropriately describe the FWD-to-TSD correlation in the range used to calculate granular overlay thicknesses.

4.6 REGRESSION ANALYSIS

As already discussed, the collected deflection datasets showed a reasonably linear correlation between the FWD and TSD values for both maximum deflections and curvature. An investigation of the significance of this relationship, and the determination of the adjustment factors for TSD to FWD deflections, was undertaken using Deming regression analysis (Austroads 2019b). An overview of the analysis process is provided in the following subsections, while more detail can be found in Austroads (2019b).

4.6.1 METHODOLOGY

Following the methodology outlined in Section 4.5.1, the location paired, temperature-corrected and load-normalised datasets were analysed using the following linear regression process:

1. Preliminary regression analysis:

   The Deming regression process was applied to the collated dataset, as well as each road's individual datasets. Any individual data points with a standardised residual magnitude larger than 2 were identified as outliers. The regression process was then reapplied to the datasets after the exclusion of any outliers.

2. Data refinement:

   The regression statistics (e.g. correlation coefficient (R) and the 95% confidence intervals (CI)) for each individual road and lane (after outlier exclusion) were then reviewed in light of the insights gained from the work described in Section 4.3 and Section 4.5.2. Any datasets previously flagged for removal (i.e. Great Eastern Highway, and the right lane of Trial Mile Road) and showing unusual regression outcomes were removed from the collated dataset to give a refined dataset.

3. Refined regression analysis:

   The Deming regression process was then applied to the refined collated dataset (combination of all suitable road datasets), to derive the regression statistics and FWD-to-TSD relationships (for all combined pavements, as well as for separated pavements types (i.e. asphalt and granular). This was done in two ways:

   a. Unconstrained intercept relationship of the form:

      (FWD 40 kN) = α x (TSD 50 kN) + β
      (where α and β are the regression coefficients for the gradient and intercept, respectively).

   b. *Constrained*[3] *intercept relationship* (forced through the point (0,0)) of the form:

      FWD (40 kN) = α x TSD (50 kN)
      (where α is the regression coefficient for the gradient).

4.6.2 PRELIMINARY REGRESSION ANALYSIS & DATA REFINEMENT RESULTS

Preliminary regression analysis of the collated dataset as well as each road's individual deflection datasets was undertaken. Any outliers identified as per Section 4.6.1 were removed before reapplying the regression analysis to the datasets (Appendix E). The resultant regression relationships for individual road datasets were reviewed alongside their respective plots of standardised residuals to confirm the suitability of the deflection data. The preliminary regression relationships are provided in Table 4.2 and Table 4.3.

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[3] The constrained regression analysis process involves the setting the intercept coefficient for the Deming regression, equal to zero and then recalculating the gradient coefficient accordingly.
The intercept coefficient is typically calculated as: β = mean(y) – (α*mean(x)) (as per Appendix B of Austroads 2019b). The coefficient for the gradient (i.e. α) is then found, when β = 0, to be: α = mean(y) / mean(x).

Table 4.2 Preliminary regression results – maximum deflection (D_0)

ID	Road	Pavement type	Lane	Model gradient (α)	Model intercept (β)	Correlation coefficient (R)	Standard error (SE) (mm)
11	Bussell Hwy	Sealed granular	Left	EXCLUDED FROM ANALYSIS			
12	Bussell Hwy	Sealed granular	Right	EXCLUDED FROM ANALYSIS			
21	Forrest Hwy	Sealed granular	Left	0.37	0.27	0.43	0.020
22	Forrest Hwy	Sealed granular	Right	1.05	0.05	0.42	0.022
31	Kwinana Fwy	Asphalt (60 mm) surfaced granular	N/A	0.12	0.15	0.19	0.006
41	Leach Hwy	Asphalt (30 mm) surfaced granular	N/A	1.24	-0.28	0.34	0.035
51	Trial Mile Road	Asphalt (60 mm) surfaced granular	Left	0.72	0.05	0.88	0.022
52	Trial Mile Road	Asphalt (60 mm) surfaced granular	Right	EXCLUDED FROM ANALYSIS			
61	Great Eastern Hwy	Sealed granular	Left	0.88	0.07	0.71	0.057
62	Great Eastern Hwy	Sealed granular	Right	0.78	0.05	0.67	0.055
71	Tonkin Hwy Extension	Sealed granular	Left (NB)	0.85	0.05	0.66	0.036
72			Left (SB)	1.03	-0.02	0.66	0.043
73	Tonkin Hwy Extension	Sealed granular	Right (NB)	0.85	0.07	0.63	0.039
74			Right (SB)	1.32	-0.11	0.65	0.045
All pavements combined				0.90	0.04	0.82	0.049

Where: Hwy = highway, Fwy = freeway, NB = Northbound, SB = Southbound, and Trial Mile Road = section of Kwinana freeway/Perth-Bunbury highway.

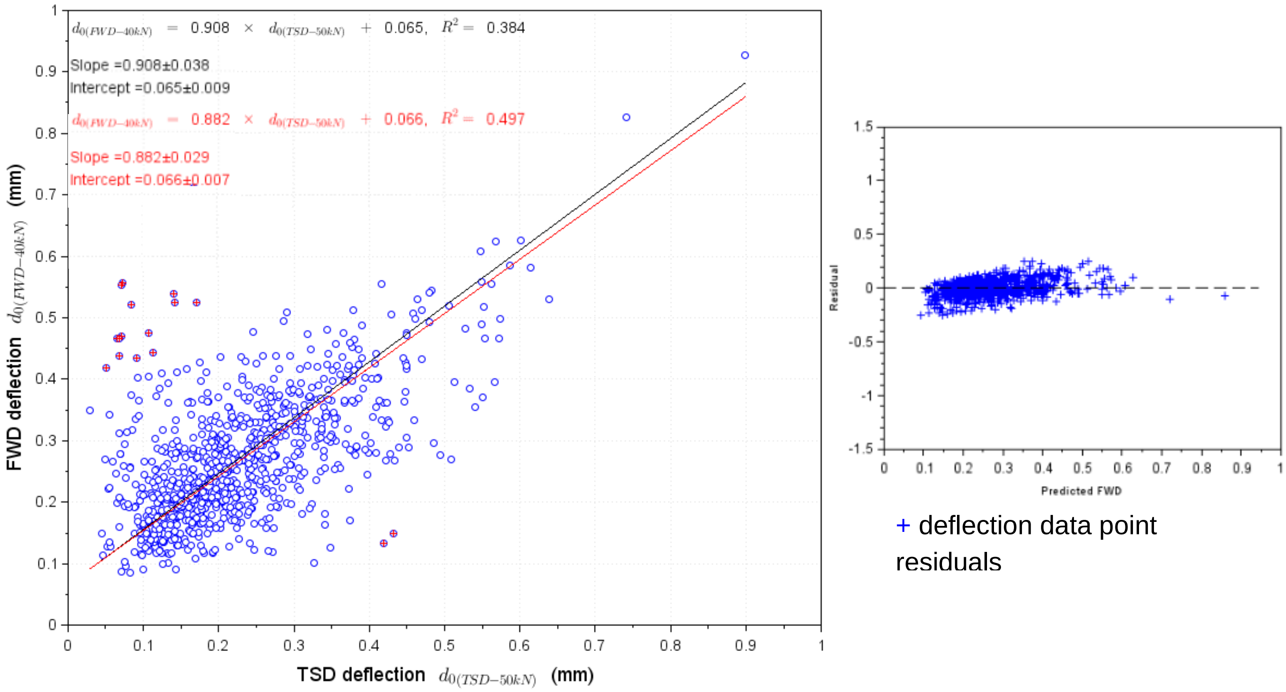
Table 4.3 Preliminary regression results – curvature (D_0 – D_{200})

ID	Road	Pavement type	Lane	Model gradient (α)	Model intercept (β)	Correlation coefficient (R)	Standard error (SE) (mm)
11	Bussell Hwy	Sealed granular	Left	EXCLUDED FROM ANALYSIS			
12	Bussell Hwy	Sealed granular	Right	EXCLUDED FROM ANALYSIS			
21	Forrest Hwy	Sealed granular	Left	1.28	-0.13	0.27	0.020
22	Forrest Hwy	Sealed granular	Right	0.81	-0.01	0.41	0.012
31	Kwinana Fwy	Asphalt (60 mm) surfaced granular	N/A	2.09	-0.05	0.22	0.007
41	Leach Hwy	Asphalt (30 mm) surfaced granular	N/A	2.55	-0.30	0.57	0.016
51	Trial Mile Road	Asphalt (60 mm) surfaced granular	Left	0.62	0.01	0.91	0.008
52	Trial Mile Road	Asphalt (60 mm) surfaced granular	Right	EXCLUDED FROM ANALYSIS			
61	Great Eastern Hwy	Sealed granular	Left	0.71	0.03	0.67	0.020
62	Great Eastern Hwy	Sealed granular	Right	0.67	0.03	0.64	0.020
71	Tonkin Hwy Extension	Sealed granular	Left (NB)	0.75	0.01	0.55	0.018
72			Left (SB)	1.08	-0.03	0.75	0.017
73	Tonkin Hwy Extension	Sealed granular	Right (NB)	0.60	0.04	0.47	0.020
74			Right (SB)	1.03	-0.02	0.75	0.017
All pavements combined				0.71	0.02	0.82	0.019

Where: Hwy = highway, Fwy = freeway, NB = Northbound, SB = Southbound, and Trial Mile Road = section of Kwinana Freeway/Perth-Bunbury Highway.

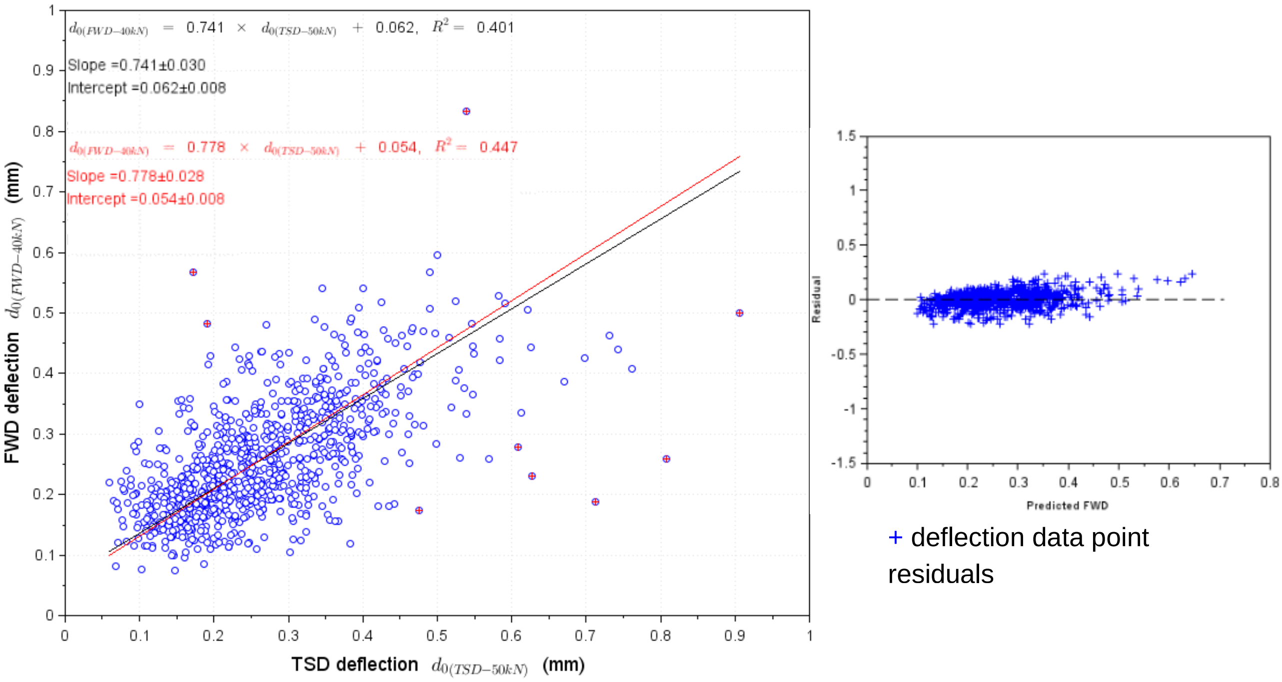
In regard to the Great Eastern Highway (both lanes) dataset that was previously flagged for potential exclusion (Section 4.5.2), the plots of the preliminary regression analysis alongside the residuals for the data are provided in Figure 4.11 to Figure 4.14. While somewhat scattered, the dataset demonstrated a strong FWD-to-TSD correlation (0.6 < R < 0.79 (Table B1 Austroads (2019b)) for both maximum deflection and curvature, with no significant bias present in the residuals. As a result, the data for the Great Eastern Highway was considered suitable for inclusion in the derivation of the TSD adjustment factor.

Figure 4.11 Great Eastern Highway, left lane – preliminary regression analysis of D_0



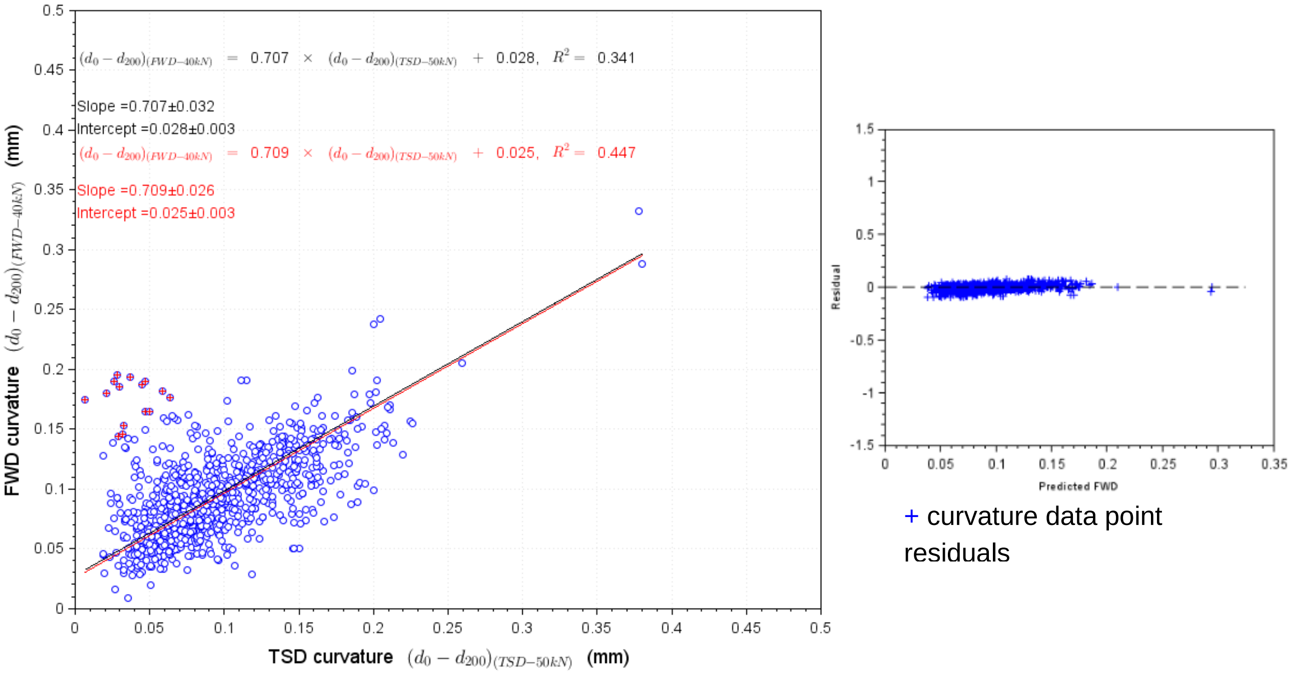
Where: *o FWD vs TSD deflection data*
– regression line on all data points
+ Identified outlier
– Regression line on data points, excluding outliers

Figure 4.12 Great Eastern Highway, right lane – preliminary regression analysis of D_0



Where: *o FWD vs TSD deflection data*
– regression line on all data points
+ Identified outlier
– Regression line on data points, excluding outliers

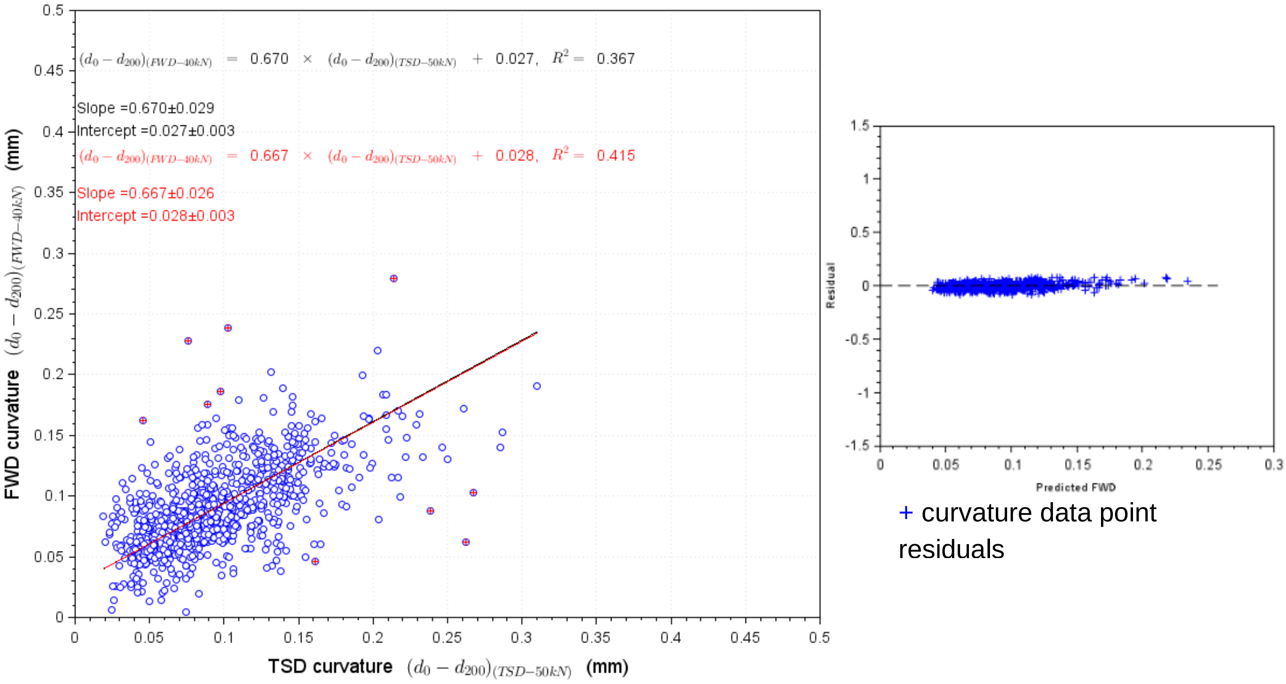
Figure 4.13 Great Eastern Highway, left lane – preliminary regression analysis of D0 – D200



Where: *o FWD vs TSD curvature data* *+ Identified outlier*

– regression line on all data points *– Regression line on data points, excluding outliers*

Figure 4.14 Great Eastern Highway, right lane – preliminary regression analysis of D0 – D200



Where: *o FWD vs TSD curvature data* *+ Identified outlier*

– regression line on all data points *– Regression line on data points, excluding outliers*

4.7 RESULTS OF REFINED REGRESSION ANALYSIS

Following data refinement, regression analyses of the refined collated datasets (i.e. all pavement types combined, and separated pavement types) was undertaken. The resultant regression relationships, both unconstrained and constrained, for the combined pavement types dataset, and the separated asphalt and granular pavement datasets were determined.

Table 4.4 to Table 4.7 provide the standard error, and regression and correlation coefficients, while graphs of the regression relationships can be found in Appendix F. The model gradients listed in Table 4.6 and Table 4.7 are the derived DSF and CSF values for each pavement type or combination.

Table 4.4 Refined and unconstrained regression results – maximum deflection (D_0)

Pavement type	Model gradient (α)	Model intercept (β)	Correlation coefficient (r)	Standard error (SE) (mm)
Asphalt	0.70	0.05	0.92	0.022
Granular	0.87	0.05	0.80	0.050
Combined	0.88	0.05	0.81	0.049

Table 4.5 Refined and unconstrained regression results – curvature (D0 – D200)

Pavement type	Model gradient (α)	Model intercept (β)	Correlation coefficient (r)	Standard error (SE) (mm)
Asphalt	0.60	0.01	0.92	0.008
Granular	0.69	0.03	0.79	0.019
Combined	0.71	0.02	0.81	0.019

Table 4.6 Refined and constrained regression results – maximum deflection (D_0)

Pavement type	Model gradient (α)	Standard error (SE) (mm)
Asphalt	0.95	0.030
Granular	1.03	0.055
Combined	1.03	0.054

Table 4.7 Refined and constrained regression results – curvature (D0 – D200)

Pavement type	Model gradient (α)	Standard error (SE) (mm)
Asphalt	0.81	0.010
Granular	0.88	0.022
Combined	0.88	0.022

4.7.1 COMPARISON OF WA AND AUSTROADS D_0 DATA

In terms of the DSF value for use in ERN16, the results of the regression analyses of the WA dataset were compared with the results obtained in AGPT05-19 (Austroads 2019b) to derive the factors given in Table 4.8 and Table 4.9

Table 4.8 Comparison of unconstrained regression models for maximum deflection (D_0)

Pavement type	WA			Austroads		
	Model	R	SE (mm)	Model	R	SE (mm)
Asphalt	y = 0.70x + 0.05	0.92	0.02	y = 0.93x + 0.04	0.78	0.14
Granular	y = 0.87x + 0.05	0.80	0.05	y = 1.06x – 0.06	0.83	0.25
Combined	y = 0.88x + 0.05	0.81	0.05	y = 1.01x – 0.01	0.85	0.21

Where: R is the Pearson correlation coefficient, SE is the standard error,
y is the predicted 40 kN FWD value, and x is representative of the 50 kN TSD data value (both in mm).

Table 4.9 Comparison of constrained regression models for maximum deflection (D_0)

Pavement type	**WA**		**Austroads***	
	Model	**SE (mm)**	**Model**	**SE (mm)**
Asphalt	y = 0.95x	0.030	y = 1.04x	0.119
Granular	y = 1.03x	0.055	y = 1.00x	0.242
Combined	y = 1.03x	0.022	y = 1.01x	0.207

Where: SE is the standard error, and
y is the predicted 40 kN FWD value, and x is representative of the 50 kN TSD data value (both in mm).

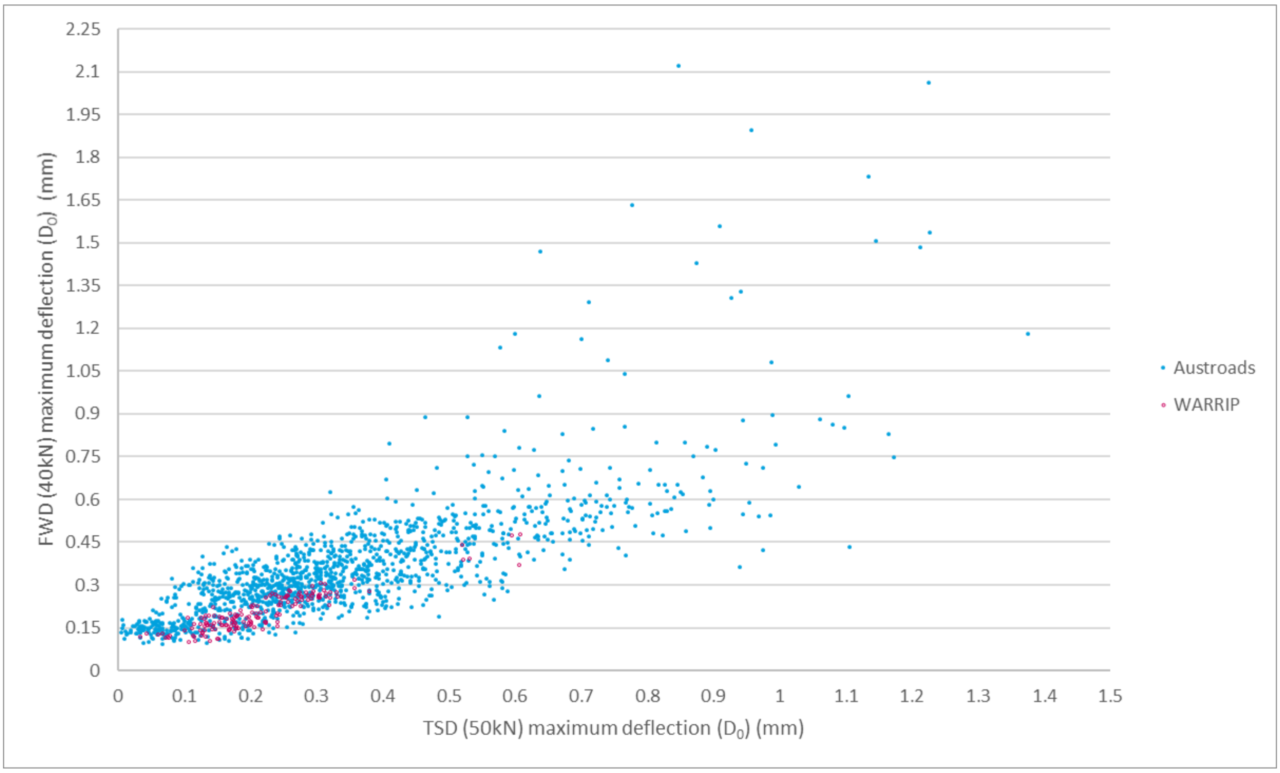
* *The constrained model for granular pavements reported in Austroads (2019b) was a direct simplification of the derived unconstrained model. However, in this study, constrained relationships were re-calculated with a forced intercept of (0,0). Additionally, Austroads (2019b) does not report the constrained regression models for asphalt pavement types only, or for combined pavement types. As such, the relationships reported here have been calculated with a forced (0,0) intercept from the Austroads datasets.*

The unconstrained regression relationships from Austroads (2019b) are noted overall to have steeper gradients, although higher standard errors for their fits. For the constrained models, however, the gradients of the fits were very similar.

Considering the spread of the WA and Austroads data (Figure 4.15 to Figure 4.17), and the derived regression relationships (Figure 4.18 to Figure 4.21, with plots of the raw datapoints provided in Appendix G) it was concluded that there was no evidence that the WA and Austroads datasets were significantly different.

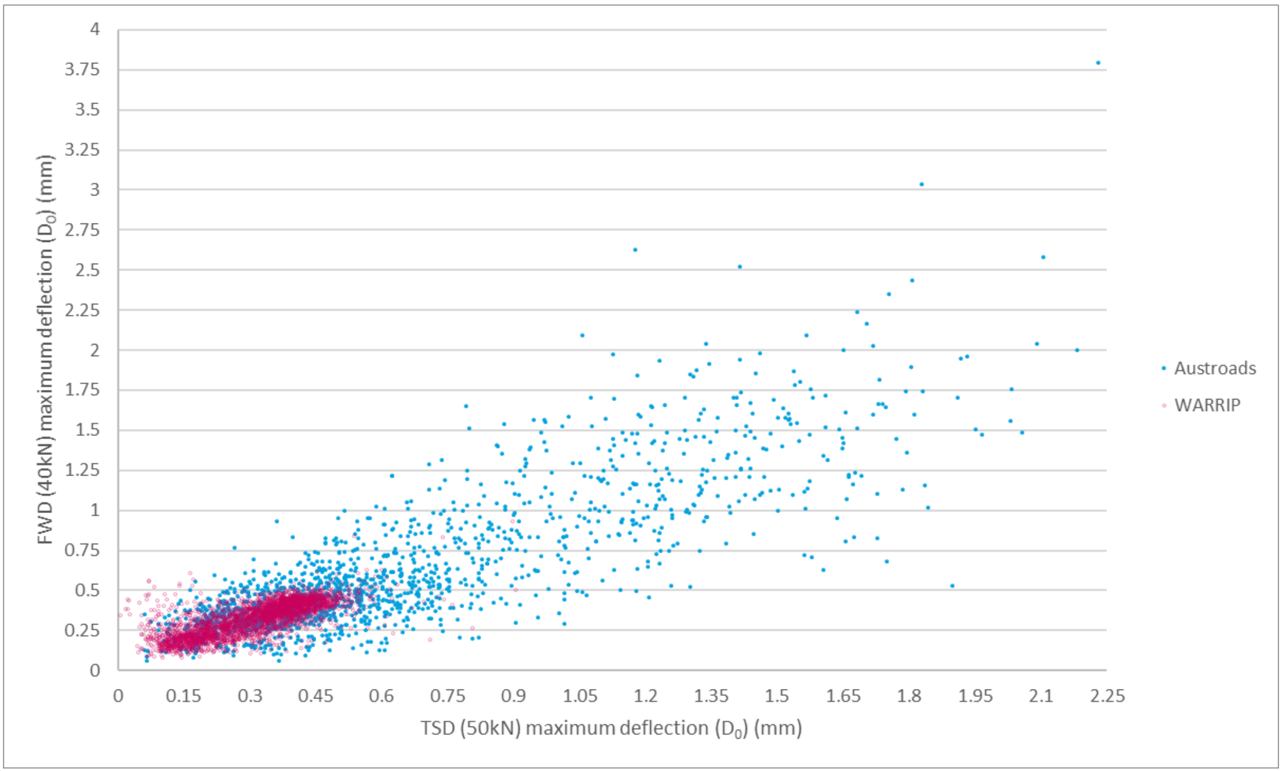
It is noted that the Austroads data used in the comparisons depicted in Figure 4.18 to Figure 4.21 includes only the individual road dataset regressions that had a sufficiently large pool of data points (e.g. > 100 points), and a linear correlation significance level higher than 95%; this is considered strong-to-very strong as per Austroads (2019b). Additionally, the individual regression models for the Kwinana freeway and Leach highway were omitted from the comparison due to the small number of data points (i.e. < 10) and correlation significance lower than 50%.

Figure 4.15 Comparison of Austroads (2019b) and WA D_0 data – asphalt pavements



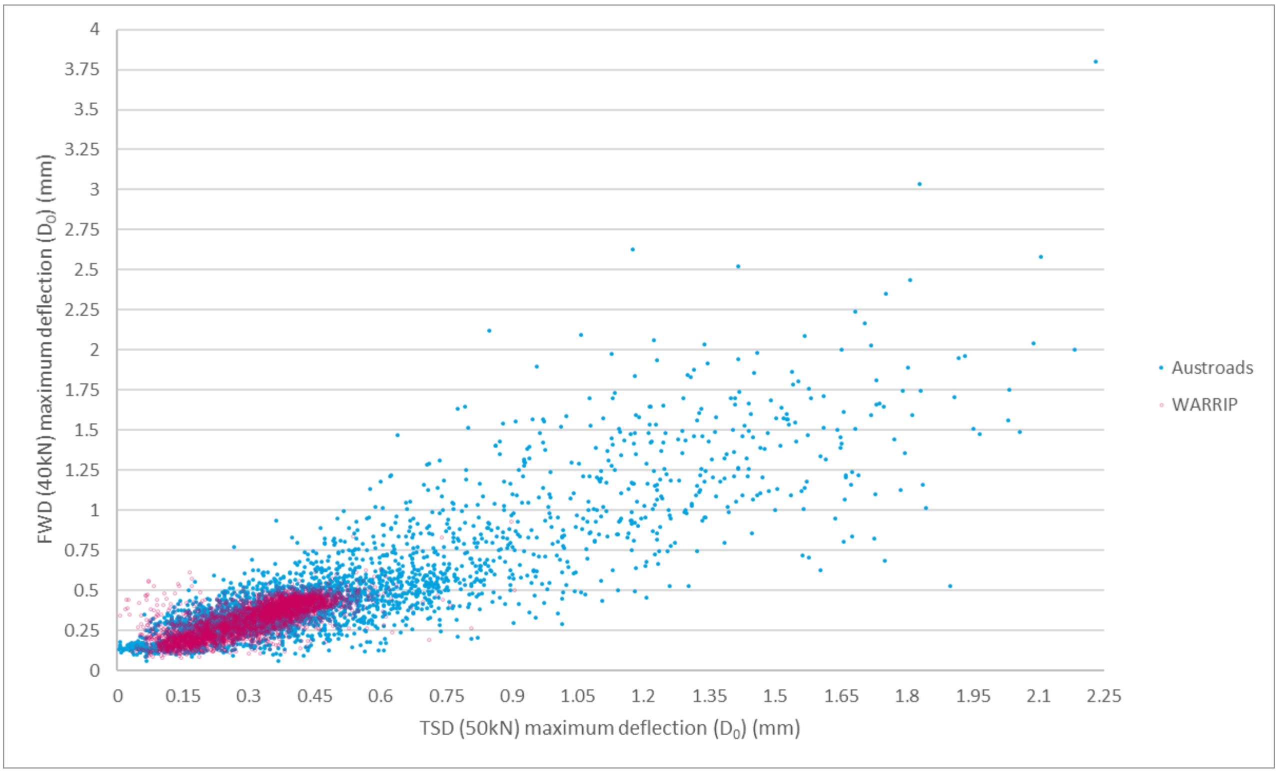
Where "WARRIP" refers to WA roads assessed in this analysis

Figure 4.16 Comparison of Austroads (2019b) and WA D_0 data – granular pavements



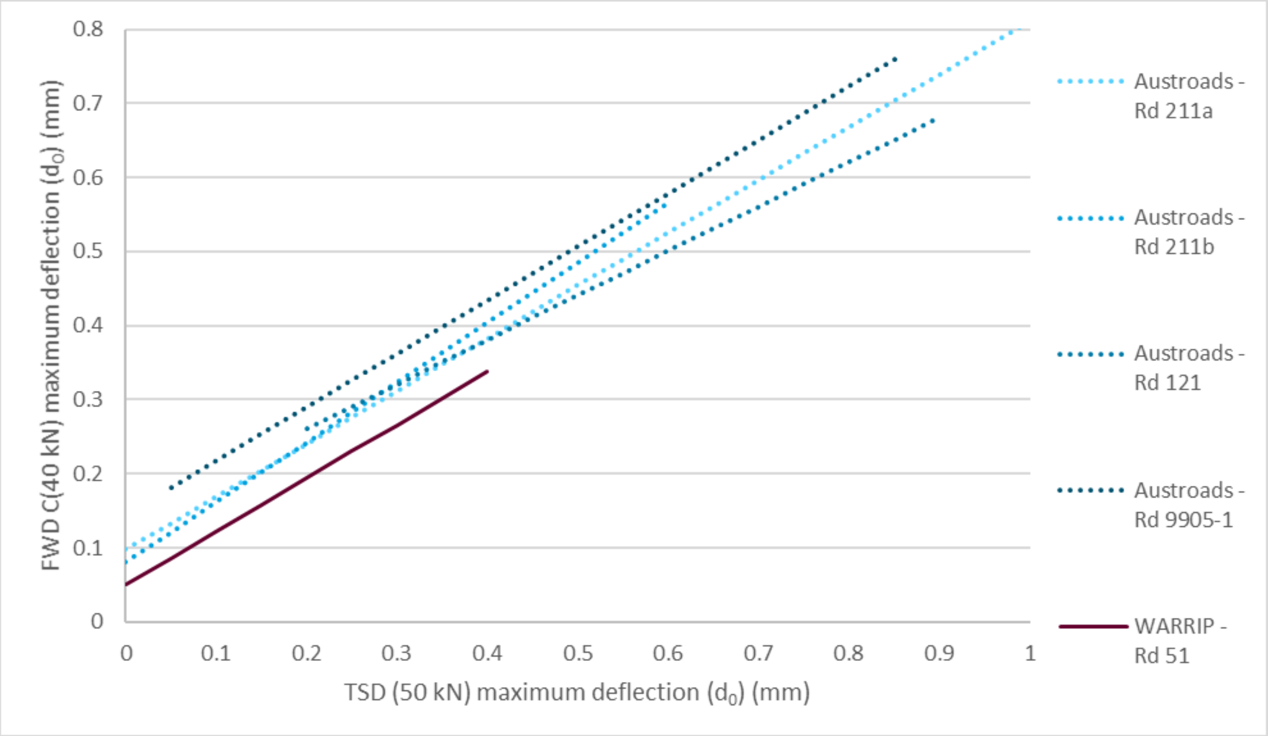
Where “WARRIP” refers to WA roads assessed in this analysis

Figure 4.17 Comparison of Austroads (2019b) and WA D_0 data – combined pavement types



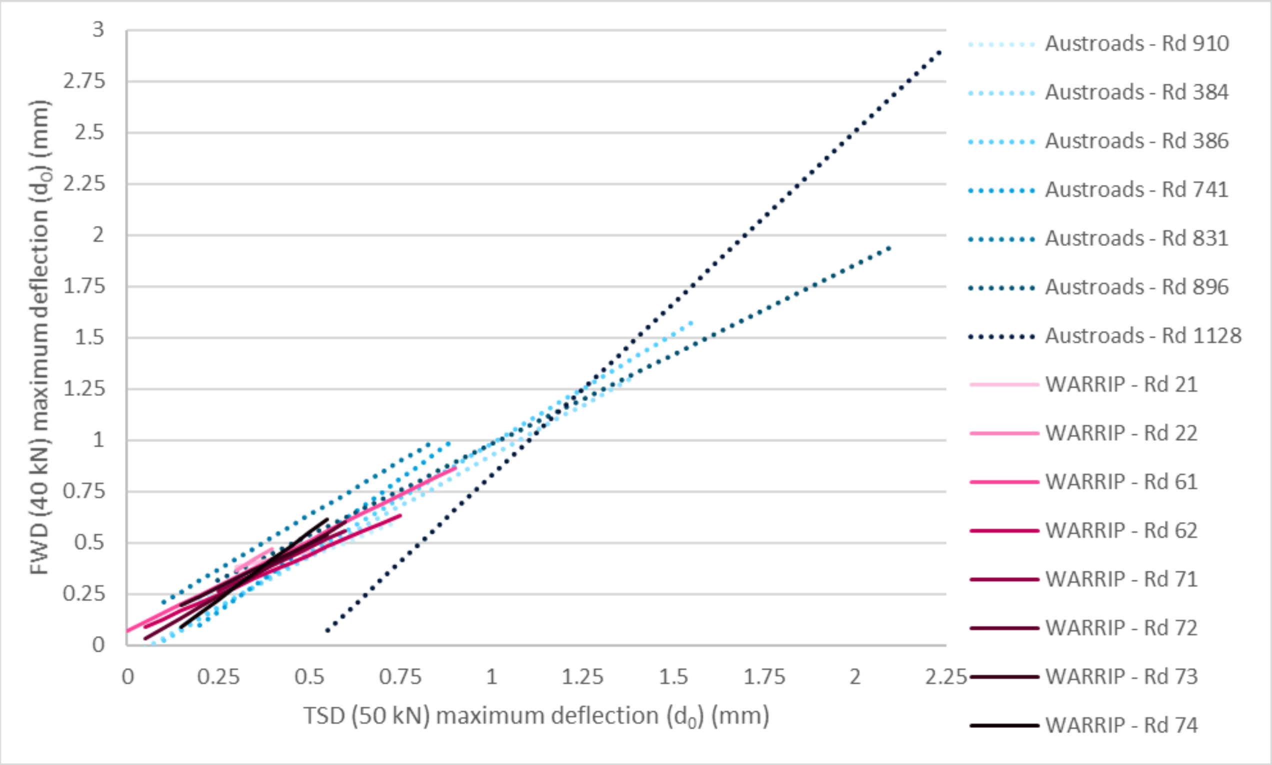
Where “WARRIP” refers to WA roads assessed in this analysis

Figure 4.18 Comparison of Austroads (2019b) and WA D_0 regression relationships – individual asphalt pavements



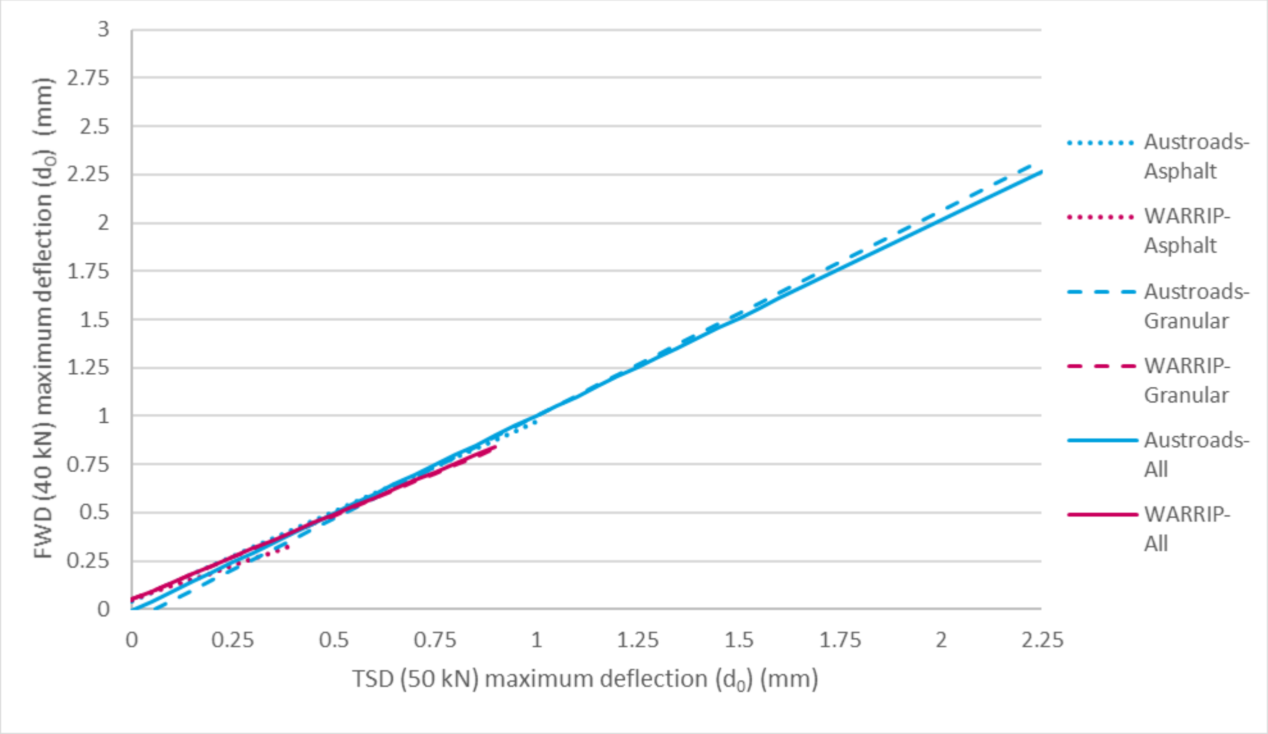
Where "WARRIP – Rd 51" refers to the WA road with ID 51 (i.e. Trial Mile Road, as listed in Table 4.1)

Figure 4.19 Comparison of Austroads (2019b) and WA D_0 regression relationships – individual granular pavements



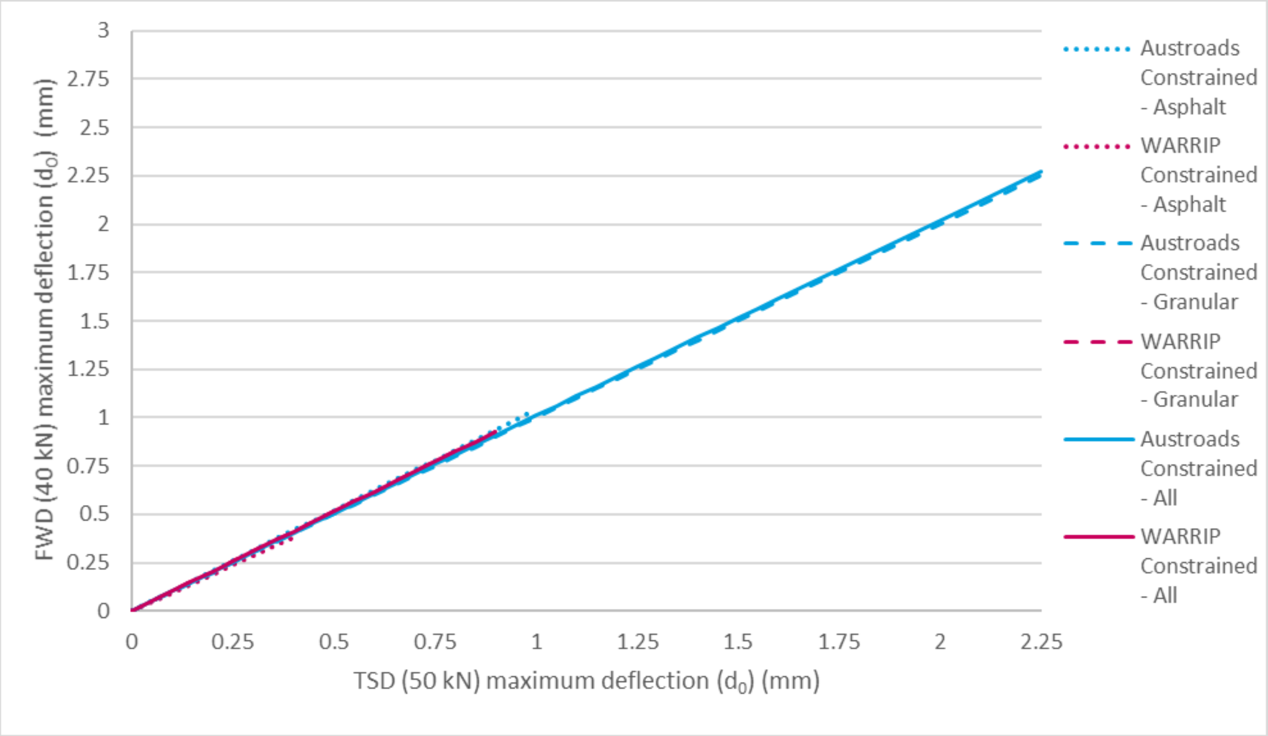
Where "WARRIP – Rd XX" refers to the WA road with ID XX (ID numbers are as listed in Table 4.1)

Figure 4.20 Comparison of Austroads (2019b) and WA D_0 regression relationships (unconstrained) – combined pavement types



Where "WARRIP" refers to WA roads assessed in this analysis

Figure 4.21 Comparison of Austroads (2019b) and WA D_0 regression relationships (constrained) – combined pavement types



Where "WARRIP" refers to WA roads assessed in this analysis

4.7.2 COMPARISON OF WA AND AUSTROADS $D_0 – D_{200}$ DATA

A further comparison was also undertaken of the WA and Austroads (2019b) curvature ($D_0 – D_{200}$) datasets. An overview of the derived relationships between the FWD and TSD curvatures are provided in Table 4.10 and Table 4.11.

Table 4.10 Comparison of unconstrained regression models for curvature ($D_0 – D_{200}$)

Pavement type	WA			Austroads		
	Model	R	SE (mm)	Model	R	SE (mm)
Asphalt	y = 0.60x + 0.01	0.92	0.008	y = 0.60x + 0.02	0.77	0.002
Granular	y = 0.69x + 0.03	0.79	0.019	y = 0.66x + 0.01	0.79	0.003
Combined	y = 0.71x + 0.02	0.81	0.019	y = 0.65x + 0.01	0.82	0.002

Where: R is the Pearson correlation coefficient, SE is the standard error,
y is the predicted 40 kN FWD value, and x is representative of the 50 kN TSD data value (both in mm).

Table 4.11 Comparison of constrained regression models for curvature ($D_0 – D_{200}$)

Pavement type	WA		Austroads*	
	Model	SE (mm)	Model	SE (mm)
Asphalt	y = 0.81x	0.010	y = 0.77x	0.029
Granular	y = 0.88x	0.022	y = 0.67x	0.057
Combined	y = 0.88x	0.022	y = 0.68x	0.147

Where: SE is the standard error, and
y is the predicted 40 kN FWD value, and x is representative of the 50 kN TSD data value (both in mm).

* *The constrained model for granular pavements reported in Austroads (2019b) was a direct simplification of the derived unconstrained model. However, in this study, constrained relationships were re-calculated with a forced intercept of (0,0). Additionally, Austroads (2019b) does not report the constrained regression models for asphalt pavement types only, or for combined pavement types. As such, the relationships reported here have been calculated with a forced (0,0) intercept from the Austroads datasets.*

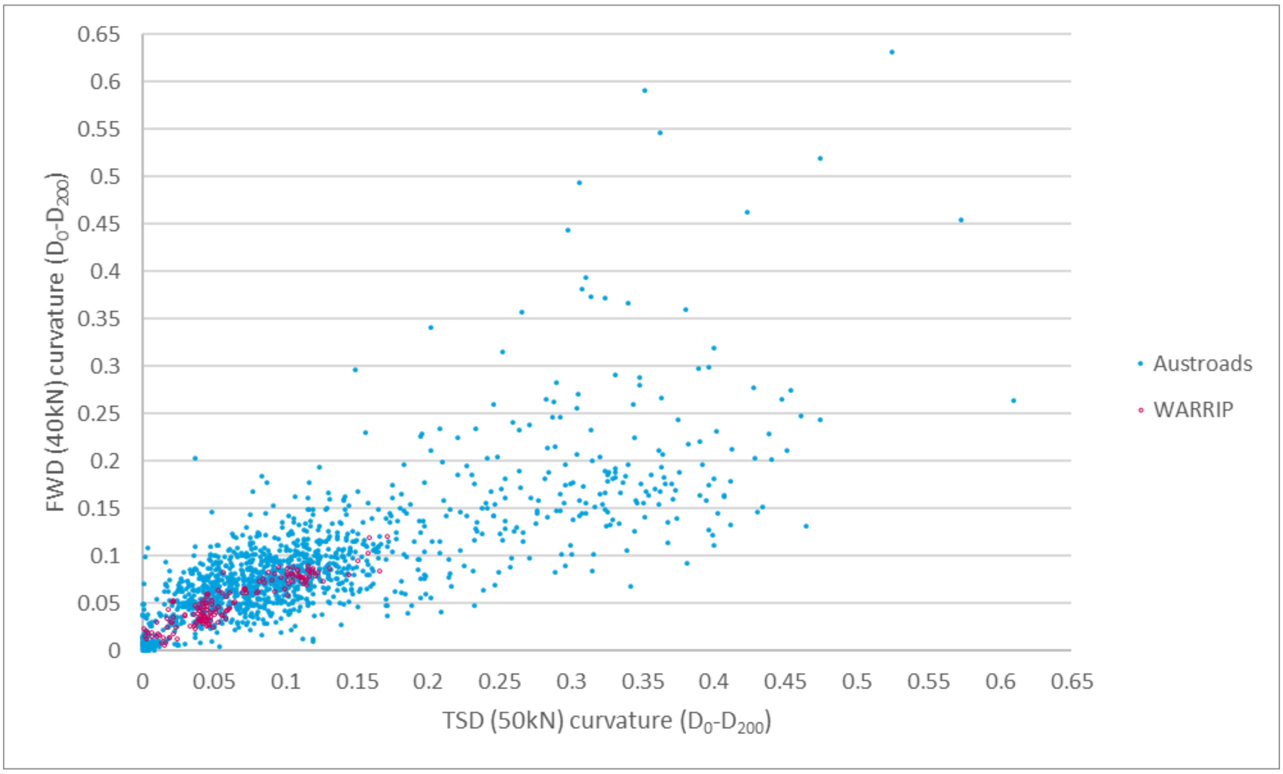
Comparison of the unconstrained curvature relationships indicates that the Austroads (2019b) and WA relationships are similar, although with slightly lower gradients and smaller standard errors for the fit. While for the constrained fits, there is a greater difference in the gradients between the projects, with smaller standard errors in the WA models.

The spread of curvature values between the projects (Figure 4.22 to Figure 4.24) and the derived regression relationships (Figure 4.25 to Figure 4.28, with plots of the raw datapoints provided in Appendix G) demonstrates that the WA data generally lies within the Austroads (2019b) datasets, although the derived relationships are less aligned than for the maximum deflections. The curvature data shows that the asphalt pavement response for Trial Mile Road is very similar to the Austroads asphalt data.

It is again noted that the Austroads data used in the comparisons depicted in Figure 4.25 to Figure 4.28 includes only the individual road datasets) that had a sufficiently large pool of data points (e.g. > 100 points), and a linear correlation significance level higher than 95%; this is considered strong-to-very strong as per Austroads (2019b). Additionally, the individual regression models for the Kwinana freeway and Leach highway were omitted from the comparison due to the small number of data points (i.e. < 10) and undesirably low correlation significance (33% for the Kwinana freeway, and 76% for the Leach highway).

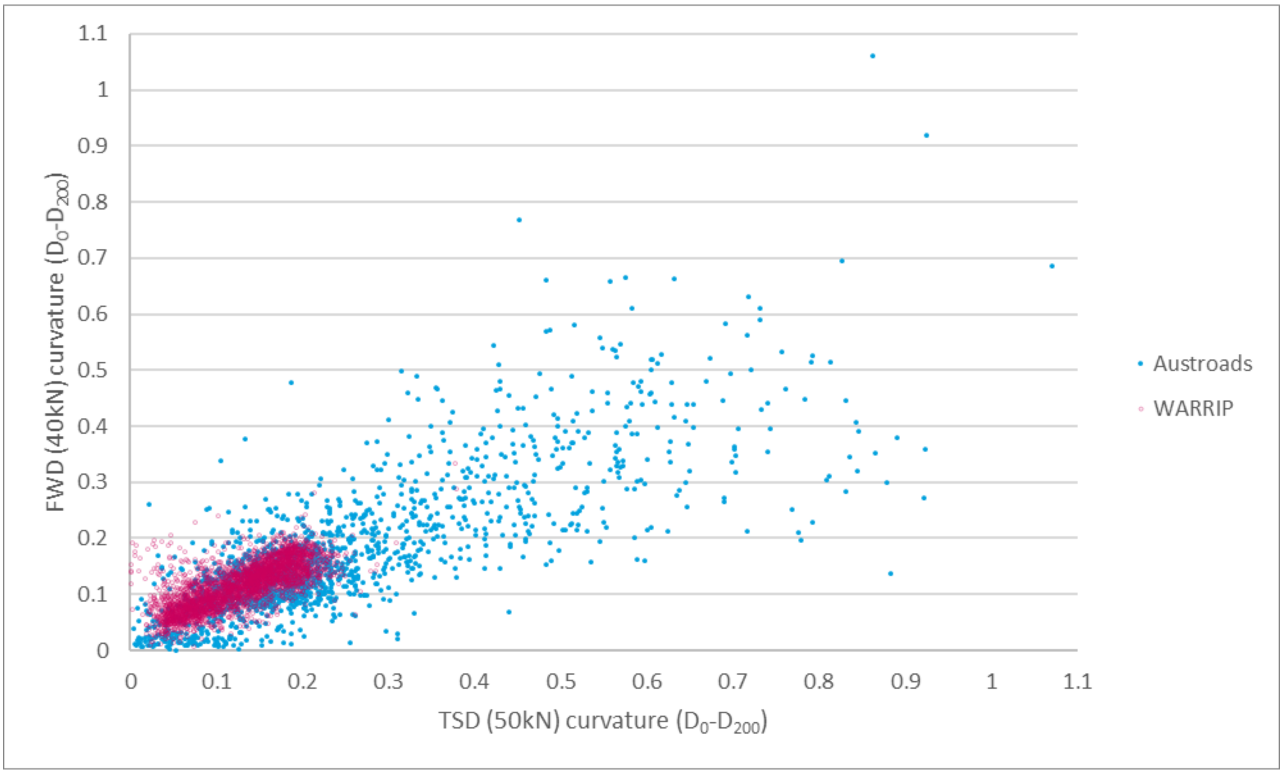
It was concluded that there was no evidence that the WA and Austroads (2019b) curvature data was significantly different. This is consistent with the findings for maximum deflections (Section 4.7.1).

Figure 4.22 Comparison of Austroads (2019b) and WA D0 – D200 data – asphalt pavements



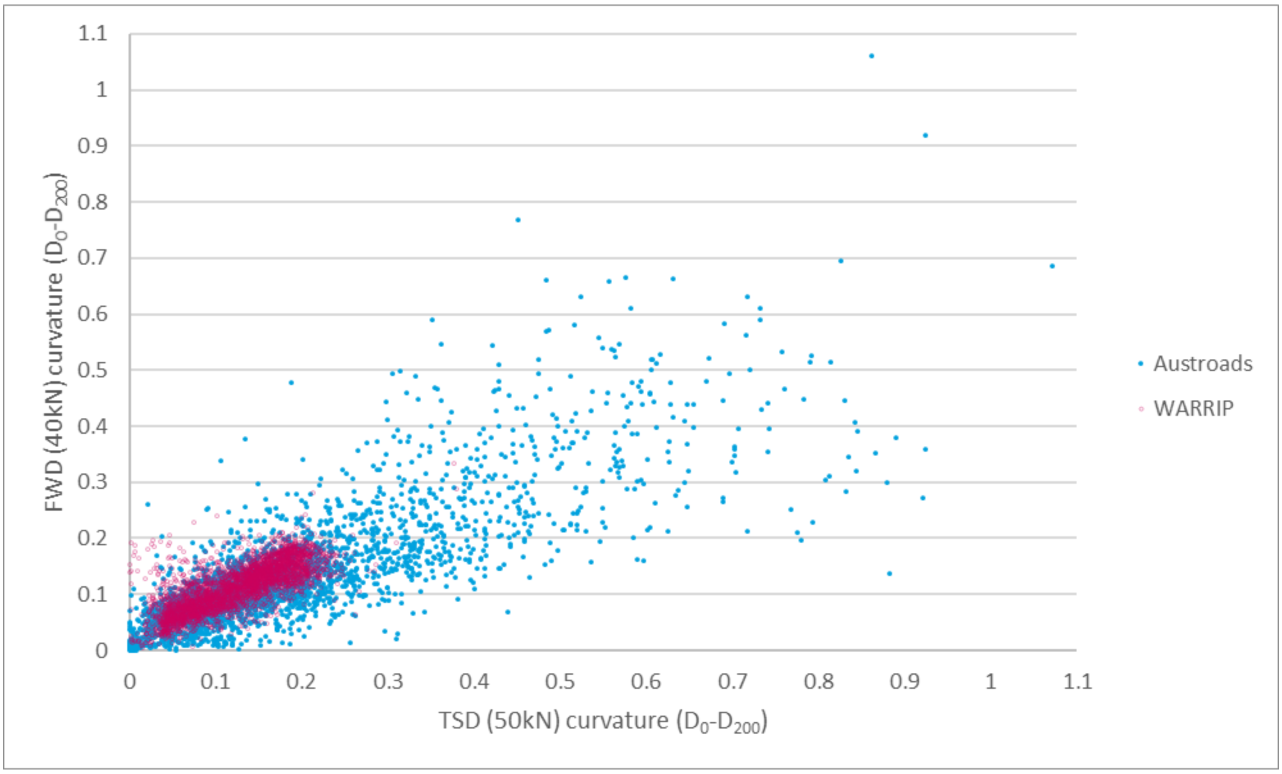
Where "WARRIP" refers to WA roads assessed in this analysis

Figure 4.23 Comparison of Austroads (2019b) and WA D0 – D200 data – granular pavements



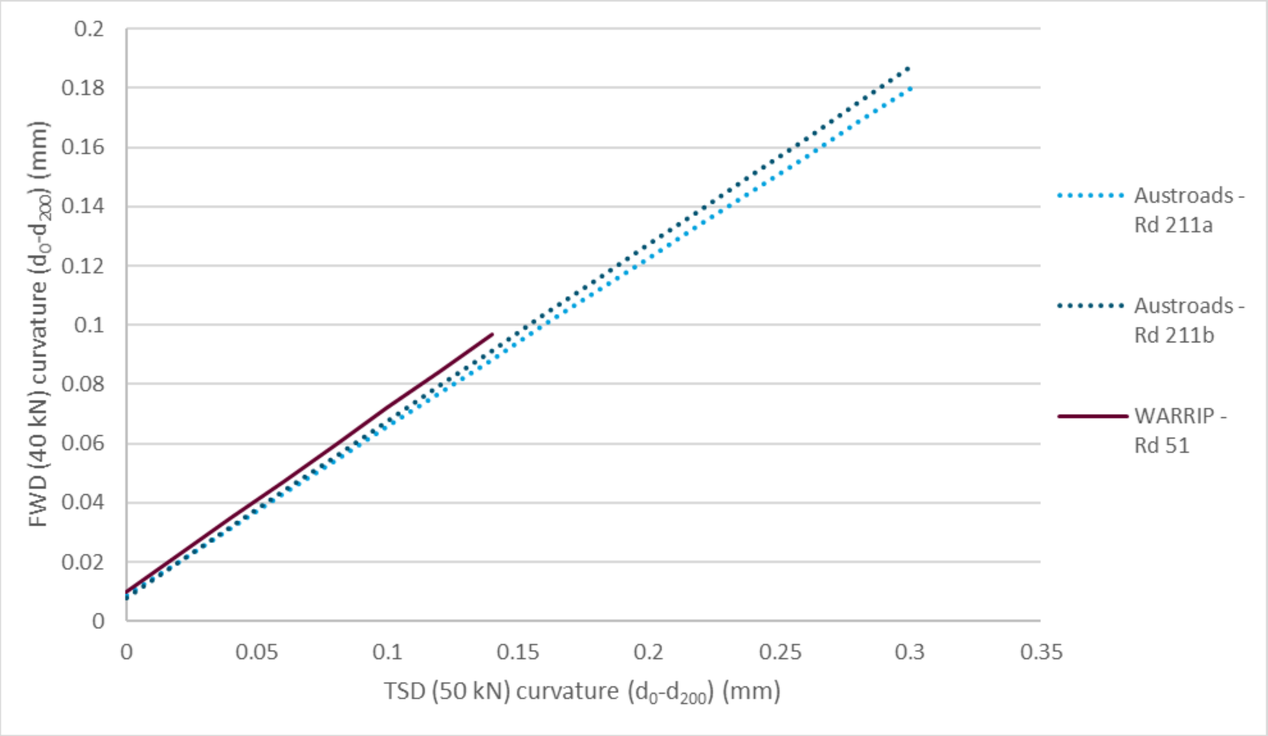
Where "WARRIP" refers to WA roads assessed in this analysis

Figure 4.24 Comparison of Austroads (2019b) and WA D0 – D200 data – combined pavement types



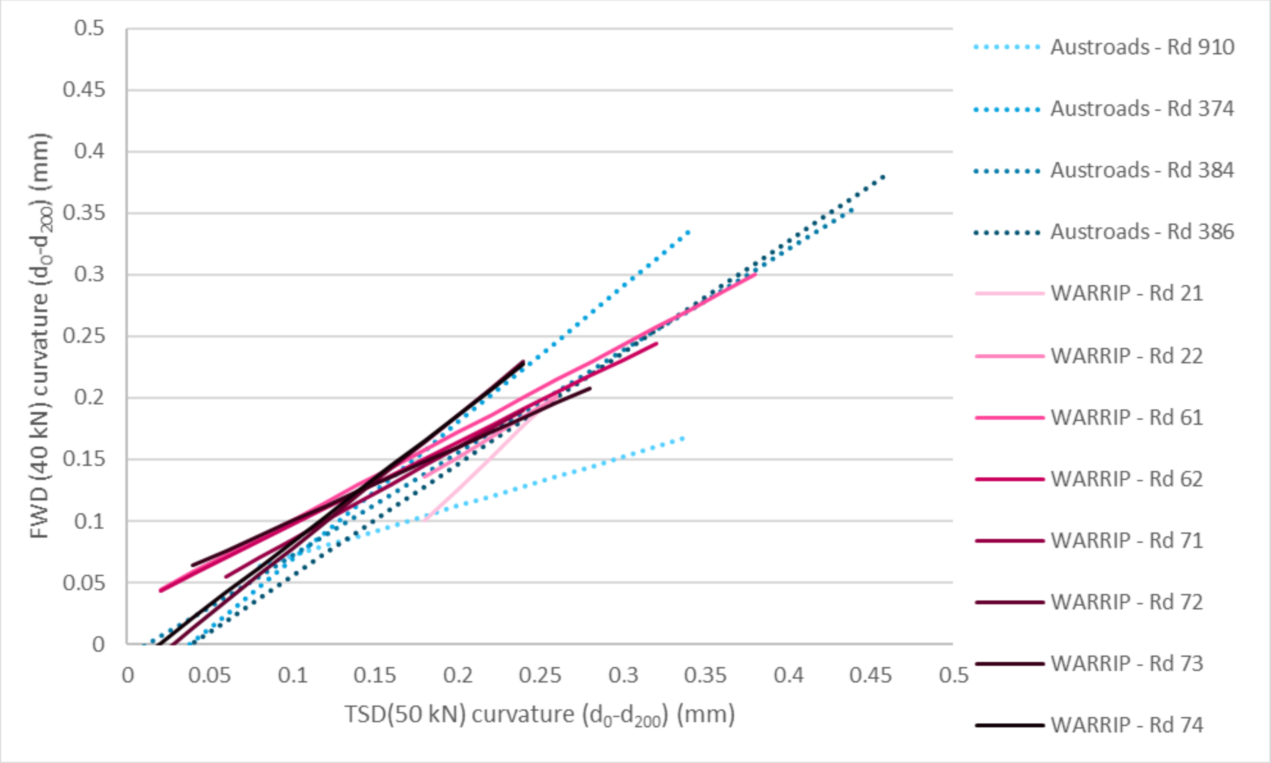
Where "WARRIP" refers to WA roads assessed in this analysis

Figure 4.25 Comparison of Austroads (2019b) and WA D0 – D200 regression relationships – individual asphalt pavements



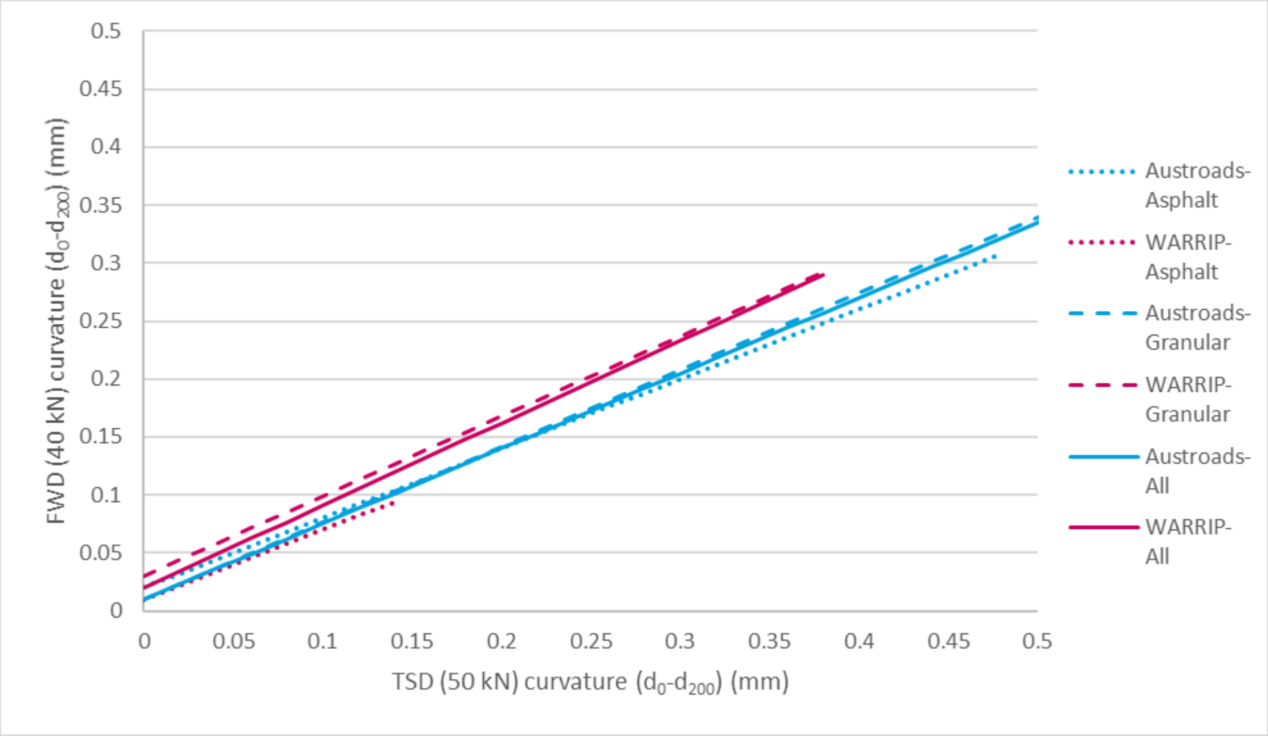
Where "WARRIP – Rd 51" refers to the WA road with ID 51 (i.e. Trial Mile Road, as listed in Table 4.1)

Figure 4.26 Comparison of Austroads (2019b) and WA D0 – D200 regression relationships – individual granular pavements



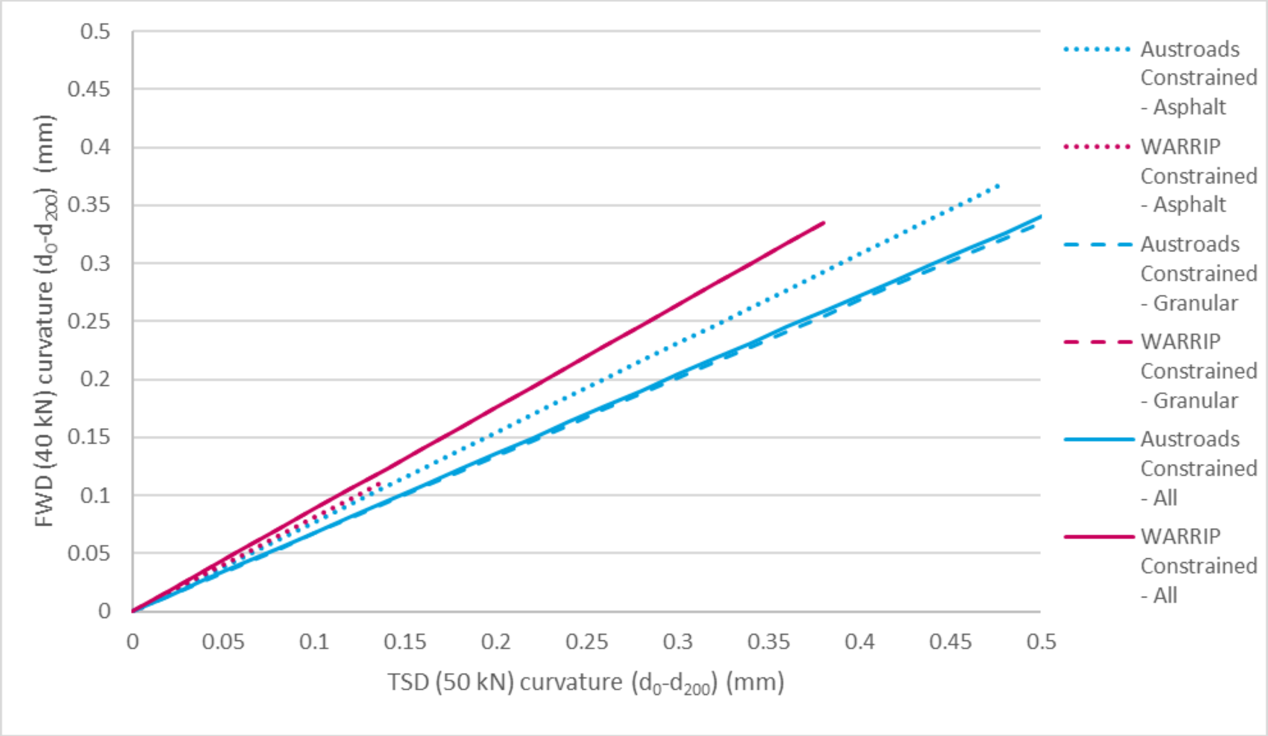
Where "WARRIP – Rd XX" refers to the WA road with ID XX (ID numbers are as listed in Table 4.1)

Figure 4.27 Comparison of Austroads (2019b) and WA D0 – D200 regression relationships (unconstrained) – combined pavement types



Where "WARRIP" refers to WA roads assessed in this analysis

Figure 4.28 Comparison of Austroads (2019b) and WA D0 – D200 regression relationships (constrained) – combined pavement types



Where "WARRIP" refers to WA roads assessed in this analysis

4.7.3 FACTORS FROM COMBINED WA AND AUSTROADS DATA

As already discussed, there was no evidence that the WA and Austroads (2019b) data were significantly different.

Based on this finding, the WA and Austroads data were combined and regression analysis (constrained) was undertaken to derive DSF and CSF values. These relationships are listed in Table 4.12:

Table 4.12 Constrained regression models for Austroads and WA combined data

Pavement type	Maximum deflections (D_0)		Curvature (D0 – D200)	
	Model	SE (mm)	Model	SE (mm)
Asphalt	y = 1.02x	0.117	y = 0.79x	0.030
Granular	y = 1.02x	0.244	y = 0.86x	0.069
Combined	y = 1.02x	0.209	y = 0.85x	0.167

Where: SE is the standard error,
y is the predicted 40 kN FWD value, and x is representative of the 50 kN TSD data value (both in mm).

Note that CSF of 0.85 is for use with FWD contact stress of 566 kPa (40 kN), which translates to a CSF of 0.85 x 700/566 = 1.05 to enable FWD curvature at a stress of 700 kPa to be estimated from measured 50 kN TSD curvatures.

4.8 PROCEDURES FOR ERN16

4.8.1 DEFLECTION STANDARDISATION FACTOR

Overall, it was concluded that the WA and Austroads (2019b) data were not significantly different. As such, the following two options were discussed with Main Roads in relation to incorporation of TSD deflection standardisation factors (DSF) in ERN16 for the design of granular overlays:

Option A: Adopt the Austroads (2019b) DSF namely:

Benkelman beam D_0 = 1.2 x TSD_{d0} (50 kN).

Option B: From the combined regression relationship in Table 4.12 and the Austroads (2019a) DSF of 1.1 to adjust the 40 kN FWD D_0 values to Benkelman beam D_0 values:

Benkelman beam D_0 = 1.02 x 1.1 x TSD_{d0} (50 kN)

rounded to: Benkelman beam D_0 = 1.1 x TSD_{d0} (50 kN).

Main Roads considered that further research is required to develop DSF appropriate for the full range of pavement types in WA, including cementitiously-modified granular pavements. Pending this research, Main Roads has decided to adopt a DSF of 1.2, consistent with Austroads (2019a).

4.8.2 CURVATURE STANDARDISATION FACTOR

Although asphalt overlay design charts were provided in the 2012 edition of AGPT05, Austroads decided to delete these charts from AGPT05-19 (Austroads 2019a). Consequently, there was no need to provide curvature standardisation factors in AGPT05-19 to enable TSD curvatures (D0 – D200) to be used in the design of asphalt overlays.

As the WA and Austroads (2019b) data were not significantly different, the following CSF was derived (Section 4.7.3) by combining the data:

$FWD_{d0-d200}$ (50 kN) = 1.05 x $TSD_{d0-d200}$ (50 kN).

The incorporation of this CSF in ERN16 was discussed with Main Roads. Given the need for Main Roads to research the DSF values, pending this research Main Roads decided not to provide CSF values to enable TSD curvatures to be used to design asphalt overlays in the first edition of ERN16. Although TSD data may be useful in identifying pavement sub-section of homogeneous strength, the thickness design of asphalt overlays in ERN16 shall be undertaken using measured FWD curvatures.

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APPENDIX A DISTRIBUTED QUESTIONNAIRES

A.1 MAIN ROADS QUESTIONNAIRE

As part of the Western Australia Road Research and Innovation Program (WARRIP), Main Roads has commissioned ARRB to prepare a Pavement Rehabilitation Manual, which will be written as a supplement to the *Austroads Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design* (AGPT05). The aim of this manual is to provide consistent direction in the selection and design of pavement treatments, ensuring learnings from each region are shared and taken into consideration. As part of this project, it was agreed that ARRB would interview Main Roads staff to determine current practices, historical learnings and potential needs.

The following questionnaire was prepared to provide ARRB with initial information on regional practices. Following the receipt of questionnaire responses, a workshop is proposed during the next Main Roads Network Managers Conference, where the contents of the proposed pavement rehabilitation manual will be discussed.

You have two options:

1. Fill out the attached questionnaire in detail and return, OR
2. Book a telephone or Skype interview with ARRB at a time suitable for yourself to discuss the questions in person.

If you intend on filling out the attached in detail please return to devina.gee@arrb.com.au by 11th October 2019.

If you would prefer to discuss the questions over the phone please advise a suitable time and date before 11th October 2019 through devina.gee@arrb.com.au ASAP.

Should you have any questions, please contact Ross Keeley (ross.keeley@mainroads.wa.gov.au) or Zia Rice (zia.rice@mainroads.wa.gov.au).

Main Roads staff name: ____________________

Region: ________________________

1. What are the most common types of treatments are used in your region to address different situations?

To be addressed	Common treatment
General shape loss	
Rutting in the wheelpath	
Cracking	
Cracking with only minor shape loss	
Proposed increase in traffic	
Other	

2. What are the triggers for rehabilitation work being initiated in your region (rutting exceeds x mm, roughness exceeds x IRI, MMIS data, maintenance expenditure exceeding, or collectively, etc.)
3. Who scopes treatment projects and budgets?
4. Who prepares the design? (internal regional staff, consultants, when is assistance from MEB required?)
5. What is required to undertake design? Is there a standard minimum requirement in terms of pavement investigation items (e.g. FWD, pavement dippings, laboratory testing, etc.)?
6. What data is available and used for the design?
7. What is/are the design methodologies adopted (e.g. CBR design chart, deflection design chart, mechanistic empirical designs, UCS to determine cement/lime content, etc.)?
8. What are the typical targeted design lives for different pavement treatments?

9. Can you identify any areas of potential improvement in regards to pavement rehabilitation treatments that would be relevant to your region (e.g. lack of standard specifications specific to rehabilitation works, difficult access to historical pavement data, lack of guidelines on how to design the thickness of overlay treatments, etc.)
10. Can you identify any specific items you would like to see included in the proposed pavement rehabilitation manual?
11. Can you provide examples of pavement rehabilitation design reports and any other relevant regional documents that you would recommend are reviewed in the preparation of the pavement rehabilitation guide? (Please attach files to your email.)
12. Please provide any other comments or important information you think may be relevant to this project.

A.2 INDUSTRY QUESTIONNAIRE

As part of the Western Australia Road Research and Innovation Program (WARRIP), Main Roads has commissioned ARRB to prepare a Pavement Rehabilitation Manual, which will be written as a supplement to the Austroads *Guide to Pavement Technology Part 5: Pavement Evaluation and Treatment Design* (AGPT05).

The aim of this manual is to provide consistent direction in the selection and design of pavement treatments, ensuring learnings from each region of Main Roads are shared and taken into consideration. As part of this project, it was agreed that ARRB would interview Main Roads staff and industry to determine current practices, historical learnings and potential needs.

The following questionnaire was prepared to provide ARRB with initial information on industry practices.

Should you have any questions, please contact Ross Keeley (ross.keeley@mainroads.wa.gov.au) or Zia Rice (zia.rice@arrb.com.au).

Please return the above questionnaire with detailed responses to zia.rice@arrb.com.au before 14th February 2020.

Thank you in advance for your collaboration!

Name: ____________________

Position/employer: ________________________

1. Do you provide input into the type of treatment for different rehabilitation situations? If so, what common treatment do you suggest/prefer for the following:

To be addressed	Common treatment
General shape loss	
Rutting in the wheelpath	
Cracking	
Cracking with only minor shape loss	
Proposed increase in traffic	
Other	

2. What data is available or provided by Main Roads for the design?
3. Is there other data which you are typically not provided with which you would prefer in order to undertake a design (e.g. FWD, pavement dippings, laboratory testing, etc.)?
4. What is/are the design methodologies adopted (e.g. CBR design chart, deflection design chart, mechanistic empirical designs, UCS to determine cement/lime content, etc.)
5. What are the typical targeted design lives for different pavement treatments?
6. Can you identify any areas of potential improvement in regards to pavement rehabilitation treatments?

7. Can you identify any specific items you would like to see included in the proposed pavement rehabilitation manual?
8. Can you provide examples of pavement rehabilitation design reports and any other relevant documents that you would recommend are reviewed in the preparation of the pavement rehabilitation guide? (Please attach files to your email.)
9. Please provide any other comments or important information you think may be relevant to this project which may not be covered by the above items.

APPENDIX B TRAFFIC LOAD DISTRIBUTION

Main Roads advised that the design charts should be calculated using its generic urban traffic load distribution as given in Table B.1.

Table B.1: Generic urban traffic load distributions

Axle group load (kN)	Axle group type					
	SAST	**SADT**	**TAST**	**TADT**	**TRDT**	**QADT**
10	0.9001	5.9100		0.8070		
20	11.9431	15.5050		1.4290		
30	14.3071	14.5340		1.2210		
40	14.4381	15.7460	1.4970	1.9910		
50	23.7261	15.6660		5.2170	4.6510	
60	28.6961	13.0310	1.2810	8.4000	9.1460	
70	5.9891	7.6930	5.0730	9.0690	10.7790	
80		4.6480	14.9400	9.0800	7.2810	
90		3.7970	20.6550	7.8010	5.3120	4.6850
100		1.7530	17.3820	5.5780	3.7340	9.1440
110		1.7290	16.7530	4.3480	2.8800	10.7770
120			14.2290	4.0880	2.4150	7.2790
130			5.6100	5.2440	1.8670	5.3100
140			1.8250	5.4340	1.7950	3.7320
150			0.7640	6.5890	1.8520	2.8780
160				7.5350	1.9070	2.4130
170				7.1900	2.2070	1.8650
180				4.9060	3.1560	1.7930
190				2.5110	4.0650	1.8500
200				1.5760	5.5330	1.9050
210					6.7400	2.2050
220					7.0980	3.1540
230					6.2900	4.0630
240					4.6190	5.5310
250					3.1560	6.7380
260					1.9970	7.0960
270					1.5370	6.2880
280						4.6170
290						3.1540
300						1.9950
310						1.5350
Total	100	100	100	100	100	100
Proportion of Each Axle Group (%)	35.8	21.9	2.5	24.6	15.2	0.01
NHVAG	ESA/HVAG	ESA/HV	SAR5/ESA	SAR7/ESA		
2.61	0.85	2.25	0.83	1.20		

Notes:

SAAT: Single axle single tyre, SADT: Single axle dual tyre, TAST: Tandem axle single tyre,
TADT: Tandem axle dual tyre, TRDT: Tri-axle dual tyre, QADT: Quad-axle dual tyre.

APPENDIX C RELATIONSHIPS BETWEEN CURVATURE AND OVERLAY FATIGUE LIFE

Shown in Figure C.1 to Figure C.21 are plots of predicted FWD $D_0 – D_{200}$ values under a plate contact stress of 700 kPa prior to placement of an overlay against the allowable traffic loading (ESA) to fatigue cracking of the asphalt overlay.

C.1 THIN OVERLAYS ON PAVEMENTS WITHOUT EXISTING ASPHALT

Figure C.1 40 mm thick asphalt overlay/inlay

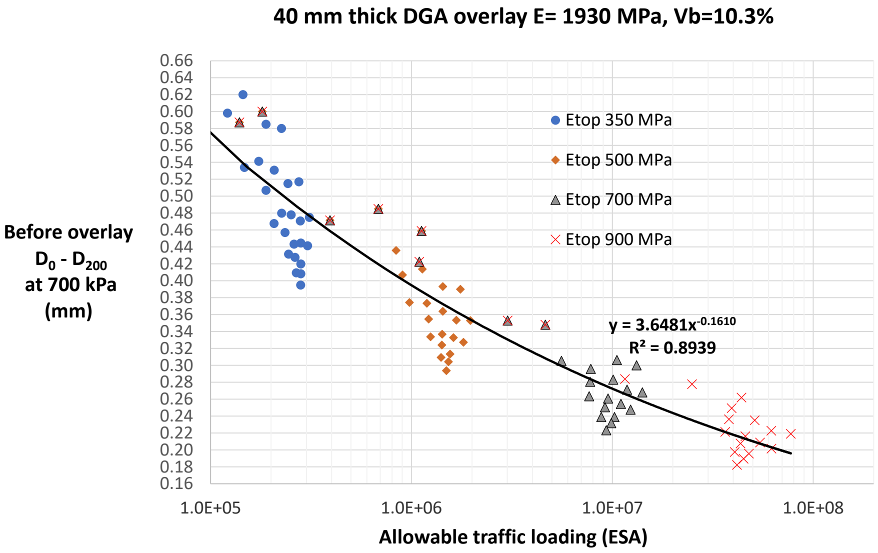


Figure C.2 50 mm thick asphalt overlay/inlay

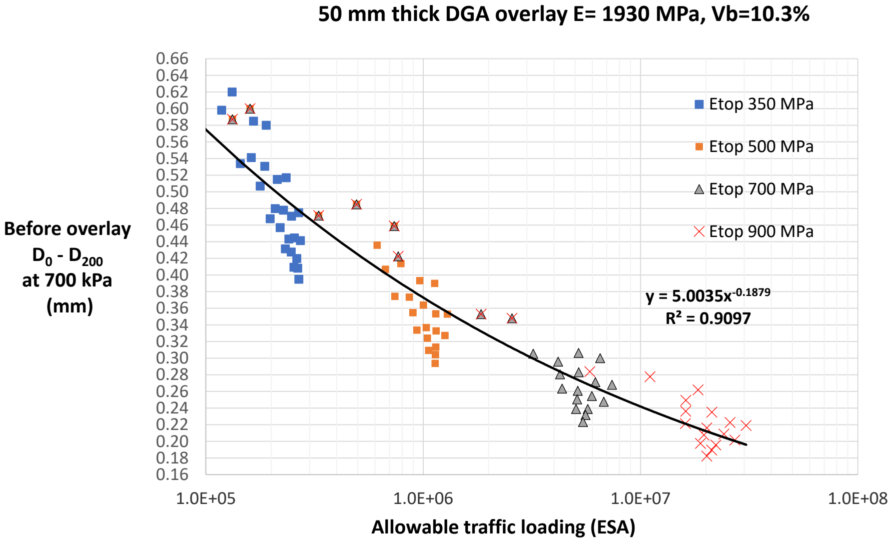


Figure C.3 60 mm thick asphalt overlay/inlay

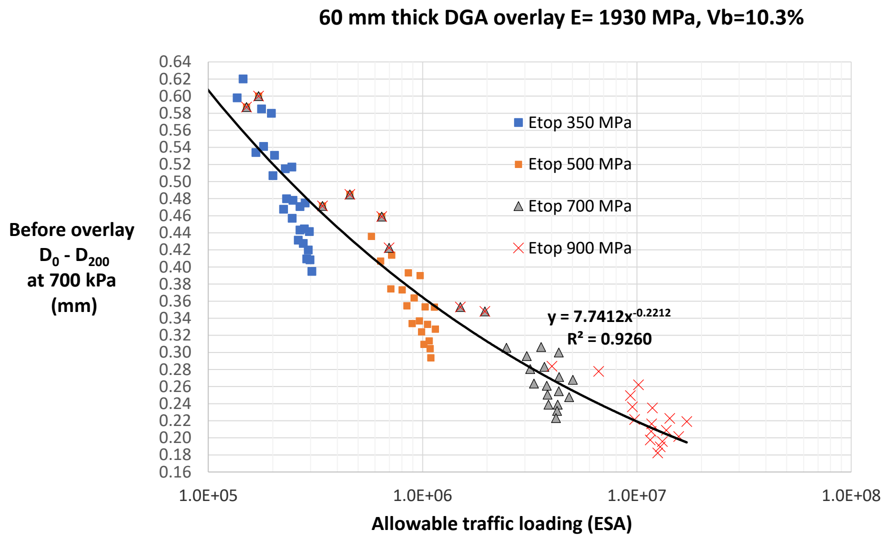
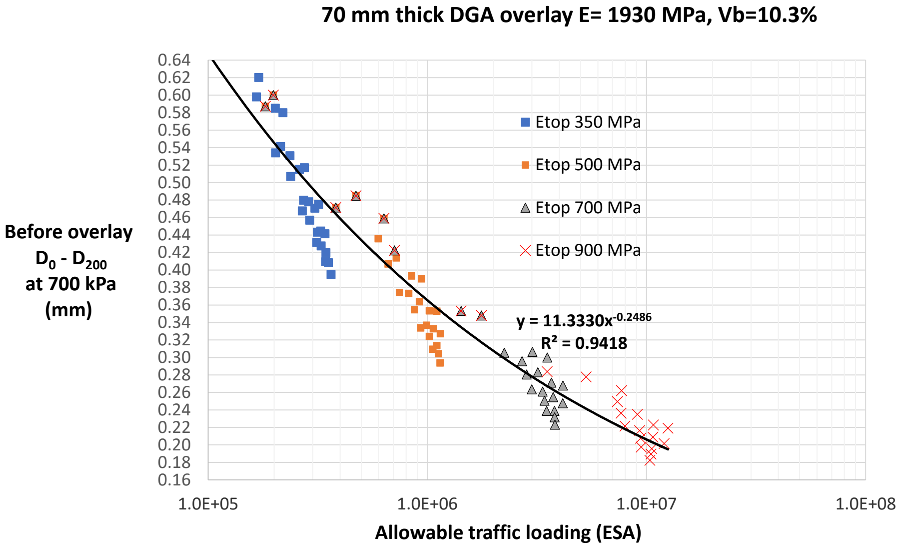


Figure C.4 70 mm thick asphalt overlay/inlay



C.2 THIN OVERLAYS ON PAVEMENTS WITH EXISTING ASPHALT

Figure C.5 40 mm thick asphalt overlay/inlay

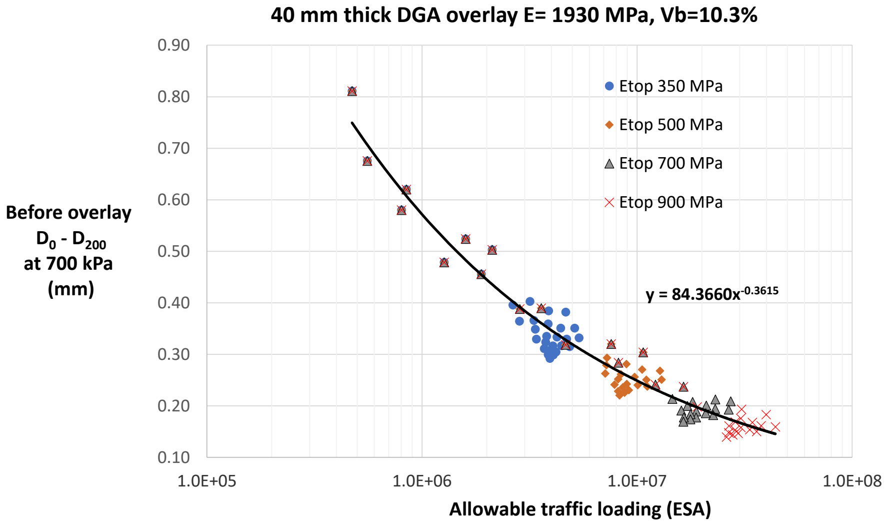


Figure C.6 50 mm thick asphalt overlay/inlay

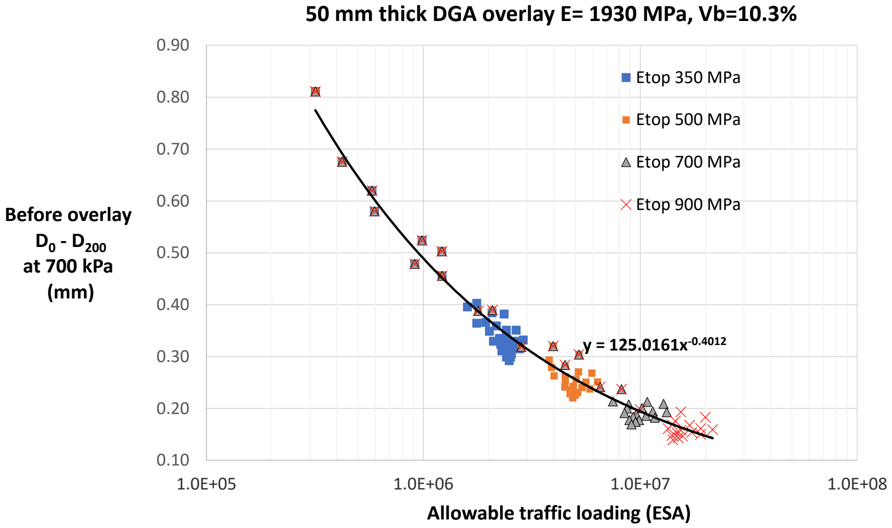


Figure C.7 60 mm thick asphalt overlay/inlay

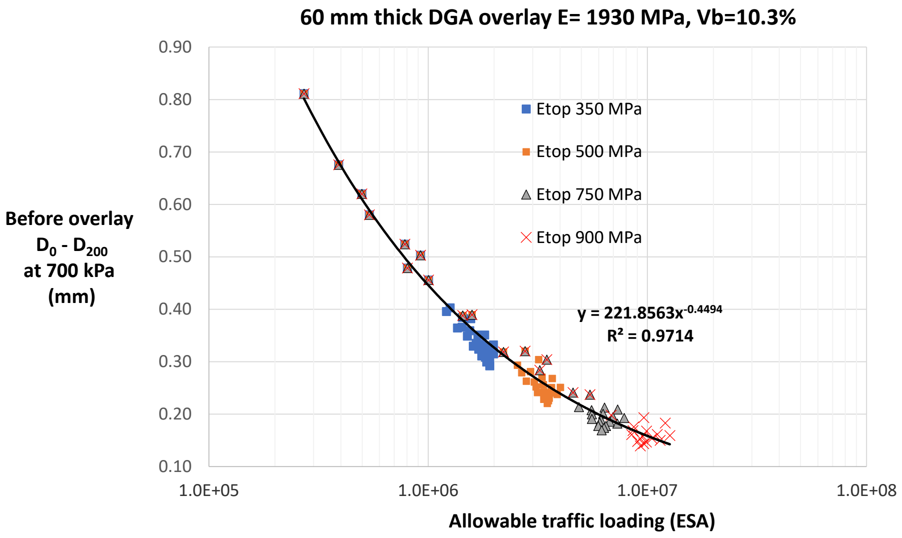
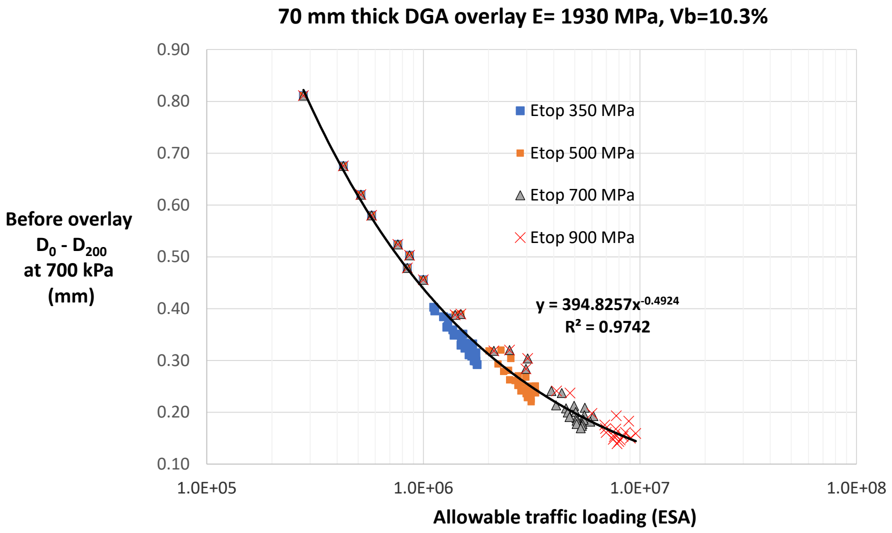


Figure C.8 70 mm thick asphalt overlay/inlay



C.3 RELATIONSHIPS DEVELOPED FOR THE THICK OVERLAY DESIGN CHART

Figure C.9 80 mm thick asphalt overlay/inlay

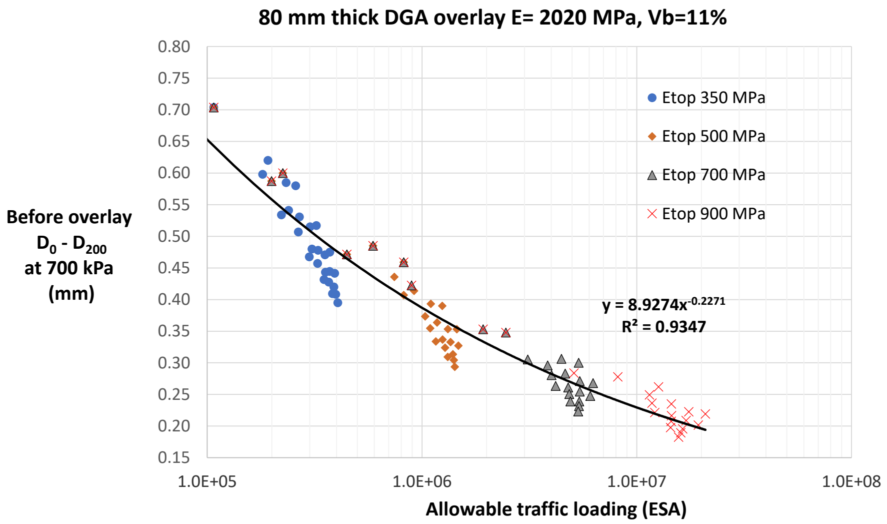


Figure C.10 90 mm thick asphalt overlay/inlay

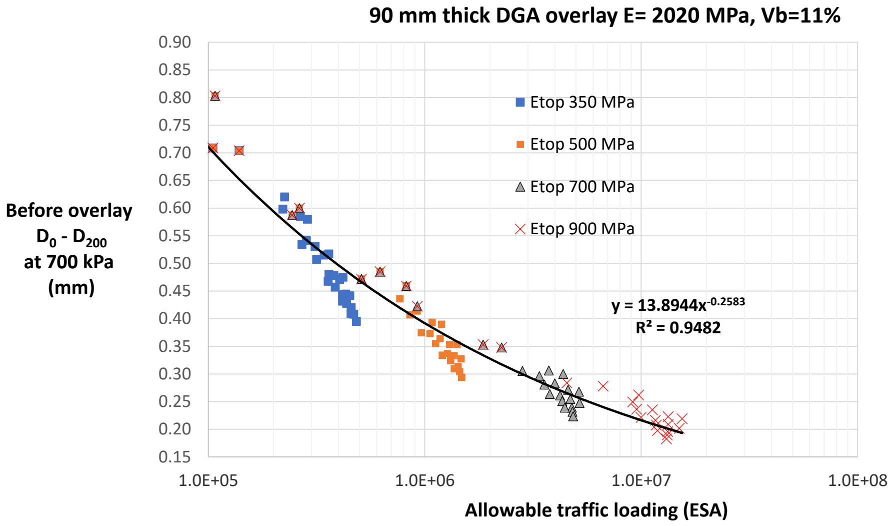


Figure C.11 100 mm thick asphalt overlay/inlay

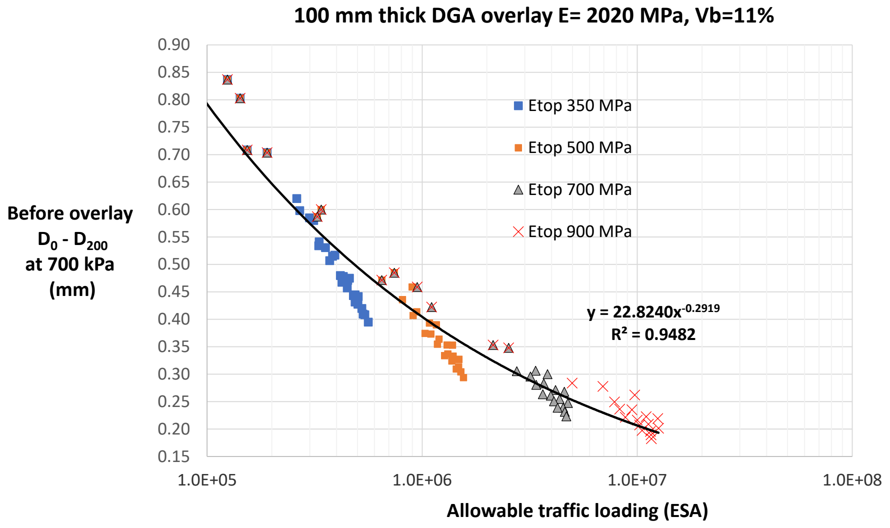


Figure C.12 110 mm thick asphalt overlay/inlay

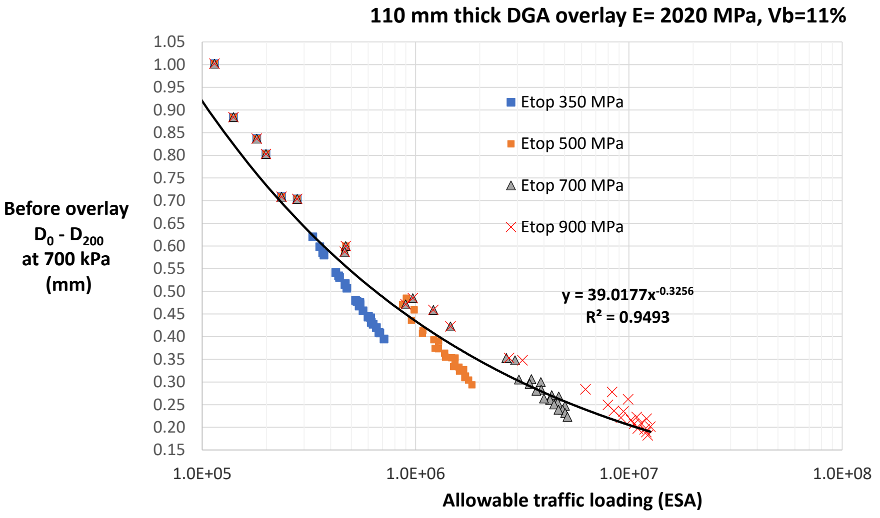


Figure C.13 120 mm thick asphalt overlay/inlay

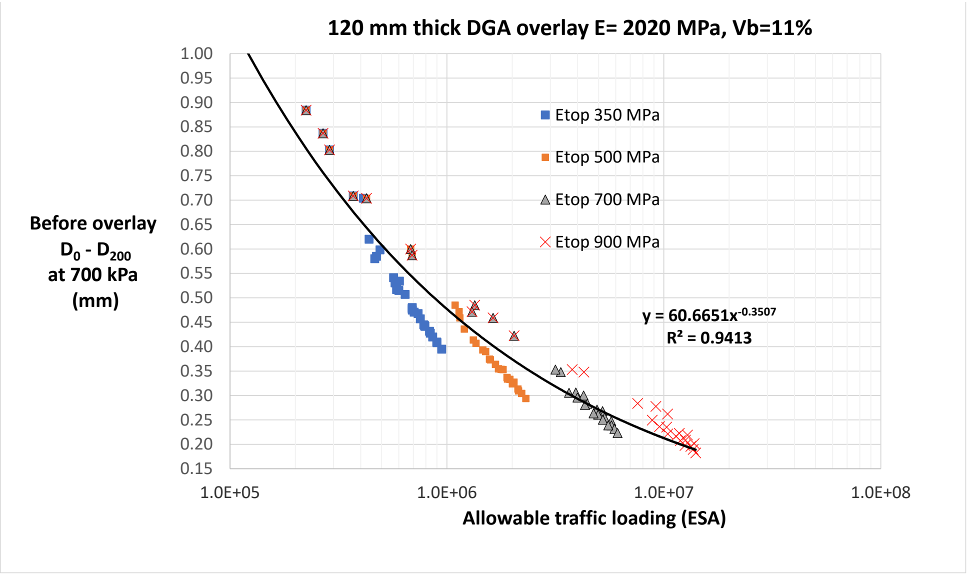


Figure C.14 130 mm thick asphalt overlay/inlay

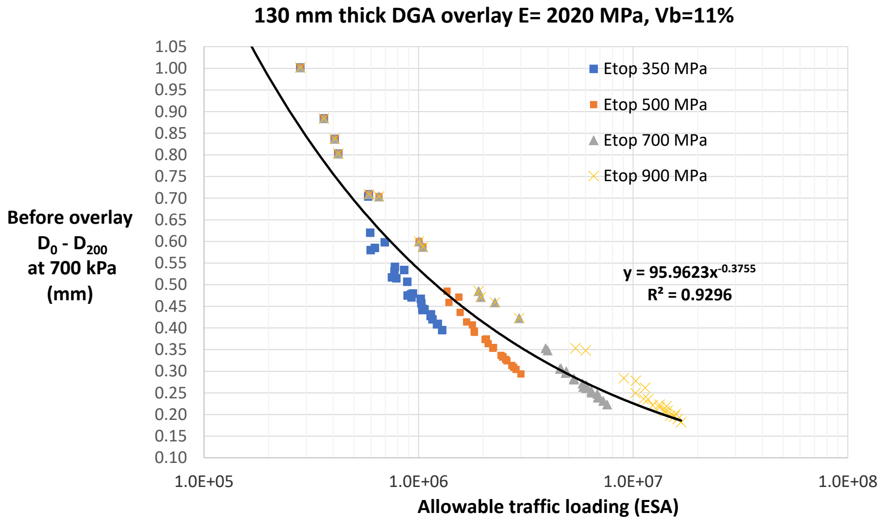


Figure C.15 140 mm thick asphalt overlay/inlay

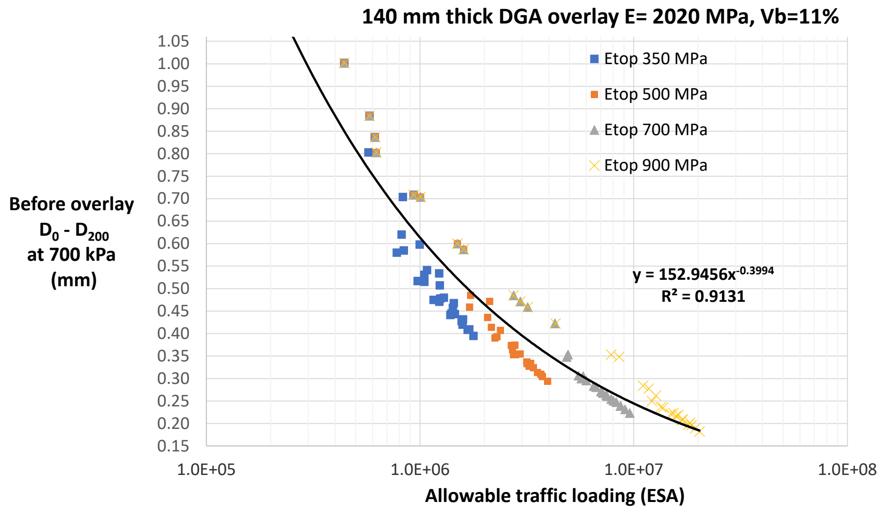


Figure C.16 150 mm thick asphalt overlay/inlay

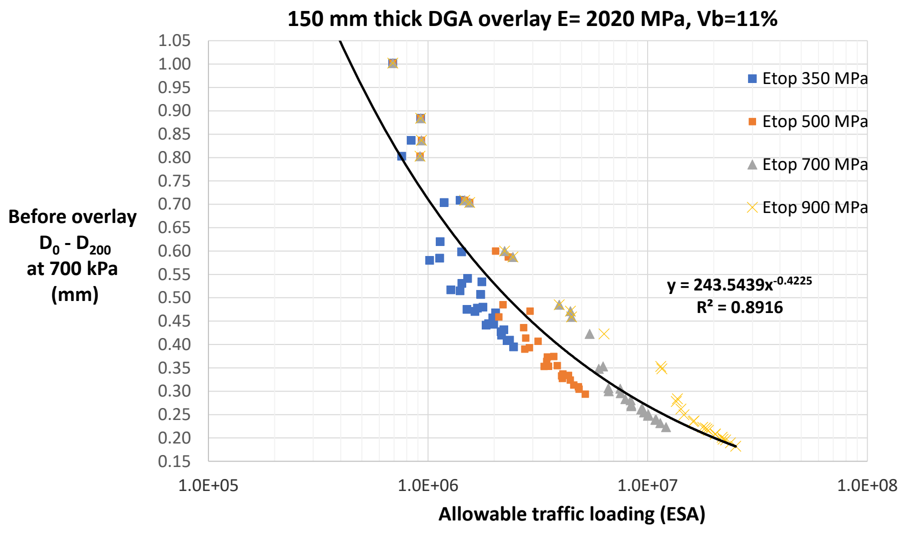


Figure C.17 160 mm thick asphalt overlay/inlay

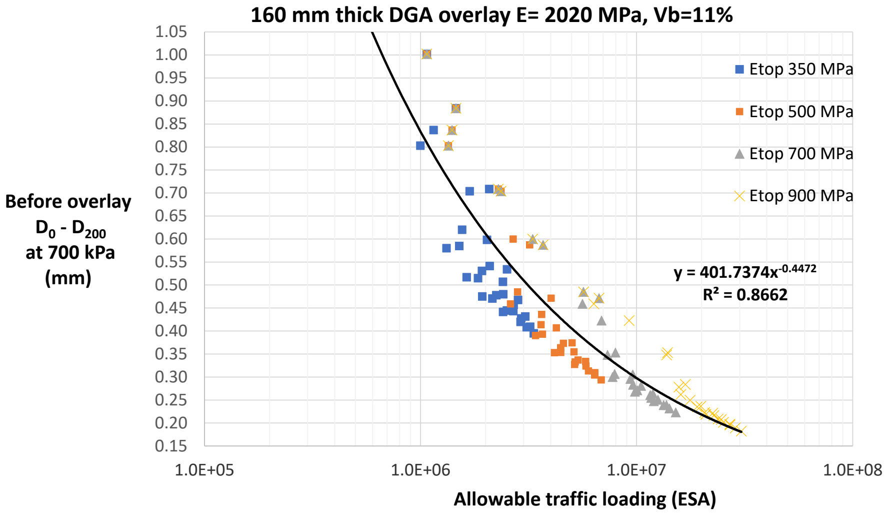


Figure C.18 170 mm thick asphalt overlay/inlay

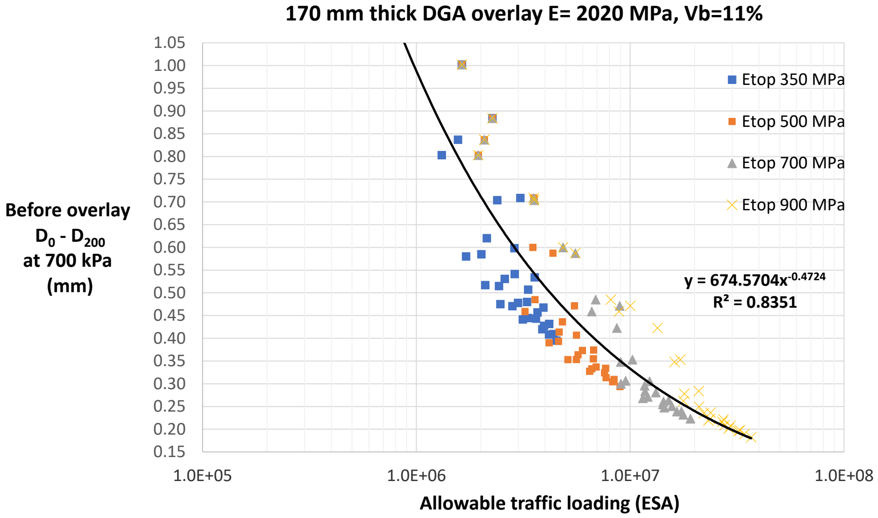


Figure C.19 180 mm thick asphalt overlay/inlay

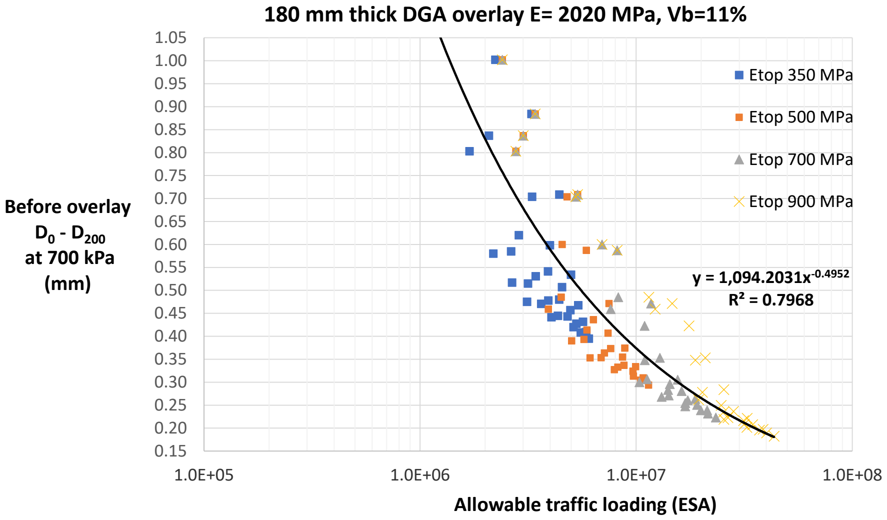


Figure C.20 190 mm thick asphalt overlay/inlay

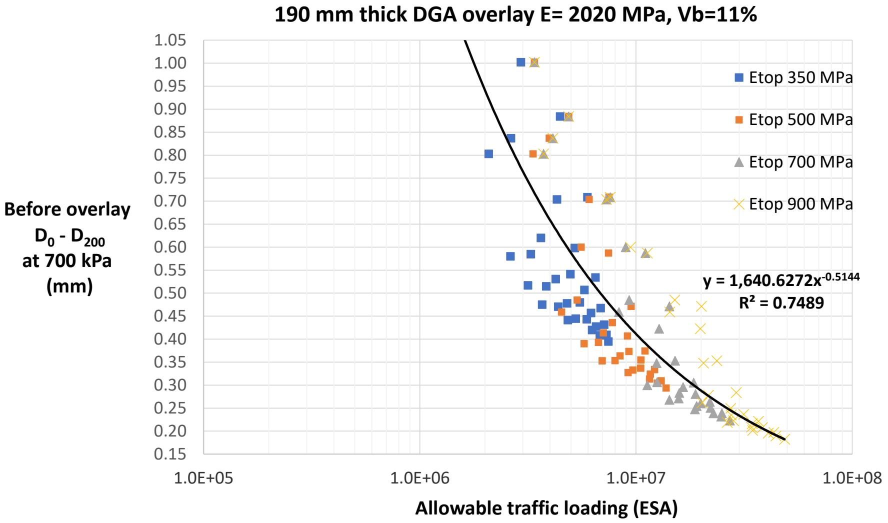
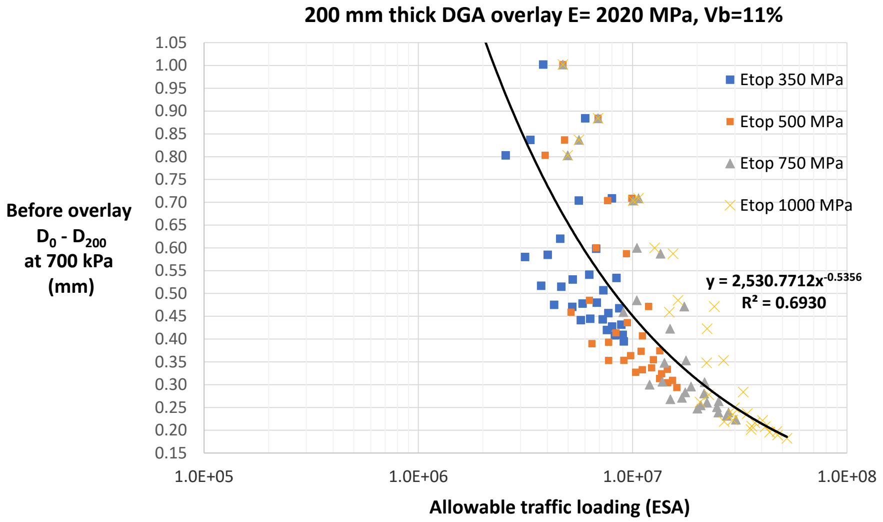


Figure C.21 200 mm thick asphalt overlay/inlay



APPENDIX D PRELIMINARY ASSESSMENT OF DEFLECTION DATA

Figure D.1 Visual inspection of FWD vs TSD maximum deflections (D_0) – Bussell Highway

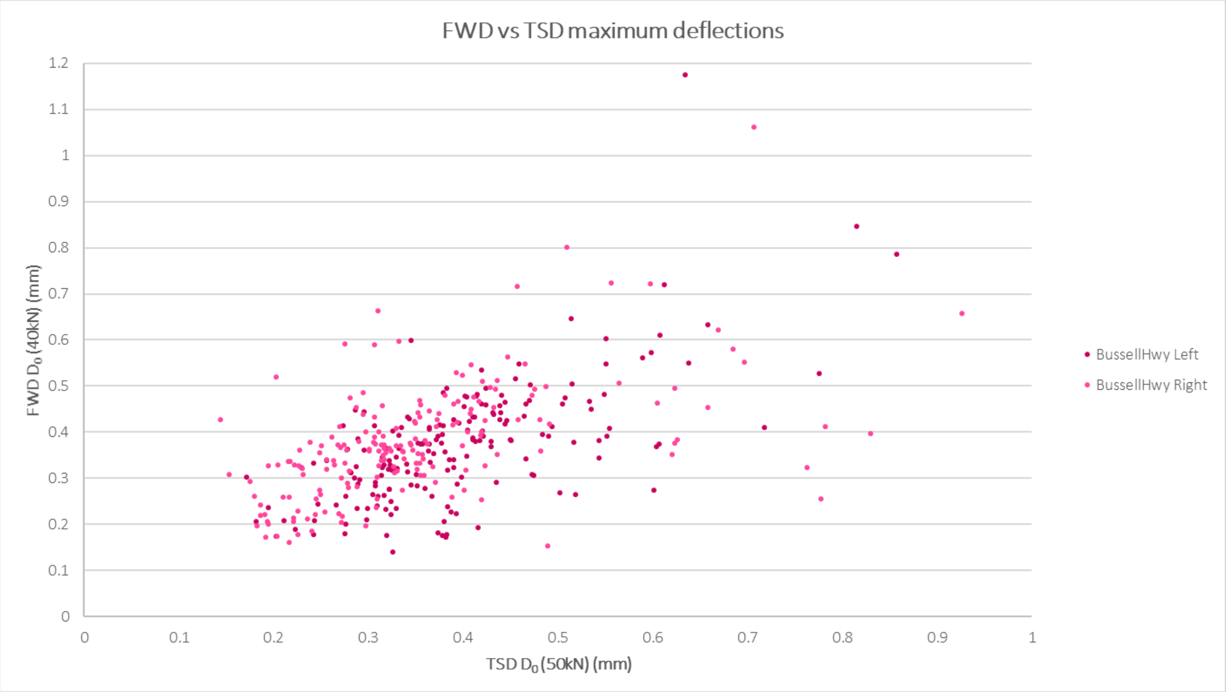


Figure D.2 Visual inspection of FWD vs TSD curvature (D_0 – D_{200}) – Bussell Highway

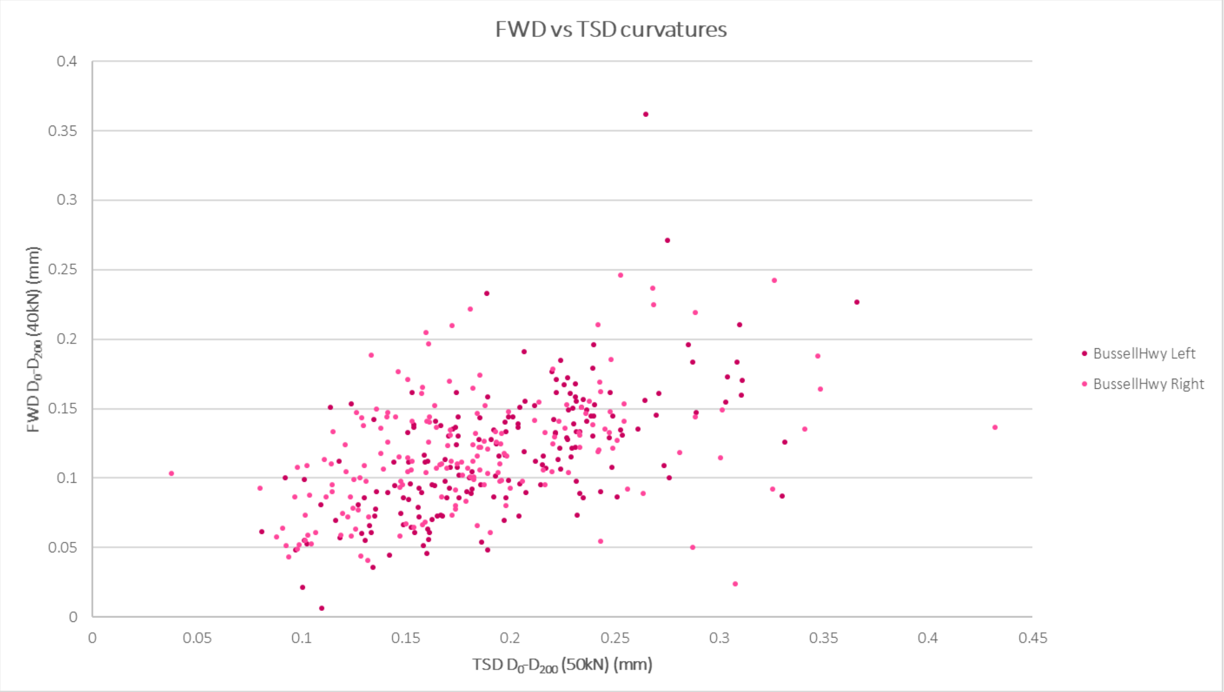


Figure D.3 Visual inspection of FWD vs TSD maximum deflections (D_0) – Forrest Highway

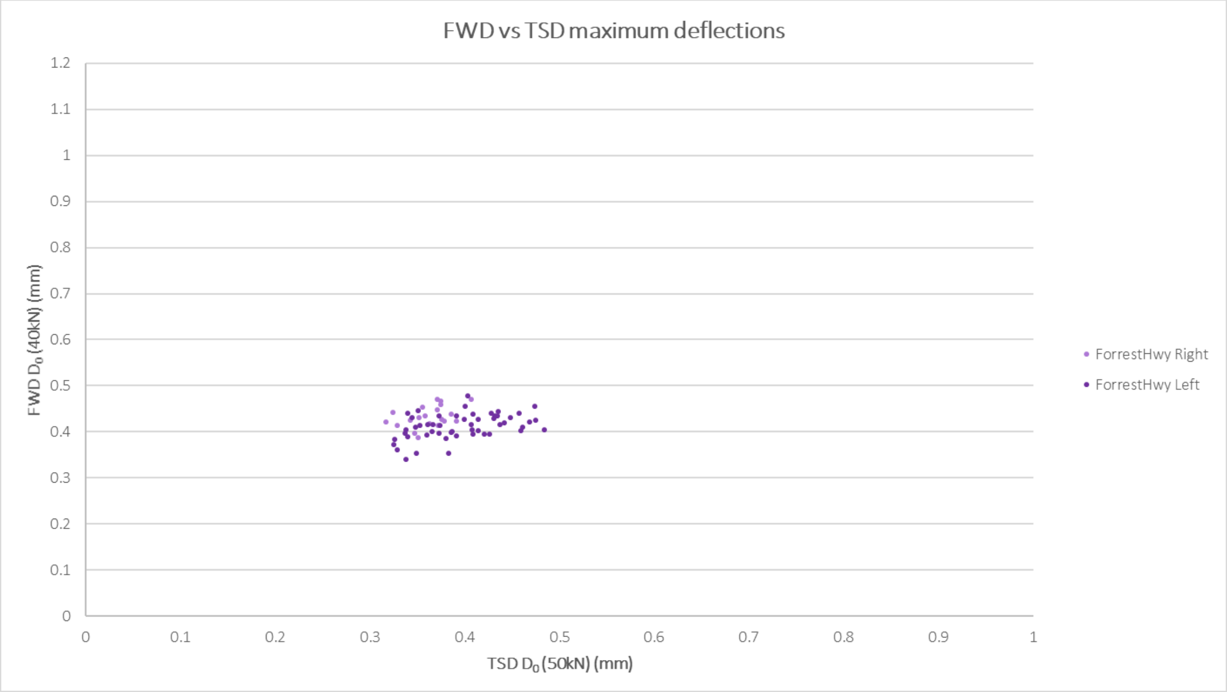


Figure D.4 Visual inspection of FWD vs TSD curvature (D_0 – D_{200}) – Forrest Highway

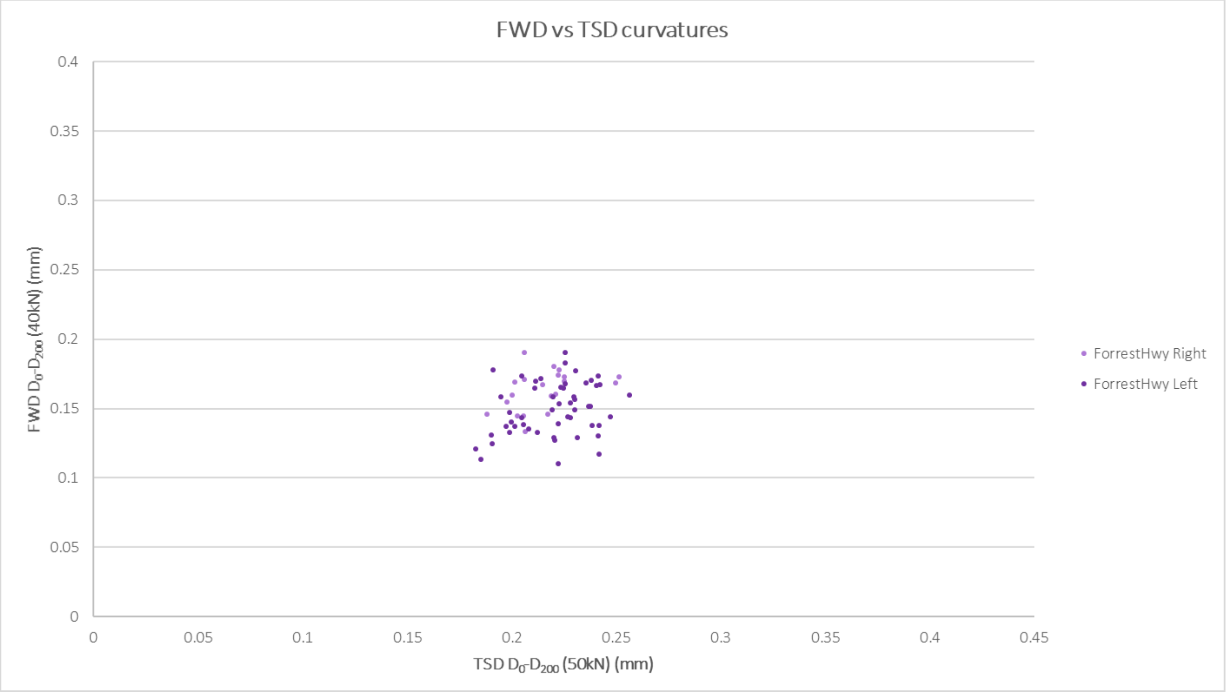


Figure D.5 Visual inspection of FWD vs TSD maximum deflections (D_0) – Kwinana Freeway & Leach Highway

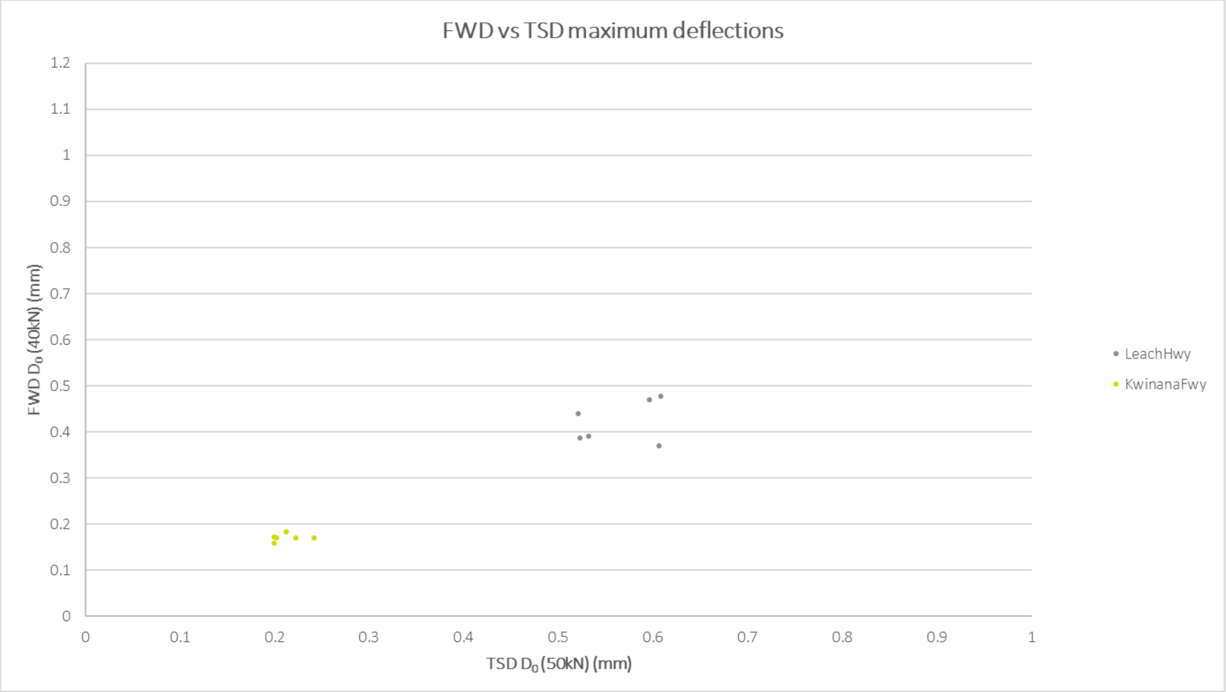


Figure D.6 Visual inspection of FWD vs TSD curvature (D_0 – D_{200}) – Kwinana Freeway & Leach Highway

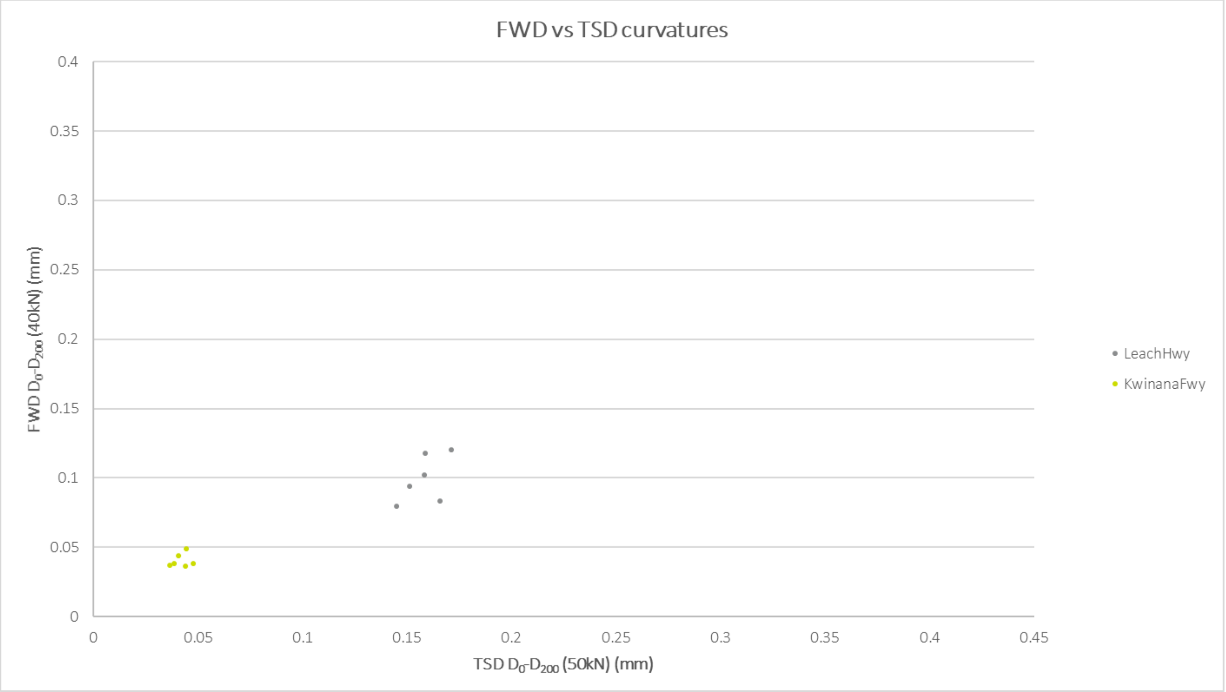


Figure D.7 Visual inspection of FWD vs TSD maximum deflections (D_0) – Trial Mile Road

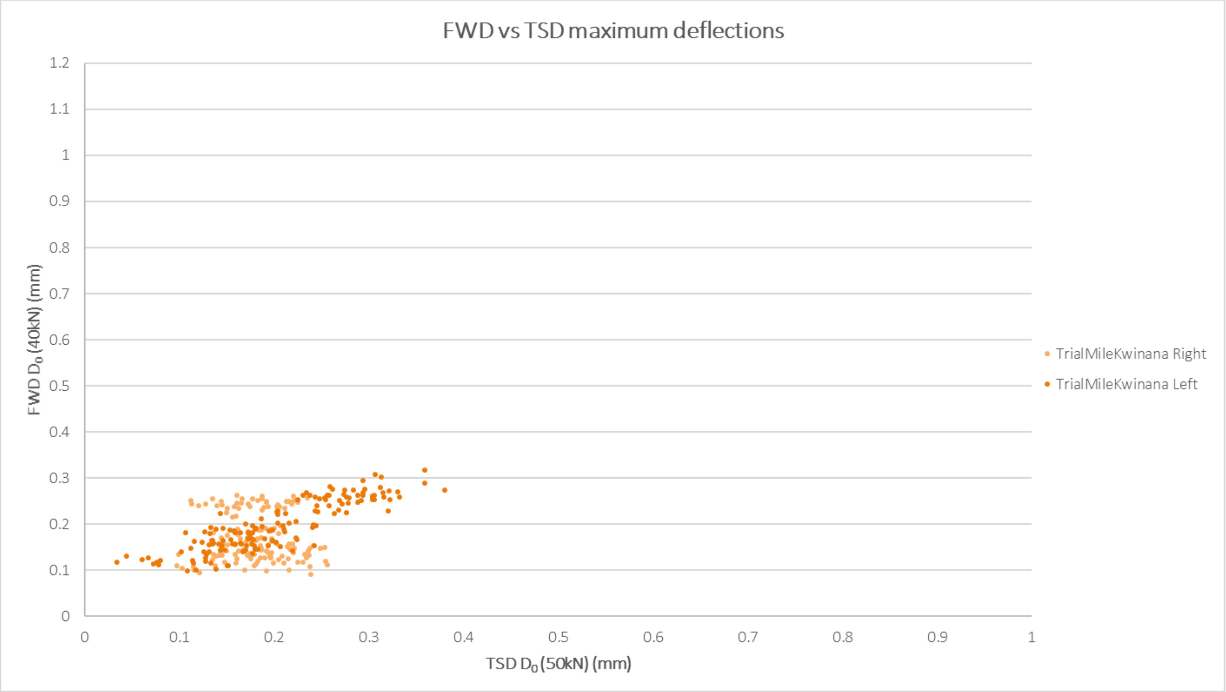


Figure D.8 Visual inspection of FWD vs TSD curvature ($D_0 – D_{200}$) – Trial Mile Road

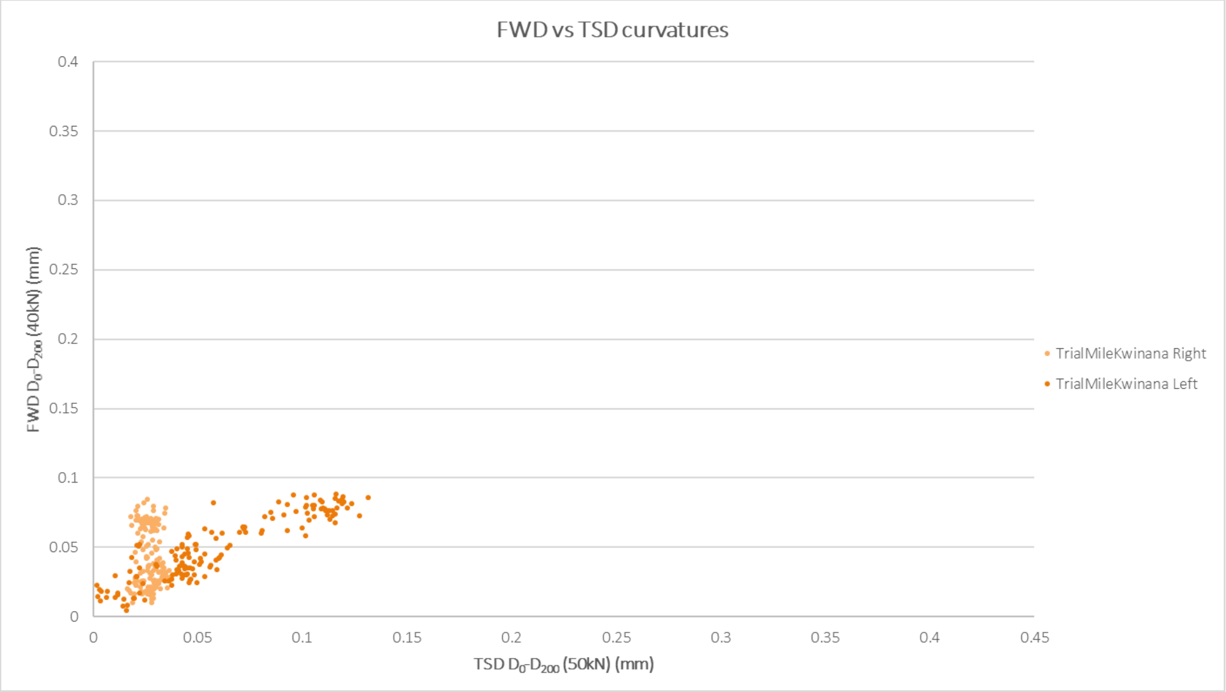


Figure D.9 Visual inspection of FWD vs TSD maximum deflections (D_0) – Great Eastern Highway

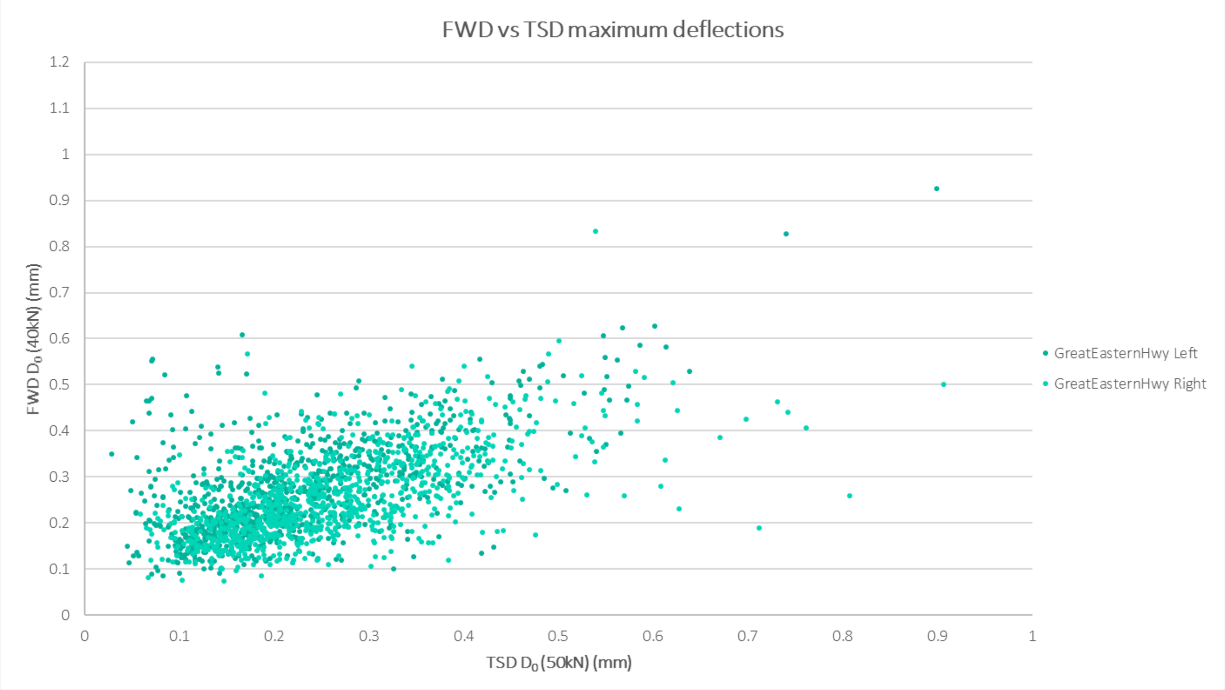


Figure D.10 Visual inspection of FWD vs TSD curvature ($D_0 – D_{200}$) – Great Eastern Highway

Figure D.11 Visual inspection of FWD vs TSD maximum deflections (D_0) – Tonkin Highway, northbound

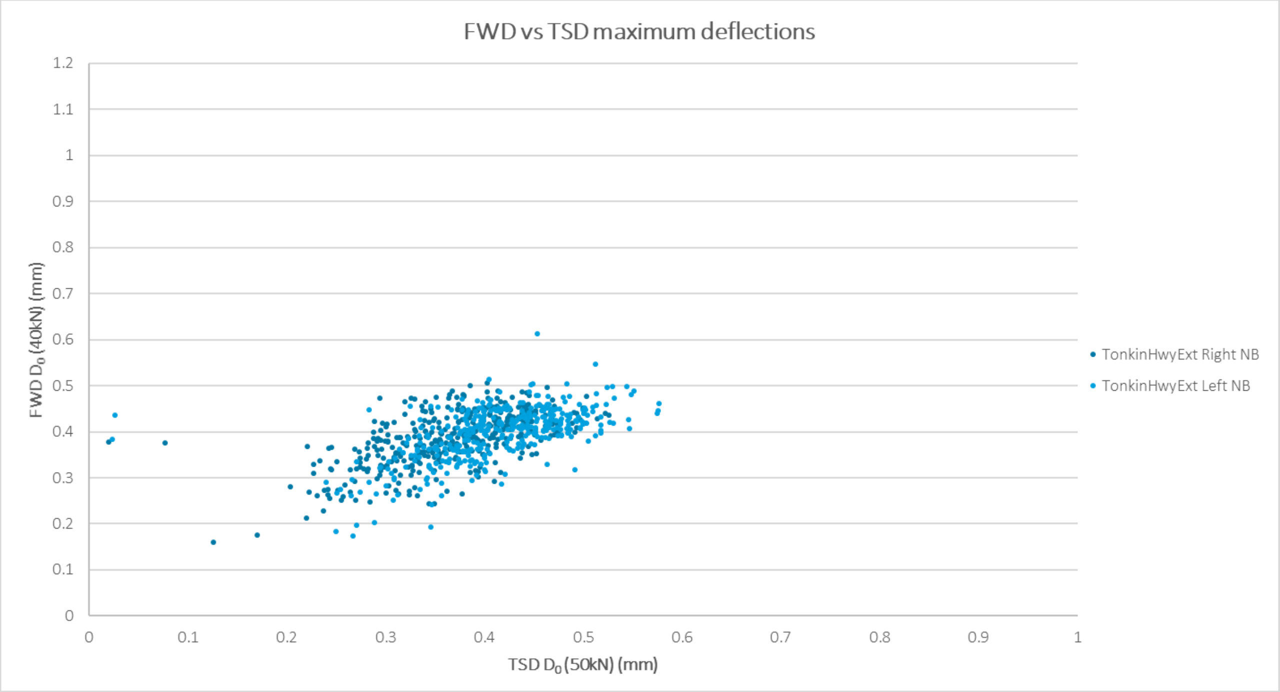


Figure D.12 Visual inspection of FWD vs TSD curvature ($D_0 – D_{200}$) – Tonkin Highway, northbound

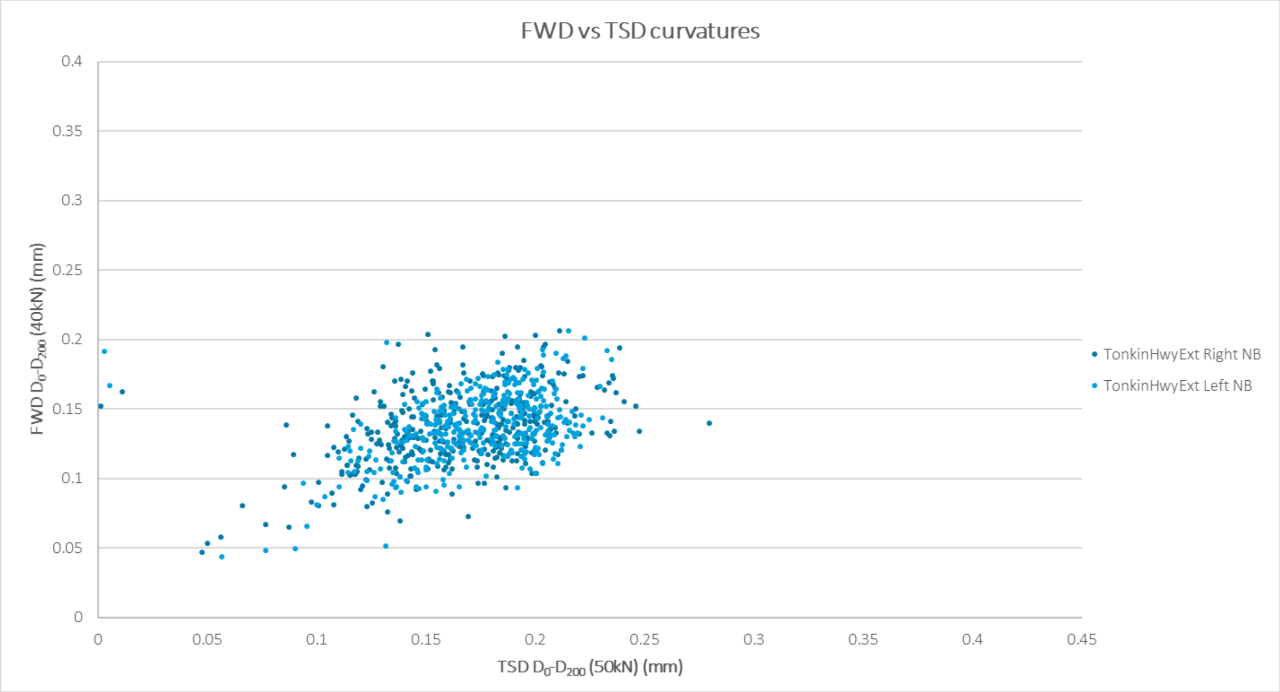


Figure D.13 Visual inspection of FWD vs TSD maximum deflections (D_0) – Tonkin Highway, southbound

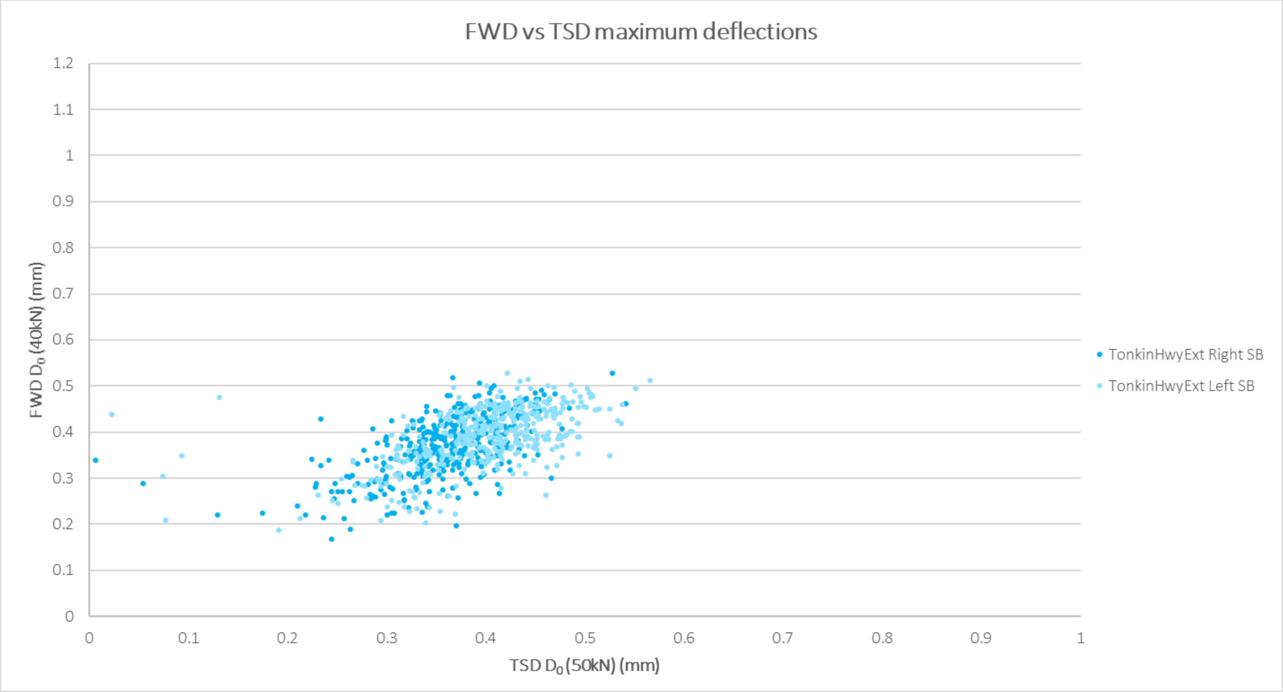
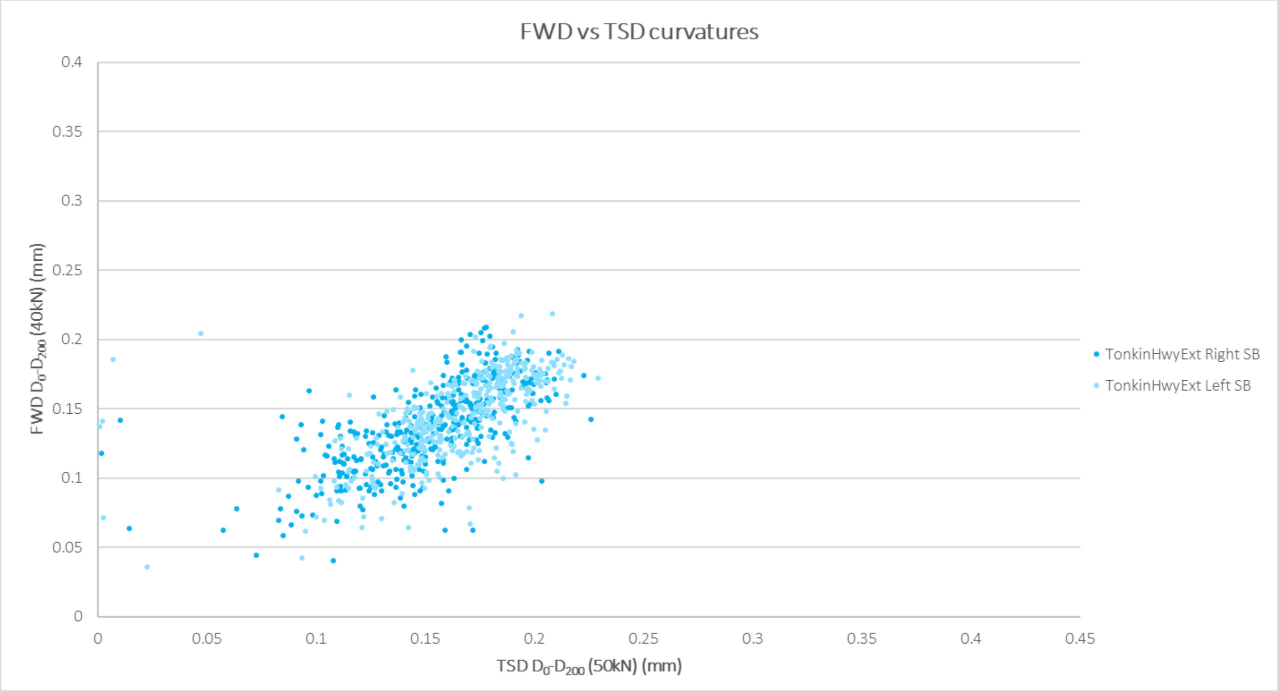
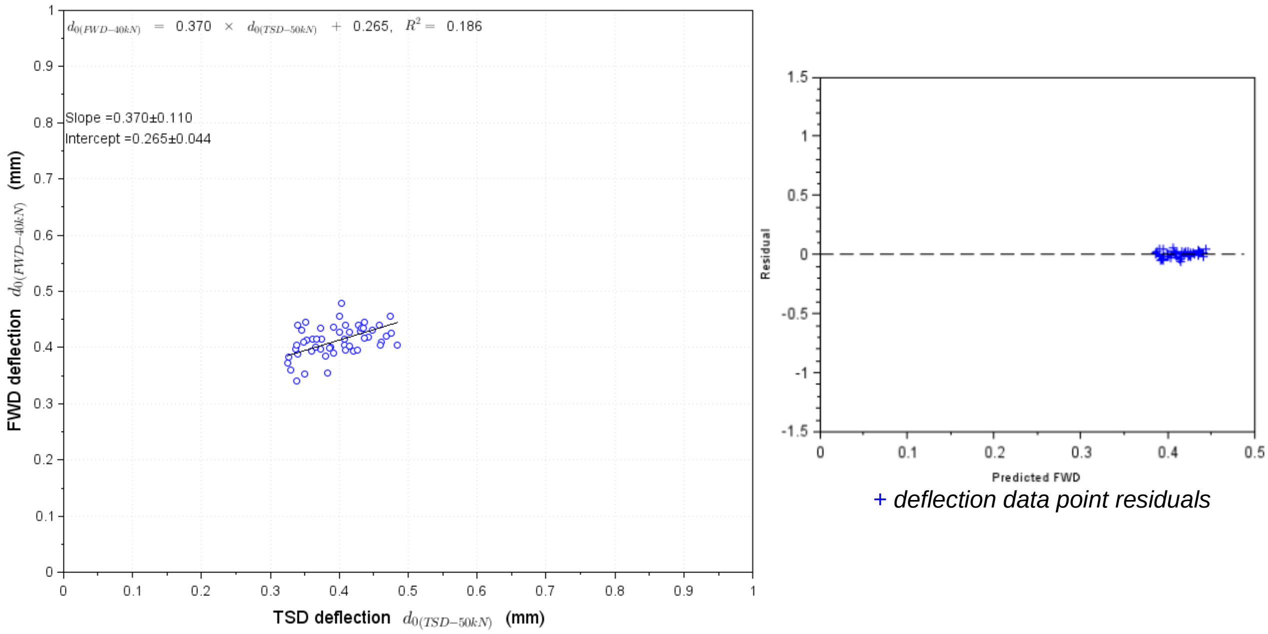


Figure D.14 Visual inspection of FWD vs TSD curvature (D_0 – D_{200}) – Tonkin Highway, southbound



APPENDIX E RESULT OF PRELIMINARY REGRESSION ANALYSIS

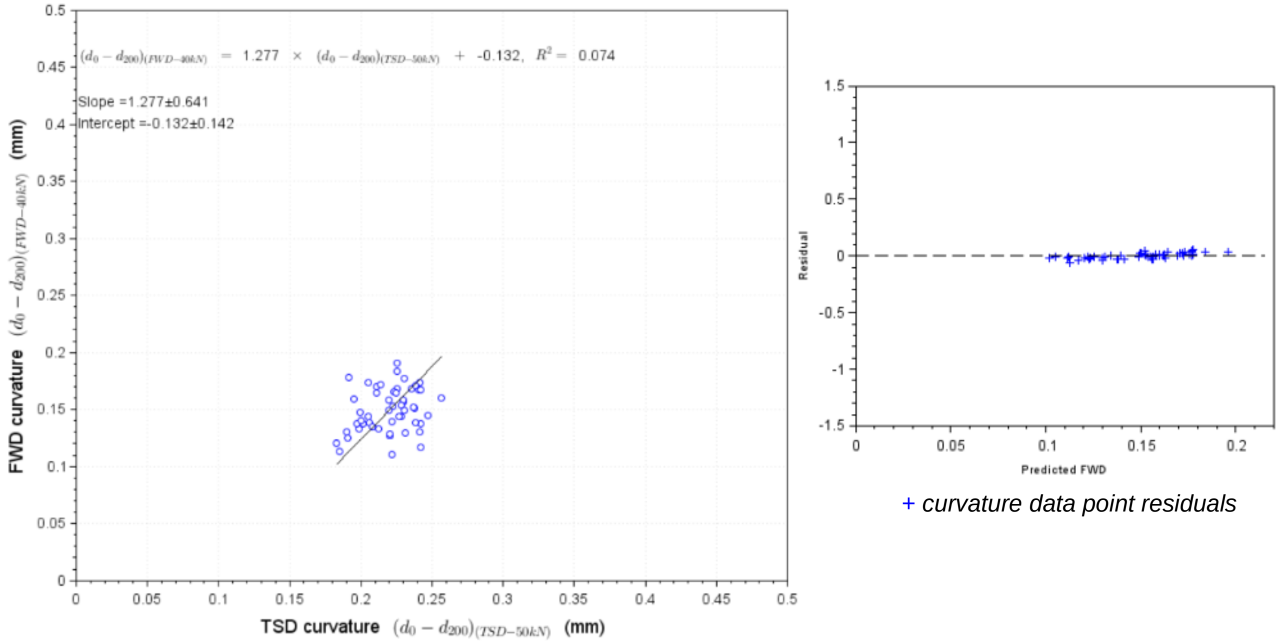
Figure E.1 Forrest Highway, left lane – preliminary regression analysis of D_0



+ *deflection data point residuals*

Where: o FWD vs TSD deflection data – regression line on all data points

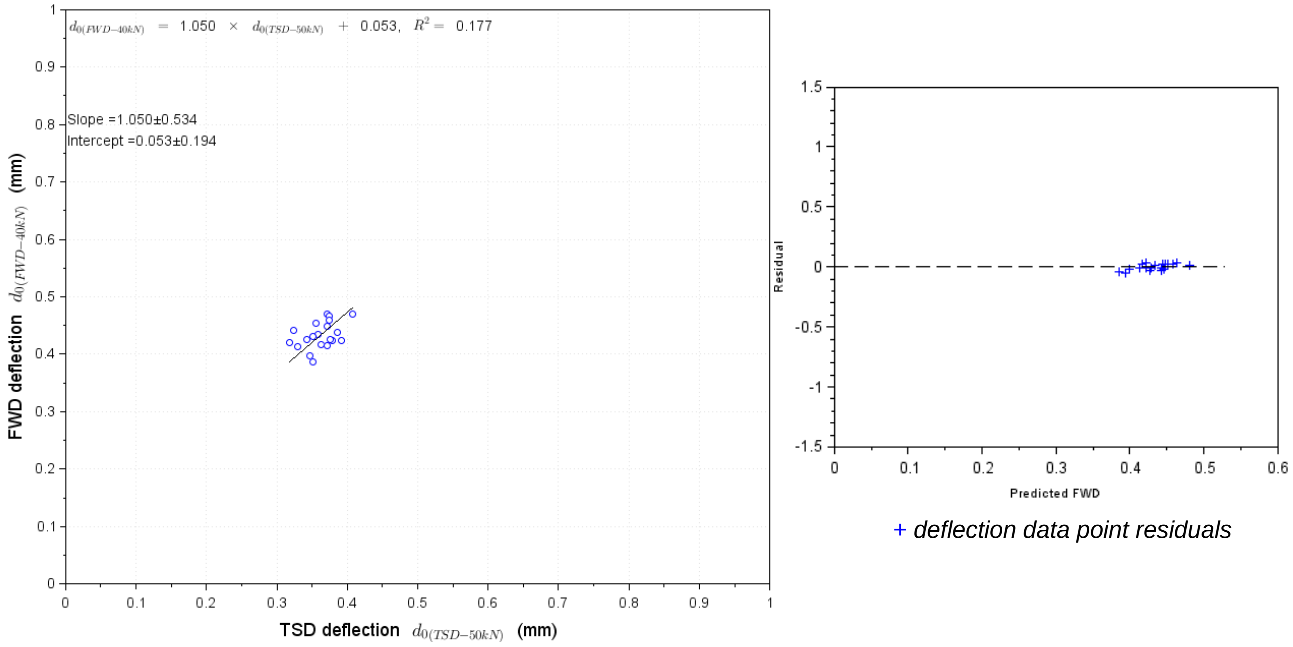
Figure E.2 Forrest Highway, left lane – preliminary regression analysis of curvature (D_0 – D_{200})



+ *curvature data point residuals*

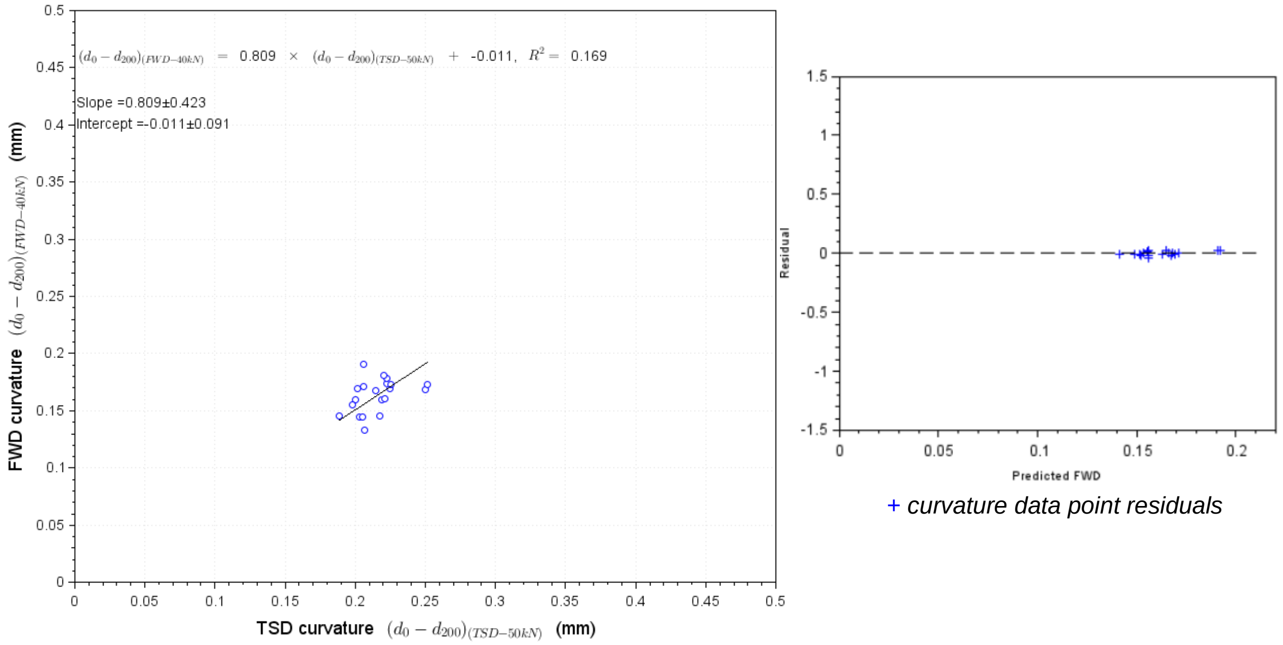
Where: o FWD vs TSD curvature data – regression line on all data points

Figure E.3 Forrest Highway, right lane – preliminary regression analysis of D_0



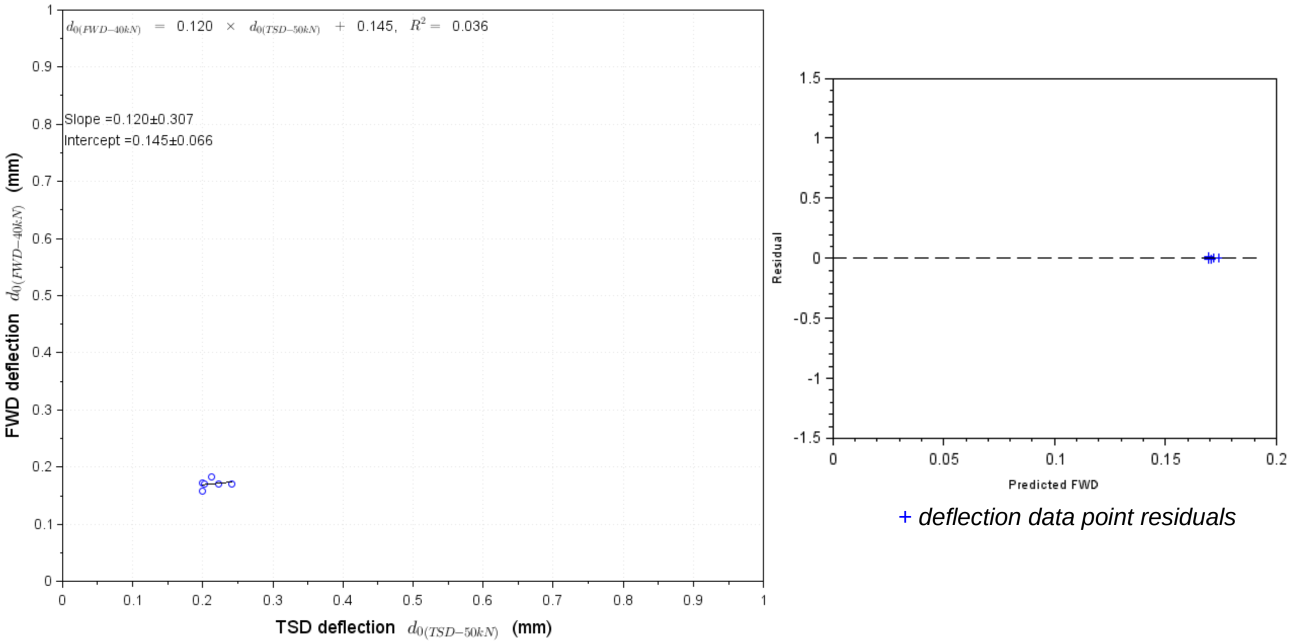
Where: *o FWD vs TSD deflection data* *– regression line on all data points*

Figure E.4 Forrest Highway, right lane – preliminary regression analysis of curvature (D_0 – D_{200})



Where: *o FWD vs TSD curvature data* *– regression line on all data points*

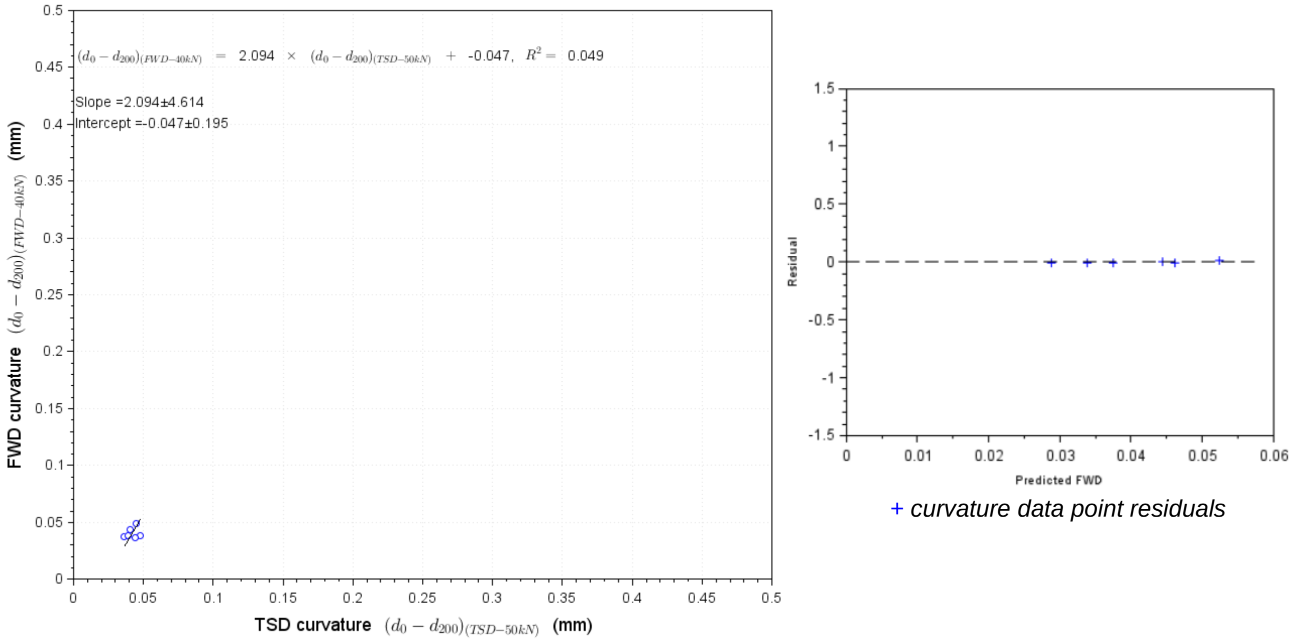
Figure E.5 Kwinana Freeway, one lane – preliminary regression analysis of D_0



+ *deflection data point residuals*

Where: *o FWD vs TSD deflection data* *– regression line on all data points*

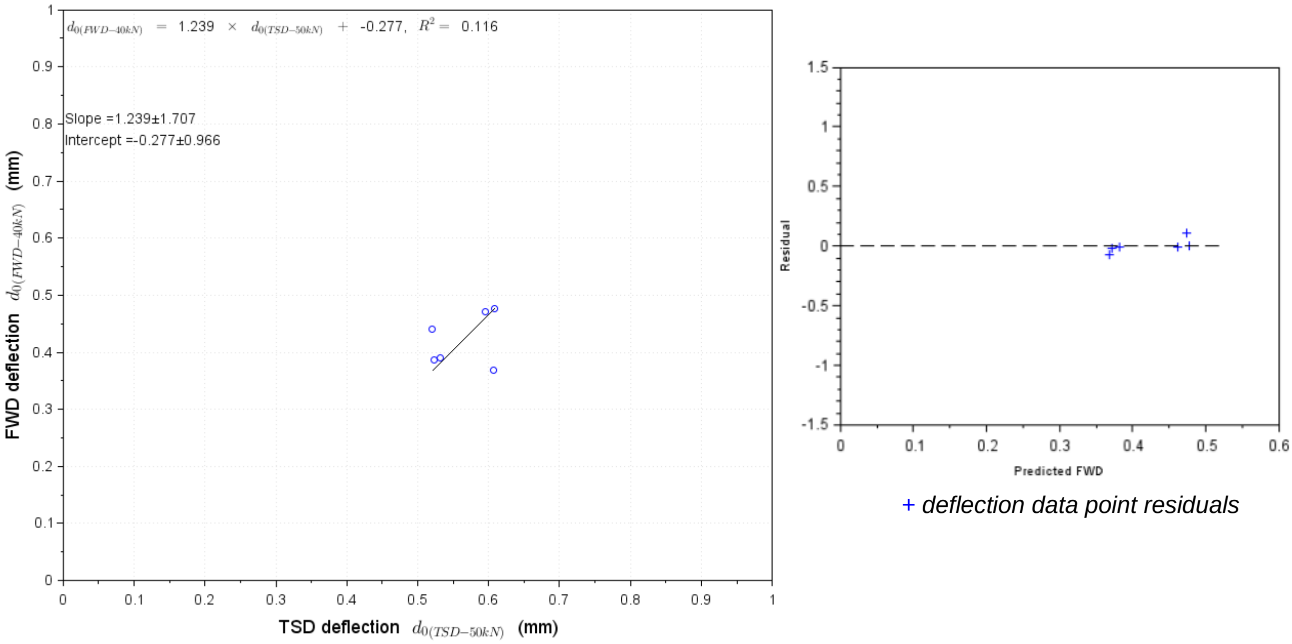
Figure E.6 Kwinana Freeway, one lane – preliminary regression analysis of curvature (D_0 – D_{200})



+ *curvature data point residuals*

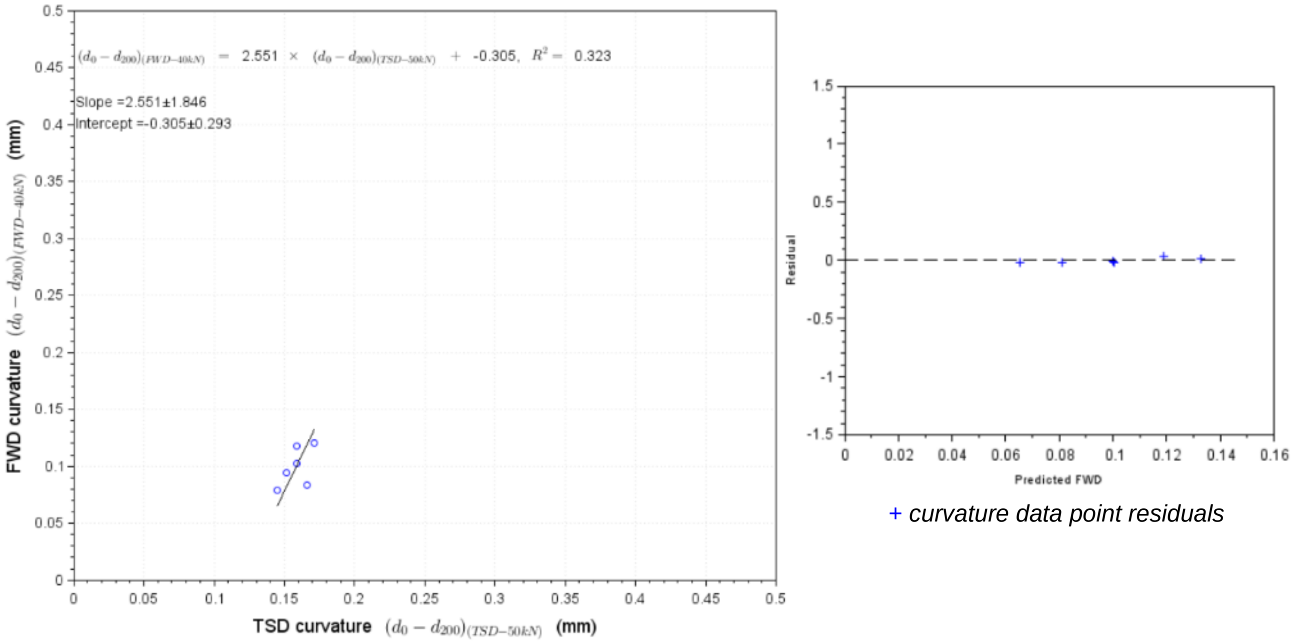
Where: *o FWD vs TSD curvature data* *– regression line on all data points*

Figure E.7 Leach Highway, one lane – preliminary regression analysis of D_0



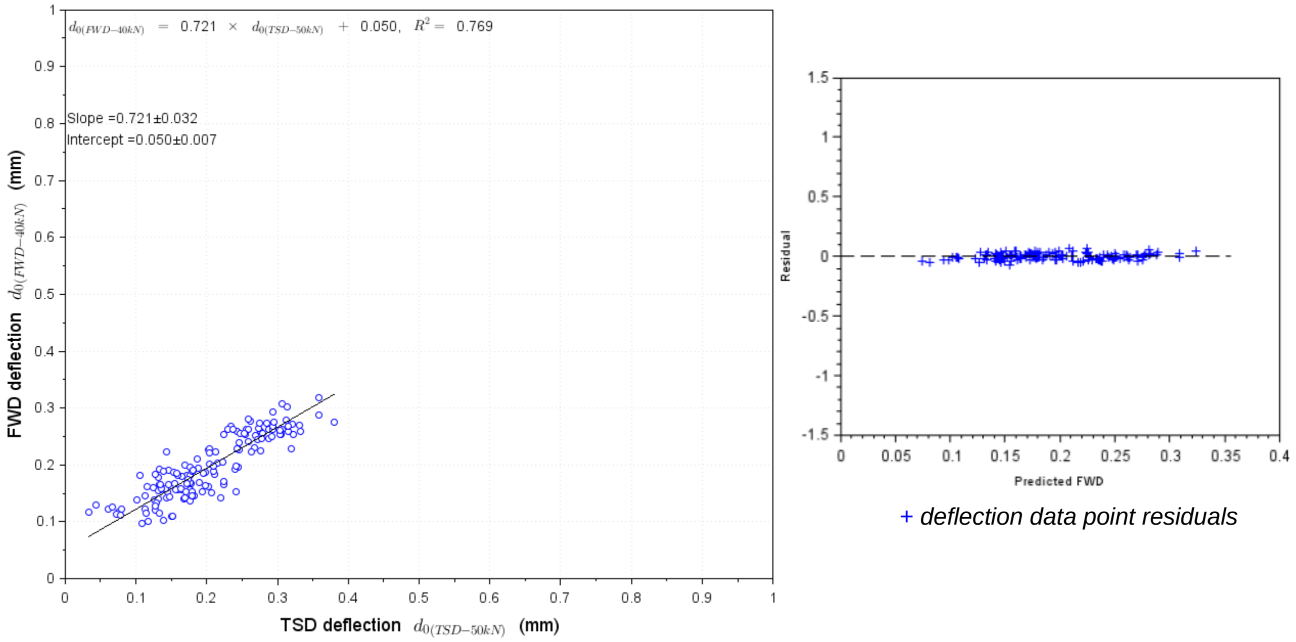
Where: *o FWD vs TSD deflection data* *– regression line on all data points*

Figure E.8 Leach Highway, one lane – preliminary regression analysis of curvature (D_0 – D_{200})



Where: *o FWD vs TSD curvature data* *– regression line on all data points*

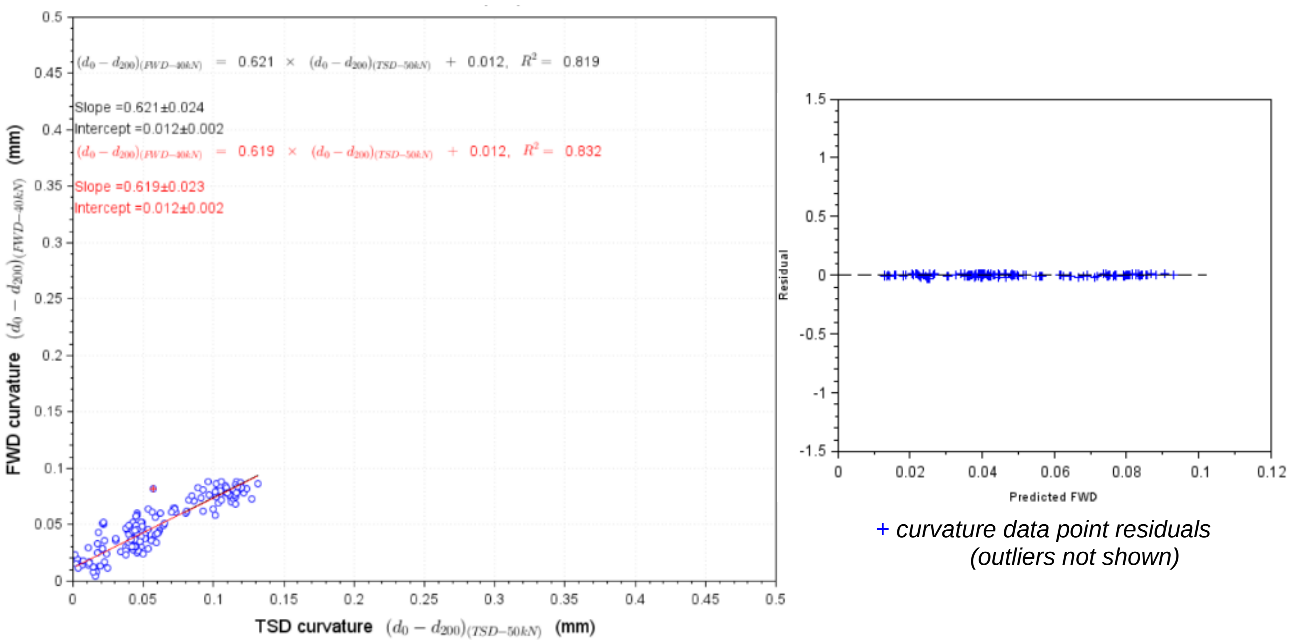
Figure E.9 Trial Mile Road, left lane – preliminary regression analysis of D_0



+ deflection data point residuals

Where: o FWD vs TSD deflection data – regression line on all data points

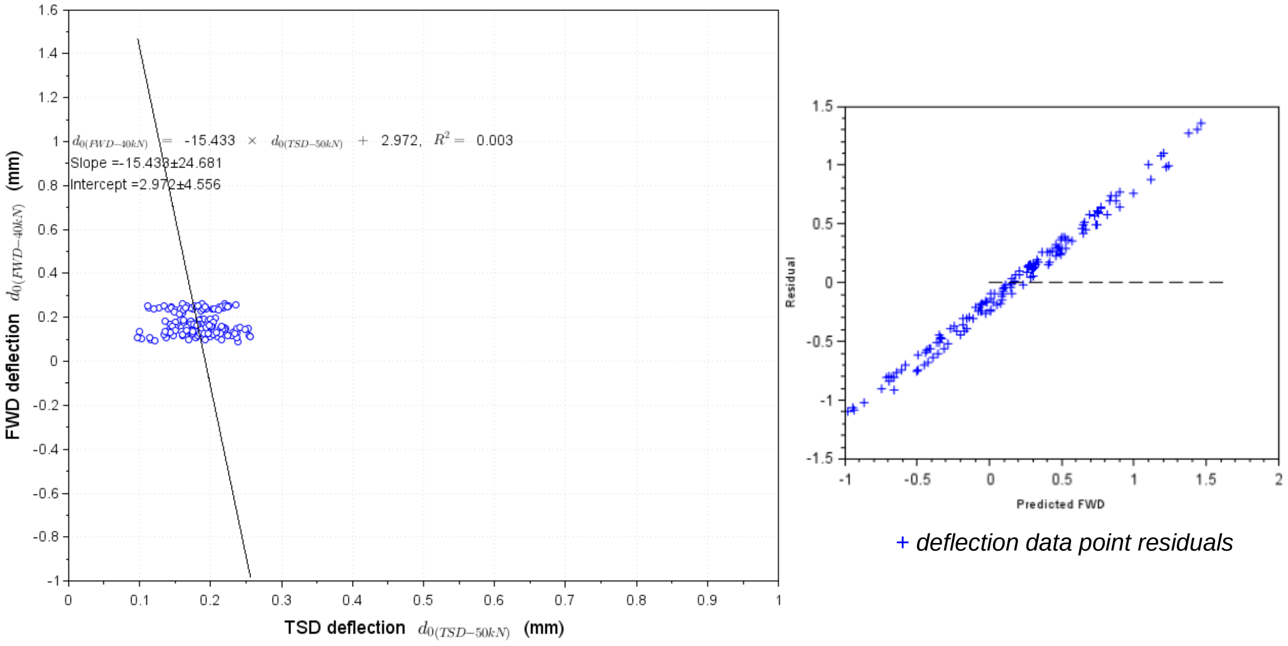
Figure E.10 Trial Mile Road, left lane – preliminary regression analysis of curvature (D_0 – D_{200})



+ curvature data point residuals
(outliers not shown)

Where: o FWD vs TSD curvature data
– regression line on all data points
+ Identified outlier
– Regression line on data points, excluding outliers

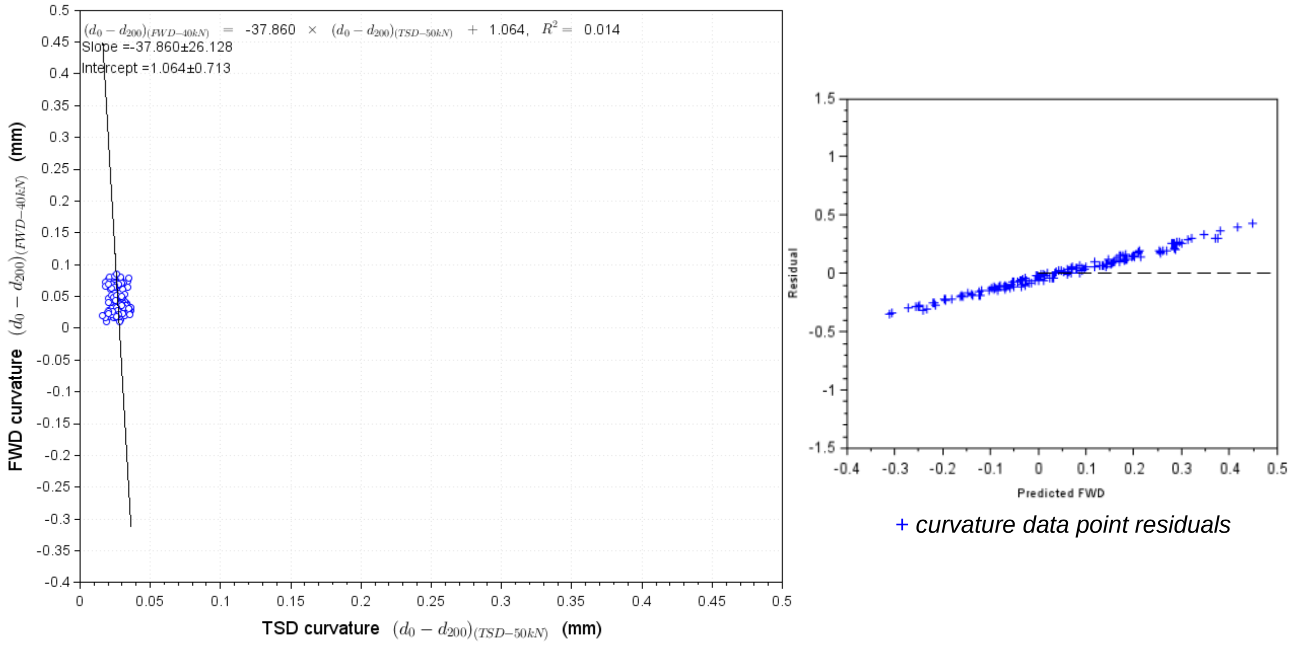
Figure E.11 Trial Mile Road, right lane – preliminary regression analysis of D0



+ *deflection data point residuals*

Where: o FWD vs TSD curvature data – regression line on all data points

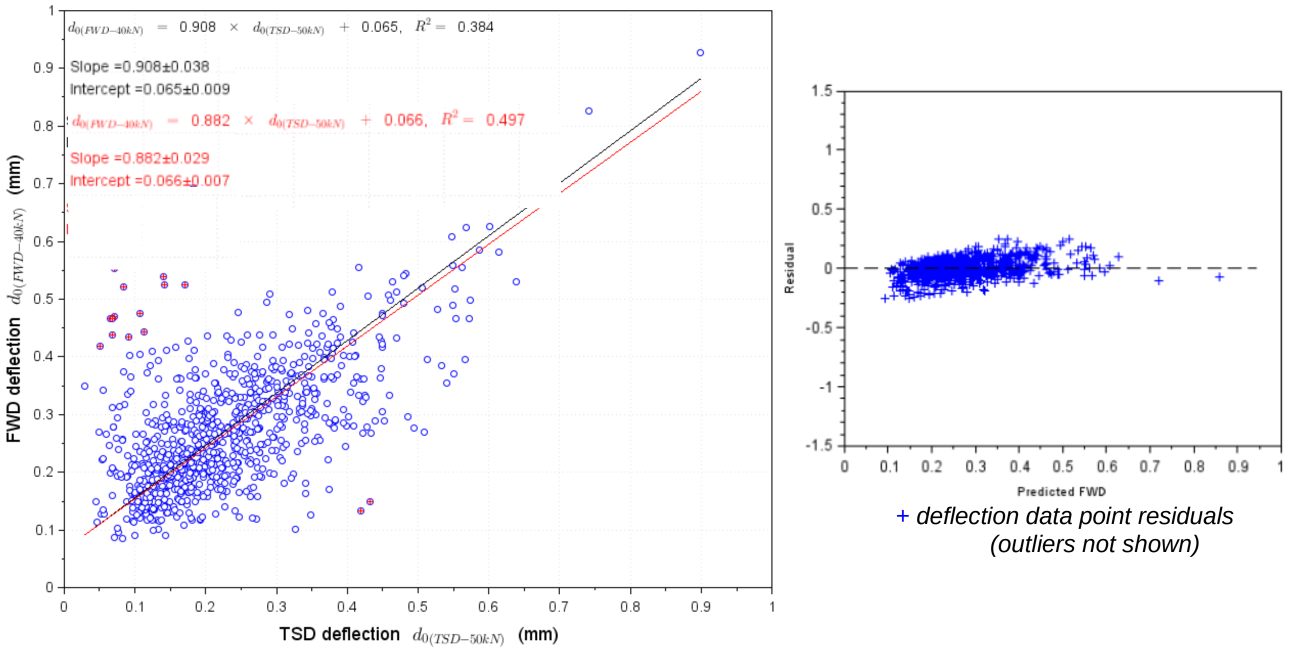
Figure E.12 Trial Mile Road, right lane – preliminary regression analysis of curvature (D0 – D200)



+ *curvature data point residuals*

Where: o FWD vs TSD curvature data – regression line on all data points

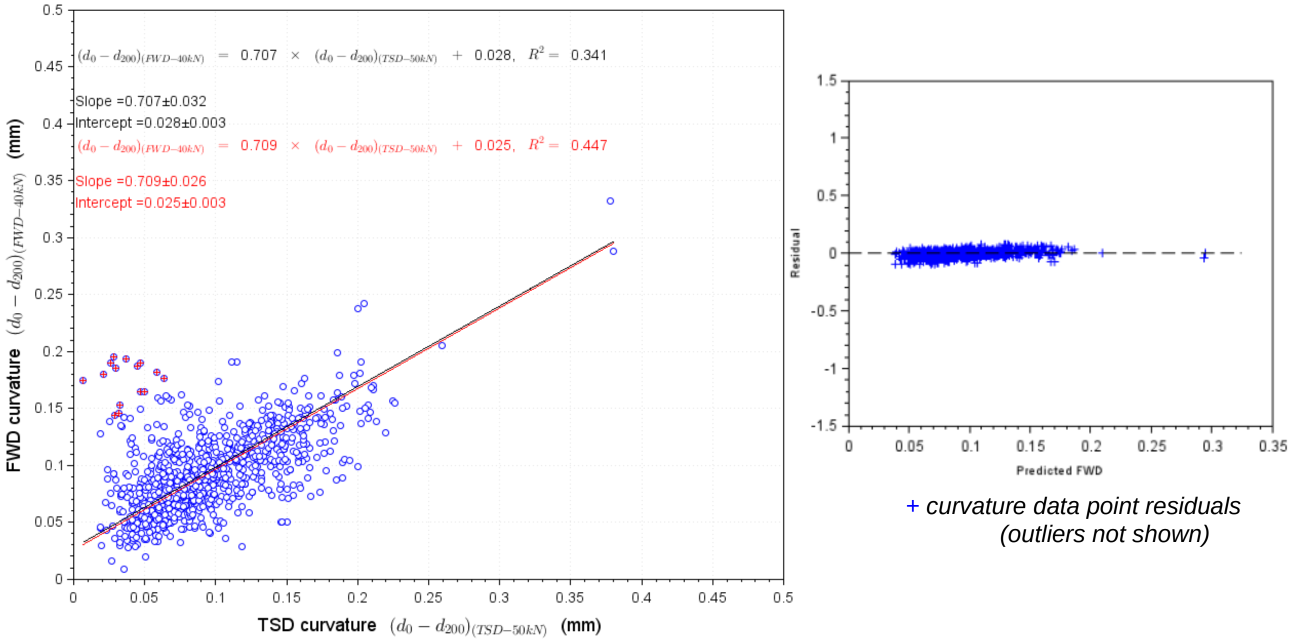
Figure E.13 Great Eastern Highway, left lane – preliminary regression analysis of D_0



\+ deflection data point residuals
(outliers not shown)

Where: *o FWD vs TSD deflection data* *+ Identified outlier*
– regression line on all data points *– Regression line on data points, excluding outliers*

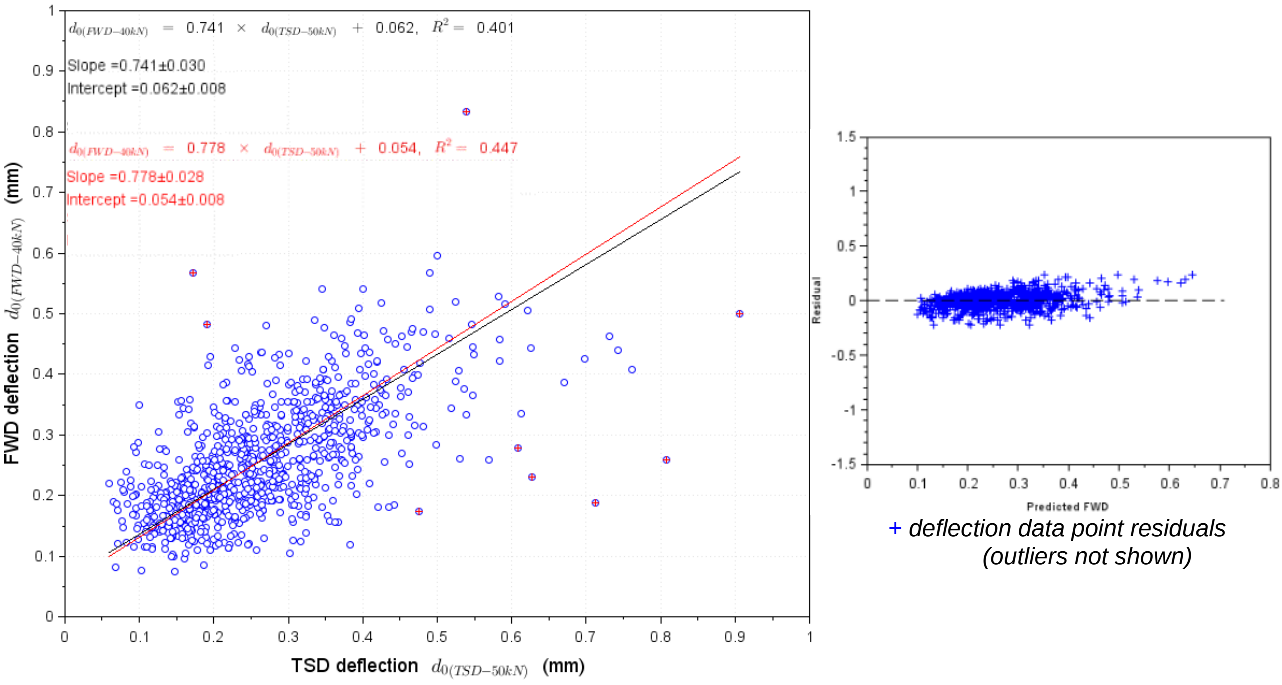
Figure E.14 Great Eastern Highway, left lane – preliminary regression analysis of curvature (D_0 – D_{200})



\+ curvature data point residuals
(outliers not shown)

Where: *o FWD vs TSD curvature data* *+ Identified outlier*
– regression line on all data points *– Regression line on data points, excluding outliers*

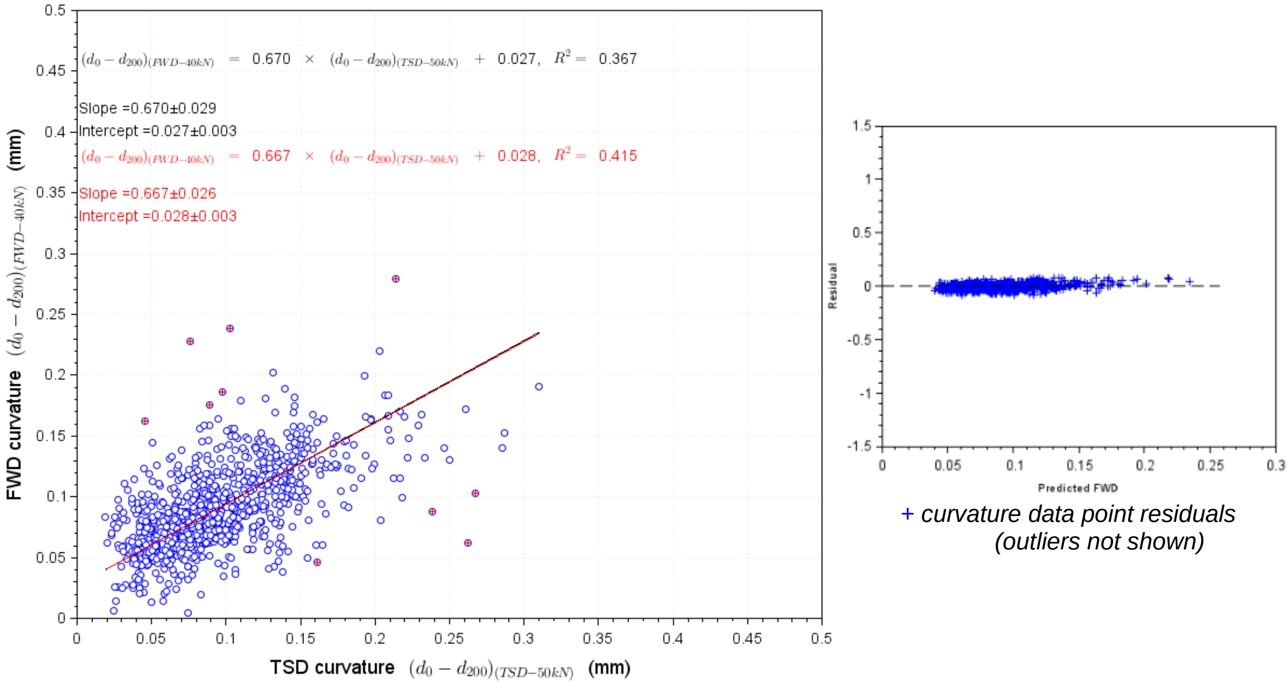
Figure E.15 Great Eastern Highway, right lane – preliminary regression analysis of D_0



+ *deflection data point residuals*
(outliers not shown)

Where: *o FWD vs TSD deflection data* *+ Identified outlier*
– regression line on all data points *– Regression line on data points, excluding outliers*

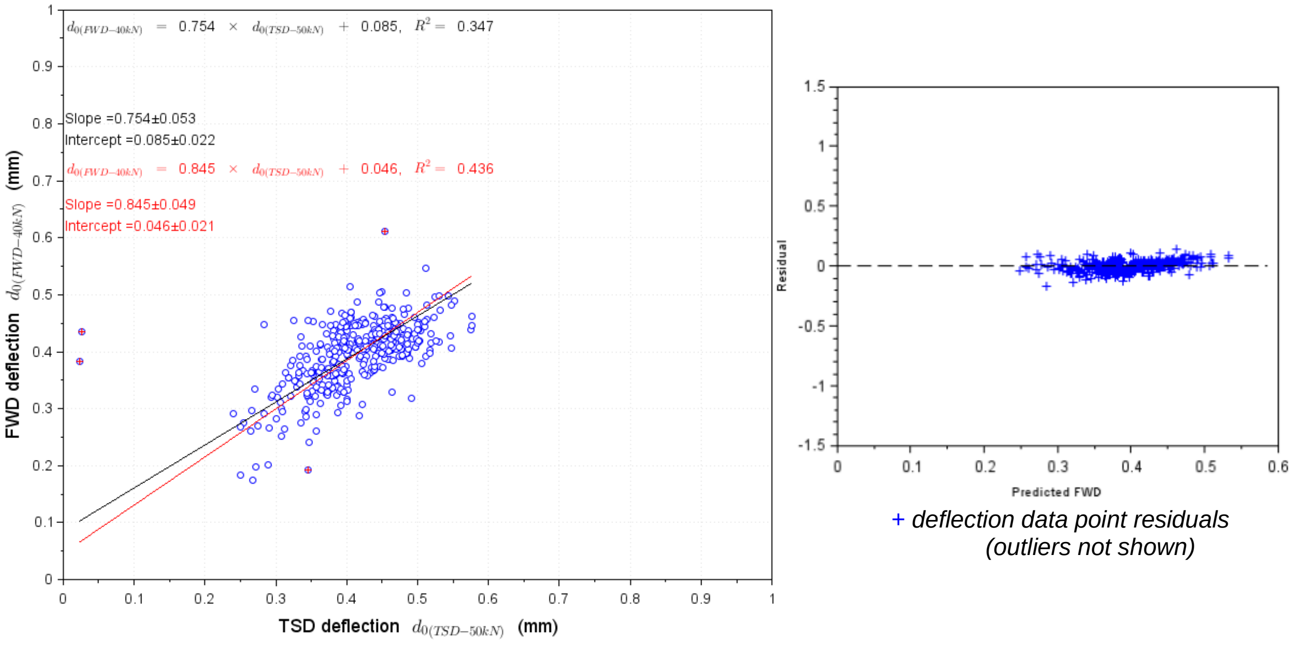
Figure E.16 Great Eastern Highway, right lane – preliminary regression analysis of curvature (D_0 – D_{200})



+ *curvature data point residuals*
(outliers not shown)

Where: *o FWD vs TSD curvature data* *+ Identified outlier*
– regression line on all data points *– Regression line on data points, excluding outliers*

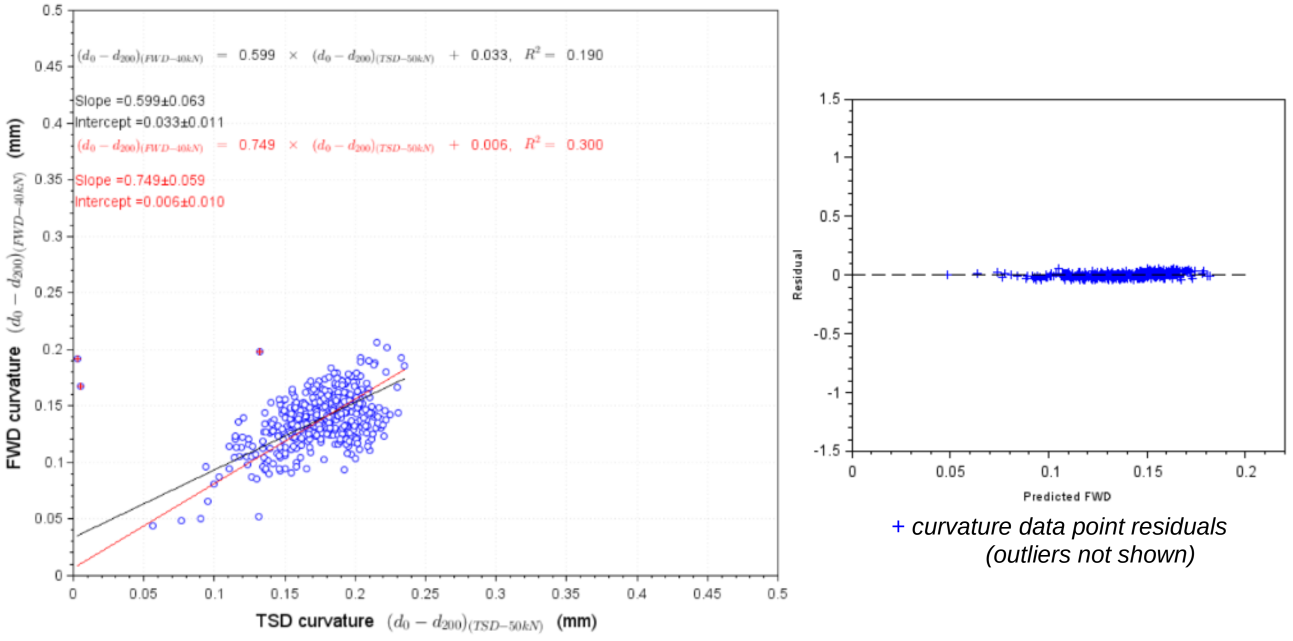
Figure E.17 Tonkin Highway extension, northbound, left lane – preliminary regression analysis of D_0



+ deflection data point residuals (outliers not shown)

Where: o FWD vs TSD deflection data
– regression line on all data points
+ Identified outlier
– Regression line on data points, excluding outliers

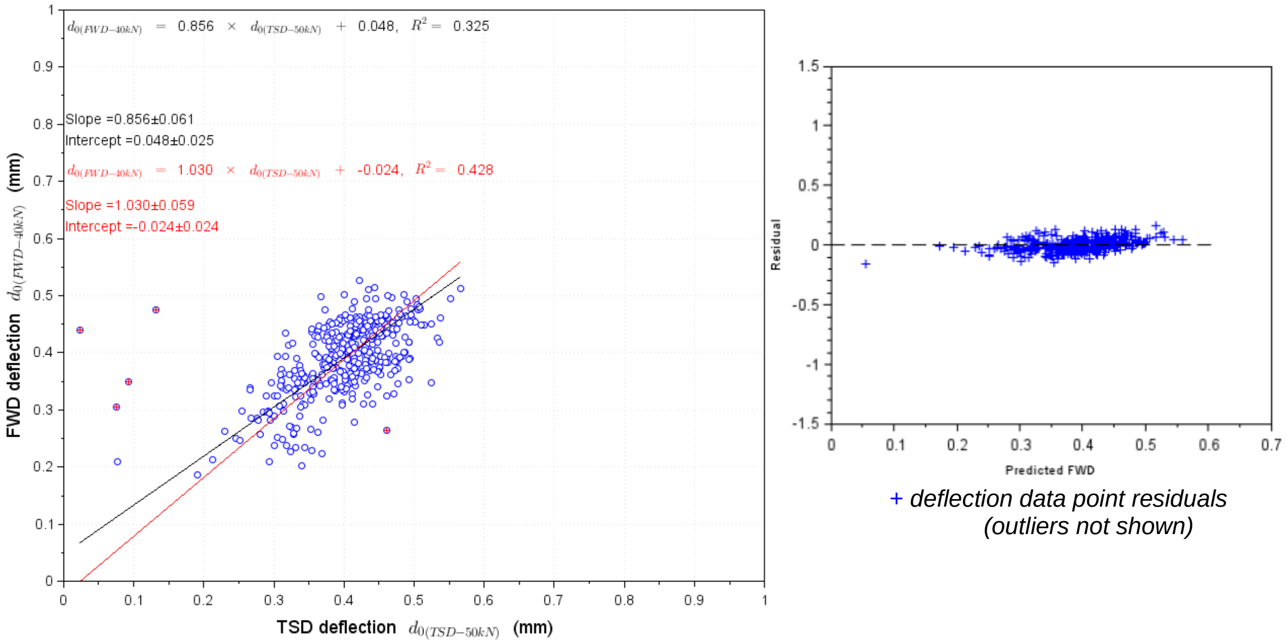
Figure E.18 Tonkin Highway extension, northbound, left lane – preliminary regression analysis of curvature (D_0 – D_{200})



+ curvature data point residuals (outliers not shown)

Where: o FWD vs TSD curvature data
– regression line on all data points
+ Identified outlier
– Regression line on data points, excluding outliers

Figure E.19 Tonkin Highway extension, southbound, left lane – preliminary regression analysis of D_0



+ deflection data point residuals (outliers not shown)

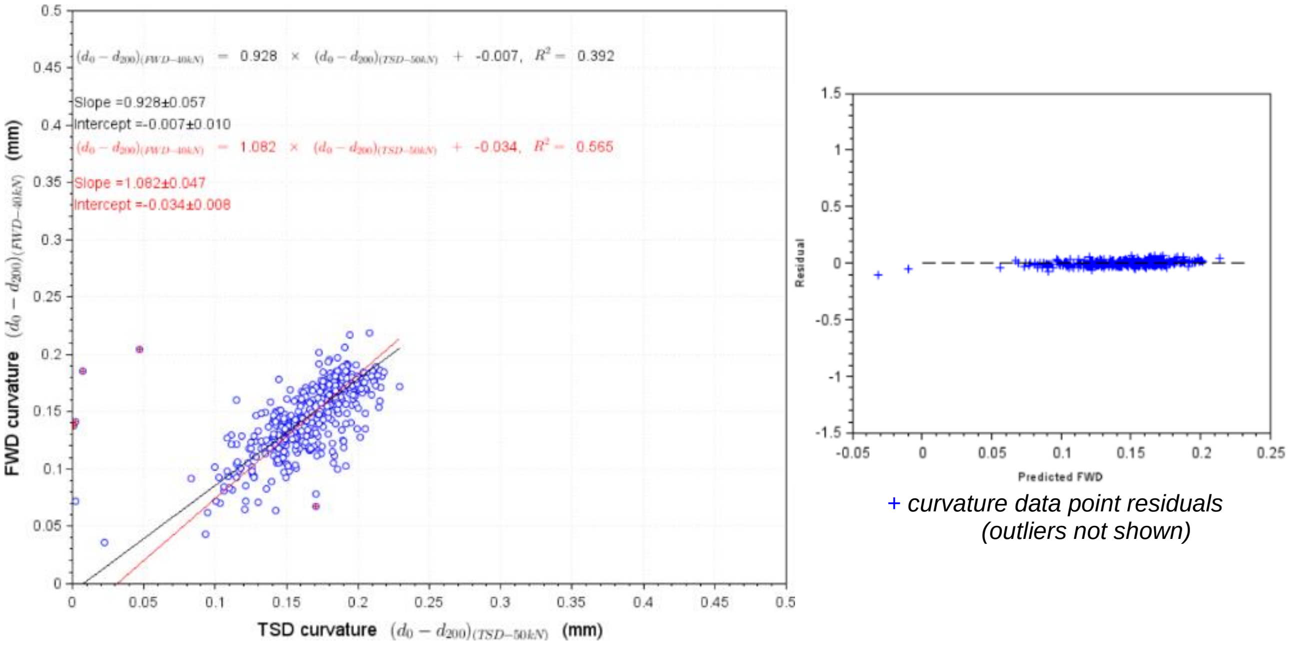
Where: o FWD vs TSD deflection data

– regression line on all data points

+ Identified outlier

– Regression line on data points, excluding outliers

Figure E.20 Tonkin Highway extension, southbound, left lane – preliminary regression analysis of curvature (D_0 – D_{200})



+ curvature data point residuals (outliers not shown)

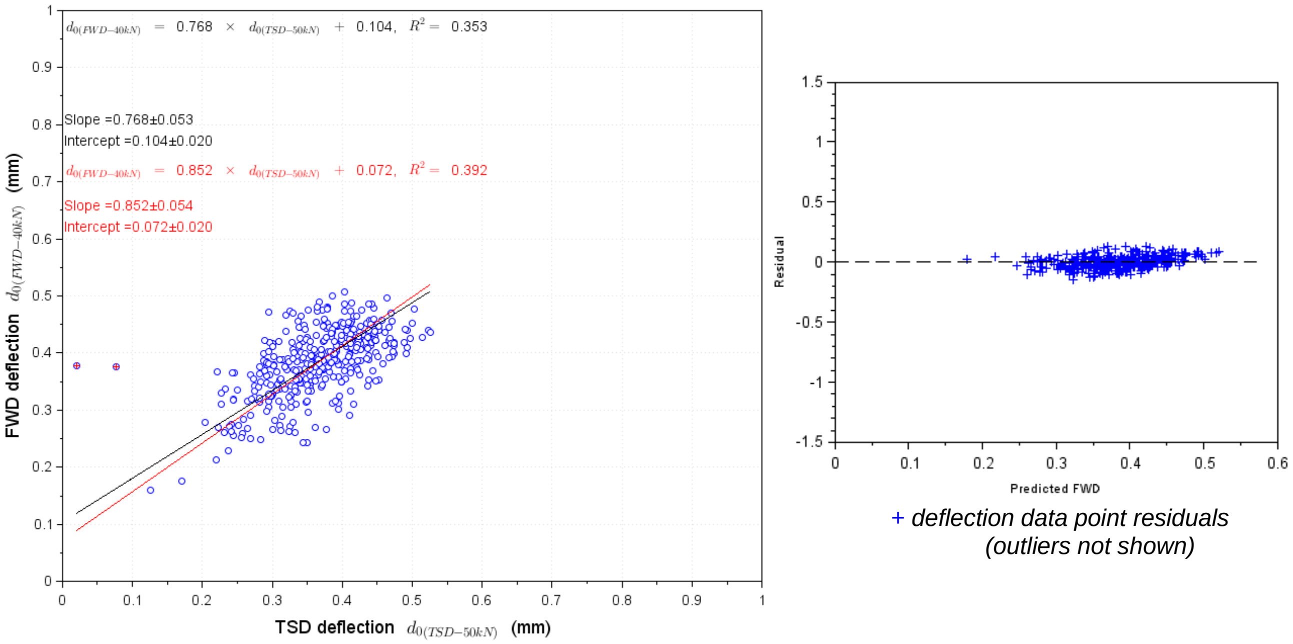
Where: o FWD vs TSD curvature data

– regression line on all data points

+ Identified outlier

– Regression line on data points, excluding outliers

Figure E.21 Tonkin Highway extension, northbound, right lane – preliminary regression analysis of D_0

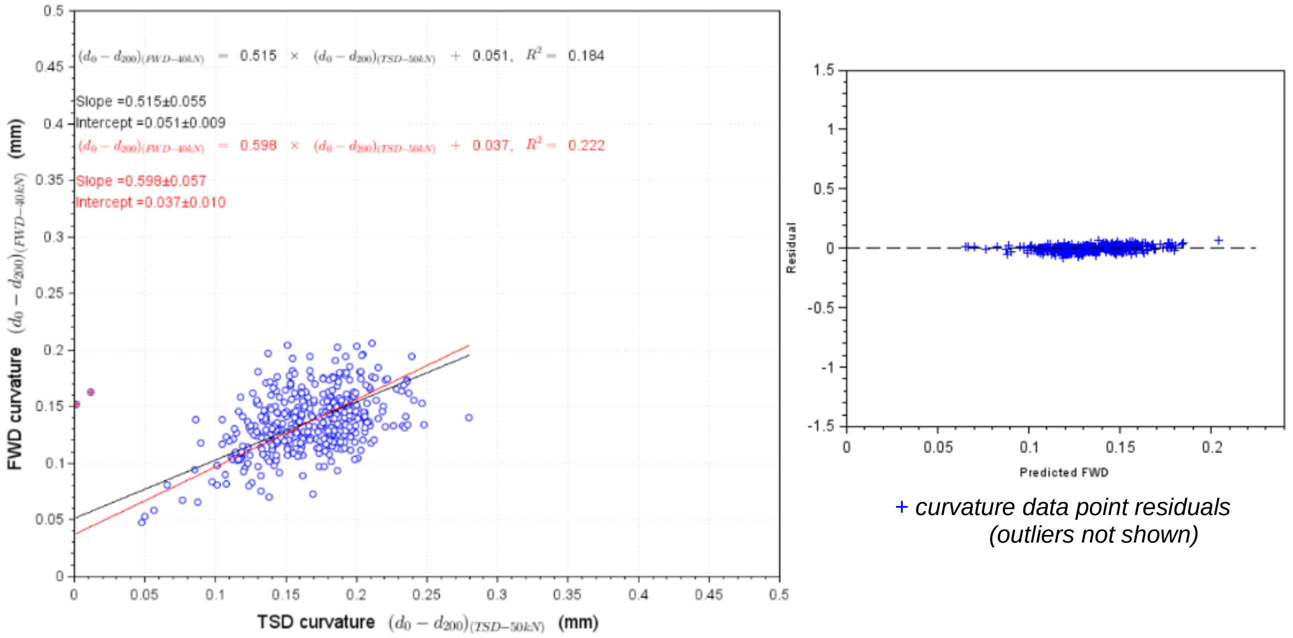


+ deflection data point residuals (outliers not shown)

Where: o FWD vs TSD deflection data *+ Identified outlier*

– regression line on all data points *– Regression line on data points, excluding outliers*

Figure E.22 Tonkin Highway extension, northbound, right lane – preliminary regression analysis of curvature (D_0 – D_{200})

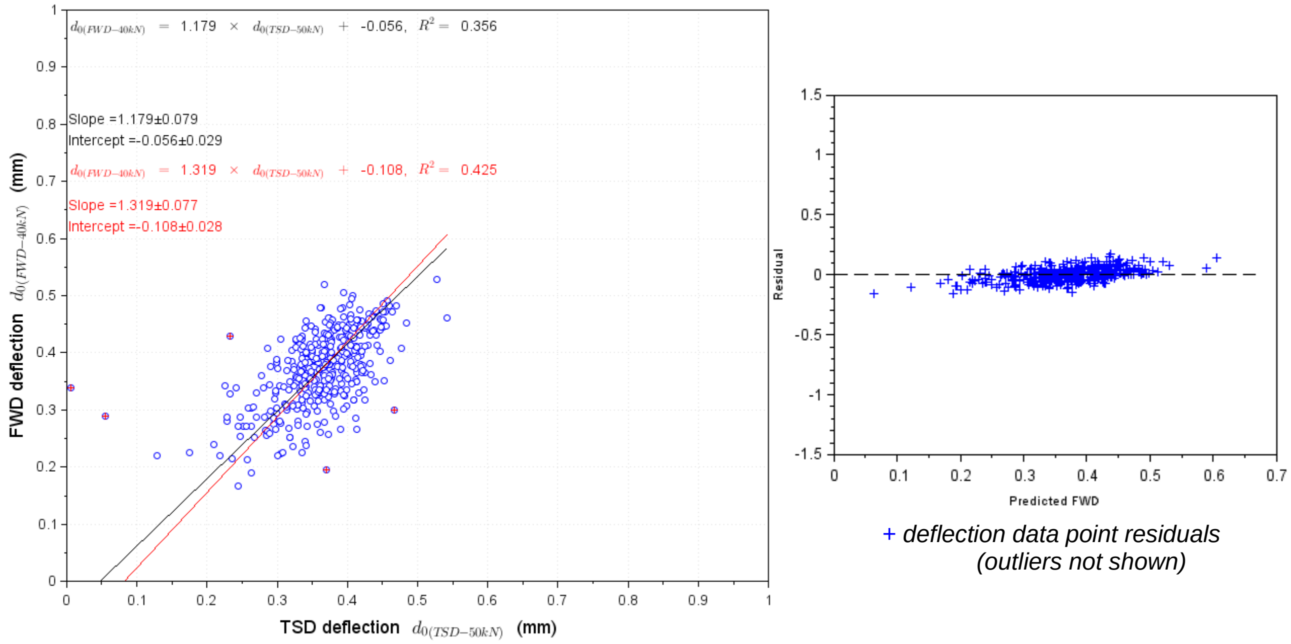


+ curvature data point residuals (outliers not shown)

Where: o FWD vs TSD curvature data *+ Identified outlier*

– regression line on all data points *– Regression line on data points, excluding outliers*

Figure E.23 Tonkin Highway extension, southbound, right lane – preliminary regression analysis of D_0

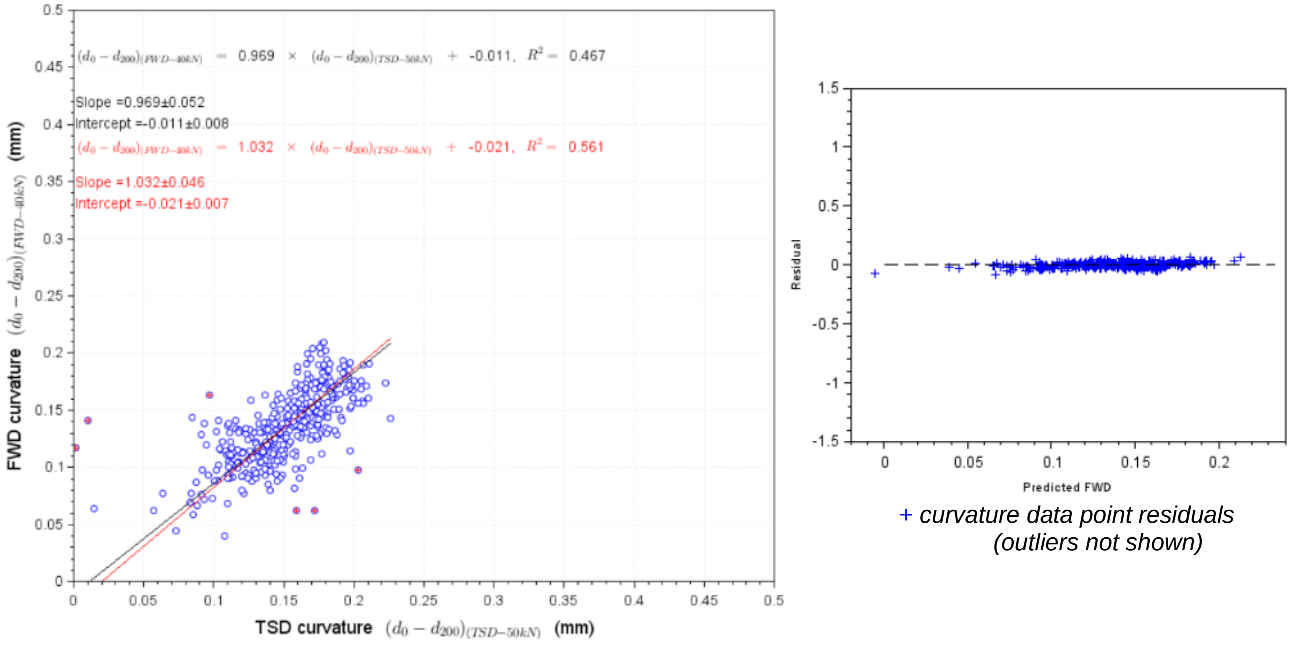


+ deflection data point residuals
(outliers not shown)

Where: o FWD vs TSD deflection data *+ Identified outlier*

– regression line on all data points *– Regression line on data points, excluding outliers*

Figure E.24 Tonkin Highway extension, southbound, right lane - preliminary regression analysis of curvature (D_0 – D_{200})

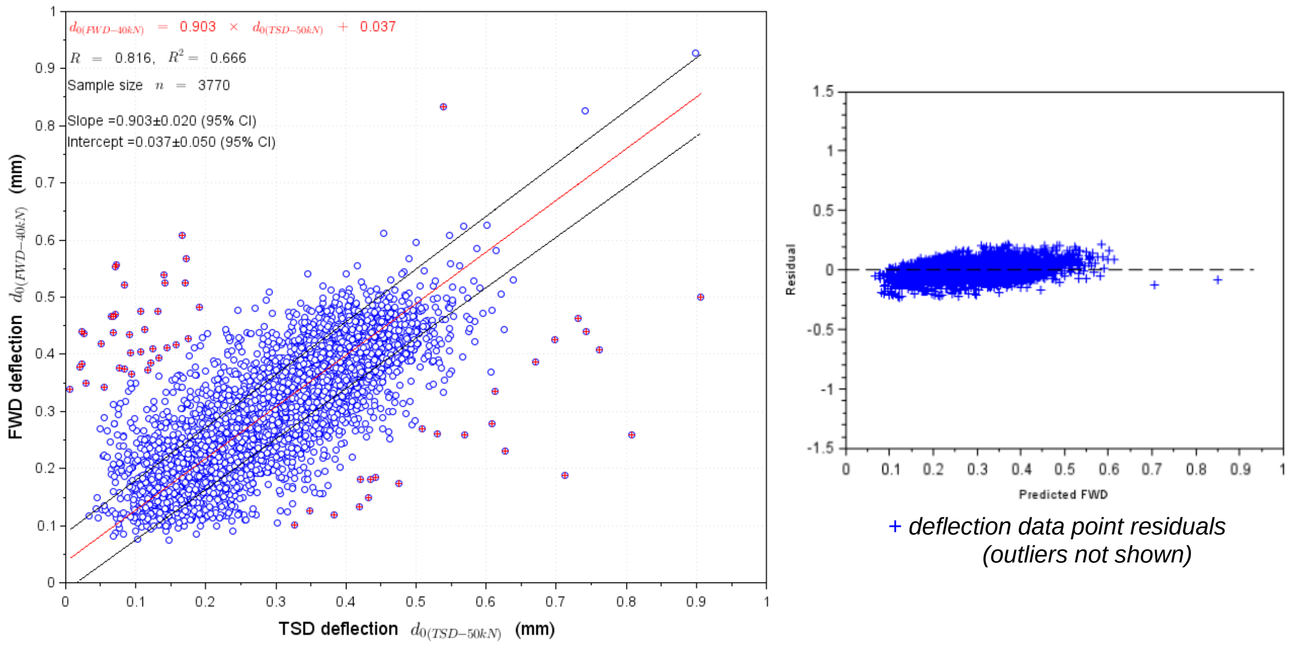


+ curvature data point residuals
(outliers not shown)

Where: o FWD vs TSD curvature data *+ Identified outlier*

– regression line on all data points *– Regression line on data points, excluding outliers*

Figure E.25 All pavements combined – preliminary regression analysis of D_0

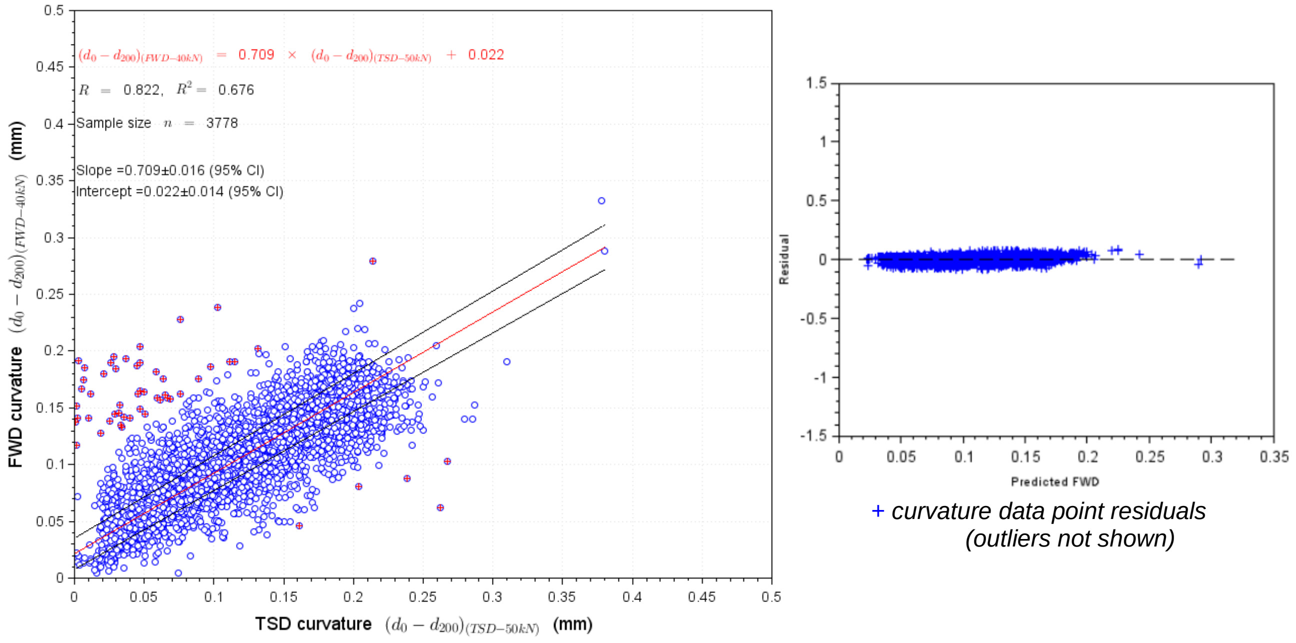


+ *deflection data point residuals (outliers not shown)*

Where: o FWD vs TSD deflection data *+ Identified outlier*

– regression line on all data points *– Regression line on data points, excluding outliers*

Figure E.26 All pavements combined – preliminary regression analysis of curvature ($D_0 – D_{200}$)



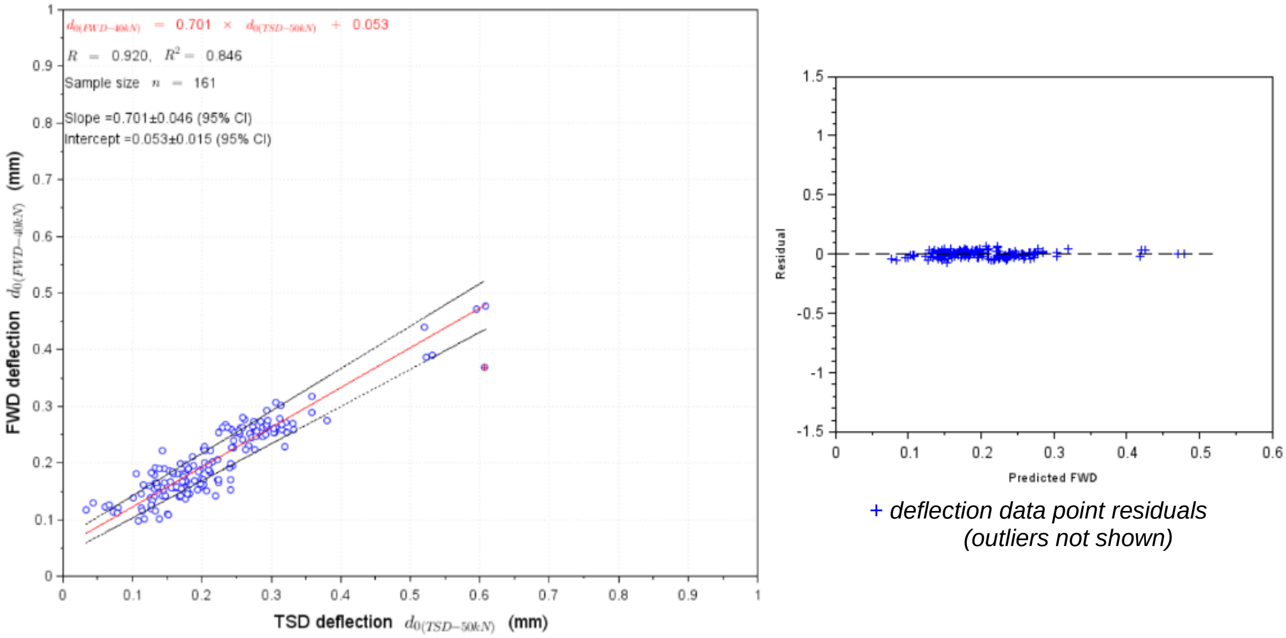
+ *curvature data point residuals (outliers not shown)*

Where: o FWD vs TSD curvature data *+ Identified outlier*

– regression line on all data points *– Regression line on data points, excluding outliers*

APPENDIX F REFINED REGRESSION ANALYSIS RESULTS

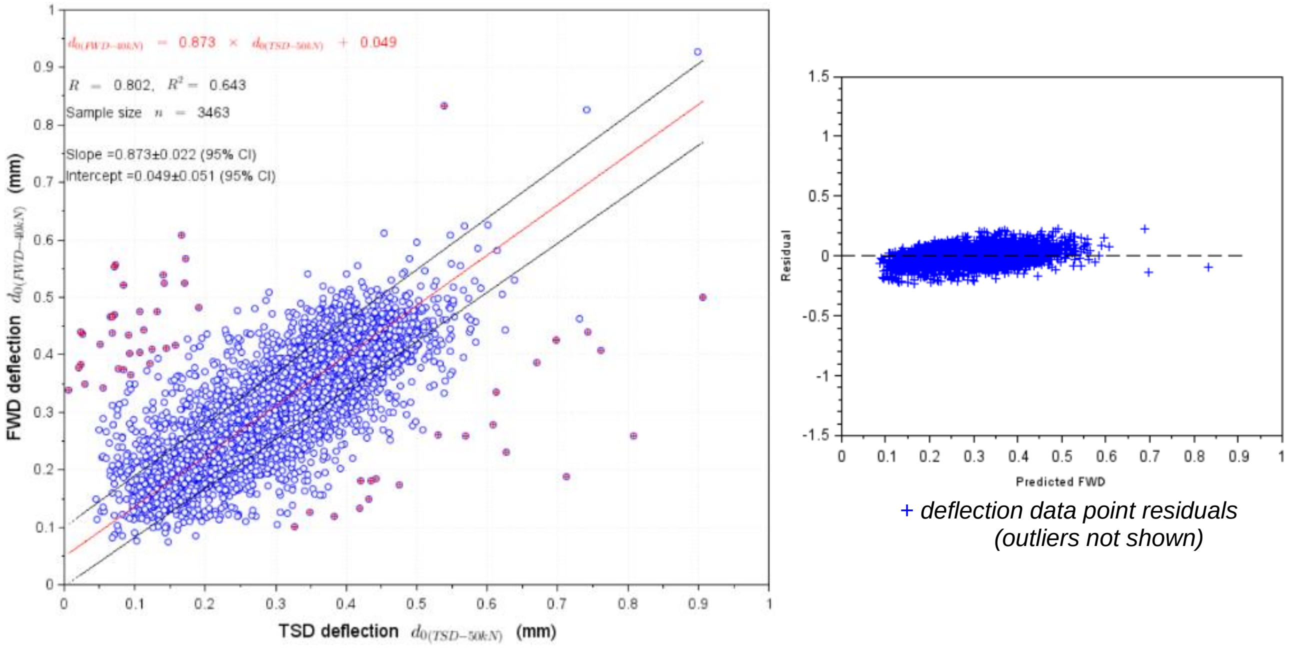
Figure F.1 Asphalt pavements – refined and unconstrained regression results for D_0



+ deflection data point residuals (outliers not shown)

Where: o FWD vs TSD deflection data
– regression line on all data points
+ Identified outlier
– Regression line on data points, excluding outliers

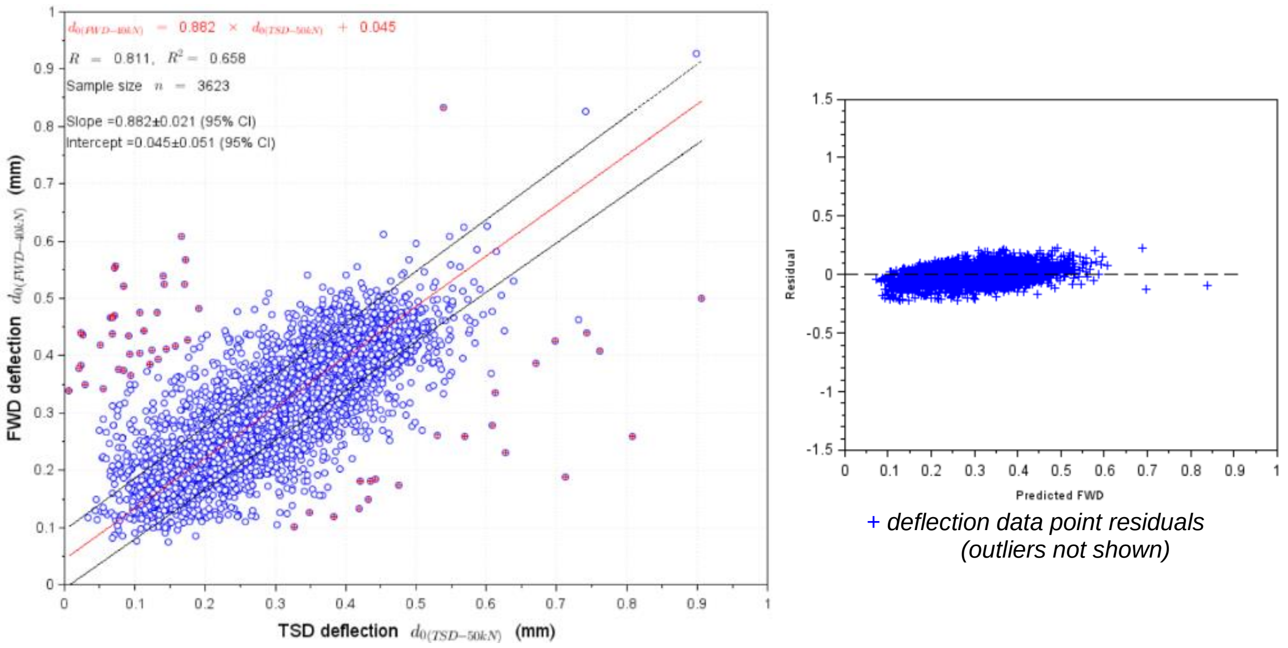
Figure F.2 Granular pavements – refined and unconstrained regression results for D_0



+ deflection data point residuals (outliers not shown)

Where: o FWD vs TSD deflection data
– regression line on all data points
+ Identified outlier
– Regression line on data points, excluding outliers

Figure F.3 Combined pavement types – refined and unconstrained regression results for D_0

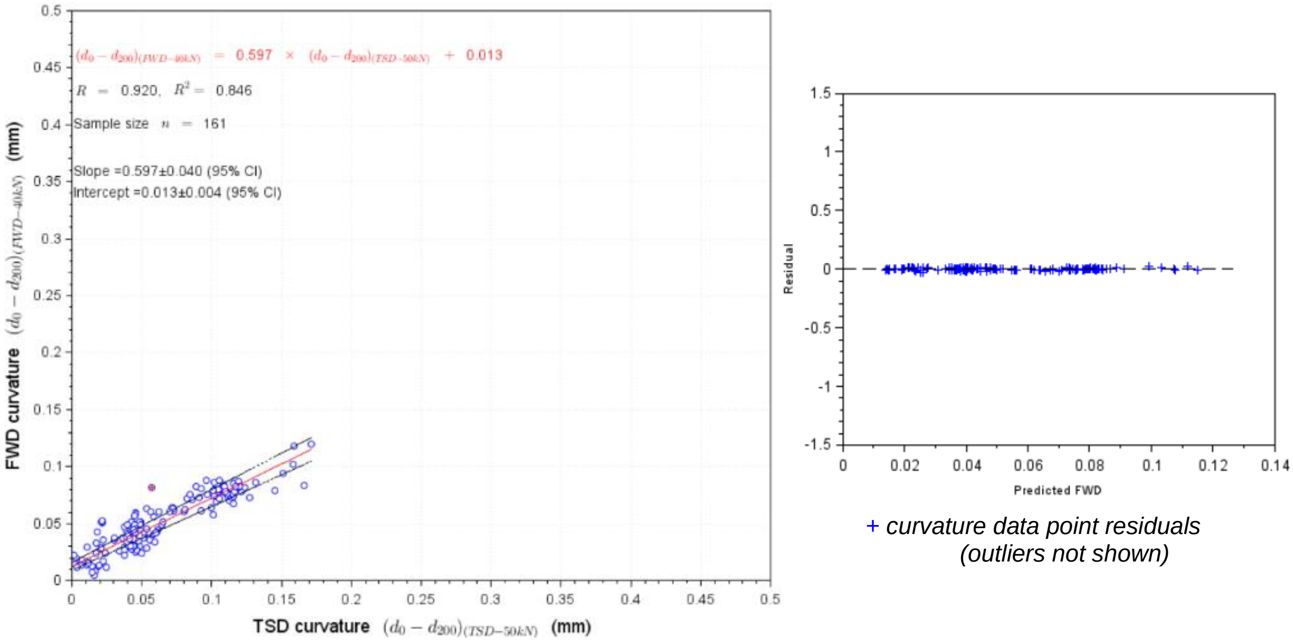


+ deflection data point residuals
(outliers not shown)

Where: o FWD vs TSD deflection data
– regression line on all data points

+ Identified outlier
– Regression line on data points, excluding outliers

Figure F.4 Asphalt pavements – refined and unconstrained regression results for curvature ($D_0 – D_{200}$)

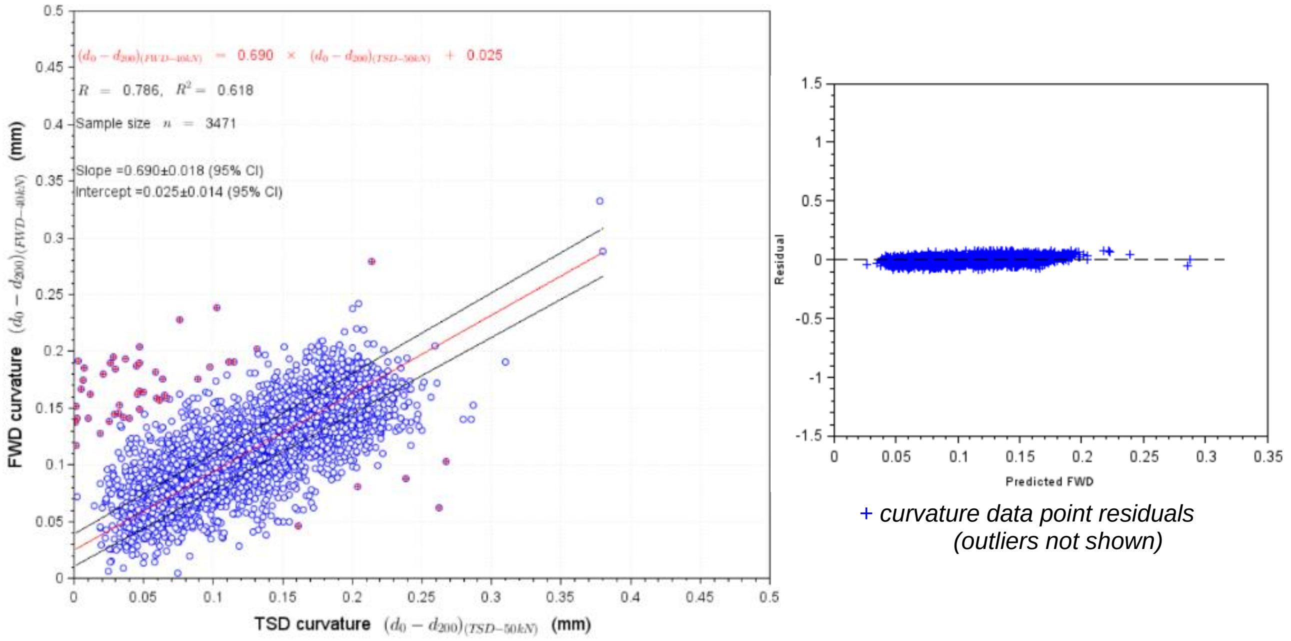


+ curvature data point residuals
(outliers not shown)

Where: o FWD vs TSD curvature data
– regression line on all data points

+ Identified outlier
– Regression line on data points, excluding outliers

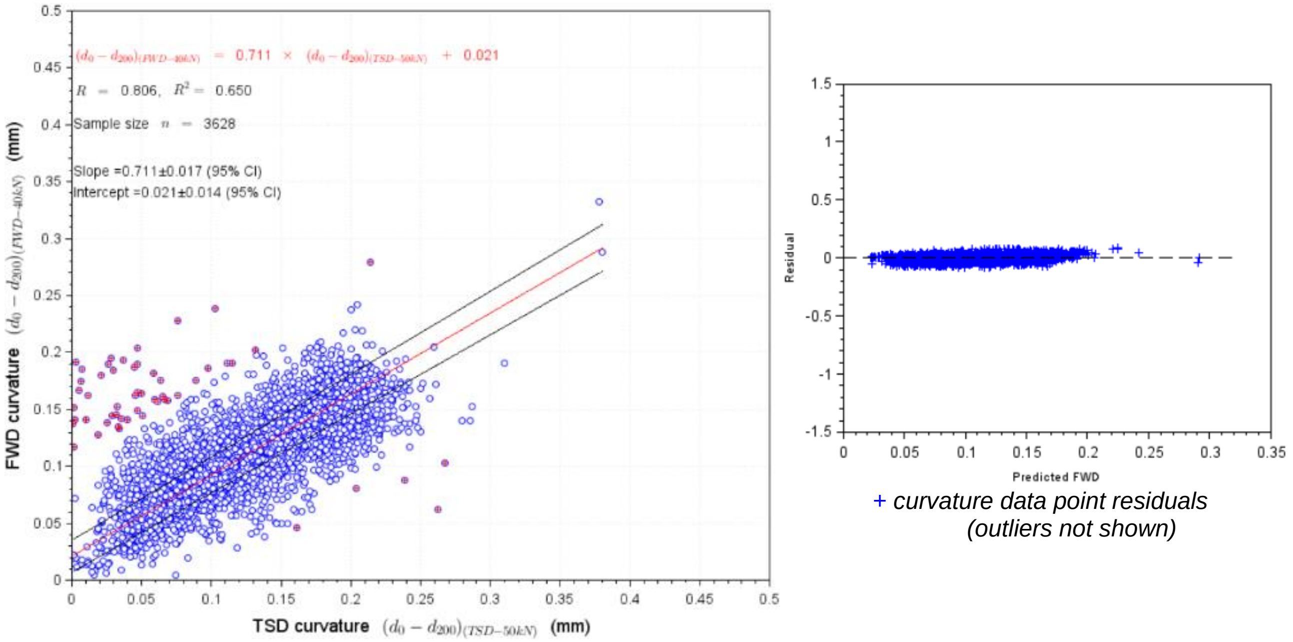
Figure F.5 Granular pavements – refined and unconstrained regression results for curvature ($D_0 - D_{200}$)



+ curvature data point residuals (outliers not shown)

Where: o FWD vs TSD curvature data *+ Identified outlier*
– regression line on all data points *– Regression line on data points, excluding outliers*

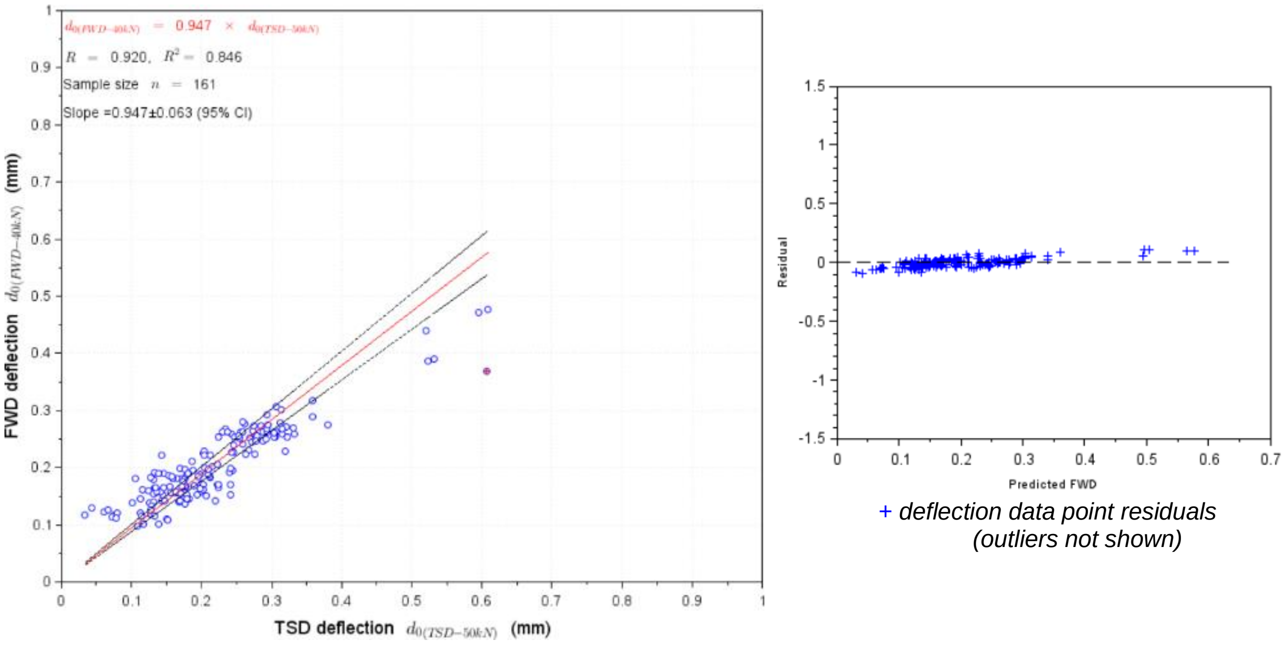
Figure F.6 Combined pavement types – refined and unconstrained regression results for curvature ($D_0 - D_{200}$)



+ curvature data point residuals (outliers not shown)

Where: o FWD vs TSD curvature data *+ Identified outlier*
– regression line on all data points *– Regression line on data points, excluding outliers*

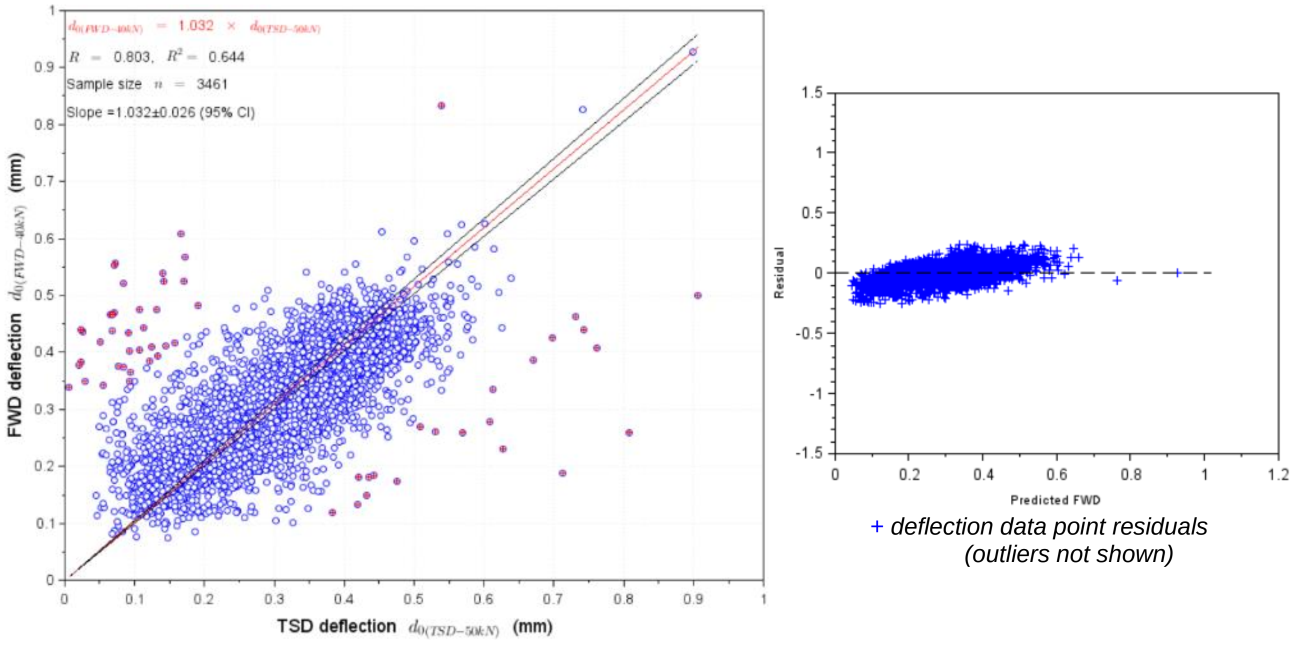
Figure F.7 Asphalt pavements – refined and constrained regression results for D0



+ deflection data point residuals (outliers not shown)

Where: o FWD vs TSD deflection data
– regression line on all data points
+ Identified outlier
– Regression line on data points, excluding outliers

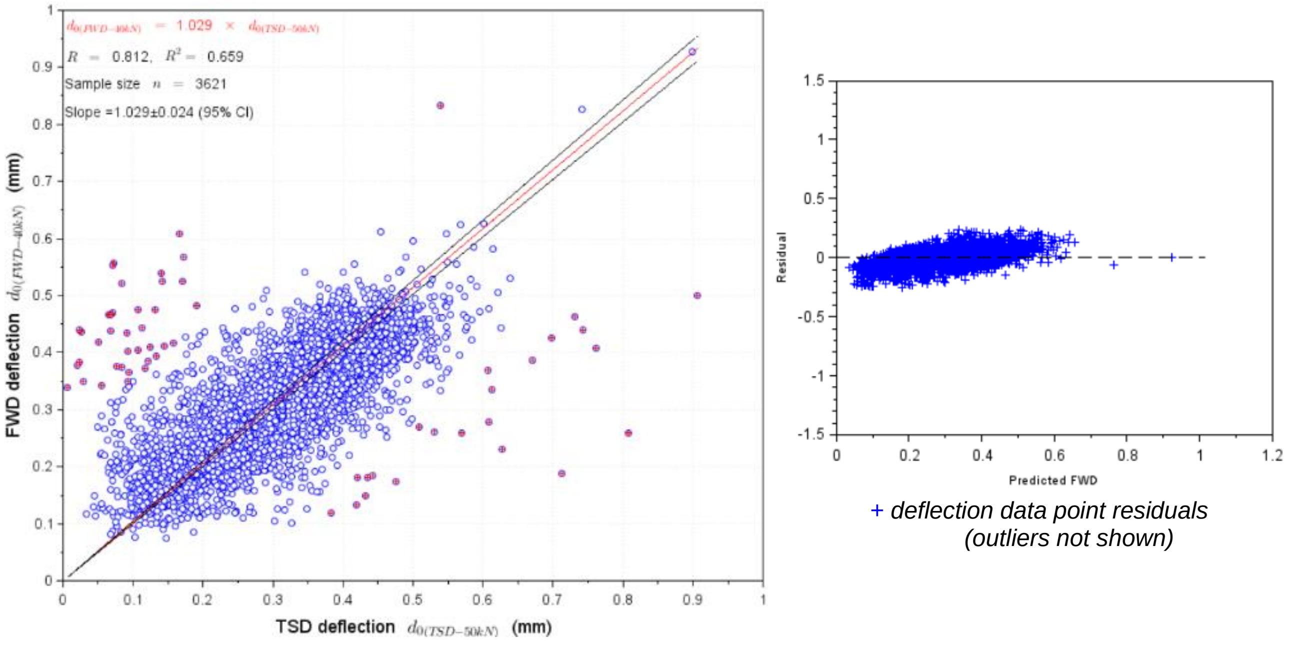
Figure F.8 Granular pavements – refined and constrained regression results for D0



+ deflection data point residuals (outliers not shown)

Where: o FWD vs TSD deflection data
– regression line on all data points
+ Identified outlier
– Regression line on data points, excluding outliers

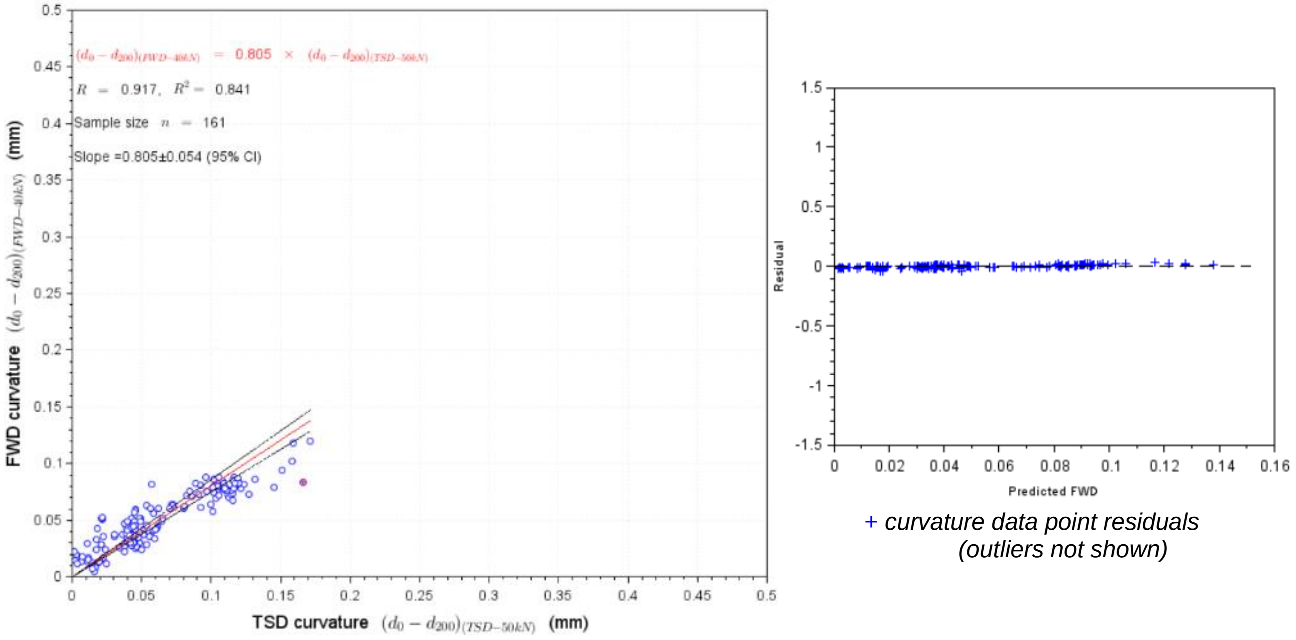
Figure F.9 Combined pavement types – refined and constrained regression results for D_0



+ deflection data point residuals (outliers not shown)

Where: o FWD vs TSD deflection data
– regression line on all data points
+ Identified outlier
– Regression line on data points, excluding outliers

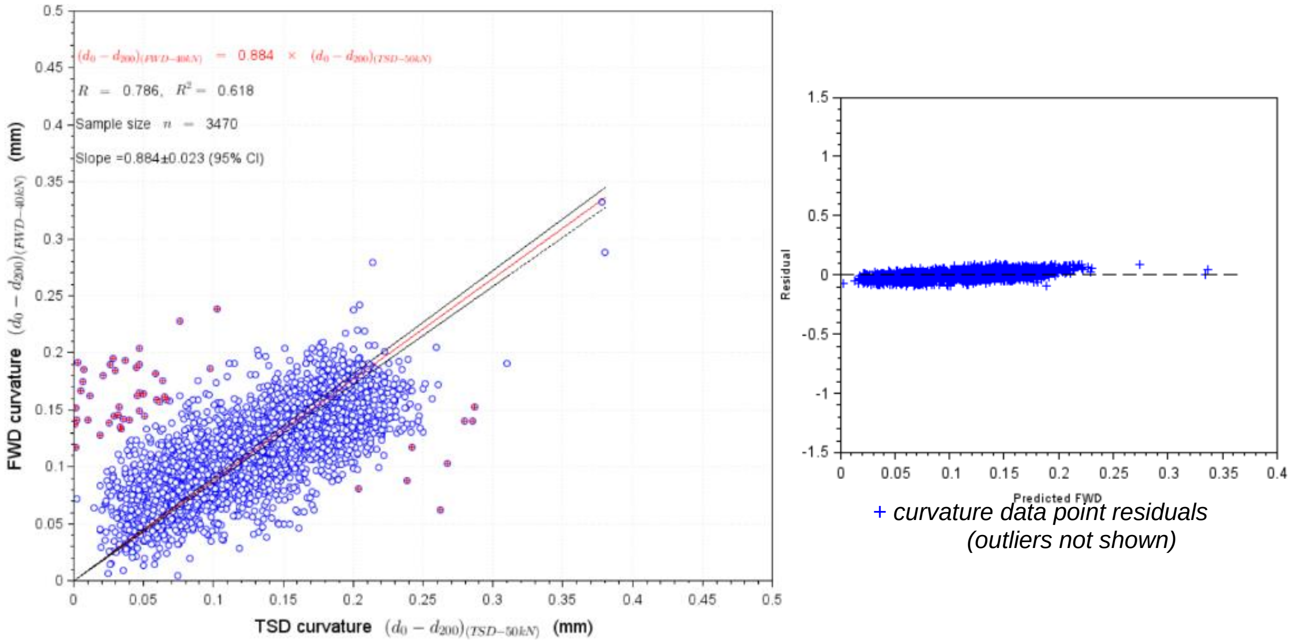
Figure F.10 Asphalt pavements – refined and constrained regression results for curvature ($D_0 – D_{200}$)



+ curvature data point residuals (outliers not shown)

Where: o FWD vs TSD curvature data
– regression line on all data points
+ Identified outlier
– Regression line on data points, excluding outliers

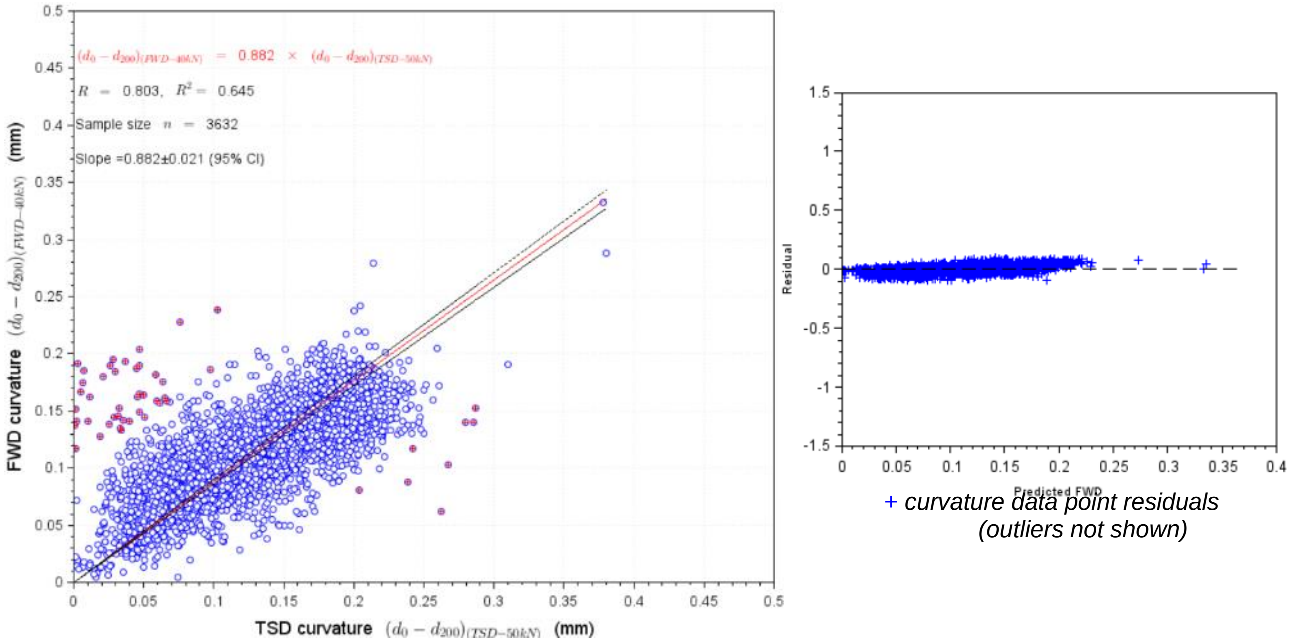
Figure F.11 Granular pavements – refined and constrained regression results for curvature (D_0 – D_{200})



+ curvature data point residuals (outliers not shown)

Where: o FWD vs TSD curvature data *+ Identified outlier*
– regression line on all data points *– Regression line on data points, excluding outliers*

Figure F.12 Combined pavement types – refined and constrained regression results for curvature (D_0 – D_{200})



+ curvature data point residuals (outliers not shown)

Where: o FWD vs TSD curvature data *+ Identified outlier*
– regression line on all data points *– Regression line on data points, excluding outliers*

APPENDIX G AUSTROADS INDIVIDUAL ROAD DATASETS FOR COMPARISON

Figure G.1 Austroads dataset for road 910, maximum deflections (D_0) – granular pavement

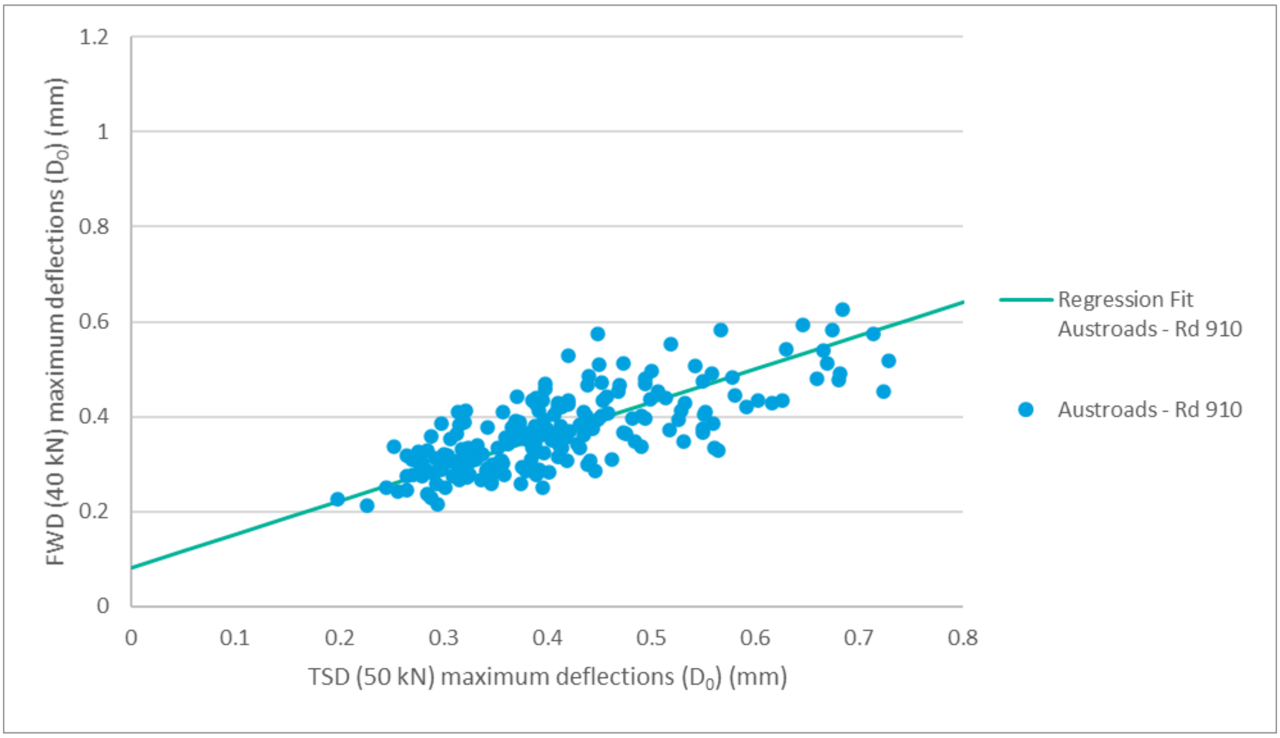


Figure G.2 Austroads dataset for road 910, curvature (D_0 – D_{200}) – granular pavement

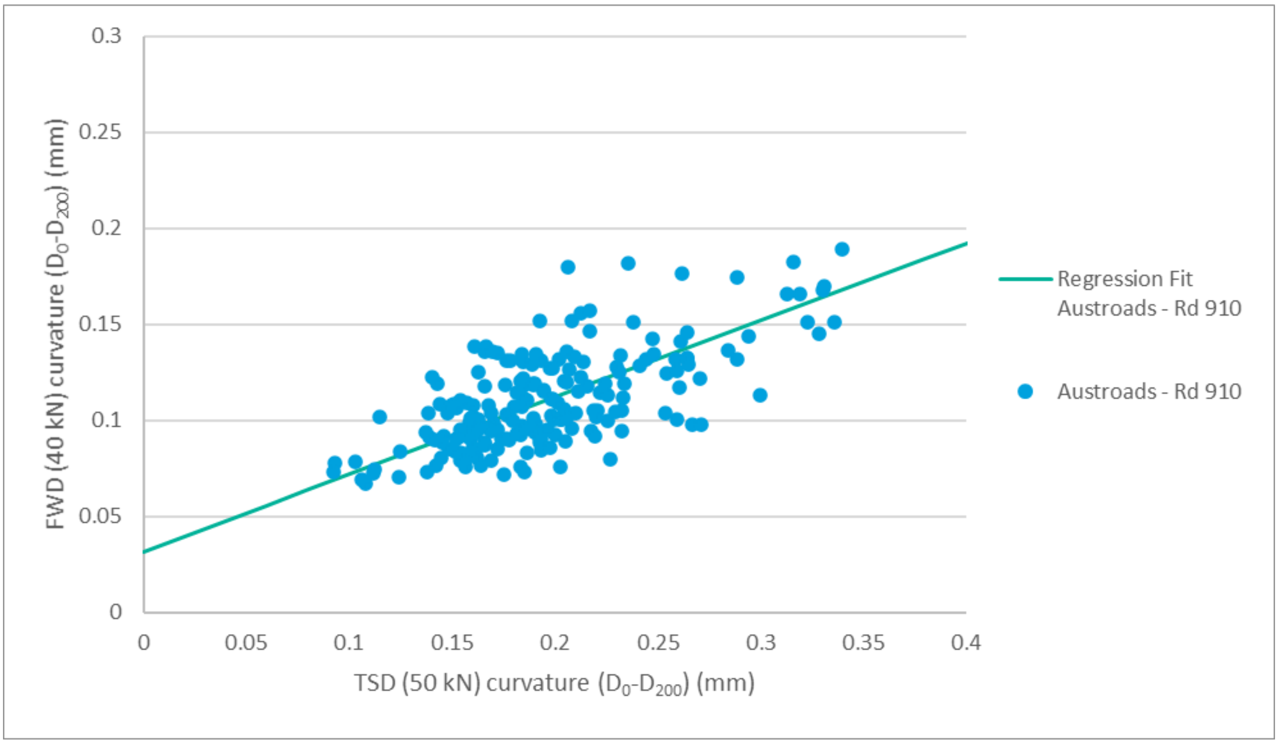


Figure G.3 Austroads dataset for road 374, maximum deflections (D_0) – granular pavement (*weak correlation, regression not used in Austroads vs. WA comparison*)

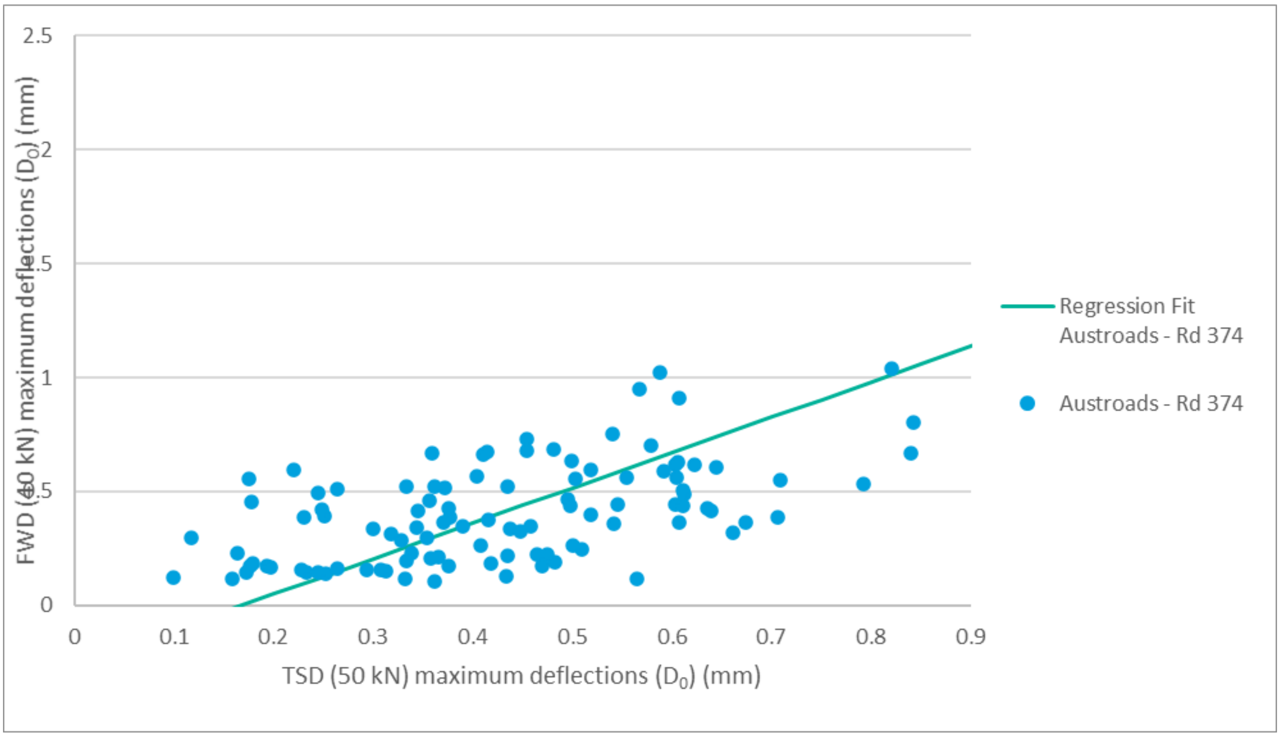


Figure G.4 Austroads dataset for road 374, curvature ($D_0 – D_{200}$) – granular pavement

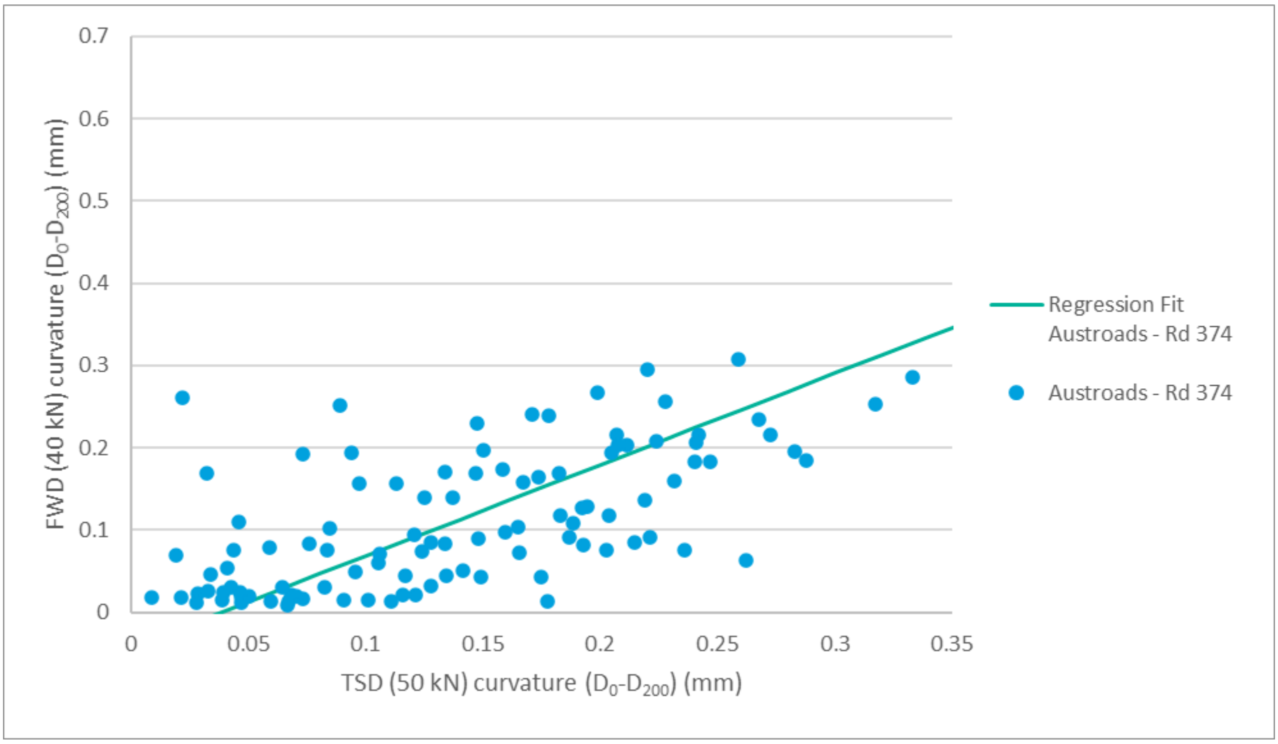


Figure G.5 Austroads dataset for road 384, maximum deflections (D_0) – granular pavement

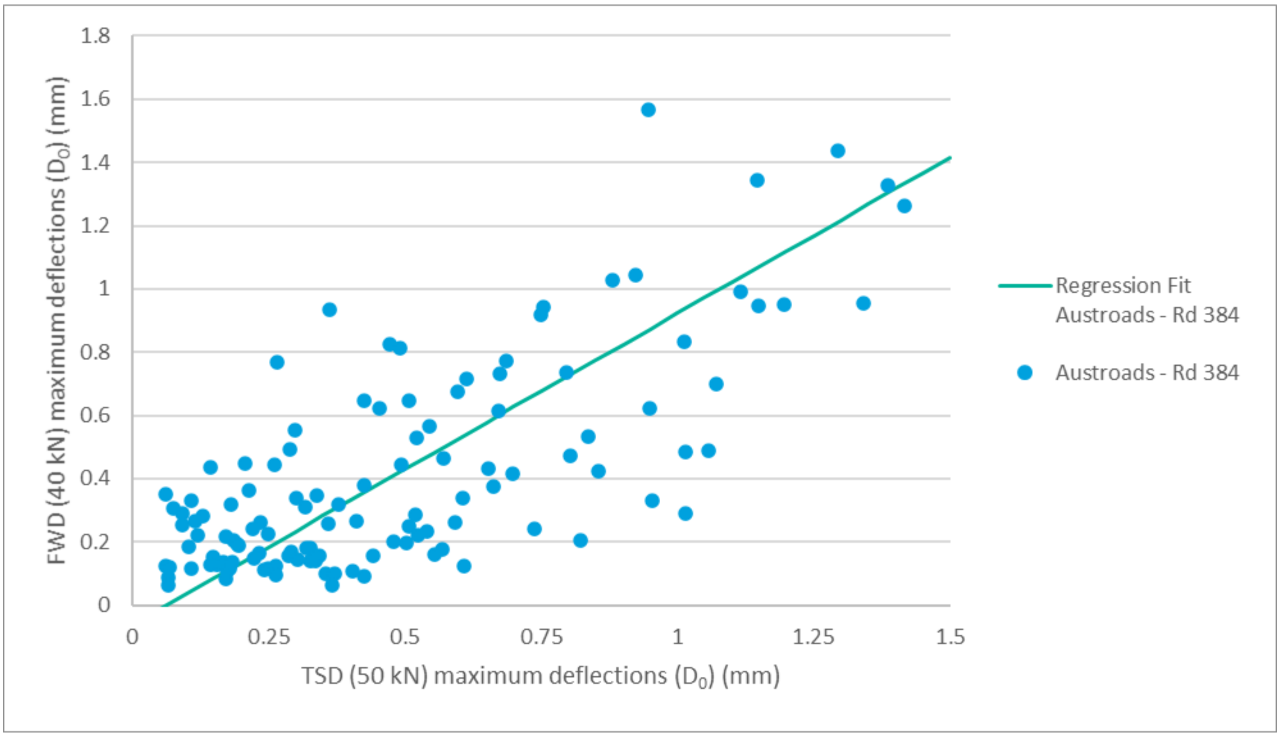


Figure G.6 Austroads dataset for road 384, curvature (D_0 – D_{200}) – granular pavement

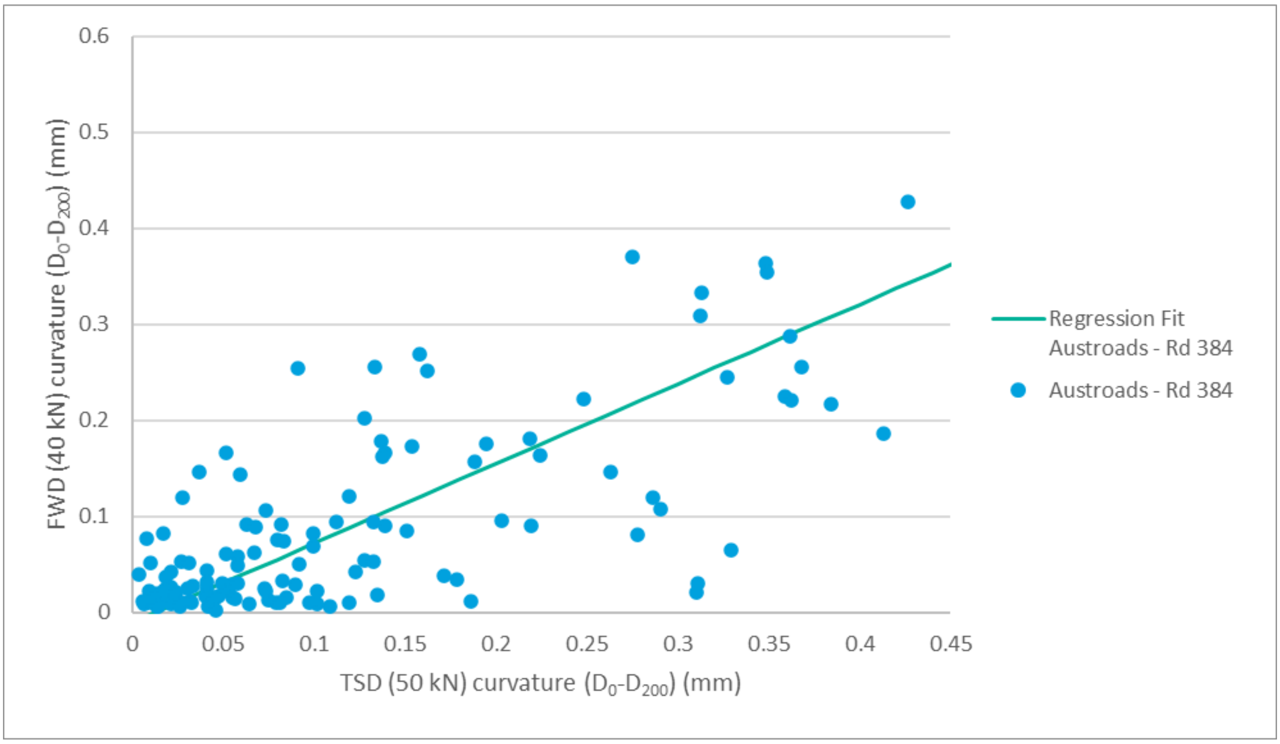


Figure G.7 Austroads dataset for road 386, maximum deflections (D_0) – granular pavement

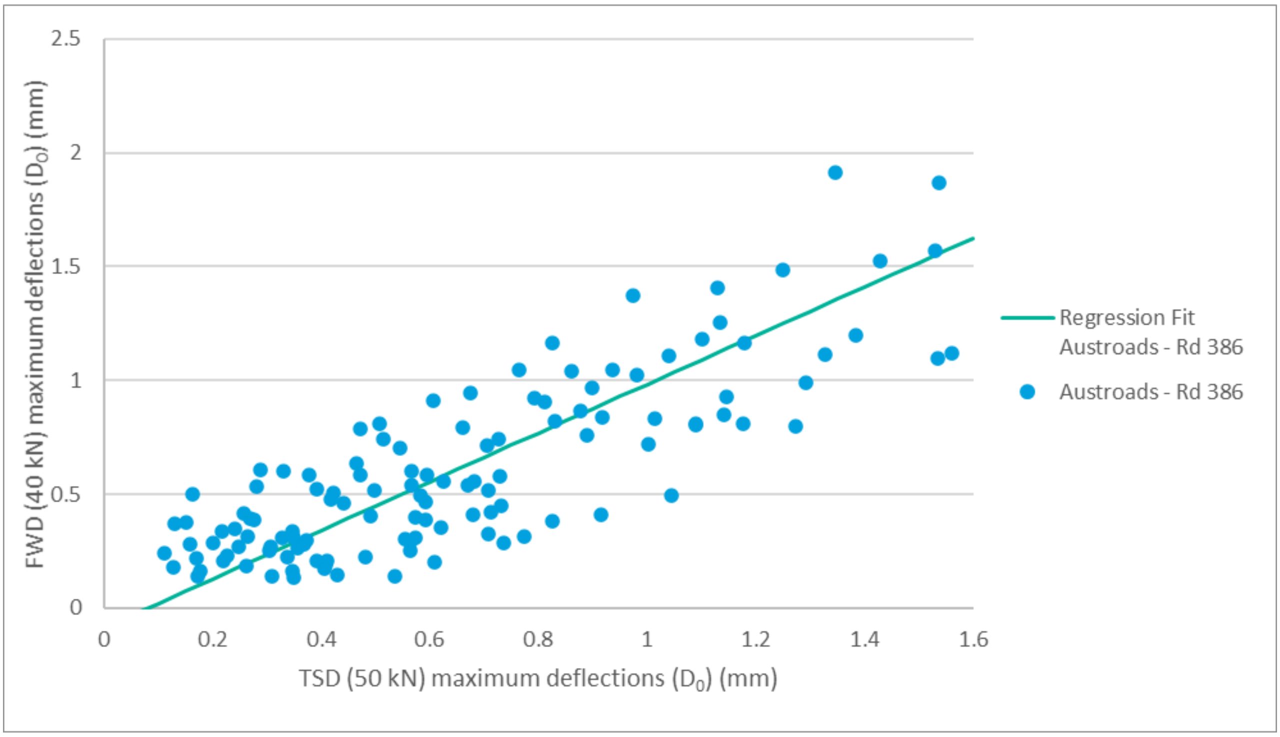


Figure G.8 Austroads dataset for road 386, curvature (D_0 – D_{200}) – granular pavement

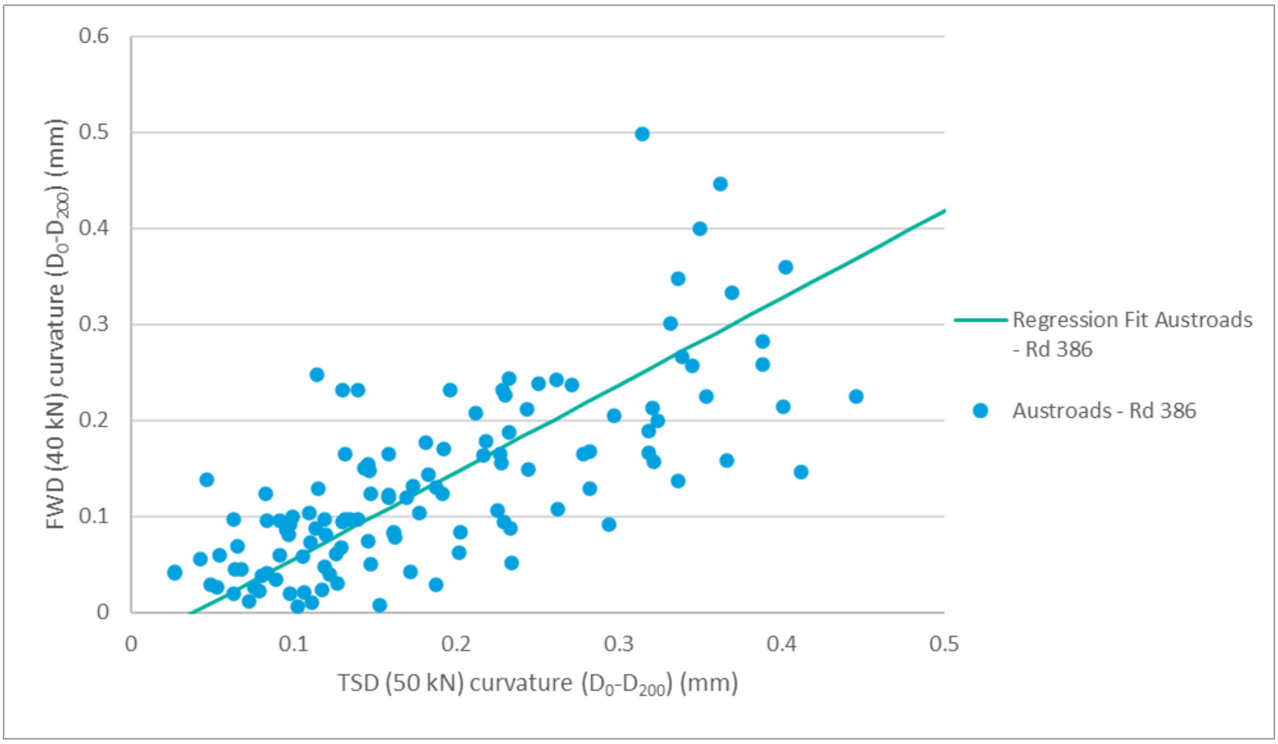


Figure G.9 Austroads dataset for road 741, maximum deflections (D_0) – granular pavement

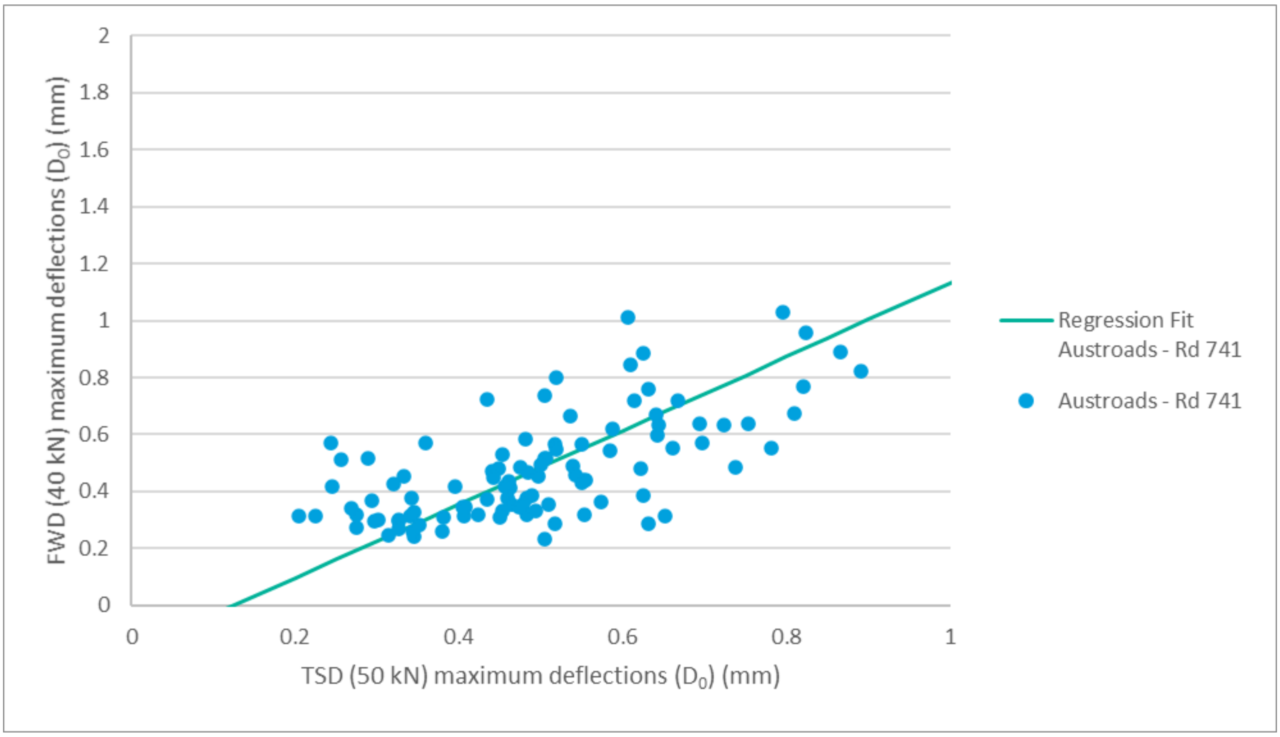


Figure G.10 Austroads dataset for road 741, curvature (D_0 – d_{200}) – granular pavement (*weak correlation, regression not used in Austroads vs. WA comparison*)

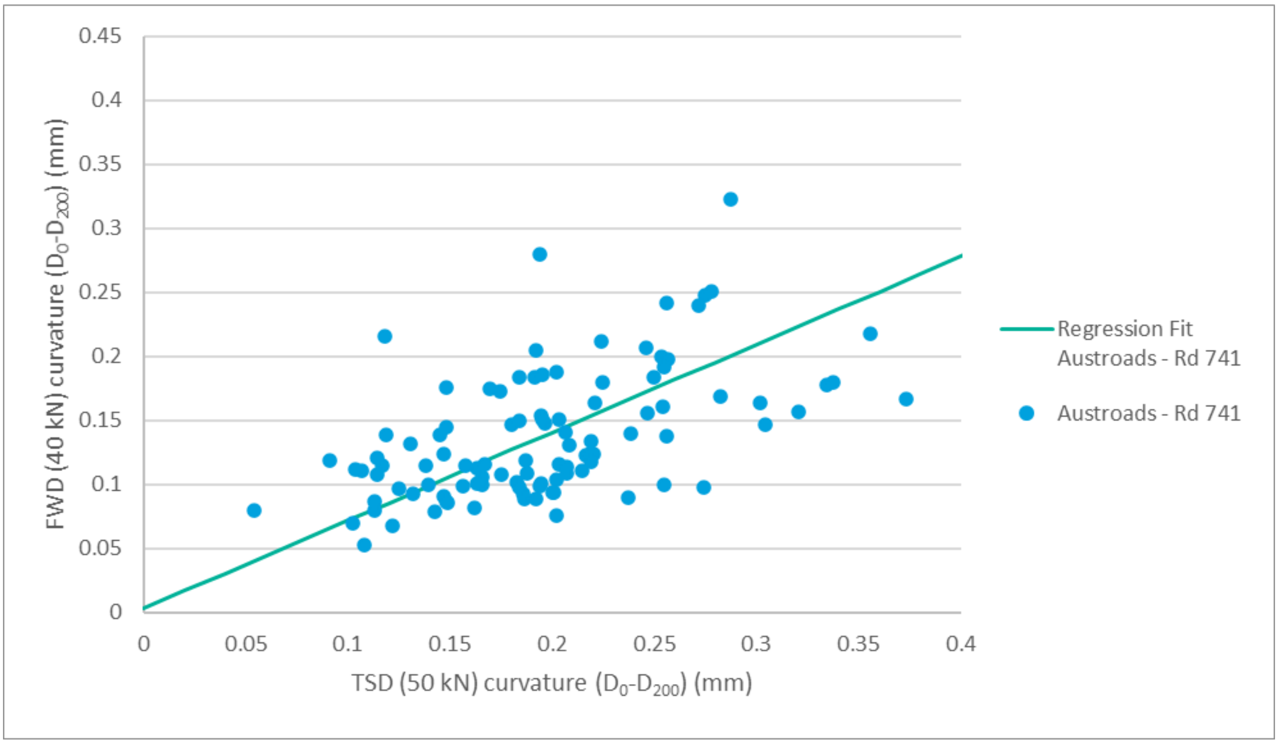


Figure G.11 Austroads dataset for road 831, maximum deflections (D_0) – granular pavement

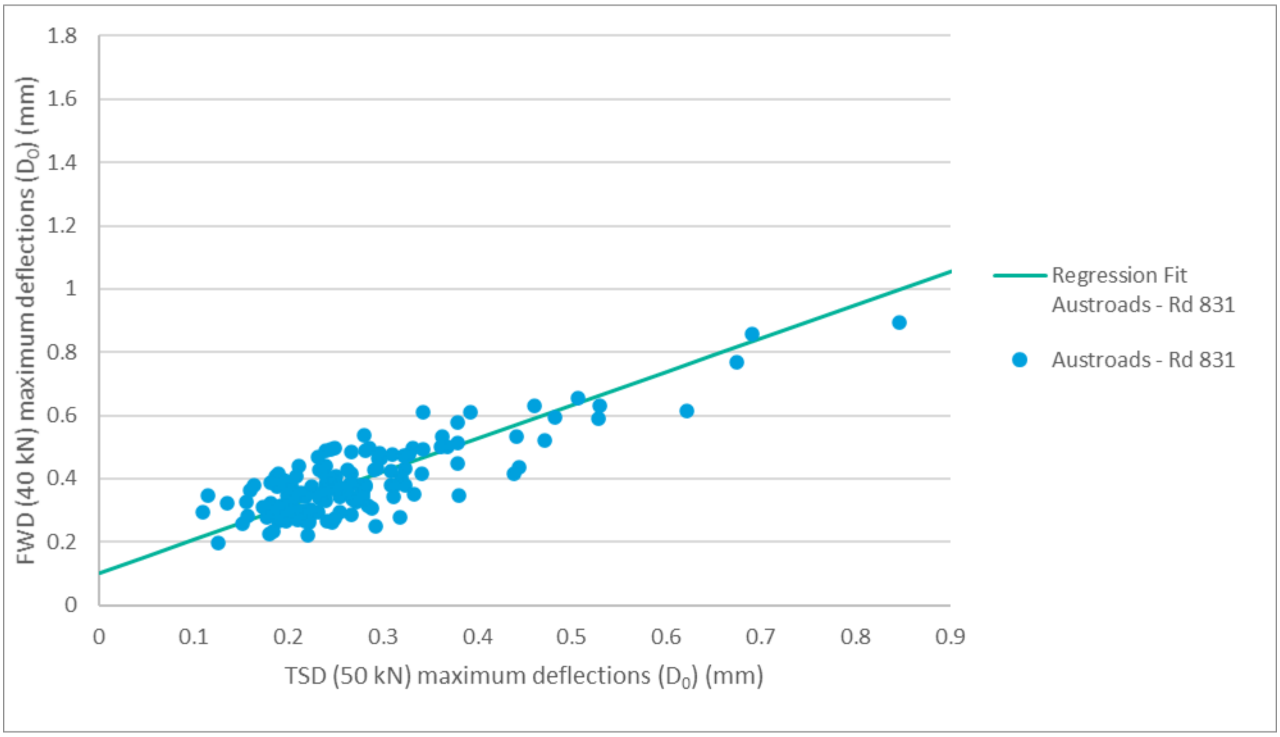


Figure G.12 Austroads dataset for road 831, curvature (D_0 – D_{200}) – granular pavement (*weak correlation, regression not used in Austroads vs. WA comparison*)

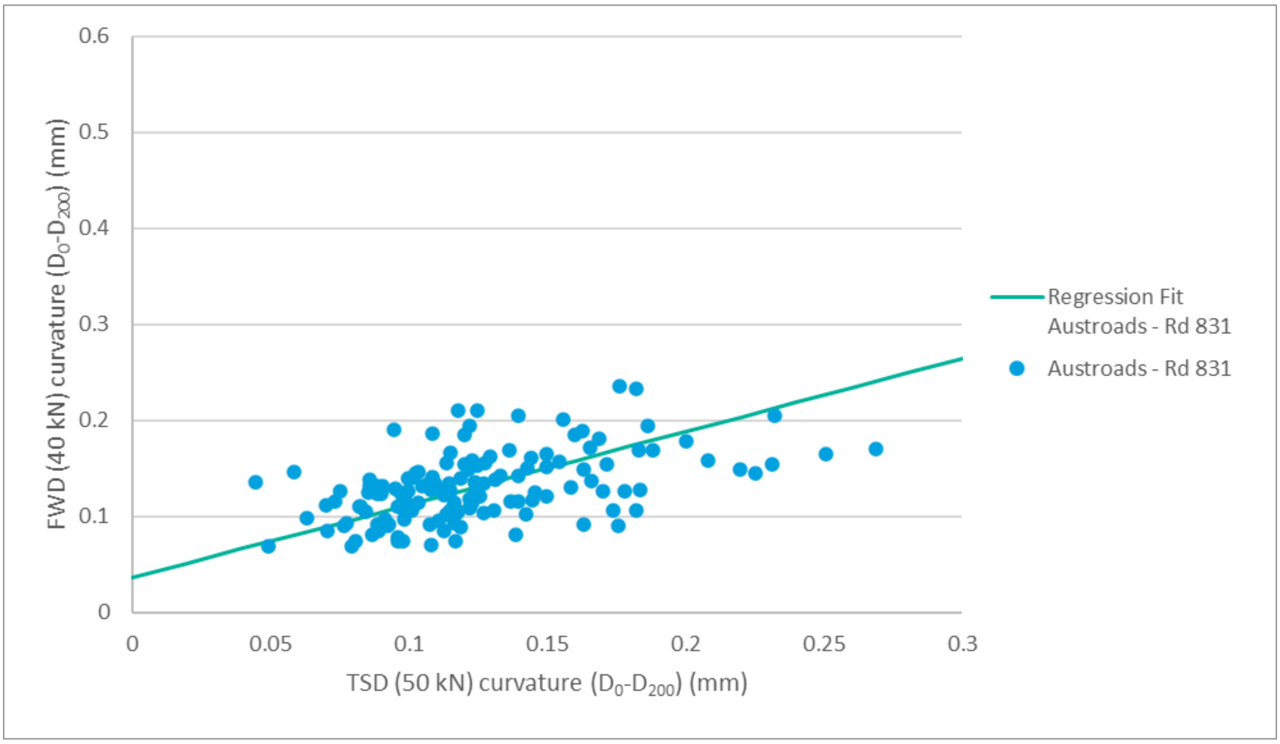


Figure G.13 Austroads dataset for road 896, maximum deflections (D_0) – granular pavement

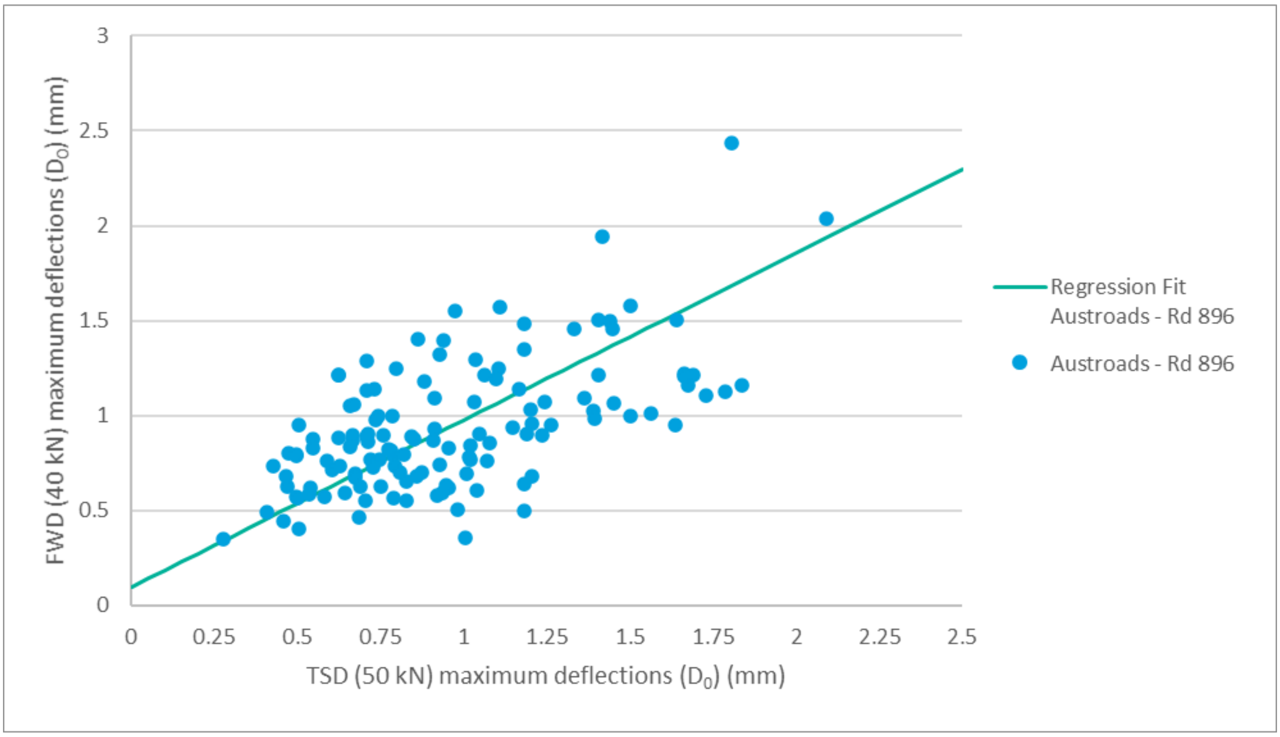


Figure G.14 Austroads dataset for road 896, curvature (D_0 – D_{200}) – granular pavement (*weak correlation, regression not used in Austroads vs. WA comparison*)

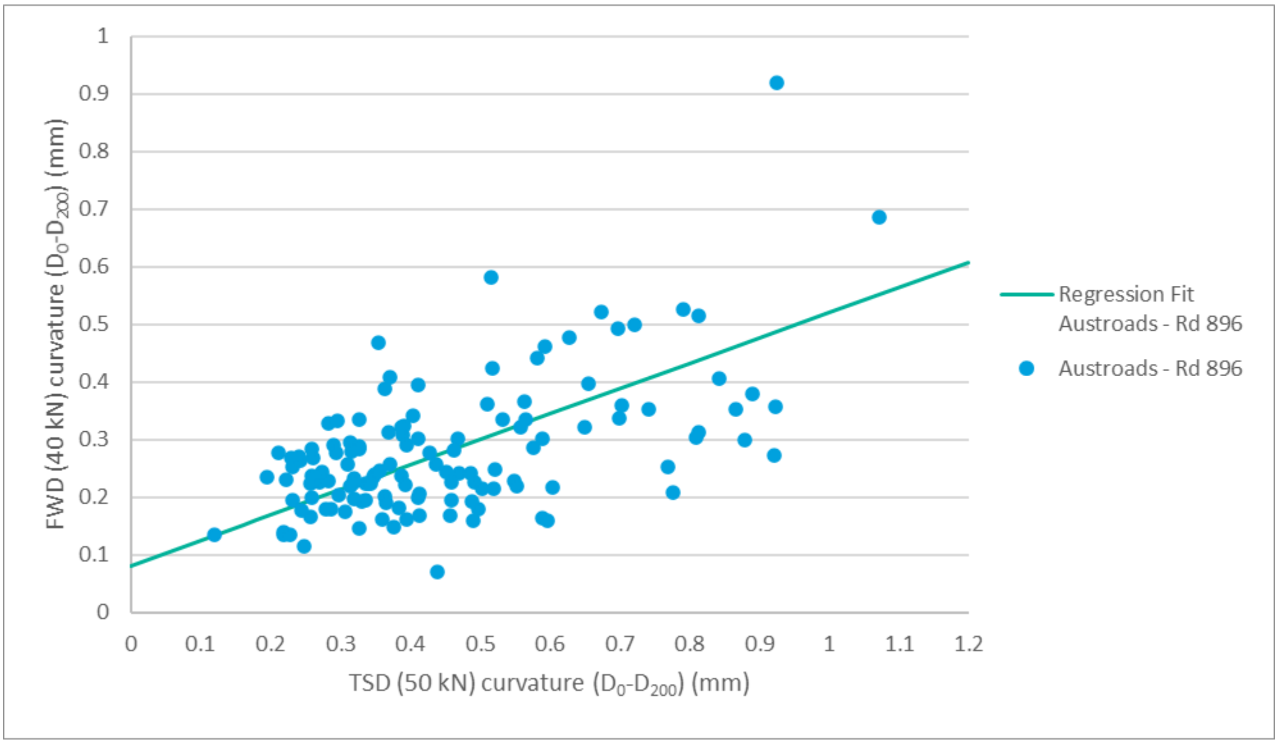


Figure G.15 Austroads dataset for road 1128, maximum deflections (D_0) – granular pavement

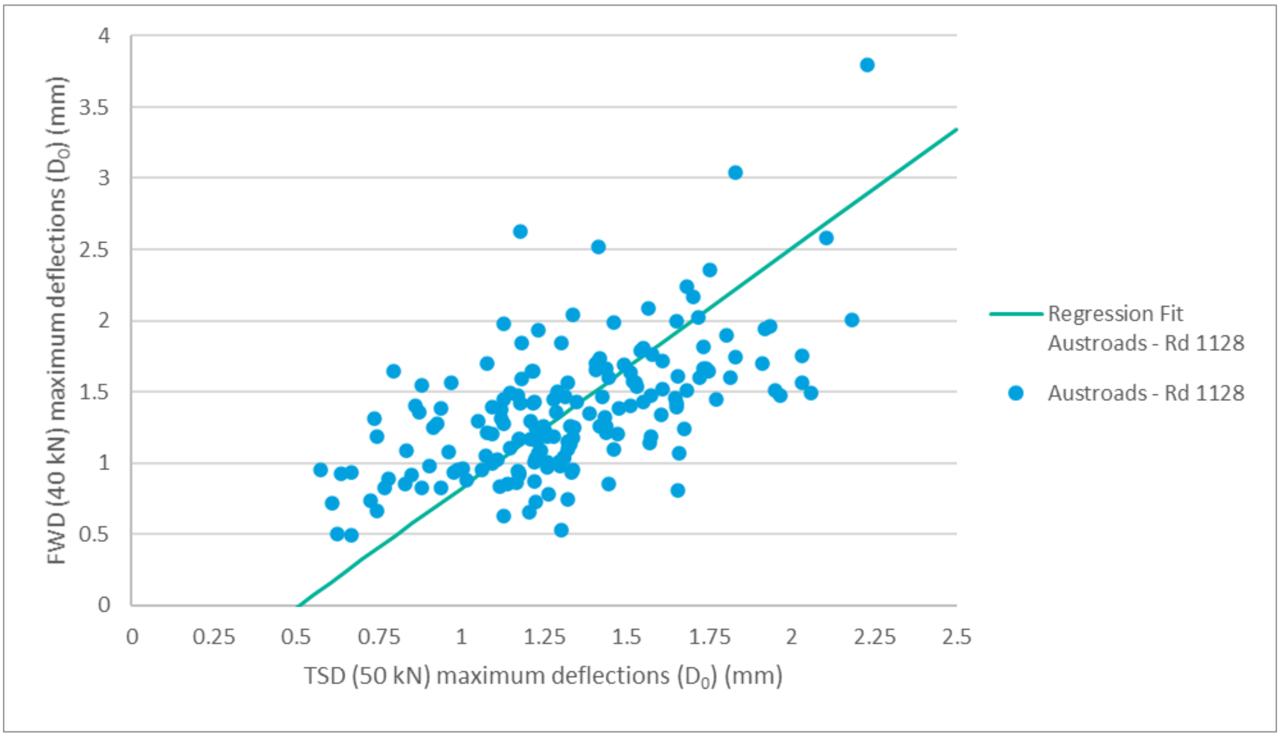


Figure G.16 Austroads dataset for road 1128, curvature (D_0 – D_{200}) – granular pavement (*weak correlation, regression not used in Austroads vs. WA comparison*)

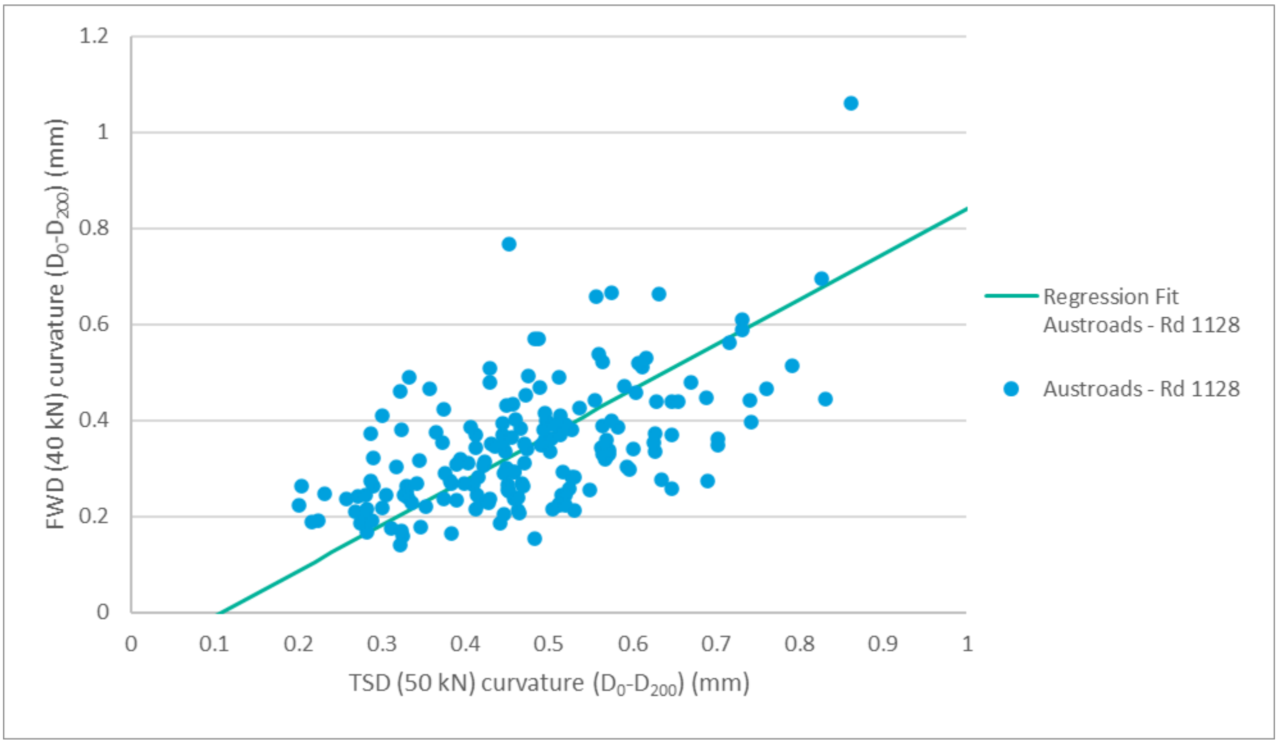


Figure G.17 Austroads dataset for road 211a, maximum deflections (D_0) – asphalt pavement

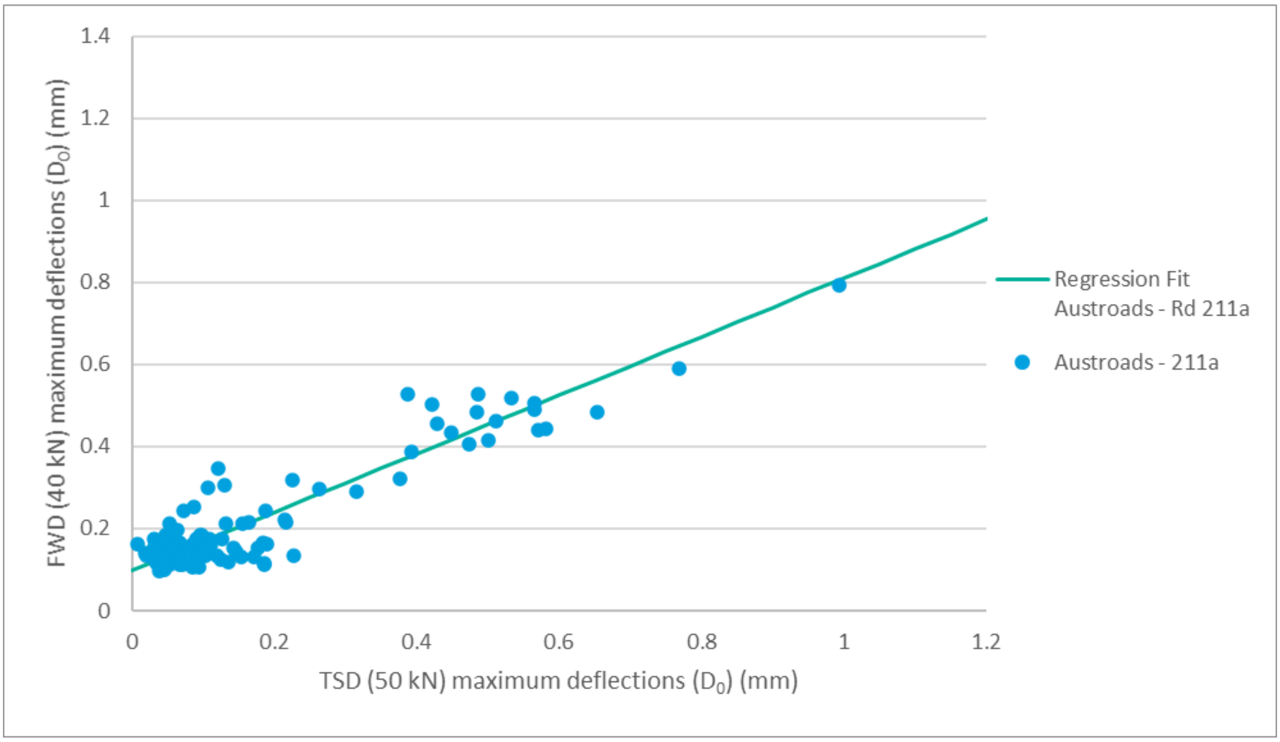


Figure G.18 Austroads dataset for road 221a, curvature (D_0 – D_{200}) – asphalt pavement

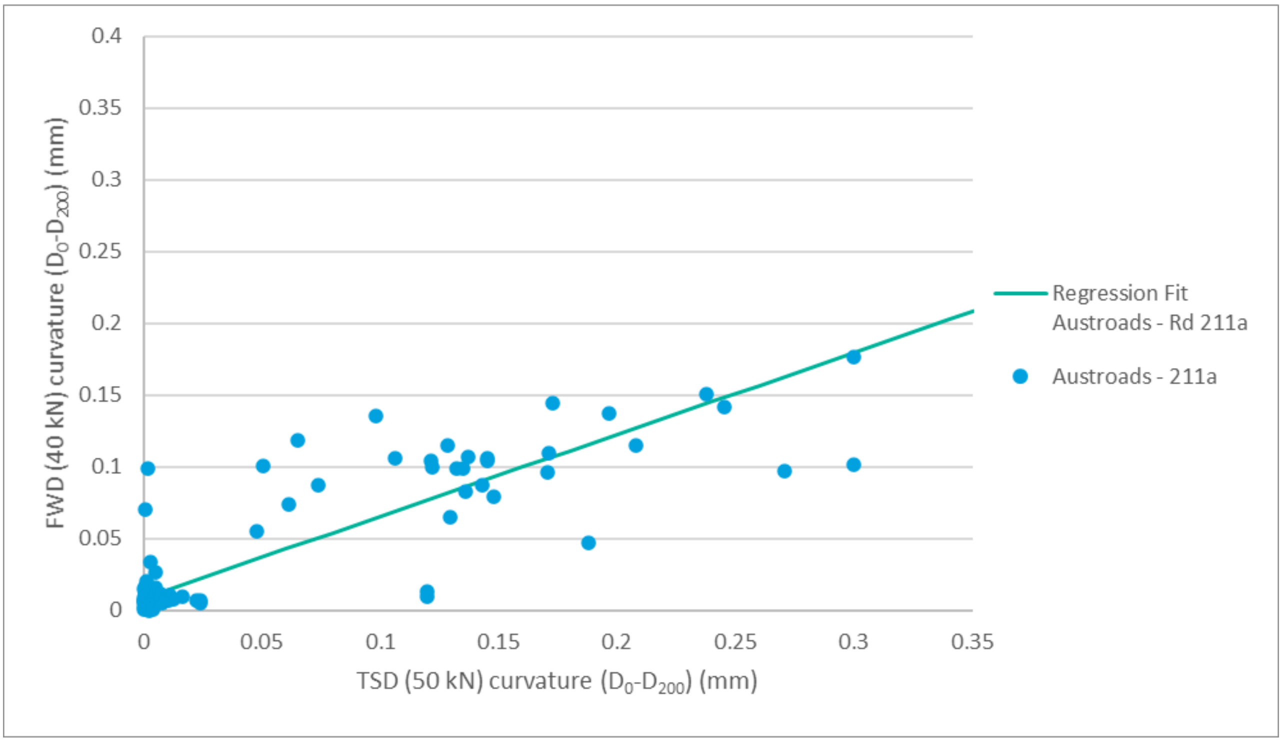


Figure G.19 Austroads dataset for road 211b, maximum deflections (D_0) – asphalt pavement

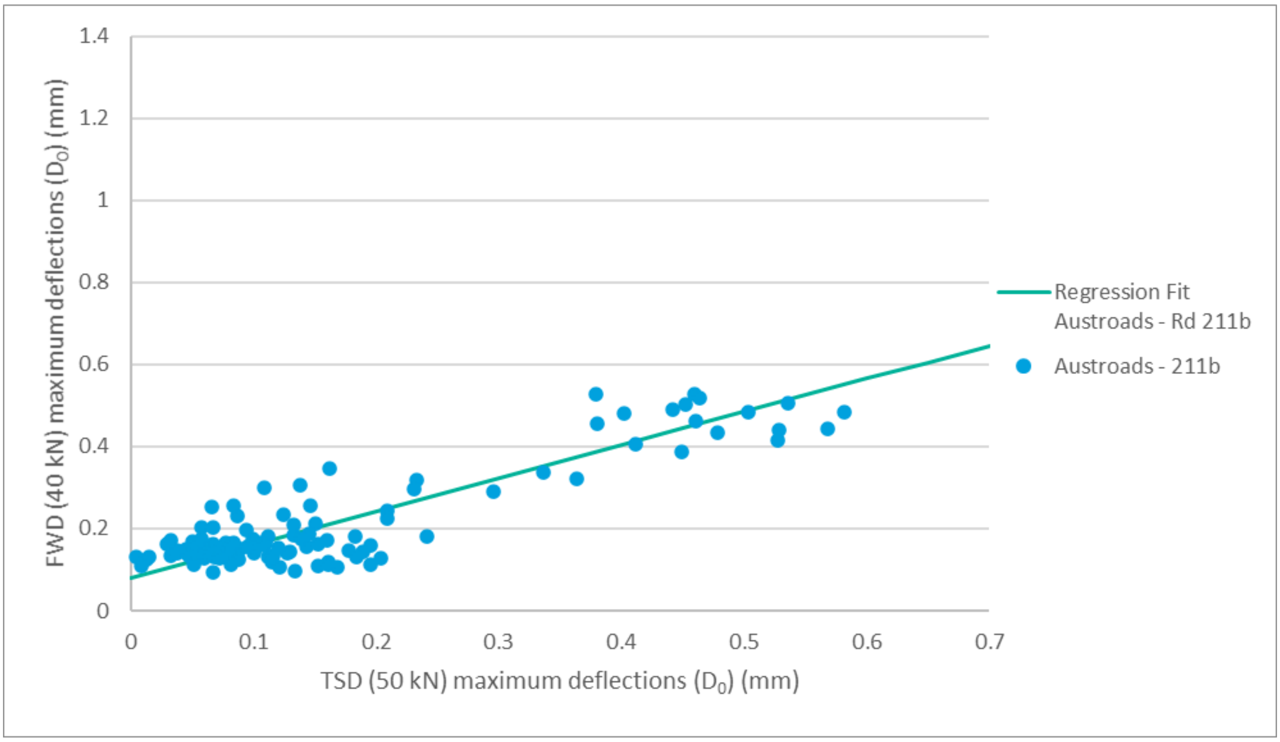


Figure G.20 Austroads dataset for road 221b, curvature (D_0 – D_{200}) – asphalt pavement

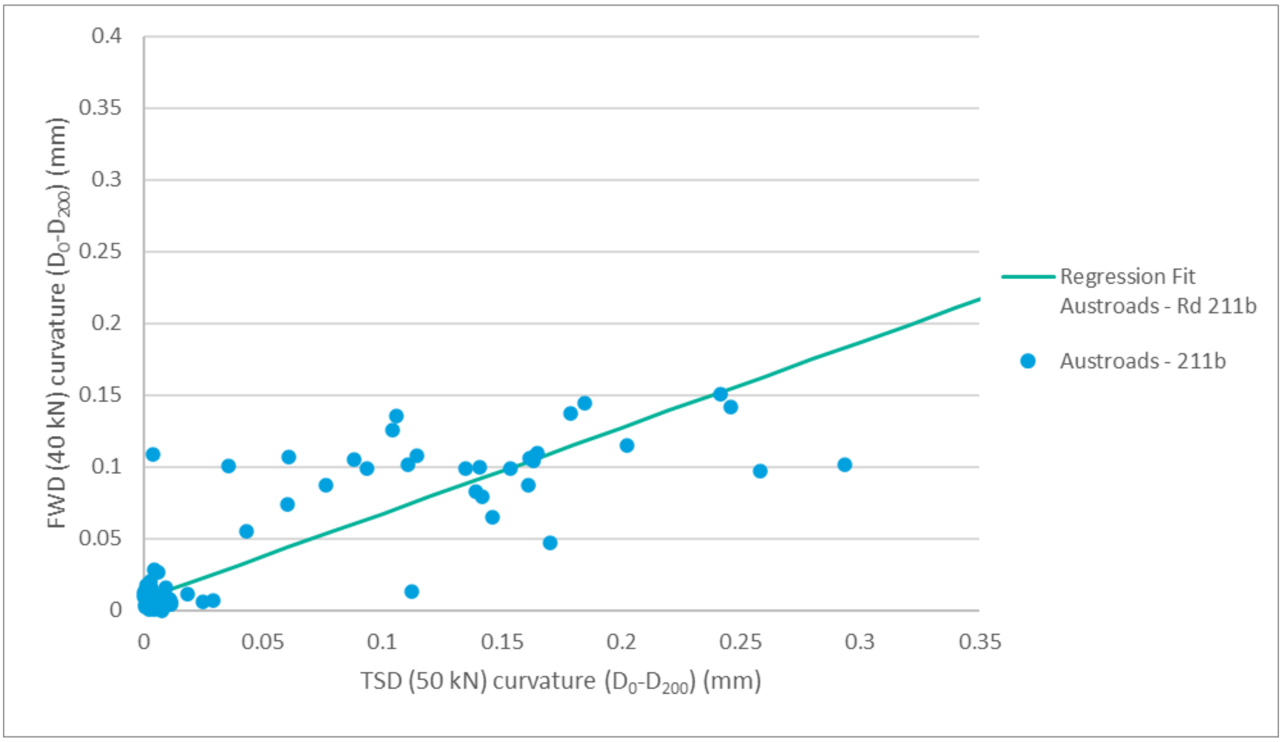


Figure G.21 Austroads dataset for road 121, maximum deflections (D_0) – asphalt pavement

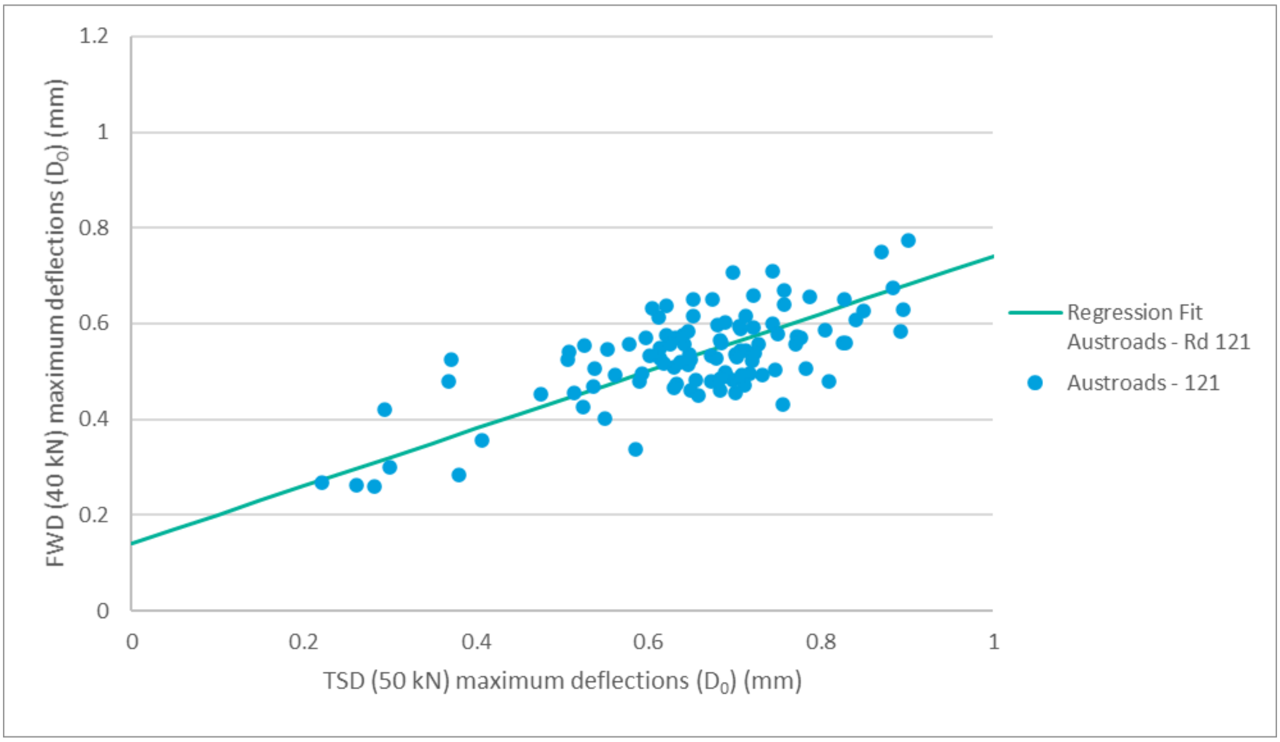


Figure G.22 Austroads dataset for road 121, curvature ($D_0 – D_{200}$) – asphalt pavement (*weak correlation, regression not used in Austroads vs. WA comparison*)

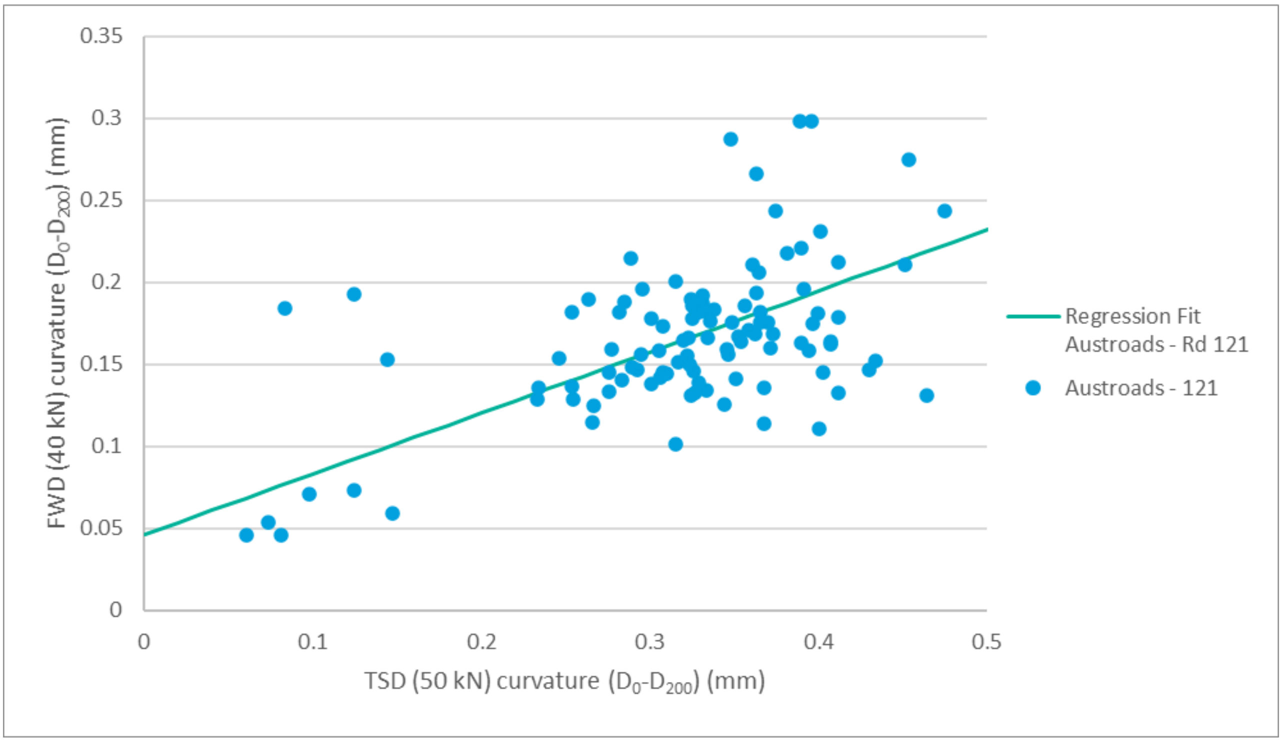


Figure G.23 Austroads dataset for road 9905-1, maximum deflections (D_0) – asphalt pavement

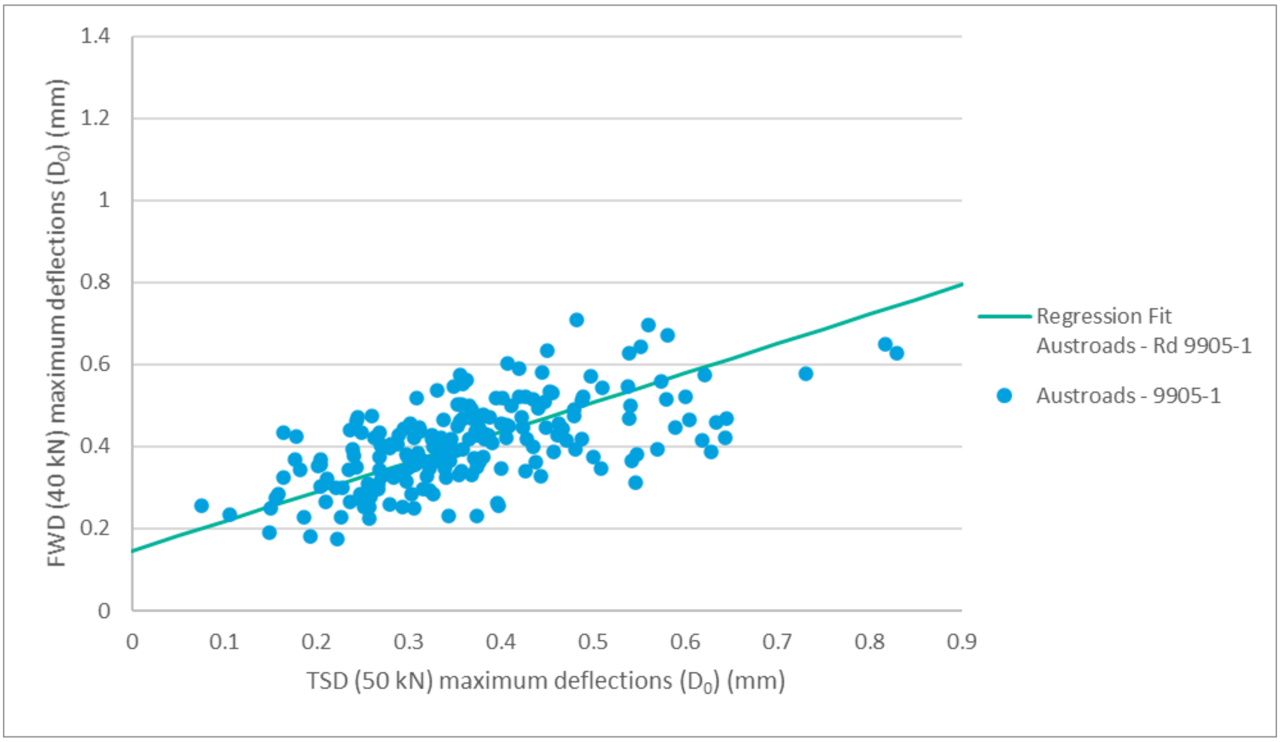


Figure G.24 Austroads dataset for road 9905-1, curvature ($D_0 – D_{200}$) – asphalt pavement (*weak correlation, regression not used in Austroads vs. WA comparison*)