Three-Dimensional Finite Element Analysis
of Concrete Pavement on Weak Foundation

A thesis submitted in fulfilment of the requirements
for the award of the degree of

Doctor of Philosophy

by

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August 2014
Declaration

This work has not previously been submitted for a degree or diploma in any university. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person except where due reference is made in the thesis itself.

How Bing Sii (Perry)
August 2014
Abstract

Essential modern transportation systems together with the high demand for sustainable pavements under applied vehicular loading have led to a great deal of research worldwide of concrete pavements. Despite progressive knowledge of concrete pavement behaviour under applied loads, concrete pavements are still subject to deterioration due to crack initiation and propagation, indicating the need for further research. Cracks can be related to fatigue of concrete or erosion of materials in sub-layers. Transverse joints in concrete pavements are the locations where most pavement distress appears, leading to deterioration of the riding quality and featuring high maintenance cost. The state of stresses in the concrete surrounding dowel bars, in dowel jointed concrete pavements, are a major factor that contribute to transverse joint distress.

A three-dimensional (3D) finite element model is developed in this study for analysing a dowel-jointed concrete pavement. The effects of different pavement and joint related parameters on the load transfer characteristics of a joint have been evaluated using the 3D finite element model. The numerical results from FE modelling are validated with classical analytical solutions of shear and moment along the dowel. Five loading cases are applied in the model to replicate realistic vehicular loadings approaching and leaving the joint. Group action of the dowel bar system has also been examined.

Research presented in this thesis aims to address the most common fatigue related distresses in concrete pavements. The thesis establishes comprehensive finite element (FE) models and analyses to determine the structural behaviour of concrete pavements
under vehicular loading. The specific structural behaviour of the pavement at the doweled joint has been investigated for the following cases: (1) pavement with and without dowels; (2) doweled joint with and without lean concrete; (3) doweled joint with and without voids; (4) undoweled and doweled joint under subgrade strength with varying California Baring Ratio (CBR); (5) doweled joint under load varying magnitude (tyre pressure); (6) variation in dowel spacing; (7) slab thickness change; (8) single and dual wheel loads; (9) dowel looseness.

Results show that the voids underneath the joint cause an increase in the vertical displacement of the concrete slab and vertical stress at concrete/dowel bar interface which may result in crushing of the concrete and dowel loosening. These studies also indicate that significant reduction in load transfer efficiency and increase in both slab and base course stresses can be expected due to small gaps varying from 0.25 to 1.25 mm between the dowels and the slabs. In the worst case, the load transfer efficiency (LTE) has been found to reduce to 11.3% and 11.6% respectively for single wheel loading and odd dual wheel loading cases. In both cases voids are presented at the base course layer with 1.25 mm gap between dowel and slabs.

The numerical results from the finite element (FE) modelling are validated with classical analytical solutions of shear and moment along the dowel. The group action of the dowel bar system is examined and useful relationships have been developed for estimation of the relative load shared by individual dowel bars. These relationships have been used to develop a prediction model for the shear force in group action of dowel bar system and
deflection at the loading nodal point. The outcomes of the prediction model agree reasonably well with the finite element results.
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List of publications

During the course of the research work, the following refereed papers have been published or submitted for publication.

Journal Publications:


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International Conference Publications:


Submitted Journal Manuscript:

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<td>3D</td>
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<tr>
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<td>American association of highway and transportation official</td>
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<tr>
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<tr>
<td>CBR</td>
<td>California baring ratio</td>
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<td>Continuously reinforced concrete pavements</td>
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<tr>
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<td>Cement &amp; concrete association of Australia</td>
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<td>D-L</td>
<td>Dense-liquid</td>
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<td>Dowel Looseness</td>
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<td>Dowel bar retrofit</td>
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<tr>
<td>DEM</td>
<td>Discrete element method</td>
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<tr>
<td>ES</td>
<td>Elastic-solid foundation</td>
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</tr>
<tr>
<td>EEL</td>
<td>Equivalent effective length</td>
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<td>ESAL</td>
<td>Equivalent single axle load</td>
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<td>Finite element</td>
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<td>FEA</td>
<td>Finite element analysis</td>
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</tr>
<tr>
<td>FWD</td>
<td>Falling weight deflect meter</td>
<td></td>
</tr>
<tr>
<td>FRP</td>
<td>Fibre reinforced polymer</td>
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<tr>
<td>FHWA</td>
<td>Federal highway administration</td>
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<td>GFRP</td>
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<tr>
<td>HVAGs</td>
<td>Heavy vehicle axle groups</td>
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<tr>
<td>IRI</td>
<td>International roughness index</td>
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<td>JPCP</td>
<td>Jointed plain concrete pavements</td>
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<tr>
<td>JRCP</td>
<td>Jointed reinforced concrete pavements</td>
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<td>Kerr-Vlasov three-parameter foundation</td>
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<tr>
<td>LTE</td>
<td>Load transfer efficiency</td>
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<td>LSF</td>
<td>Load safety factor</td>
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<td>LC</td>
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<td>MPS</td>
<td>Maximum principle stress</td>
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<td>MEL</td>
<td>Maximum edge length</td>
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<tr>
<td>SAST</td>
<td>Axle single tyre</td>
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<td>SADT</td>
<td>Single axle dual tyre</td>
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<td>SA</td>
<td>Single axle</td>
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<td>TADT</td>
<td>Tandem axle dual tyre</td>
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<td>Triple axle dual tyre</td>
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<td>Tandem axle</td>
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<td>TR</td>
<td>Triple axle</td>
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<tr>
<td>$TT_{HH}$</td>
<td>Vertical displacement along transverse joint</td>
<td></td>
</tr>
<tr>
<td>RTA</td>
<td>Road and traffic authority</td>
<td></td>
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<tr>
<td>QADT</td>
<td>Quad axle dual tyre</td>
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<tr>
<td>$r_o$</td>
<td>Radius of the loaded area</td>
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<tr>
<td>$\mu_s$</td>
<td>Poisson’s ratio of foundation</td>
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<tr>
<td>$E_s$</td>
<td>Elastic modulus of foundation</td>
<td></td>
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<tr>
<td>$\nabla^2$</td>
<td>Laplace operator</td>
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<tr>
<td>$D_s$</td>
<td>Flexural rigidity of an imaginary plate in the Winkler foundation</td>
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<tr>
<td>$G_b$</td>
<td>Shear modulus of foundation</td>
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<tr>
<td>$\phi$</td>
<td>Angle of internal friction</td>
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<tr>
<td>CIF</td>
<td>Coefficient of internal friction</td>
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<tr>
<td>$\delta_U$</td>
<td>Deflection of the unloaded slab</td>
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<tr>
<td>$\delta_L$</td>
<td>Deflection of the loaded slab</td>
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<tr>
<td>$L$</td>
<td>Embedded length of the dowel</td>
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<tr>
<td>$Z$</td>
<td>Width of joint</td>
<td></td>
</tr>
<tr>
<td>$d$</td>
<td>Dowel diameter</td>
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<tr>
<td>$P_t$</td>
<td>Shear force on the dowel</td>
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<tr>
<td>$E$</td>
<td>Young’s modulus of the dowel</td>
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<tr>
<td>$I$</td>
<td>Moment of inertia of the dowel bar</td>
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<tr>
<td>$\beta$</td>
<td>Relative stiffness of a dowel embedded in concrete</td>
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<tr>
<td>$K_0$</td>
<td>Modulus of dowel support</td>
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<td>$d$</td>
<td>Diameter of the dowel</td>
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<tr>
<td>$\sigma$</td>
<td>Stresses in concrete</td>
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<tr>
<td>$y$</td>
<td>Deflection of the dowel</td>
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<tr>
<td>$Mo$</td>
<td>Bending moment on dowel at face of concrete</td>
<td></td>
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<tr>
<td>$f'c$</td>
<td>Ultimate compressive strength of concrete</td>
<td></td>
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<tr>
<td>$\ell_r$</td>
<td>Radius of relative stiffness (mm)</td>
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<tr>
<td>$E_c$</td>
<td>Elastic modulus of concrete pavement in MPa</td>
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<td>$h$</td>
<td>Thickness of concrete pavement in mm</td>
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<td>$\mu$</td>
<td>Poisson’s ratio of concrete pavement</td>
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<td>$K$</td>
<td>Modulus of subgrade reaction in MPa/mm</td>
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<td>$S$</td>
<td>Dowel spacing</td>
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<tr>
<td>$k$</td>
<td>Spring constant</td>
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<tr>
<td>$\Delta$</td>
<td>Relative displacement between loaded and adjacent concrete slabs</td>
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<tr>
<td>$Y_o$</td>
<td>Deflection at the face of the joint</td>
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<tr>
<td>$\lambda$</td>
<td>Form factor, equal to 10/9 for solid circular sections</td>
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<tr>
<td>$G$</td>
<td>Shear modulus of dowel</td>
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<tr>
<td>$\sigma_a$</td>
<td>Allowable bearing stress</td>
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<tr>
<td>$P_w$</td>
<td>Applied wheel load</td>
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<tr>
<td>$P_t$</td>
<td>Load transferred across the joint</td>
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<tr>
<td>$h_e$</td>
<td>Effective slab thickness</td>
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<tr>
<td>$X_{na}$</td>
<td>Neutral axis distance from top of concrete pavement layer in mm</td>
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<td>$F$</td>
<td>Shear shape factor for the steel dowel</td>
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<tr>
<td>$\Delta_{conc}$</td>
<td>Deflection of the concrete</td>
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</table>
e  natural logarithm base

x  distance along dowel from face of concrete
1.1 Background

Concrete pavements have been constructed in many parts of the developed and the advanced developing countries over the last fifty years. This type of pavement construction is usually selected by pavement engineers for roads subjected to heavy traffic loading. The cost of construction and maintenance for concrete pavement is high and therefore it is essential that this road asset is properly monitored during the life of the pavement. The use of concrete pavement without an asphalt top layer dates back to Scotland in 1865 (Croney, 1998). Concrete pavement technology has been embraced in Australia since the 1970s (Cruickshank, 1981).

Concrete pavements were traditionally designed based on theoretical equations developed by Westergaard (1926, 1927, 1933, 1939, 1943 and 1947). The mid-edge bottom-up transverse fatigue cracking was the only failure mode of the concrete pavements considered in the mechanistic design guides. Initiation and propagation of other fatigue related cracks in concrete pavements, led to the development of mechanistic-empirical approaches in concrete pavement design guides (AASHTO, 2003). Vehicular loads have been considered as static loads in concrete pavement design guides. Magnitude and configuration of vehicular loads together with environmental effects have a significant effect on induced tensile stresses within concrete pavements (Yu, 1998 and Hiller, 2002).
Since a variety of axle group configurations are employed in heavy vehicle industries and across countries, further study is required to determine the interrelationship between concrete pavement distresses and pavement responses to the applied loads. Furthermore, the fact that structural responses of concrete pavements may be affected by the frequency and speed of vehicular loads (Izquierdo, 1997) has not yet adequately considered in concrete pavement design guides.

Three major types of joints have been widely used to obtain slab discontinuity; these are contraction joints, expansion joints, and construction joints. Contraction joints are formed by introducing a weakened plane into the concrete by sawing a groove into a concrete while it is in its curing process, and allowing a crack to form at that plane. Expansion joints are created when an intersection is needed between the pavement and adjacent road structures, and in many cases within pavements. This is achieved by forming a full depth gap in the concrete slabs. Construction joints are also used between paving lanes, or when it is necessary to stop the paving construction.

When joints have no ability to transfer the load across the two slab boundaries, each slab edge must bear the fully applied load at a time. This case produces not only high dynamic tensile stresses in the concrete slab, but also large compressive stresses at the foundation layers in addition to increasing the pavement roughness and diminishing the riding quality. To overcome this problem, three means of load transfer mechanisms at the transverse joints have been widely used. These are dowel bars, aggregate interlock, and keyways. A good understanding of the mechanical behaviour of dowel bars and induced stresses at their interface with concrete is of outmost importance for the development of
feasible and effective doweled joints. Contact stresses between dowel bars and concrete are of major importance for improvement in the load transfer efficiency (LTE).

Concrete pavement systems typically consist of a jointed plain, jointed reinforced or continuously reinforced concrete top layer. A jointed system (either plain or reinforced) has longitudinal and transverse joints between slabs. Typically, the longitudinal joints are parallel to the direction of travel and transverse joints are perpendicular to it. Tie bars are required at longitudinal joints and dowel bars at transverse joints to assist in load transfer under vehicular loadings. Transverse joints in plain concrete are generally placed at a length of 3.70 to 6.10 m and for jointed reinforced concrete this length can reach 15 m. Reinforced systems (either jointed or continuous) use meshed reinforcing steel to control shrinkage cracks in concrete slabs. Several studies (Hiller, 2002 and 2005) have indicated that critical stresses within the concrete slab are more likely to occur closer to the transverse joints than the longitudinal joints especially for negative temperature gradients and widened lane widths.

Figure 1.1 (a) shows a typical concrete pavement system. The thickness of the concrete slab is 250 mm. Joints are placed in the concrete slab to reduce cracking and provide movement of concrete under vehicular loading and thermal gradients. Joints are usually created in the concrete slab by two means: (1) leaving a gap between the adjacent concrete slabs before pouring; (2) making a cut after a larger area of concrete is set. Typically the joint is 20 mm in width. In the first case, dowel bars may only be installed in the pavement system during concrete pouring but not cutting, as illustrated in Figure
1.1 (b). In the second case, when cutting the concrete slab, the initial cut (i.e. 10 mm in width) extends through the entire thickness of the concrete slab. A second cut (i.e. 20 mm) may then be made only to mid depth (i.e. 125 mm) of the slab, see Figure 1.1 (c). Once the joint is cleaned of excess concrete debris a backer rod (i.e. rolled and compressed foam) is installed at roughly 15 mm from the top concrete surface. A silicone sealant is then placed on top of the backer rod inside the joint, to help reduce moisture ingress (see Figures 1.1 (b) and (c)).

![Concrete pavement system diagram](image-url)
Dowel bars aid load transfer across transverse joints in concrete pavements. The load transfer efficiency (LTE) is determined by comparing displacement at the joint for the loaded and unloaded slabs. It has been reported that a LTE of less than 60% would necessitate load transfer restoration of the pavement (Zollinger, 2008). These cylindrical mild steel bars typically have a length and diameter of 460 mm and 32 mm, respectively. Dowel bars are secured within a wire basket which is pinned to the base course prior to laying the concrete slab. The dowel bars located at mid depth of the concrete slab are positioned parallel to the direction of travel (see Figure 1.1 (a)). Under a vehicular loading in the vicinity of the concrete slab joint, dowel bars immediately under the applied load assume a major portion of the load with other dowel bars assuming progressively lesser amounts (Friberg, 1938). The most severe loading occurs when the vehicle is on the edge of the concrete slab on both the approach and leave sides, as illustrated in Figure 1.1 (a). Previous studies (Yoder, 1975 and Shoukry, 2002) have evaluated pavement performance with dowel bars and showed that the normal stress in the vertical direction is an indicator of fatigue fracturing at concrete/dowel bar interface.

The base course layer is positioned in between the concrete slab and subgrade by spreading and compacting to a 95% relative compaction. The base course extends roughly 300 mm beyond the edge of the concrete slab thus providing more support and preventing concrete failure (Yoder, 1975). The base course layer helps to control pumping, frost action, drainage, shrink and swell of the subgrade, and expedition of construction (Huang, 2010). Construction aggregate is a composite material which may consist of sand, gravel, recycled concrete and geosynthetic aggregates. Subsurface voids
(see Figure 1.1 (a)) form within the base course as a result of moisture ingress which loosens and subsequently weakens the base course. Moisture may enter the base course through cracks on the concrete slab and leaking drainage systems. Voids may also form close to cracks or joints due to water infiltration from the pavement surface. Once developed, voids can be detected through the use of a Ground Penetrating Radar (GPR) and Impact-Echo, or by core drilling. Subgrade forms the bottom layer of the pavement system and is generally compacted and can be stabilised through the addition of asphalt, cement or lime.

A concrete pavement is designed to provide safe and long-lasting road surfaces. However, fatigue and deterioration (i.e. distress) from repeated vehicular loading over time are the most common observed failure mechanisms and major attributors to pavement maintenance and rehabilitation costs. Thus, a good understanding of pavement distress is crucial to the successful design and operation of road infrastructure. Warping deformation of the concrete slab is a characteristic phenomenon under environmental and repeated vehicle loads (Quintus, 1993) which may lead to void formation due to the accumulated plastic deformation and subsequent disengagement of the base course from the concrete. Distress of the pavement in the form of joint deterioration or cracking also contributes to void formation by allowing moisture infiltration. The combination of distress and layer voids will further reduce the pavement load carrying capacity. Friberg (1938) and Quintus (1993) stated that distress is influenced mostly by compressive stress and that the first sign of deterioration is the formation of transverse or longitudinal cracking within the concrete slab. It has also been reported that transverse and
longitudinal cracking are more common than D-cracks, corner cracks and meander cracks (Chen, 2002 and Machemehl, 2005). It is generally understood that the aim of joint design for concrete pavement is to reduce transverse and longitudinal cracking (CCA, 2004). Transverse cracks are typically due to shrinkage of the concrete layer from low temperatures, reflective crack caused by cracks beneath the surface layer and top-down cracking (Shoukry, 2002). Longitudinal cracks may be due to incorrect joint orientation and subsequent reflective cracking of layers (Zollinger, 2008).

One of the main causes of pavement failures is the deterioration and gradual weakening of the pavement components at the joints. Repetitive loading produces micro cracks in the slab which will propagate through the slab due to both external loadings and erosion in the cracks, and develops voids in the pavement sub layers. It can be monitored in field by using Falling Weight Deflectometer (FWD) deflection test. During the evaluation both approach and leave tests are taken (the definition of approach and leave testing corresponds to the direction of normal traffic flow are shown in Figure 1.2). Christopher (2013) indicated that the FWD deflection sensor time history response data provides the best data for quantifying pavement joint looseness caused by joint openings and off-set slack. In the new rigid pavement design, a layer of Cement Treated Base (CTB) or Lean Concrete Base is introduced between the wearing surface and the base course to provide resistance to erosion and pumping, as well as better foundation support in the pavements.
Chapter 1: Introduction

The main purpose of this research is to develop finite element models of a concrete pavement section using commercial finite element software – Strand7 (2004) and study the response of the model under simulated static wheel loads. The method of finite element analysis can produce accurate, reliable approximate solutions over shorter time than more rigorous, closed form analysis. It can be used for analysing complex concrete pavement structures such as dowel joints. It enables distress prediction on the level of stresses and strains at the vicinity of the joints, and assesses the performance of concrete pavement with lean concrete base (LCB) and with cavity (voids).

The Finite Element Method (FEM) is a numerical method of analysis for stresses and deformations in structures of any given geometry. The structure is discretised into ‘finite elements’ connected through nodes. The type, arrangement and total number of elements affect the accuracy of the results. FEM has become one of the most successful engineering computational methods and most useful analysis tools since the 1960s.

![Concrete pavement system with FWD](image)
(Ergatoudis, 1968 and Rzemieniecki, 1969). It is showing overwhelming capability and versatility in concrete pavement evaluation.

1.2 Problem statement

Transverse joints in rigid pavements are the locations where most pavement distress appears, leading to deterioration of the riding quality and escalating maintenance cost. The state of stresses in the concrete surrounding dowel bars, in dowel jointed concrete pavements, is a major factor that contributes to transverse joint distress. Review of previous studies indicates that many researchers were primarily concerned with identifying the compressive bearing stress on the top and the bottom of the dowel. Being small compared to the allowable bearing stress of concrete, transverse joint failure is often attributed to improper construction practice. It was found that significantly large tensile stresses develop in the concrete on both sides of the loaded dowel. The magnitude of the tensile stress approaches the tensile modulus of rupture of concrete causing tensile cracks to develop on both sides of dowel bar.

Many studies have been conducted on pavement distresses by using conventional analytical approaches such as Westargaard theory or using some tedious iteration process to calculate the load transfer across the joints. However, there are many limitations associated with these approaches. The theory is based on the 2D model of a concrete slab presented by thick plate element resting on Winkler foundation which is a series of uniformly distributed springs. It is applicable only if the analysed structure is continuous with consistent material property. A concrete pavement section is composed of a concrete
slab followed by two or three sub layers and the load transfer device at the joint. Each of these components has its own material properties and behaves in a completely different manner. Therefore, general differential equations cannot represent the actual behaviour of the whole pavement structure. Moreover, the load transfer and the existence of joint opening, cracks and gaps between slabs and the effect of pumping or voids in the sub layers cannot be analysed.

The most serious problem in concrete pavements is crack generation and propagation (Hossain, 2003). Cracking occurs in the first few days after placement due to plastic shrinkage and the reaction between cement and aggregates in the setting time (Nawy, 2001). It then spreads over the pavement surface and propagates deeper in the concrete due to external factors such as drying shrinkage and fatigue. Current concrete pavement guides normally require the provision of reinforcement to minimise the development of secondary cracks in the pavement surfaces. Research on the effects of different factors, such as environmental effects, and dynamic response of concrete pavements have been performed in the past based on point, wheel, single axle and tandem axle loadings. Results of dynamic analyses, conducted in the past, showed that static loads produce greater tensile stresses in concrete pavements than dynamic loads.

Despite the fact that many studies were conducted to achieve a better understanding of the mechanical behaviour of the joints, it appears that transverse joints in concrete pavement are the locations where most pavement distresses could be observed. Premature distress is often observed around rigid pavement joints (Sargand, 2000). The loss in ride
quality for many jointed plain concrete pavements (JPCP) is primarily associated with joint related distresses such as faulting and spalling (Buch, 1995). Long-Term Pavement Performance (LTPP) studies indicate that dowel jointed plain concrete pavements (DJPCP) performs better than those without dowels (Titus-Glover, 1998). Field surveys showed that spalling is often found at locations along the transverse joints, indicating that spall development is due to a combination of traffic action and daily variations in pavement temperature (Senadheera, 1995). An expanded review of JPCP distresses could be found in Strategic Highway Research Program (SHRP, 1993). Previous parametric studies related to identifying the variables, which significantly affect the performance of transverse joints, indicated that the dowel concrete interaction takes a major place among those influencing the load transfer efficiency (Ozbeki, 1985). Therefore, it is necessary to have an in-depth understanding of the state of stresses and strains that develop at the dowel-concrete interface and their distributions around loaded dowel bars. These stresses are believed to have a significant contribution in the descent of the dowel/concrete contact modulus, and consequently the deterioration of the load transfer efficiency at the joint. These stresses need to be thoroughly analysed. A great improvement of the joint performance could then be achieved, once these stresses are reduced to a minimum level, through careful design of the transverse joint and the load transfer mechanism.

1.3 Research objectives

This research reports on an analytical investigation of load transfer across doweled joints under various loading cases and design conditions using three-dimensional (3D) finite element models.
1. The pavement behaviours to be investigated include
   - Pavement with and without dowels;
   - doweled joint with and without lean concrete;
   - doweled joint with and without voids;
   - undoweled and doweled joint under subgrade strength with varying California Baring Ratio (CBR);
   - doweled joint under load varying magnitude (tyre pressure);
   - variation in dowel spacing;
   - slab thickness change;
   - single and dual wheel loads;
   - dowel looseness.

2. The group action of the dowel bar system is examined and useful relationships have been developed for estimation of the relative load shared by individual dowel bars. These relationships have been used to develop a prediction model for the shear force in group action of dowel bar system and deflection at the loading nodal point. The outcomes of the prediction model agree reasonably well with the finite element results.

3. The numerical results from the finite element (FE) modelling are validated with classical analytical solutions of shear and moment along the dowel.

In this study, finite element models are constructed with two concrete slab segments composed of brick elements and the dowel bars are modelled using beam elements. The present model is capable of accounting for the influence of dowel under various
parameters investigated in this study in relation to the load transfer efficiency. 3D Brick element can capture severe deformation gradients in the concrete slab under multiple wheel loads, which is impossible with classical approaches using Kirchhoff plate elements (Kim, 2000). Further, brick elements used in supporting layers count for heterogeneous material properties of each layer. Hence, this approach can provide more accurate displacement fields, which influence the stress response of the concrete slab, than the classical approaches with Winkler foundation (Kim, 2000). In addition, the FE mesh density can be easily varied with regard to the stress gradient to improve mesh efficiency. Above all, this approach can directly evaluate the amount of load transfer across doweled joint by computing the shear force in the beam elements. Therefore, one can observe dowel shear force distributions for various parameters mentioned above and determine the number of engaged dowels as well. The 3D finite element models are therefore expected to be more realistic and versatile to represent a jointed concrete pavement with dowel bar system.

1.4 Outline of thesis

Following the introduction in this chapter, Chapter 1 also includes the problem statement; research objectives and outline of the thesis.

In chapter 2, a comprehensive literature reviews on behaviour and deteriorations of concrete pavements have been carried out to collect adequate information on factors affecting performance of concrete pavements. This includes information on concrete
pavement cross section, loadings, analysis of concrete pavements, and concrete pavement distresses. The 2004 revision of the Austroads and the American Association of Highway and Transportation Official (AASHTO, 2003) methods for designing concrete pavements are presented. While the Austroads (2004) method is based on a PCA method, the AASHTO (2003) method was developed in accordance with a mechanistic-empirical approach.

Chapter 3 represents the FE modelling and evaluation of the parametric studies. Three dimensional modelling strategy and analysis are discussed. The results of the modelling and discussion of the analysis are presented for South East Queensland Motorways including Reedy Creek and Gold Coast regions. The analysis results and discussions on parametric studies for three and four layers concrete pavement are as follows: pavement with and without dowels; doweled joint with and without lean concrete; doweled joint with and without voids; undoweled and doweled joint under subgrade strength with varying California Baring Ratio (CBR); doweled joint under load varying magnitude (tyre pressure); variation in dowel spacing; slab thickness change; single and dual wheel loads; Finally, the summary and comparison with existing observation are presented.

Chapter 4 discusses an effect of dowel looseness in response to jointed concrete pavement. Three different loading scenarios are applied to the dowel looseness model. Four cases with gaps vary from 0 to 1.25 mm between the dowel and slabs are investigated. An effect of dowel looseness in load transfer efficiency and stresses on both slab and base course are discussed in this chapter.
Chapter 5 presents a prediction model for the prediction of the shear force in the group action of dowel bar system and a deflection model to predict the deflection at the loading nodal point. The numerical solutions from F.E modelling for shear force and moment along the dowel directly under the applied load have been validated with the analytical solutions of Timoshenko (1925) and Friberg (1938) are presented in chapter 6.

Chapter 7 presents conclusion and recommendation for further research. Relevant figures to support the findings discussed in chapter 3 are presented in Appendix A.
Chapter 2: Literature Review

2.1 Introduction

A rigid concrete pavement system is composed of numerous discrete concrete slabs, longitudinal and transverse joints, and dowels. Longitudinal joints are provided for construction convenience and transverse joints are provided to control cracks caused by thermal deformation and drying shrinkage of the concrete slab. Despite those benefits, the joint often reduces the load carrying capacity of the concrete slab near the edge and results in pavement damage under repeated wheel loads (Huang, 1993 and Tayabji, 1983). Field observations, as well as experimental tests, have shown that doweled joints had better performance (Colley, 1983). The first use of smooth round steel dowels for the purpose of transferring the load to the adjoining slab was reported in a pavement constructed near Newports, between two army camps in 1917-1918 (Tabatabaie, 1979). The diameter of dowel is equal to one-eighth of the slab thickness and dowels being spaced at 300 mm on centres (Huang, 1993). A great deal of research has been devoted to assessing the amount of load transfer across a doweled joint. An intact joint is known to transfer more wheel load to adjacent slabs than a damaged joint. Unfortunately, there is no way to directly measure the shear force in a dowel with available sensor technology. Various indirect measures have been developed to estimate the load transfer over doweled joint. Among them, the displacement-based load transfer efficiency has been widely used. The LTE is defined as the ratio of displacement of the unloaded slab to that
of the loaded slab at a joint. Although LTE can easily be measured in the field with a
Falling Weight Deflectometer, it does not correlate well with actual load transfer across a
joint. Rather, it gives an implication of the magnitude of damage at the joint due to the
pumping and funnelling (dowel looseness). Austroads (2004) developed design guides
based on PCA method and AASHTO (2003) developed design guides based on
mechanistic-empirical approach are presented.

2.2 Concrete pavement

2.2.1 Subgrade-foundation models

The response of the supporting soil medium under the pavement is an important
consideration for engineering application. To accurately evaluate this response, it is
essential to know the complete stress-strain characteristics of the foundation. Describing
the stress-strain characteristics of any given foundation is usually governed by the
complex soil conditions, which are nonlinear behaviour, irreversible, and time-dependent.
These soils are generally anisotropic and nonhomogeneous. Finite Element models were
developed to simulate soil response under vehicular loading and boundary conditions.
The assumptions are necessary for reducing the analytical rigor of such a complex
boundary condition problem. Two of the most common applied assumptions are linear
elasticity and homogeneity.
Dense-Liquid (DL), spring, and Winkler foundation

In the dense-liquid foundation model, the foundation is seen as a bed of evenly spaced, independent, linear springs.

The model assumes that each spring deforms in response to the vertical stress applied directly to the spring, and does not transmit any shear stress to the adjacent springs. The external loads (p), applied on any point is given by Equation 2.1.

\[
P = kw
\]  \hspace{1cm} (2.1)

Where \( k \) is the modulus of subgrade reaction, \( w \) is a displacement of foundation.

No transmission of shear forces means that there are no deflections beyond the edges of the plate or slab. Westergaard (1926; 1927; 1933; 1939; 1943 and 1947) applied these pavement support systems in his studies. The liquid idealization of this foundation type was derived for its behavioural similarity to a medium using Archimedes’ Buoyancy principle.

In the field, the \( k \)-value is determined using data obtained from a plate-loading test performed on the foundation using a 30-inch-diameter plate (Ioannides, 1985). The load is applied to a stack of 1-inch-thick plates, until a specified pressure (p) or deflection (\( \Delta \)) is reached.
The k-value is then computed as the ratio of the pressure to the corresponding deflection,

\[
k = \frac{P}{\Delta}\quad (2.2)
\]

Another method for obtaining a k-value for use in analysis is by back calculation from measured deflections of the slab surface obtained from non-destructive tests; using devices such as falling weight deflectometers (FWD).

**Elastic-Solid foundation (ES)**

The elastic-solid foundation model or Boussinesq subgrade treats the soil as a linearly elastic, isotropic, homogenous material that extends semi-infinitely. It is considered a more realistic model of subgrade behaviour than the dense-liquid model, because it takes into account the effect of shear transmission of stresses to adjacent support elements. Consequently, the distribution of displacements is continuous; that is, deflection of a point in the subgrade is due to stress acting at that particular point, and also is influenced by decreasing by stresses at points further away.

Various solutions were available in the literature, such as the Boussinesq solution, Equation 2.3 (Boussinesq, 1860).

\[
W = \frac{2(1-\mu_s^2)pr_0}{E_s}\quad (2.3)
\]
Where, \( W \) is a displacement of foundation surface at the centre of loaded area (inches), \( p \) is a contact pressure (psi), \( r_o \) is a radius of the loaded area, \( \mu_s \) is a Poisson’s ratio of foundation, and \( E_s \) is an elastic modulus of foundation.

Because of its mathematical complexity, the solid-foundation model is less attractive than the dense-liquid foundation model. Unlike the dense-liquid foundation model, where the governing equations are differential, the elastic foundation model requires solving integral or integro-differential equations. The continuous nature of the displacement function in the elastic-solid model also means that this model cannot accurately simulate pavement behaviour with discontinuities in the structure, especially for slabs supported on natural soil subgrades. The limitation of this model is unsuitable for predicting slab response at edges, corners, cracks, or joints with no physical load transfer. The elastic-solid foundation model considers the shear force interaction of different elements in the foundation. Although it improves on Winkler foundation model by considering shear forces in the foundation, field tests showed inexact solutions for many foundation materials. Foppl (1909) reported that the surface displacements of foundation soil outside the loaded region decreased faster than the prediction by this model. AASHTO (2003) shows that both DL and ES idealization exhibit discrepancies with the result that real conditions ignore the shear strength in the subgrade soil between springs.
Improved models using a modified Winkler foundation

Dense-liquid and elastic-solid foundation models represent two extremes of actual soil behaviour. The dense-liquid model assumes complete discontinuity in the subgrade and is better for soils with relatively low shear strengths. In contrast, the elastic-solid model simulates a perfectly continuous medium and is better for soils with high shear strengths. The elastic response of a real soil subgrade lies somewhere between these two extreme foundation models. In real soils, the displacement distribution is not continuous; neither is it fully discontinuous. Deflection under a load can occur beyond the edge of the slab, and goes to zero at some finite distance. To link the gap between the dense-liquid and elastic-solid foundation models, these foundation models can be improved in one of two ways:

- Starting with the Winkler foundation and assuming, some interaction among spring elements
- Starting with the elastic-solid foundation, assuming simple expected displacements or stresses

A disadvantage with these models is the lack of guidance in selecting the governing parameters.

Hetenyi foundation: Hetenyi (1946; 1950) suggested achieving interaction of independent spring elements by embedding an elastic beam in two-dimensional cases and by embedding a plate in the material of the Winkler foundation in three-dimensional cases. It is assumed that the beam or plate deforms only in bending. Equation 2.4 shows
the relation between contact pressure \( p \) and deflection of foundation surface \( w \) for three dimensional cases.

\[
P = kw + D_s \nabla^2 w \tag{2.4}
\]

Where \( p \) is contact pressure (psi), \( \nabla^2 \) is the Laplace operator and \( D_s \) is the flexural rigidity of an imaginary plate in the Winkler foundation, representing interaction of independent spring elements.

**The two-parameter foundation (TP):** Pasternak (1954) considered shear interactions in the spring elements of a Winkler foundation by connecting the ends of the springs with a beam or plate consisting of incompressible vertical elements that deformed only by transverse shear. Under this assumption, Equation 2.5 shows the relation between the contact pressure \( p \) and deflection of foundation surface \( w \) (inches).

\[
P = kw + G_b \nabla^2 w \tag{2.5}
\]

Where \( G_b \) is a shear modulus of foundation.

**“Genelized” foundation by Venckovskii:** In this foundation model, in addition to the Winkler hypothesis, Venckovskii (1958) assumed that the applied moment \( M_n \) is proportional to the angle of rotation. Equations 2.6 and 2.7 describe this analytically.

\[
P = kw \tag{2-6}
\]
\[ M_n = k_1 \frac{dw}{dn} \]  

(2.7)

Where \( n \) is any direction at the point in the plane of the foundation surface, and \( k \) and \( K_1 \) are the corresponding proportionality factors.

**Improved models by using a modified elastic-solid foundation (ES)**

**Reissner foundation:** Assuming that the in-plane stresses throughout the foundation layer (2.8) are negligibly small.

\[ \sigma_x = \sigma_y = \tau_{xy} = 0 \]  

(2.8)

And that the horizontal displacements at the upper and lower surfaces of the foundation layer are zero; Reissner (1958) obtained the relationship in Equation 2.9 for the elastic case

\[ c_1 w - c_2 \nabla^2 w = p - \frac{c_2}{4c_1} \nabla^2 p \]  

(2.9)

Where \( c_1 = \frac{E_s}{H} \)  
\( c_2 = \frac{HG}{3} \)  
\( G = \frac{E_s}{2(1+\mu)} \)  

(2.10)

To apply the Reissner model to the case in which elastic modulus \( E_b \) varies linearly with the depth of foundation, Horvath (1983) developed a modified Reissner model.
\[ C_1 w - C_2 \nabla^2 w = p - C_3 \nabla^2 p \] (2.11)

Where, \( C_1, C_2 \) and \( C_3 \) are constants which are functions of elastic modulus, \( E_b \) and thickness \( H \) of the foundation.

**Beam-Column-Analogy foundation:** From an elastic continuum, Horvath (1989) developed a Pasternak-type, beam-column-analogy foundation model as

\[ p = \frac{E_s w}{H} \frac{HG}{2} \nabla^2 w \] (2.12)

With this model, Horvath (1992) analysed the mat-supported Chemistry Building at Massachusetts Institute of Technology in Cambridge, Massachusetts. The comparison of computed and observed settlements showed that this model provided good agreement with the observed behaviour.

**Kerr-Vlasov Three-parameter foundation (KV)**

Kerr (1964) recommended the use of another physical interpretation of this model as a shear layer placed above a TP foundation. The method is based on several analytical solutions resulting from different practically important boundary conditions. The KV foundation leads to a banded stiffness matrix, without predicting infinite soil pressures under the free edge of the slab, as ES and TP would.
To determine the stiffness matrix for KV, a special 8-noded, 24-degree-of-freedom element was introduced. The first four nodes are placed at the top of the upper (DL) springs, while the other four nodes are positioned at the top of the TP foundation.

**Drucker-Prager model**

Drucker-Prager (1952; 1957) is a yield criterion, where the yield surface does not change with progressive yielding. As a result, no hardening rule is considered in this model and material is assumed to be elastic-perfectly plastic. The Drucker-Prager criterion can be expressed as:

\[
\alpha_{DP} I_1 + \beta_{DP} = \sqrt{J_2}
\] (2.13)

Where \( I_1 \) is the first invariant of the stress tensor and can be calculated from Equation 2.14, \( J_2 \) is the second invariant of the stress deviator tensor and can be calculated from Equation 2.15.

\[
I_1 = \sigma_1 + \sigma_2 + \sigma_3
\] (2.14)

\[
J_2 = \frac{1}{6} [ (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2]
\] (2.15)

The Drucker-Prager, material constants, \( \alpha_{DP} \) and \( \beta_{DP} \), can be determined by contributing the Mohr-Coulomb criterion into the method. As a result:
\[ \alpha_{DP} = \frac{2 \sin \varphi}{\sqrt{3} (3 - \sin \varphi)} \]  

(2.16)

\[ \beta_{DP} = \frac{6 C_{IF} \cos \varphi}{\sqrt{3} (3 - \sin \varphi)} \]  

(2.17)

Where, \( \varphi \) and \( C_{IF} \) are angle and coefficient of internal friction respectively.

The material fails under the applied load if the magnitude of failure functions becomes less than or equal zero. The failure function is:

\[ F_{DP} = \alpha_{DP} I_1 + \beta_{DP} \sqrt{J_2} \]  

(2.18)

### 2.2.2 Subbase

This layer is constructed over the subgrade and under concrete slab. Austroads (2004) recommends the use of a lean-mix concrete (LMC) or a bonded subbase with a characteristic 28-day compressive strength of not less than 5 MPa. Bonded subbase includes cement stabilised crushed rock, dense-graded asphalt or rolled lean concrete (Austroads, 2004).

The subbase has a thickness of 125 to 150 mm (Austroads, 2004). The aims of producing a subbase under the concrete slab are:

- To resist erosion of the subbase and limit “pumping” at joints and slab edges;
• To provide uniform support under the pavement;
• To reduce deflection at joints and enhance load transfer across joints (especially if no other load transfer devices are provided, such as dowels); and
• To assist in the control of shrinkage and swelling of high volume change subgrade soils.

2.2.3 Debonding layer

A debonding layer is placed between the concrete slab and subbase to avoid the early age cracking due to plastic and drying shrinkage induced by the tensile stress at the interface of the concrete slab and subbase.

Wimsatt (1987) and Wesevich (1987) have concluded that a classical friction model cannot be used in concrete pavement systems as friction force between concrete slab and subbase because each layer has a different adhesion, shear, and bearing. Their studies result indicated a friction between the concrete slab and the subbase is greater than that calculation based on the classical friction model. Wimsatt (1987) and Wesevich (1987) introduced the concept of frictional stress in concrete pavement analysis instead of using coefficient of friction. This concept is highly independent of concrete slab thickness and bearing stress. However, Austroads (2004) recommends coefficients of friction of 1.5 to 3 are adopted between the concrete slab and the subbase.
Suh (2002) has come up with several methods to reduce the friction force in concrete pavements such as the use of polythene sheets or 40 mm asphalt bond breaker. He has concluded that the use of a single layer polythene sheet can reduce the coefficient of friction to 1.2.

The bonded boundary condition keeps the concrete slab and subbase together with no vertical separation, a fully unbonded boundary condition allows them to be separated under tensile force without inducing any frictional force between these layers. A partially bonded boundary condition, keeps the concrete slab and the subbase together for a certain frictional force. Yu (1998) indicated that friction between the concrete slab and the subbase is sufficient to produce bonded behaviour even if polyethylene sheets are not placed between them. Tarr (1999) stated that an unbonded condition could only be achieved by using a double layer of polyethylene sheets.

### 2.2.4 Concrete slab

The concrete slab is the top layer in the concrete pavement system. It contains a concrete with a recommended minimum compressive strength of 32 MPa (Austroads, 2004) to ensure the durability of the wearing surface and to provide sufficient flexural strength to avoid deteriorations in pavements under repetitive vehicular loading. The recommended concrete slab thickness was traditionally considered to be between 200 mm and 250 mm (RTA, 1991).
The principal types of cementitious concrete pavements are:

- Jointed plain concrete pavements (JPCP);
- Jointed reinforced concrete pavements (JRCP);
- Continuously reinforced concrete pavements (CRCP); and
- Steel fibre reinforced concrete pavements (SFRCP)

### 2.2.4.1 Jointed plain concrete pavement (JPCP)

There are two main categories of JPCP suitable for Australian conditions, (1) slabs 4.2 m long, with undowelled skewed joints, and (2) Slabs 4.5 m long, with dowelled square joints. However, longitudinal joints should be provided to limit slab widths to about 4.3 m and should be tied up to a maximum total tied width of about 15 m (Austroads, 2004).

### 2.2.4.2 Jointed reinforced concrete pavement (JRC)
in the concrete slab length. AASHO (1962) reported most JRCPs show a good performance over long periods.

2.2.4.3 Continuously reinforced concrete pavement (CRCP)

In a CRCP pavement, sufficient continuous longitudinal steel reinforcement is provided to induce transverse cracking at random spacing of about 0.5 to 2.5 m, and no contraction joints are required. Transverse reinforcement is provided to support the longitudinal steel and is designed in accordance with the subgrade drag theory. Transverse steel also provides “insurance” in the event of unplanned longitudinal cracking. A longitudinal joint should be provided to limit slab widths to about 4.3m and should be tied up to a maximum total tied width of about 15 m (Austroads, 2004).

2.2.4.4 Steel fibre reinforced concrete pavement (SFRCP)

Steel fibre reinforced concrete (SFRCP) provides increased resistance to cracking in both odd-shaped and acute cornered slabs and therefore is ideally suited to areas with a high proportion of slabs of irregular shape. At intersections and roundabouts it will often be the only viable concrete pavement option because of the increased flexibility it allows in the design of the joint layout. Transverse joints in SFRCP are typically square and undowelled, at maximum spacings of 6.0 m (Austroads, 2004).
2.2.5 Joints

Joints are locations where geometric discontinuities are included in concrete pavement slab. Concrete pavements are subjected to volumetric changes primarily due to temperature and moisture variations. If this volume change is restricted, then cracking distortion, as a result crushing due to excessive stresses can occur.

Skokie (1992), the primary functions of joints in concrete pavements are:

- to provide crack control and to provide space and freedom of relative movement between adjacent slabs;
- to provide load transfer;
- to facilitate construction without serving any structural purpose

Tabatabaie (1979) the primary mode of load transfer across a joint is by shear. Snyder (1988) as a wheel load approaches a joint or slab discontinuity, a portion of wheel load is transferred from one side of the joint to other by means of aggregate or “grain interlock” or mechanical devices like dowel bars. Joints are planes of weaknesses in concrete pavement systems. The stresses and deflections at joints should be of major concern to the pavements design engineer. According to (Kelleher, 1989) in terms of the economical point of view, the distance between joints should be long enough to minimise the number of load transfer devices but short enough to eliminate transverse cracking. Byrum (1994) indicated that joint opening is the key factor in stress distribution around the joint. CCAA (1999) a variety of joints such as, isolation joints, contraction joints, construction joints, expansion joints and longitudinal joints is used in pavement constructions.
2.2.5.1 Isolation joint

The isolation joint allows elements, which have been located on opposite sides of joints, to work independently and separately. It should be applied in all construction where movement or vibration of one structural element can strongly influence the structural behaviour of the adjacent elements. For instance, isolation joints are used in machinery foundations to isolate them from other construction parts because of vibration.

2.2.5.2 Contraction joint

The contraction joint is used particularly in concrete pavements to control the random drying shrinkage cracking by inducing the concrete slab to crack at the contraction joints. The induced stress due to contraction or expansion, which can generate a crack in the concrete pavement, will be relieved by allowing the pavement to move independently between contraction joints. The concrete slab is weakened in the contraction joint by forming or cutting a groove to ensure that shrinkage cracking occurs at the contraction joints. Concrete pavements are normally categorised according to configuration of transverse contraction joints in the concrete slab.
2.2.5.3 Construction joint

Construction joints are used to separate areas of concrete placed at different times. The construction joints may perform in the longitudinal and transverse directions of the traffic lane.

2.2.5.4 Expansion joint

Expansion joints are used in constructions where the ratio of length to width is high and effect of the depth can be ignored. The main reason for utilizing an expansion joints is to avoid failure due to thermal and moisture movement in the structural elements. Expansion joints are widely used in bridges and long length or width structures.

2.2.5.5 Longitudinal joints

Longitudinal joints are placed between paving lanes. Weakened plane joints are normally used at the centre of two or more pavement lanes when cast in a single pour. Such joints allow for the reduction of stresses induced due to temperature (curling), moisture (warping), and loading (vehicular). The horizontal movement of these joints is restrained by incorporating tie bars. Keyed joints or a combination of tie bar and keyed joints may be employed in the longitudinal joints depending on the subgrade and subbase strengths and density of vehicular loads passing along the pavement close to the longitudinal joints.
2.2.6 Load transfer devices

Load transfer across joints in a concrete pavement is accomplished mainly by: (a) aggregate interlock; (b) dowel bar action; or (c) combination of the two mechanisms. In the aggregate interlocking mechanism, the load is transferred by shear interaction between individual aggregates at a joint. This type of load transfer mechanism is effective for slabs having small joint width and short joint spacing. For a pavement with heavy traffic volume, mild steel dowel bars are placed across transverse joints to transfer the load. The dowel bars transfer load without restricting the horizontal joint movement caused by thermal and moisture contraction and expansion. They also help in maintaining the horizontal and vertical alignments of slabs. Dowel bars and tie bars are the most common mechanical devices that are widely used in pavement constructions compared with other mechanical devices such as double-vee shear devices and studded plates.

2.2.6.1 Dowel Load transfer system

Teller (1958), the first installation took place in the winter of 1917–1918 between two army camps near Newport News, Virginia, where four 19 mm diameter bars were used across the 6100 mm pavement width with 2 dowels per 3048 mm travel lane.

FHWA (1983), dowel plays an important role in transverse joints particularly when the pavement is not reinforced. Dowels are typically round with a diameter of 1/8 of concrete slab thickness and 350 mm to 460 mm long. Dowels are placed at mid-depth of the base.
with an even space of 300 mm centre to centre and perpendicularly to the transverse joints. Since one side of the dowel is always coated by a debonding layer, longitudinal movements of the concrete slabs on both sides of the transverse joint are not restrained. Using dowel bars across transverse joints as load transfer devices has been utilised for nearly 100 years.

Numerous dowel bar studies and tests were conducted by (Westergaard, 1928 and 1938), (Bradbury, 1932), (Teller, 1935, 1936, and 1943), and others. Repeated load testing of dowels in slabs performed at the Bureau of Public Roads labs in the 1950s led to the development of design recommendations that eventually became the standard in the United States in the 1960s and 1970s.

Teller (1958) concluded that the minimum embedment required to achieve maximum load transfer was found to be 8 dowel diameters for dowels up to 19 mm diameter. These recommendations were for dowels in expansion joints with widths up to 19 mm. The results also shown that decreasing the joint width would decrease the dowel bending, bearing stresses and deflections, resulting in much better structural performance.

Darter (1985) studies have shown a decreased joint faulting with larger dowel diameter; in fact this lowers dowel-concrete bearing stress. There have also been efforts to improve dowel bar design through the use of alternate shapes other than round to further reduce dowel-concrete bearing stresses.
Crovetti (1999) studied the constructability and potential cost-effectiveness of three kinds of dowel materials for a variety of slab thicknesses. The dowels used in the study were fibre reinforced polymer (FRP) composite dowels, solid stainless steel dowels and hollow-core mortar-filled stainless steel dowels. The results indicated:

- Load transfer efficiencies were reduced in all test sections particularly in the FRP composite dowel test sections.
- A general uniformity of load transferring among sections, showed by Ride quality surveys.

Eddie (2001) Glass Fibre Reinforced Polymer (GFRP) dowel bars are a possible maintenance-free alternative in corrosive environments. They potentially reduce the overall life-cycle-cost of pavements.

Harvey (2003) and Bischoff (2002) used a different Dowel Bar Retrofit (DBR) plans showed that joint performance is not affected by types of dowel bars. The DBR is a technique to rehabilitate jointed concrete pavements where joint faulting is the problem.

Byrum (1994) showed that there is an optimum dowel size depending on the applied load and boundary condition of concrete pavements.

Alireza (2013) introduced a new load transfer system called Hinged Dowel System (HDS), the following show the advantage of HDS over traditional dowel system
- Accommodates movements of the concrete pavement slabs due to thermal expansion and contraction as well as safe rotation at the joints,
- reduced potential for curling damage due to flexibility given to the slab to rotate near the joint area,
- reduction of shear stress in concrete slab by approximately 15 percent (finite element analysis showed),
- misalignment of dowel bars can be prevented,
- reduce the risk of shear cracking and joint failure in concrete pavement,

2.2.6.2 Dowel load transfer system designs

Guo (1996) reported that dowel bars transfer load through both shear and bending action. However, many researchers have shown that the primary load transfer mechanism is shear and bending action can be superfluous.

Circa (1920) reported the earliest dowel load transfer system designs were performed by “opinioneering” and later designs were developed based on combinations of analytical work and the experience gained from previous studies.

The following sections describe the factors that should be taken into consideration in an analysis of dowel load transfer systems.
2.2.6.3 Dowel diameter

Dowel diameter does strongly affect the behaviour and performance of the dowel joints pavement system. Increased dowel stiffness can reduce peak and differential deflections and reduces dowel-concrete bearing stresses. As a result, these will reduce the rate of development of joint faulting. Dowel diameter requirements may different depending on dowel spacing across a joint and are often significantly greater when lower modulus materials are used as dowels. The diameter of a round dowel also directly controls the primary aspects of the dowel’s structural capacity of shear, bending, and tension, although these are seldom of concern for typical design loads and steel dowels of the sizes normally used today.

2.2.6.4 Dowel bar length

Dowel bar length practices have evolved from those used in the late 1920 that typically featured the use of 19 mm dowels measuring 900 mm in length and spaced 450 mm to 900 mm apart. By the late 1930s, 610 mm dowel lengths were more common, and the benefits of using larger diameters and closer spacings were beginning to be recognized.

Timoshenko (1925) started an analytical design of pavement dowel, which developed the original analysis of dowel bars embedded in concrete by considering the dowel as semi-infinite length.
Friberg (1938) showed that the effect of cutting the dowel at the second point of contraflexure resulted in a net change in the maximum bearing pressure at the face of the concrete of less than 0.25 percent. Based on this finding, he concluded that dowel lengths could be further reduced.

The results of laboratory and field studies, including work begun at the Bureau of Public Roads in 1947, led the American Concrete Institute Committee 325 in 1956 to recommend the use of 450 mm long dowels spaced 300 mm apart. These recommendations were given for steel dowels between 19mm and 32mm in diameter used in pavements with thicknesses between 150 mm and 250 mm.

Timoshenko (1925) indicated that dowel lengths were originally selected to be long enough to ensure that the resulting bearing stresses at the joint face would be very close to values that would be obtained with dowels of semi-infinite length. They do not seem to be based on the results of data that relate dowel embedment length to dowel performance, although such data has been available since at least 1958.

Teller (1958) first published the results of the Bureau of Public Roads repeated shear load testing of full-scale pavement joints. The results of repeated load tests, determined that the length of dowel embedment required to develop maximum load transfer for 19 mm dowels could be achieved with an embedment of about 8 dowel diameters (150 mm) while 25 mm and 32 mm dowels required only 6 diameters of embedment (150 mm and
190 mm, respectively. Their test data suggest that even shorter embedment lengths (4
dowel diameters or less) may still result in acceptable performance.

Khazanovich (2009) performed a laboratory study of dowel misalignment conditions and
found that the shear capacities and relative displacements of 32 mm and 38 mm diameter
steel dowels were probably acceptable, even when embedment was reduced to 100 mm or
less.

Burnham (1999) evaluated the field performance and behaviour of several pavement
joints on a Minnesota concrete pavement where the joints were not sawed at the proper
locations, resulting in reduced embedment lengths. He concluded that “a minimum dowel
bar embedment length of 64 mm is needed to prevent significant faulting and maintain
reasonable load transfer efficiency across a joint.”

2.2.6.5 Dowel alignment requirements

ARA (2005) reported that most states have adopted the Federal Highway Administration-
recommended limits on dowel rotation of 6.35 mm of dowel bar length or two percent
noted by (FHWA, 1990). However, there was no evidence that this level of tolerance was
required to ensure good field performance. Poor dowel alignment does not necessarily
result in the development of slab cracking and spalling. If a single joint locks up, adjacent
joints may provide sufficient stress relief to prevent the development of distress.
ACPA (2005) noted that the number of consecutive joints that must lock to produce distress depends on many other factors, such as climate conditions, pavement structural design, concrete properties and restraint provided by the slab-subbase interface.

Fowler (1983) found similarly poor dowel alignment conditions on two comparable portions of I-20 located less than one mile apart from each other in Georgia, but a recent condition survey of those sections (ARA, 2005) found one in excellent condition while the other exhibited substantial cracking. ACPA, recommends limiting dowel rotational misalignment to three percent of the bar length based on NCHRP Synthesis 56 (ACPA, 1998; NCHRP, 1979). It is assumed that a properly designed and manufactured dowel basket will hold the dowels in positions that assure adequate rotational alignment and stability.

2.2.6.6 Vertical translation

Khazanovich (2009) analysed field performance data to compare faulting and load transfer efficiency (LTE) at joints with dowels centred within 6.35 mm of slab mid-depth with those of joints with dowels that were more than 25 mm closer to the pavement surface. They found no statistically significant differences in faulting and LTE between the two groups. They also performed laboratory tests of single dowels and conducted finite element analyses to examine the effects of concrete cover and dowel diameter on the shear capacity of the dowel-concrete system. Recalling the maximum design shear loads in the critical dowel, it is clear that significant vertical dowel translation will still
provide sufficient shear capacity for typical design load conditions. Khazanovich (2009) further suggest that concrete cover exceeding 3.5 times the dowel diameter provides no significant increase in shear capacity.

Odden (2003) conducted full-scale repeated load testing performed at the University of Minnesota confirmed that the reduction of performance associated with reduced dowel cover was minimal. Three epoxy-coated steel dowels were retrofit in the wheel paths of each of two 190 mm thick concrete slabs—at mid-depth in one slab and with 50 mm of cover in the other. The results, presents LTE measurements obtained over more than 10 million application of a 40 kN simulated wheel load, and shows that both installations provided good performance and exhibited similar rates of deterioration, although the shallow cover installation had slightly lower LTE values.

\[2.2.6.7 \textbf{Longitudinal translation}\]

Khazanovich (2009) also compared faulting and LTE data for joints with dowels that were placed with their centres within 12.5 mm of the joint versus those placed with more than 50 mm of longitudinal translation. They found no statistically significant differences in faulting and LTE between the two groups. In laboratory shear pull tests of dowels with varying amounts of embedment. Khazanovich (2009) also found no significant loss of shear capacity until embedment length fell to 100 mm and embedment lengths of as little as 50 mm provided shear capacity of more than 22.25 kN which is more than sufficient for the critical dowel under typical highway design conditions. However, the initial
stiffness of the dowel concrete system decreased by 60 percent or more when dowel embedment decreased to 76 mm or less, which would result in higher differential deflections and increased potential for pumping and faulting. Khazanovich (2009) also concluded that the combined effects of low concrete cover and low embedment length was greater than either of these two individual misalignment effects.

2.2.6.8 Dowel spacing and number of dowels

Dowel spacing of 300 mm has been standard practice since the 1950s and providing each dowel with sufficient shear capacity without creating a fracture plane along the line of dowels. There is evidence to suggest that spacing of 150 mm or less may result in the formation of a failure plane through the dowels.

Khazanovich (2009) recommended a misalignment problem practice by improved anchoring of the baskets; a more reliable solution is to place the outside dowel 230 mm to 300 mm from the pavement edge and longitudinal joint and can be shown to result in only a small increase in pavement corner stress.
2.2.6.9 Epoxy coatings

Most pavement dowels have been made primarily of carbon steel, which will readily corrode. Dowel corrosion can cause and accelerate the rate of development of several types of pavement distress. For instance, when dowel corrosion begins at the joint and progresses back into the adjacent slabs, the gap (looseness) between the concrete and dowels increases the effective width of the joint, slab deflections and stresses increase, and load transfer is reduced. A second corrosion-related distress mechanism is the expansion of corrosion products around the dowel, which can cause severe joint spalling or the formation and deterioration of mid-panel cracks.

Carbon steel dowels have typically been coated with grease, paint, epoxy, or plastic to inhibit corrosion. The epoxy provides a barrier between the steel and corrosive elements. Additional materials, such as grease or oil, are often applied to the epoxy coating to act as a bond breaker between the dowel and the concrete to facilitate horizontal joint movement in response to temperature and moisture changes.

Mancio (2008) found no significant difference in the degree of corrosion protection provided by either Standard Specification for Epoxy-Coated Steel Reinforcing Bars (ASTM) A934 or (ASTM) A775 of these types of epoxy in dowel bar applications. Epoxy coating of dowels is relatively inexpensive, and this treatment has been the most widely used corrosion protection treatment for dowel bars. However, factors such as environmental conditions, coating properties and durability, construction practices can
influence the long-term performance of epoxy coating. These types of coatings have sometimes proven unreliable for long performance periods in locations where de-icing salts are used, because small defects in the coating may provide a corrosion initiation site, reducing the dowel performance. Coating thickness specifications call for sufficient thickness that normal variability in coating does not result in areas with coating that are too thin. Standardization of these items will help to make the dowel manufacturing process more efficient and will improve the field performance of epoxy-coated dowels.

2.2.7 Load transfer efficiency (LTE)

The LTE of a jointed pavement is expressed as a percentage of the ratio of the maximum deflection of the unloaded slab ($\delta_{ul}$) to that of the loaded slab ($\delta_l$). That is

$$\text{LTE} = \frac{\delta_{ul}}{\delta_l} \times 100$$  \hspace{1cm} (2.19)

LTE depends upon a number of dowel-joint parameters such as diameter, length and spacing of dowels, modulus of dowel support, dowel looseness, concrete slab thickness, joint opening, and properties concrete and subgrade.

Khazanovich (2001) reported a comprehensive field study, deflection measurements carried out using a falling weight deflectometer over a period of time and seasonal variation.
Jeong (2001) expressed LTE in terms of total stiffness ratio (J factor), which is a function of dowel and aggregate interlocking parameters. Modulus of dowel support (K) and dowel looseness play important roles in the load transfer mechanism of a jointed pavement. Estimation of these parameters is generally done by back-calculating from the measured load transfer efficiencies of the joint. LTE is an important factor in performance of transverse joints and cracks.

Harvey (2003) reported that LTE did not change by changing the traffic volume on the sections reinforced with DBR. Furthermore, the LTE was less sensitive to temperature changes. In addition to the use of suitable dowels and the improvement of subgrade strength can improve the LTE as reported by (Hossain, 1996).

Ashish (2011) concluded that estimated LTE (measured by FWD) values from static loading were lower by 38% than those from dynamic loads.

2.2.8 Differential deflection

Differential deflection is another important factor, which provides further information on joint performance. It is the value of difference between the edges of adjacent slabs and presents the vertical distance between the loaded and unloaded slabs on the opposite sides of the joint. It is computed by using

\[ DD = \delta_L - \delta_U \]  

(2.20)
LTE does not correlate with the amount of differential deflection. Therefore, different values of the LTE can result in the same value of the DD. The differential deflection defines the sensitivity of the pavement to impact loads, applied at the edge of transverse joints of unloaded slab. The impact load results in further deterioration or joint faulting. Popehn (2003) stated that DD becomes an important factor when dynamic behaviour of rigid pavement is investigated.

2.3 Loading

Concrete pavement analyses are normally based on vehicular loading. Byrum (1994) showed that highway slabs are predominantly in the upward curled condition. Byrum (1994) and Hiller (2005) reported that environmental effects together with built-in temperature curl result in different failure types of concrete pavements.

2.3.1 Traffic loads

Austroads (2004), states that the traffic on the roads contains a wide range of vehicular loading from bicycles to triple road trains. It is known that under vehicular loading, the structural response of concrete pavements is affected by the axle configurations and magnitude of the applied wheel loading. Traffic wander upon the pavement has a significant effect on concrete pavements. Critical axle group locations upon pavement were determined in the past to capture maximum pavement response. Ongle (2004)
vehicular induced tensile stress occurs at the deepest surface layer of the concrete slab particularly when the load is applied at longitudinal joints.

### 2.3.1.1 Truck speed and load frequency

Speed and frequency of vehicular loads are two significant factors that may affect dynamic response of concrete pavements. Allowable truck speed varies from country to country. Achenbach (1975) showed that the critical truck speed is between 85 and 100 percent of propagation velocity of a transverse displacement wave through the pavement system. Maximum truck speed on the highways in Australia is restricted to 120 km/h. Gillespie (1993) showed that the load frequency is between 0 and 20 Hz based on frame bending vibration mode frequency for trailers and tractors and also reports that for trucks, load frequency is 4.6 Hz for a speed of 58 km/h and 6.5 Hz for a speed of 82 km/h. Monismith (1998) found that the load frequency is between 2 and 15 Hz based on a trucks’ suspension vibration frequency.

### 2.3.1.2 Traffic wander

Packard (1985) assumed that only 6 per cent of the traffic passes along the edge area of the traffic lane. The edge area is defined in a transverse distance of 600 mm from longitudinal joints or edges. AASHTO (2003) showed that the number of transverse cracking in the pavement is rapidly increased by a decrease in distance between outer tyre edge and pavement edge (longitudinal joints) for truck passing along the pavement.
Lennie (2005) studies results showed the volume of the traffic passing along the edge area in the state of Queensland is much higher than (Packard, 1985) assumed.

2.3.1.3 Location of axle group on pavement

Packard (1985) discovered that a confined single lane pavement under a single and tandem axle loads showed that the critical location of axle groups upon pavement is at the middle of longitudinal joint between transverse joints to capture maximum tensile stress and next to the corner to measure the maximum slab deflection in jointed concrete pavements. Consideration of a confined traffic lane was to reduce the possibility of erosion of subbase and subgrade materials. Based on these results, the maximum slab deflection and the maximum induced tensile stress in concrete pavements are calculated by applying the vehicular loads at the corner and at the middle of the longitudinal joint respectively. This consideration is to prevent the bottom-up transverse fatigue cracking in the pavement. AASHTO (2003) implemented research to determine effects of adjacent concrete slab panels and traffic lanes on the loaded concrete slab using finite element analysis (FEA). Results indicated that adjacent traffic lane has no effect on the maximum induced tensile stress of the loaded concrete slab. Consideration of different concrete slabs in the longitudinal direction has also no major effect on the maximum induced tensile stress. Since results of the FEA on a single concrete slab was highly sensitive to a change in thickness of concrete slab, temperature fluctuation and other pavement characteristics, AASHTO (2003) also recommends the use of a single lane with three
concrete slabs in the longitudinal direction. This recommendation is compatible with that considered by (Packard, 1985).

2.3.1.4 Axle group types

A single axle load with dual tyres was employed in the work of (Tayabji, 1983), (Packard, 1983), (Tayabji, 1983), (Smith, 1990), (Yu, 1997), (Shoukry, 2007), (Minkwan, 2008), (Swati, 2009) and (AASHTO, 2003). Gillespie (1993) and William (2000) used the Equivalent Single Axle Loading (ESAL). ESAL was based on load characteristics derived from single axle dual tyres. The effect of tandem axle loads on concrete pavement response was investigated by (Tayabji, 1983), (Packard, 1983), (Minkwan, 2008) and (AASHTO, 2003). Triple axle loads were used in the work of (Packard, 1985) and (AASHTO, 2003).

Austroads (2004) adopted the PCA method to suit for Australian conditions. The PCA method was extended to consider different types of axle groups including Single Axle Single Tyre (SAST), Single Axle Dual Tyre (SADT), Tandem Axle Single Tyre (TAST), Tandem Axle Dual Tyre (TADT), Triple Axle Dual Tyre (TRDT), and Quad Axle Dual Tyre (QADT).

![Axle group types considered in Austroads 2004b](image-url)

*Figure 2.1: Axle group types considered in Austroads 2004b*
2.3.1.5 Suspension system

Gillespie (1993) stated that depending on the level of pavement roughness, structural performance of concrete pavements under moving vehicular loading is significantly affected by suspension type of a given axle group.

Blanksby (2006) conducted a survey study of suspension systems, including air and mechanical suspensions in triple axle groups used by heavy vehicles passing along the Hume Highway in Australia. His finding showed that the possibility of non-uniform load distribution between axles in a given axle group, i.e. TRDT, is high particularly in the presence of the mechanical suspensions. The mean percentage of load shift between axles is about 1 to 2 per cent in the air suspension and between 20 to 40 per cent in the mechanical suspensions.

2.3.1.6 Distance between axles in a given axle group

Packard (1985) used a distance of 1220 mm to 1372 mm between axles in a triple axle group and 1270 mm for distance between axles in a tandem axle group in his research study. Gillespie (1993) and AASHTO (2003) recommended a distance of 1295.4 mm between axles to capture the critical pavement response. Kim (2002) and Hiller (2002) considered 1320 mm for the same purpose in their research works. RTA (1998) Road and Traffic Authority of Australia allow a variation between 1000 mm and 1600 mm for the triple axle group and between 1067 mm and 1633 mm for the quad axle group.
2.3.1.7 Axle width and space between dual tyres

Lee (2001) adopted Packard (1985) by assumed the distance between the centres of dual tyres and axle width as 305 mm and 1829 mm, respectively. William (2000) considered them to be 205 mm x 1520 mm in his FE model. Kim (2002) considered these distances to be 330 mm and 1880 mm in his research study. Hiller (2002) assumed them to be 340 mm and 1850 mm, respectively in his study analysis. Shoukry (2007) considered them to be 305 mm and 1815 mm, respectively in his FE model. Minkwan (2008) used the distance 343 mm and 1562 mm in his FE model.

AASHTO (2003) considered a distance of 305 mm between the centres of dual tyres but increased the axle width to 2134 mm. Austroads (2004b) recommends 330 mm and 1800 mm for distance between the centres of dual wheels and axle width, respectively. Figure 2.2 summarises the variations of axle width and distance between the centres of dual tyres according to various sources.

![Diagram](image)

Figure 2.2: Transverse section of a typical SADT (Austroads, 2004b)
2.3.1.8 Tyre-Pavement contact stress

Lippmann (1985); De Beer (1997); and Douglas (2000) investigated different types of tyre pressure and reported that contact stress is not uniform particularly when the tyre inflation pressure is low.

Marshek (1986) and Gillespie (1993) results also showed that tyre-pavement contact stress is not uniform, but a change in the distribution of tyre contact stress has no significant effect on concrete pavement response. To simplify the analysis procedure, tyre contact stress is, hence, assumed to be uniform in concrete pavements.

2.3.1.9 Tyre-Pavement interface shape

Handson (1988) stated that the tyre pavement contact area is more rectangular than circular. Gillespie (1993) also claimed that tyre imprint is not a rectangle; however, the assumption of rectangular tyre contact area does not significantly change the responses of the concrete pavements.

Shackel (1993) showed that tyre imprint has an elliptical shape but to simplify the analysis process, he recommended the use of a rectangular shape of an equivalent area. A variety of dimensions for tyre imprint have been used despite researchers having considered a rectangular tyre-pavement interface.
Packard (1985) considered a rectangular tyre contact shape of 178 mm × 254 mm. Handson (1988) showed that width and length of tyre imprint in an 11R24.5 tyre are 196 mm and 250 mm respectively. Gillespie (1993) suggested using a square shape of 203 mm × 203 mm for dual tyres and a rectangular shape of 203 mm × 229 mm for single tyres.

Douglas (2000) provide a graphic interrelationship between contact patch length and wheel load based on an experimental study of different tyre inflation pressures for radial tyres. The result of this study showed that an increase in tyre pressure decreases the contact length. However, the decrease in tyre contact length is relatively small when tyre pressure exceeds 480 kPa. The contact patches length for a wheel load of 22.5kN decreases from 240 to 230 mm when tyre pressure increases from 480 kPa to 690 kPa.

Kim (2002) used a rectangular shape of 203 mm × 178 mm in his research analysis model. Swati (2009) used a rectangular shape of 160 mm x 234.4 mm in his model.
2.3.2 Shrinkage - Loss of moisture content

Ongel (2004) the top layer of the concrete slab is exposed to solar radiation and wind, it dries and cures faster than other layers within the depth of the concrete slab and consequently results in non-uniform shrinkage which is the reason for concrete slab warping and top-down cracking.

Reddy (1963) recommended the use of equivalent night time temperature gradients between 0.065 and 0.13°C / mm in concrete pavement analysis to represent the effects of drying shrinkage in concrete pavement responses. In fact, the effect of drying shrinkage on concrete pavement is similar to effects of night time differential temperature.
Rasmussen (1998) states that shrinkage is assumed to linearly decrease from the top surface layer of the concrete slab towards the mid-depth of the concrete slab. In fact, a full shrinkage occurs at the top surface layer and no shrinkage exists below the mid depth of the concrete slab.

Rania (2011) drying shrinkage at the slab surface was observed to increase with time during the 2-year period of his study and the rate of increase in slab curvature attributable to drying shrinkage is larger for the unrestrained slabs than for the restrained slabs. Some of this drying shrinkage is reversible, as can be seen by the seasonal fluctuations in slab curvature, with shrinkage and slab curvature decreasing during wet seasons and increasing again during drier seasons. Restraint provided by the dowel and tie bars have a substantial effect on reducing slab curvature attributable to long-term drying shrinkage.

2.4 Concrete pavement analysis

2.4.1 Analytical approach

2.4.1.1 Analytical solution

Westergaard (1926, 1927, 1933, 1939, 1943 and 1947) used the classical thin-plate based theoretical models to analyse rigid pavement. Westergaard assumed the concrete pavement as a homogenous, isotropic, elastic, thin slab resting on a Winkler foundation.
Three most critical loading positions such as the interior, edge, and corner have been identified in his study. Westergaard made the following simplifying assumptions in his analysis,

- The foundation acts like a bed of springs (dense liquid foundation model);
- There is full contact between the slab and foundation;
- All forces act normally to the surface where shear and frictional forces are negligible;
- The semi-infinite foundation has no rigid bottom;
- The slab is of uniform thickness, and the neutral axis is at its mid-depth;
- The load is distributed uniformly over a circular contact area;
- For corner loading, the circumference of the circular area is tangential to the edge of the slab;
- The concrete pavement acts as single semi-ininitely large, homogenous, isotropic elastic slab with no discontinuities.

Westergaard theory has following limitations,

- Stresses and deflections can be analyse only for the interior, edge and corner loading conditions;
- Shear and frictional forces on slab surface are ignored;
- The Winkler foundation extends only to the edge of the slab;
- The theory does not account for unsupported areas resulting from voids or discontinuities;
- Multiple wheel loads cannot be analyse;
• Load transfer between joints or cracks is not considered.

The thin plate based theoretical models for structural analysis of concrete pavement did not develop much further after Westergaard findings.

Hogg (1938) assumed the subgrade as a semi-infinite elastic-solid and then developed an analytical model for determining the stresses and deflections of a concrete slab under the action of a single load by using the elastic properties of subgrade. Bradbury (1938) developed an analytical solution or analysing of concrete pavements with discontinuities subjected to differential temperature gradients. Hence the Westergaard method can not be used in concrete pavements with discontinuities.

Reissner (1945 and 1950) developed a thick-plate theory to analyse two problems: (1) the problem of torsion of a rectangular plate, and (2) the problems of plain bending and pure twisting of an infinite plate with a circular hole. His theory is regarded as stress based shear deformable theory as it is based on assumed stress variation through the plate thickness.

Pickett (1951) modified Westergaard’s solution for the design of concrete pavement by using influence chart. Because of the complexities of the mathematics involved this model did not received much attention from the researcher. Mindlin (1951) proposed another formulation to account for shear deformation based on a proposed displacement field through the plate thickness.
Hu (1981) further extended Reissner’s theory and developed a set of basic equations for thick plates that are simpler to solve than the Reissner’s equations. Ioannides (1985) revised the original Westergaard equations for edge loading cases in his research study. Huang (1993) developed an equation to convert effects of dual tyres to single tyre with circular tyre pavement contact shape. Hence all Westergaard’s equations are based on single tyre with a circular tyre pavement contact area.

Shi (1994) developed a theoretical solution to the problem of a rectangular thick plate with four free edges and supported on Pasternak foundation. The Fundamental equations for the problem were established by applying Reissner thick-plate theory and solved by applying the method of superposition.

Fwa (1996) further extended this solution into analysis of concrete pavement and found differences existed in both stresses and deflections between thick-plate solutions and Westergaard’s solutions. Since no discontinuities were considered in the Westergaard method, effects of the LTE at joints or cracks on pavement performance were not addressed.

Mohamed (1997) developed an analytical method for estimating the curling induced stresses in the concrete pavement based on nonlinear temperature gradients. Ioannides (1998) adopted the plate theory; therefore the concept of equivalent temperature distributions was then introduced in concrete pavement analysis. A plate consisting of one or more layers resting on an elastic foundation was investigated. The layers used in
his study were plate layers with no separation and compressible layers with possible separation using the Totsky model. Consequently, mathematical formulations for analysis of a typical concrete pavement subjected to a linear function, a quadratic function or multi linear function of differential temperature together with arbitrary wheel load were developed.

2.4.1.2 Analytical studies of dowel stresses

Bradbury (1932) was the first attempt to calculate the compressive bearing stress induced by a dowel bar. By adopting Timoshenko’s (1925) analysis, assuming the dowel was to be an infinite beam and concrete slab resting on Winkler foundation. Bradbury developed formular to calculate the dowel length required for allowable shear, bending and bearing stresses. The analysis developed by Timoshenko stated that the general expression for the deflection of the bar gives a wave curve having gradually diminishing amplitude as the distance from the applied load increases. Assuming that the supporting medium is an elastic material, it would follow that the intensity of pressure on the bar at any point is proportional to the deflection at that point. The bearing stress on concrete ($f_c$) was given by Bradbury

$$f_c = \frac{25p(\ell + 1.5z)}{2\ell^2d}$$

(2.21)

Where: $P$ = the shear on the bar,

$\ell$ = the total embedded length of the dowel,
Friberg (1938) was the earliest researcher addressed the dowel/concrete interface stresses in considering dowel groups. Based on Timoshenko’s analysis, Friberg indicated that the maximum deformation of concrete under the dowel ($y_o$) could be expressed as:

$$y_o = \frac{P_t}{4\beta^3EI} (2 + \beta z)$$  \hspace{1cm} (2.22)

Where:  \[\begin{align*}
P_t &= \text{the shear force on the dowel,} \\
w &= \text{the joint width,} \\
E &= \text{Young’s modulus of the dowel,} \\
I &= \text{the moment of inertia of the dowel bar, and} \\
\beta &= \text{the relative stiffness of a dowel embedded in concrete expressed as:}
\end{align*}\]

Where $K$ is the modulus of dowel support, Tabatabaie (1978) suggested $K$ values should be ranging from $3 \times 10^5$ pci to $32 \times 10^6$ pci. A typical value of $1.5 \times 10^6$ pci is commonly used by researchers in their analysis. $d$ is the diameter of the dowel.

$$\beta = 4\sqrt{\frac{Kb}{4EI}}$$  \hspace{1cm} (2.23)
Figure 2.4: Basic relationship for dowel stresses, developed by (Timoshenko, 1925), pressure exerted on a loaded dowel

This relation takes the form:

\[ \sigma = ky \]  \quad (2.24) 

Where: 
\( \sigma \) = the stresses in concrete,
\( y \) = the deflection of the dowel.

\[ Y = \frac{e^{-\beta x}}{2\beta^2 EI} [P_t \cos \beta x - \beta M_0 (\cos \beta x - \sin \beta x)] \]

\( M_0 \) = bending moment on dowel at face of concrete = 0.5wP_t

\( w \) = width of joint opening

The computed value for bearing stress should then be compared to the allowable bearing stress of the concrete (f_b) recommended by the American Concrete Institute (ACI) subcommittee 325 where:
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\[ f_b = \left( \frac{4 - d}{3} \right) f'_c \]  

(2.25)

In which: \( f'_c \) = the ultimate compressive strength of concrete in (psi).

Friberg also found that the maximum negative moment in the slab for both interior and edge loadings occur at a distance of 1.8 \( \ell_r \) from the loaded dowel where \( \ell_r \) the radius of relative stiffness defined by is:

\[ \ell_r = \sqrt[4]{\frac{E_c h^3}{12(1 - \mu^2)k}} \]  

(2.26)

Where: \( \ell_r \) = Radius of relative stiffness (mm)

- \( E_c \) = elastic modulus of concrete pavement in MPa
- \( h \) = thickness of concrete pavement in mm.
- \( \mu \) = Poisson’s ratio of concrete pavement
- \( K \) = modulus of subgrade reaction in MPa/mm

Tabatabaie (1979) conducted a factorial analysis using two dimensional finite element (2DFE) program ILLI-SLAB by varying slab thickness, subgrade k-value, load positioning, and joint width opening. In his study, Tabatabaie proposed the following formulation to determine the maximum bearing stress induced in concrete:

\[ \Sigma_{\text{max}} = \gamma \left( \frac{800 + 0.68E}{D^4} \right) \left( 1 + 0.355J_0 \right) sP \]  

(2.27)
Where: $P =$ applied wheel load,

$J_o =$ the width of joint opening,

$E =$ the modulus of elasticity of the dowel bar,

$D =$ the diameter of the dowel bar,

$\Gamma =$ load location coefficient

$= 0.0091$ for edge loads

$= 0.0116$ for protected corner loads

$= 0.0163$ for unprotected corner loads

$s =$ dowel spacing.

The critical bearing stress equation established by (Friberg, 1938) was modified by (Ioannides, 1990) coupling earlier theoretical investigations and observations from collected results from finite element studies, as following:

$$
\sigma_b = \frac{K(2 + \beta \omega)}{4 \beta^3 EI} xP_T xTLE x f_{dc}
$$

Where: TLE = Transferred Load Efficiency = $P_T / P_t$, typical assumption is 45%

$P_T =$ the total load transferred from the loaded to the unloaded side of the joint along its entire length,

$P_t =$ the total externally applied load,

$f_{dc} =$ the portion of a load carried by the dowel that is subjected to the largest shearing force,

$f_{dc} = s/e$ for edge loading, and

$f_{dc} = 2s/(e+s)$ for corner loading,
\[ e = \text{the effective length (the length along which the dowels are effective to transfer the load)} = 1.0 \ell_r, \text{ and} \]
\[ s = \text{the dowel spacing}. \]

2.4.2 Numerical approach-FEM (past and current FEM techniques)

2.4.2.1 Background

It is quite a prohibitive task to obtain analytical closed-form solutions for concrete pavement structures because of complexities associated with geometry, boundary conditions, and material properties. With the rapid growth in computer capabilities, the analysis of such complex problems using numerical technique is possible. The most commonly used numerical techniques for analysing concrete pavement structures are: (1) discrete element method (DEM), (2) finite element method (FEM)

Discrete element method (DEM)

Hudson (1966) was the first application of DEM for concrete pavement analysis. The subgrade was idealized as a Winkler foundation. The effects of joints in this model were taken into consideration by reducing the original bending stiffness of the slab at those locations where a joint existed.
Vora (1970) modified the (Hudson, 1966) model by including element of different sizes, anisotropic skew slabs, and semi infinite elastic solid subgrade. The major disadvantages of DEM formulations are that elements of varying sizes are not easily incorporated into the analysis, and that special treatment is needed at the free edge where stresses cannot be determined uniquely.

**Finite element method (FEM)**

David’s (1999) finite element method was first employed to model the response of rigid pavements in the early 1970. Hrenikoff (1941) was the first researcher to develop a concept of finite element technics for aircraft analysis using truss and beam elements. Courant (1943) solved the torsion problem in analyse and the use of triangular elements was then incorporated into the FE method. Further research on finite element technics by (Argyris, 1960) and (Turner et. al., 1956) represented the new version of FE method. The term “finite element” was used by Clough in 1960. The concept of (Reddy, 1993) finite element techniques are extensively used to model complex engineering problems in several areas such as structural, mechanical, electrical, geological and thermal.

Finite element analysis packages can be divided into two categories, general purpose finite element programs such as ABAQUS (Zhou, 2011) and ANSYS (Swati 2009), which analyse nonlinear dynamic problem and specific finite element programs developed for concrete pavement analysis using the classical thin plate theory or three dimentional (3D) solid elements. EVER.FE (Davids, 1999), this three dimensional finite
element analysis software was jointly developed by the universities of Maine and Washington to simulate the behaviour of jointed plain concrete pavements under axle group loads and environmental effects.


2.4.2.2 Simulation of doweled joints

Four primary approaches to modelling doweled joints have been reported in the literature:

(1) Tabatabaie (1978), Cook (1989), Zhang (1994), and Bathe (1996) employed Timoshenko beam elements (also known as bar elements) directly connected to plate elements;

(2) Huang (1973 and 1985), Tia (1987) and Mahboub (2004) employed elastic spring elements directly connected to plate elements;


Approaches (1), (2) and (3) have been widely used in the pavement community with various classical two dimensional (2D) finite element (FE) analysis program (resting on elastic or Winkler foundation). Approach (4) has been introduced to the pavement community recently.

Approach (1) was the first attempt to simulate the behaviour of doweled joints. The main purpose of the dowel is to transfer the wheel load to the adjacent slab through shear force. Due to the importance of shear deformation, Timoshenko beam elements are used to simulate the behaviour of the dowels. In this approach, a beam element directly connects Kirchhoff plate elements belonging to two adjacent concrete slabs. The exposed part of the dowel has a length to depth ratio less than 1.0, because the dowel diameter is often larger than the joint width. Using Timoshenko beam elements may be a suitable approach to simulate the behaviour of an intact doweled joint, but not a damaged doweled joint. Damaged dowel jointed pavement often involves the dowel casing through the phenomenon called dowel looseness (DL). In this circumstance, the concrete slab no longer provides strong support for the embedded portion of the dowel. Such a casing failure begins at joint and gradually propagates inside the concrete slab. As a result, load transfer efficiency will decrease and significant increases in both slab and soil stresses can be expected.
Bathe (1996) and Cook (1989) employed Timoshenko beam elements to simulate dowels. They were directly connected to continuum solid elements, which simulate the concrete slabs, for intact joint. Zhang (1994) used the same approach that adopted in 2D plain strain analysis for rigid highway pavement in the MN-ROAD project. The entire length of the dowel is simulated by seven beam elements so that the rotation field can be adequately resolved. Therefore, the load transfer action does not include artificial springs. The dowel shear force is directly transferred to concrete slab.

In approach (2), the dowels are simulated by elastic spring elements directly connected to plate elements over the joint. Therefore, the dowel cannot resist bending. The amount of transferred shear force is determined by

\[ V = K \Delta \]  

(2.29)

Where \( V \) is the dowel shear force, \( K \) is the spring constant and \( \Delta \) is the relative displacement between loaded and adjacent concrete slabs.

Huang (1973 and 1985), dowel bars were modelled as linear elastic spring elements placed at the joint directly connecting the adjacent slabs. The stiffness of joint was represented by a shear spring constant. Tia (1987) modelled the moment transferability of a dowel bar by introducing a series of shear and torsional springs across the joint. Mahboub (2004) used nonlinear springs to model the dowel bars.

Approach (3) uses Timoshenko beam elements to simulate the dowels, but they are indirectly connected to plate elements by using elastic springs. Approach (2) and (3) can
simulate dowle looseness through the elastic deformation of spring elements. However, the behaviour of the doweled joint is dominated more by the artificial spring constant than it is by the mechanical properties of the dowel and concrete slab. The contact force acting between the beam (dowel) and plate elements (slab) is determined by the artificial elastic spring constant. Further, these approaches always require calibration of the artificial spring constants with FWD test measurements. Often, calibrated spring constants show a wide range of variation, from 21 to 10000 kPa (Ioannides, 1992).

Huang (1973) and Tabatabaie (1980) stated that the embedded portion of the dowel bar were not modelled and the spring constants were derived by considering the embedded portion of the dowel as an infinitely long beam resting on a Winkler foundation. Tabatabaie (1980) modelled dowel bar as an elastic beam element across the joint. In model, the relative deformation of the dowel bar and surrounding concrete was represented by the stiffness of a vertical spring connecting the two. Tayabji (1986) employing 2D finite element programs that rely on medium thick plate elements to discretize the slabs have typical modelled dowel bars as discrete beam elements spanning between adjacent slabs. To account for the effect of dowel slab interaction, springs are placed between the dowels and slabs, the spring constants are derived by considering the embedded portion of the dowel as an infinitely long beam resting on a winkler foundation.

Ioannides (1992) developed a technique where a uniform joint stiffness is used to replace the individual dowels. The joint stiffness is a function of effective modulus of dowel
support, which can be back calculated from measured load transfer efficiencies. Zaman (1995) employing dynamic 2D finite element analysis explicitly considered dowel looseness and reported small gaps ($\leq 0.2$ mm) were found to have a large effect on pavement response.

Guo (1995) modified Nishizawa’s model ideal, with new model consisting of two bending beams embedded in concrete and connected by a shear bending beam. Dowel concrete interaction was modelled by providing suitable stiffness to the springs connecting the dowel bar and concrete within the slab. DL assumed to be uniform for all dowels, was captured by using a bilinear stiffness model (Guo, 1994). It was also assumed that under load, all dowels would come into contact with the surrounding concrete at the joint face at the same time. Advancement to this model was made by the introduction of contact elements at all possible contact points between the dowel and the concrete (Zaman, 1995) to account for the effect of dowel-concrete interaction along the dowel length and to model DL. Dowel bars were modelled as bending elements.

Ioannides (1992) and Brill (2000), a non-dimensional composite joint stiffness was introduced to represent the characteristics of a joint having both the dowel bar system and the aggregate interlocking load transfer mechanism. Nishizawa (2001) developed a model simulating dowel bar by considering its finite length. The two segments embedded in the concrete were modelled as bending beams of finite length in an elastic medium. The middle dowel bar segment was modelled by a standard bending beam. For very small joints, the middle segment was ignored.
Approach (4) is suitable for simulation of both intact and damaged dowel joints because it uses continuum solid elements for both the dowel and concrete slab. It simulates their interaction through frictional contact. The detailed stress and strain distribution within a dowel and the interaction between the dowel and the concrete slab can be observed from this approach. Furthermore, pavement damage can be simulated by using plastic constitutive models for the concrete in the vicinity of the dowel or by specifying the funnel geometry at the outset. A much finer mesh is necessary to simulate the dowel and concrete casting, while a coarser mesh is adequate to model the far field behaviour of the concrete slab and other parts of pavements.

Channakeshava (1993) modelled dowel bars as 3D beam elements. Dowel-concrete interaction effect was captured by appropriately selecting the stiffness of the springs around the dowels. The joint portion was analysed separately using refined mesh. The stiffness of the springs was selected based on experimental observations.

Crovetti (1994 and 1996), and Hammons (1995) based on the theory assumed the following equation in order to analyse the concrete pavements:

\[
\delta_L + \delta_U = \delta_E \tag{2.30}
\]

\[
\sigma_L + \sigma_U = \sigma_E \tag{2.31}
\]
Where $\sigma$ and $\delta$ are induced stress and deflection respectively, and the subscripts $L$, $U$, and $E$ are used for loaded slab, unloaded slab, and at the edge of joint when the joint load transfer capability is zero.

Guo (2003) showed that the use of the above assumption results in a good approximation provided that the slab has full contact with the subbase. Dowel pavement interaction is another significant factor affecting concrete pavement responses as dowel looseness decreases the magnitude of the LTE at the transverse joints (Zaman, 1995) and leads to joint faulting associated with erosion of subbase and subgrade materials. The main reason behind dowel looseness is the fatigue phenomenon in concrete due to high induced bearing stress at dowel-pavement interface caused by repeated loads (Channakeshava, 1993 and Zaman, 1995). A contact element (Zaman, 1995) or gap element (Channakeshava, 1993) between dowel bar and concrete can be used to simulate effect of dowel looseness on concrete pavements. Consequently, the concrete slab under the load can deflect equally in correspondence to dowel looseness before transferring the load to adjacent concrete slab.

Hammons (1997) employed the effective joint stiffness concept of (Ioannides, 1992) in 3D finite element models of rigid pavements, alleviating the difficulty of meshing bending and solid elements. However, uniform dowel spacing is implicit in this approach. This is common for newly constructed pavement but rarely for retrofitted joints where dowels are typically located in the wheelpaths (Hall, 1993).
Bhattacharya (2000) modelled dowel bars by three-nodal beam-column elements. To represent the interaction between the dowel bar and the concrete in the unloaded portion of the slab, the beam-column elements were connected to the surrounding concrete by a series of linear spring elements.

Davids (2000) employed 3D finite element analysis of jointed concrete pavements with an emphasis on the effect of gaps between the dowels and slabs on pavement response. His finite element modelling of dowel load transfer is briefly presented and relies on an embedded formulation of a quadratic beam element (Davids, 1997). It allows for the efficient and rigorous consideration of dowel looseness. The element formulation is then extended to permit the inclusion of a nonlinear, 3D bond slip law between the dowels and the slabs.

The two-dimensional (2D) FE model developed by (Nishizawa, 1989) was modified later by (Nishizawa, 2001) with the development of a 3D combination element to represent the dowel bar. The segments embedded in the concrete were represented by solid elements while the middle portion, the segment between the two slabs was represented by 3D beam elements.

William (2001) presented a new approach by representing dowel bars as eight-noded solid brick elements. The modelling requires very fine mesh in order to account for the mechanism of dowel contact with the surrounding concrete. Dowel-concrete interaction was modelled by considering friction between the two. Recently, embedded beam
element formulation was used for modelling dowel bars in the analysis of concrete pavement (Davids, 2003 and Kim, 2003). The embedded element formulation has the advantage of allowing dowels to be precisely located within the slab irrespective of the slab meshing. Dowels were modelled as quadratic beam elements.

In the model proposed by (Davids, 2000), dowel-concrete interaction was captured either by specifying DL around the dowels or by providing springs sandwiched between the dowel and the concrete. Dere (2006) showed that the beam elements were connected with the surrounding concrete by vertical and horizontal springs both on the loaded and the unloaded sides. However, DL was not modelled.

Swati (2009) employed 3D finite element to model the mechanism of load transfer by dowel bar system. Eight noded 3D brick elements, with three translational degrees of freedom per node were used to model a concrete slab as well as the base support layers. The subgrade was modelled as Winkler foundation. The concrete slab and the base layers were assumed to be linear, elastic and isotropic. Dowel bars were modelled as 3D beam elements having six degrees of freedom per node. The interaction between the dowel and the concrete, modulus of dowel support (K) was modelled using contact elements. The dowel concrete friction value considered was 0.05 (William, 2001). The effect of DL was modelled by providing a gap around the dowel bar within these contact elements.
2.4.2.3 Mechanism of load transfer by dowel bar system

Dowels bars transfer the load across a pavement joint primarily by shear action. When one panel of the pavement is loaded, the panel is deflected along with the dowels connecting the loaded panel with an adjacent panel and in the process the dowels transfer part of the load to the unloaded panel.

The load transfer mechanism between the dowel and the concrete is a complex phenomenon. This mechanism depends mainly on a parameter known as the modulus of dowel support (K), the value of which can be determined by load testing (Yoder, 1975). A high modulus of dowel support value indicates a good contact between the surrounding concrete and the steel dowel. However, with repeated application of wheel loads, the contact between the surrounding concrete and the dowel deteriorates, particularly where the bar is seated and in the vicinity of the face of the joint. At these locations, the concrete may be crushed over time under repeated loading when subjected to high bearing stresses. As the crushed concrete particles are displaced, voids are created around the dowels causing dowel looseness (DL). The amount of looseness may vary along the length of the dowel bars. However, near the joint face, the looseness is generally more than at the other locations of the dowel bar. DL is generally composed of two parts-initial looseness and looseness from enlargement of the socket under repetitive loading (Bush, 1996).
In real life pavement, looseness in dowels may occur on any side of the joint due to repeated application of wheel loads on both sides of the pavement. The efficiency of a joint in transferring the applied wheel load depends on a number of dowel-joint parameters like modulus of dowel support, dowel diameter, embedded length of dowel, dowel spacing, DL, joint opening, properties of both steel and concrete, and also to a lesser extent on sub grade strength. In experimental work reported by (Guo, 1995) on the performance of dowels under repetitive loading, it was observed that DL, produced by an imperfect fit or void between the dowel and the surrounding concrete (initial looseness), greatly affects load transfer efficiency (LTE), maximum deflection, critical stresses, and the rate of pavement deterioration. A deteriorated joint under repetitive loading ultimately leads to erosion under the dowel. In another experimental study performed by (Buch, 1996), it was found that DL is affected by the type, texture, and shape of the aggregate particles used in the concrete slab, the bearing stress of concrete, and by the load magnitude and the number of load cycles.

The dowel bars placed across a joint act as a group to transmit the applied wheel loads to the adjacent slab. The dowel bar placed immediately below the load takes the major portion of the applied load with the adjacent dowels transferring progressively smaller loads. Friberg (1938) was the first to analyse the group action of the dowel bars based on Westergaard’s work (Yoder, 1975). The analysis showed that for a group of dowel bars, the maximum load is transferred by the dowel immediately below the load and the load transferred by other dowels decreases linearly up to a distance of \( 1.8 \ell_r \) from the maximum loaded dowel, where “\( \ell_r \)” is the radius of relative stiffness (Westergaard,
In another study by (Tabatabaie, 1980) reported by (Kelleher, 1989), only the dowels within a distance of \(1.0 \ell_r\) from the centre of the load were considered to be effective in transferring the major part of the load and a linear approximation was made to estimate the individual dowel shear forces.

Guo (1996) studied the dowel group action and concluded that the ratio of the distance up to which the dowels are effective in transferring the applied wheel load, termed as equivalent effective length (EEL) to radius of relative stiffness \((\ell_r)\), is not a constant but depends on some of the dowel parameters. Bhattacharya (2000) and Kim (2003) using the 3D FEM showed that the distribution of dowel shears is parabolic in shape.

### 2.4.2.4 Model of debonding layer

Totsky (1981) has contributed to finite element analysis of concrete pavements with problems associated with bonded and unbonded boundary conditions. The multilayered pavement system was modelled as a series of springs and plates. While the plate elements model the bending, the springs accommodate the direct compression between layers. Khazanovich (1998) developed the method of stiffness calculation.

Davids (2003) showed that the debonding element is capable of transferring the shear stress along the interface of the concrete slab and the subbase. It is meshed in accordance with the meshing size of the concrete slab and the subbase. A bilinear constitutive relationship was considered to define the characteristics of this element under the applied
loads. The debonding layer can be defined by introducing initial distributed stiffness and slip displacement. A free separation under tension occurs between the concrete slab and subbase when unbonded boundary condition is selected.

2.4.2.5 Static analysis

Analysis of concrete pavements under the applied loads was widely performed in the past based on static vehicular loading. Pavement configurations and properties of each layer associated with magnitude, configuration and position of vehicular loading were the main factors governing concrete pavement responses. Vehicular loading was considered as wheel loads, axle loads or axle group loads positioned at the transverse joint or at the corner of the concrete slab.

2.5 Concrete pavement distresses

Concrete pavement distresses can be related to the fatigue of the concrete slab or erosion of the subbase and subgrade materials. Since erosion of the subbase and subgrade materials is one of the most significant distresses in jointed concrete pavements, some fundamental information on erosion of the subbase and subgrade materials are highlighted.
2.5.1 Fatigue damage of concrete slab

Deterioration and delamination defect in concrete pavements can be considered as a tensile failure. Cracks can occur at any location within the pavement where tensile stresses exceed the concrete flexural strength. Tensile stresses are generally induced in a concrete pavement due to the bending action of the concrete slab under repetitive vehicular loading.

Fatigue of the concrete slabs may result in transverse, corner, longitudinal or punch-out cracking. Cracks are formed from the top surface layer towards the bottom surface layer of the slab (top-down cracking) or from the bottom surface layer towards the top surface layer of the slab (bottom-up cracking).

Bottom-up, mid-edge transverse cracking under repetitive vehicular loading is the most common failure mode of concrete pavements. It is the only fatigue failure mode of the concrete slab considered in the PCA method.

Heath (2003) reported that many jointed concrete pavements suffer from corner and longitudinal cracking, differential temperature together with different boundary conditions between the concrete slab and subbase were considered in concrete pavement analysis to investigate the process of concrete pavement deteriorations.
Zhou (2011) concluded that primary cracks are prone to occur especially in the edge load case and localized crushed concrete and looseness of dowels due to stresses concentration may result in joint distresses.

2.5.1.1 Fatigue failure modes in jointed plain concrete pavements (JPCP)

Buch (2004) investigated the structural response of jointed concrete pavement under single axle (SA), tandem axle (TA), triple axle (TR), quad axle (QA), multi axles, together with differential temperature using influence stress line approaches. The results showed that mid-edge loading causes bottom up cracking for SA, TA, TR and QA. Ongel (2004) stated that the possibility of bottom-up transverse cracking increases in the absence of load transverse devices at longitudinal joints.

Hiller (2005) used the influence stress lines to determine the critical location of fatigue damage under truck loads in a typical California concrete pavement with permanent built-in curling. They found that the critical damage location in the absence of environmental effects for a load transfer efficiency of 70% was at the bottom surface layer of the mid slab edge.

AASHTO (2003) noted that tensile stresses at the bottom surface layer of concrete slab are also induced due to daytime differential temperature gradients. AASHTO (2003) states that in order to reduce the extension of the bottom-up transverse cracking in JPCP, the following recommendations are considered:

- Increase slab thickness
• Reduce joint spacing
• Use widened slabs
• Use concrete mix with lower CTE
• Provide tied concrete shoulder
• Use concrete with a higher strength
• Use a stabilized subbase

Channakeshava (1993) concluded that loss of support due to night time temperature gradients or erosion of subbase and subgrade materials at joints associated with the corner load position lead to corner cracking. Byrum (1994) noted that these contributed to the influence function lines in analysis of jointed concrete pavement under environmental effects and wheel load. Their results indicated that maximum stress occurs at some distance away from the joint when the load passes across the joint.

Heath (2000) noted that cracks are usually initiated and distributed in the top surface layer of the concrete slab and then propagated downward in the concrete slab due to environmental effects before applying any vehicular loading.

Heath (2000) and Khaznovich (2001) studies results showed that cracks may also occur near the corner of the slab when concrete pavement is subjected to a combination of loads and environmental effects.

Beckemeywe (2002) studies results concluded that a combination of excessive upward slab curling, loss of slab support and repetitive vehicular loadings is the main reason for
top-down cracking. Hanson (2002) using finite element analysis of a JPCP in hot weather conditions subjected to a truck load showed that maximum vehicular induced tensile stress shifts toward mid-length of concrete slab due to loss of support. Hiller (2002) stated that longitudinal cracking can also be explained by excessive differential drying shrinkage.

Panda (2002) carried out experimental study on crack sensitivity to different factors showed that coarse aggregate could modify the crack generation and propagation by about 30 percent. Popehn (2003) studies concluded that joint faulting is due to either concrete slab settlement or erosion of subbase or subgrade materials at the bottom of the concrete slab. It usually occurs in the jointed concrete pavements due to repeated heavy vehicular loading crossing transverse joints. Joint faulting increases width of joints and consequently decreases the LTE.

AASHTO (2003) stated that the critical load condition in top-down cracking is caused by a combination of axle group loads when applied on the opposite ends of a slab simultaneously and in the presence of high negative temperature gradients. The crack is initiated at the top layer of the slab close to longitudinal joints and midway between transverse joints. Lower permanent built-in curling after concrete placement can modify resistance of the slab to top-down cracking. AASHTO (2003), provision of dowels and shoulders, the use of thicker dowel, the use of shorter joint spacing, the use of none erodible materials in subbase and subgrade or the use of an appropriate drainage system in sublayers can effectively restrict joint faulting.
Buch (2004) studies results shown that an increase in the subgrade reaction and using thicker concrete slab can result in decreases of the induced tensile stresses. Their findings also observed the corner loading results in top-down cracking for SA, TA, TR and QA. Hiller (2005) noted that top-down transverse cracking near the mid slab edge was the critical failure mode in the presence of a night time differential temperature of -16.5 °C.

Shoukry (2007) concluded that the maximum principal stress is a better indicator of the overall state of stress in the slab, especially when the combined effects of thermal and axle loads are considered. It is shown that the stresses induced due to the combined effect of axle loading and temperature gradients are well below the modulus of concrete rupture, therefore they are not the primary cause of premature failure of concrete slabs.

Rania (2011) stated that on the basis of slab curvatures calculated from slab temperature and moisture measurements, it was found that slabs are predominantly curled upward within a few months after paving, resulting in top-down fatigue cracking (rather than bottom-up fatigue cracking that has traditionally been assumed in concrete pavement design). His results were confirmed using curvatures computed from strain measurements, which showed the slabs were curled upward 99% of the time.

Christopher (2013) concluded that looseness is caused by typical horizontal pavement joint openings and vertical off-sets that develop at slab ends. Just a very small vertical off-set or fault along a joint line can result in significant differences in load transfer and deflections of slabs for loading on the side of a joint compared to the other.
2.5.1.2 Fatigue failure modes in continuously reinforced concrete pavement (CRCP)

Zwerneman (1995) reported that installation of transverse reinforcements near the edge of longitudinal joints has a strong effect on crack width and the magnitude of induced stress in concrete. Air temperature and air content significantly affect crack spacing.

Chen (2001) concluded that deformation under the applied loading was as follows:

- The ratio of reinforcement and the elastic modulus of the foundation soil have no significant effects on deflection of CRCP under the load.
- Pavement deflection is considerably affected by concrete slab thickness and configuration of the applied load.
- The location of reinforcement has little effect on the pavement responses.

Nam (2003) carried out a sensitivity analysis of CRCP using the CRCP-10 computer program to determine effects of different variables on the CRCP performances.

The following are his findings:

- An increase in concrete thickness, concrete flexural strength and diameter of reinforcement increases the crack spacing. In fact, crack spacing decreases with an increase in the concrete CTE and reinforcement ratio.
- Diameter of reinforcement and vertical stiffness of underlying layers have no significant effect on the CRCP behaviour.
Selezneva (2004) stated that transverse cracks result from the stress-relief process caused by volumetric change in concrete due to drying shrinkage and seasonal thermal strains in the CRCP within 2 years after construction.

Punch out is another major structural distress associated with CRCP. Punch out results in a longitudinal crack due to a high tensile stress occurring at the top of the concrete slab and 1016 mm to 1524 mm away from its edge, when axle load is located near the longitudinal edge of the slabs and between two closely spaced transverse cracks. The induced tensile stress has a reverse relationship with LTE in the transverse cracks and directly related to loss of support along the edge of the concrete slab.

Kim (2000) showed that induced tensile stress at the top surface layer of the concrete slab above transverse reinforcement can be relatively higher than that at the top surface layer of the centre of concrete slab.

AASHTO (2003), consideration of longitudinal reinforcement to decrease the crack width and consequently to increase the LTE, decreasing the concrete CTE, placing the reinforcement bars above mid-depth of concrete slab, thicker concrete slab, providing concrete shoulder, use of stabilizes subbase, reducing built-in curling after placement and the use of stronger concrete are predominant factors controlling the CRCP punch out distress.
Selezneva (2004) reported that there is a relationship between transverse cracks, critical stress in longitudinal crack, and punch out development. He also developed a mechanistic-empirical structural design procedure for punch out in the CRCP.

### 2.5.2 Erosion of subbase and subgrade materials

Erosion of subgrade or subbase materials is one of the main distresses in concrete pavement, which results in the loss of support due to voids under the slab beneath. Initial voids caused by curling or warping of the concrete slab or even plastic deformation of subgrade materials. Voids can form by pumping transportable subbase and subgrade materials due to deflection of concrete slab in the presence of water.

Larralde (1984) has developed a method of analysis of concrete pavement to study the erosion problems known as PMARP. Barlow (1994) has developed another method of analysis of concrete pavement to consider the erosion problems known as Iowa pumping model IAPUMP. Zhong (2010) established a formula of calculating the deflection under different void models. Considering rigid pavements as thin elastic slabs, displacement calculation model is established under the condition of foundation void resting on Winkler foundation, based on the thin plate theory of elastic foundation.

Tang (1992) concluded that voids are extremely detrimental for slabs to bear stress, especially in the angle and the side of the site. Under the influence of the wheel loads, the stress state is similar to cantilever beam, which produces too much stress, strain and
vertical deformation, and easily cause rupture and fragmentation damage in the cement concrete slabs. Bahatti (1996) stated that erosion happens through longitudinal and transverse joints at the beginning of the pavement operation and later in the pavement life, through cracks caused by repetitive vehicular loading and other related environment factors.

2.5.3 Spalling

Zollinger (1994) reported that spalling is another concrete pavement distress occurring due to delamination stress produced by repetitive vehicular loading, climate change and the presence of moisture in the delamination zone.

Wang (2000) observed that it often occurs at both longitudinal and transverse joints, approximately 150 mm away from the joint or the crack.

2.6 Concrete pavement design guides

The first thickness design based on fatigue process in 1933, Portland Cement Association (PCA) adopted Older’s empirical equations (1924) for calculation an induced stresses at the corner of a concrete slab subjected to a wheel load.

The Portland Cement Association revised the PCA method in 1951 based on results of several research projects studies in the different states of the USA. The Pickett’s
equations for stress calculation at the corner of a concrete slab were taken into
consideration. An effect of slab thickness, wheel load, tyre pavement contact area, and
modulus of subgrade reaction were emphasised in this PCA version.

Fatigue failure modes were considered in the 1933 version, new data on fatigue curve,
which showed that static loads affect concrete pavement stress more than moving loads,
and Pickett-Ray influence charts were then taken into account in the PCA method version
published in 1966.

Meyerhof (1962) developed a yield line theory to determine a concrete pavement
response under point loads when the load is positioned at the interior area of the concrete
pavement. His theory was based on a circular failure shape around the point load due to
positive and negative curvature on opposite sides of the failure interface.

AASHTO (1972) published a pavement design guide based on the evaluation of results
collected from road tests conducted by the state highway agencies. The concept of
equivalent single axle load (ESAL) was introduced to simplify the design procedure in
the AASHTO method.

Packard (1985) results on thickness design of concrete highway and street pavement were
then taken into account in the latest version of the PCA method published in 1984. The
concept of erosion of subbase and subgrade materials was taken into consideration in
concrete pavement damage process in this version.
Austroads (1992) extended the PCA method to suit Australian conditions, the recent revision of Austroads (2004) and AASHTO (2003) pavement design guides will be highlighted in the following sections.

2.6.1 Austroads 2004

In 2004, Austroads released a revision of the 1992 Guide for concrete pavement design based on the work of (Packard, 1985), also known as the PCA method. PCA methods have been widely recognized by pavement engineers during the past decades, current Australian practices and materials were taken into consideration in the 2004 Guide (Vorobieff, 2001).

The Guide provides a mechanistic procedure for calculating the required concrete slab thickness for JPCP, JRCP, CRCP and Steel Fibre Concrete Pavement (SFCP). The use of the Guide was restricted to those pavements whose dimensions (distance between transverse and longitudinal joints) were less than those provided in Table 2.1.
Table 2.1: Distance between joints (m) for different types of concrete pavement

<table>
<thead>
<tr>
<th>Type of Concrete Pavement</th>
<th>Availability of dowel</th>
<th>Distance between Transverse Joints</th>
<th>Instruction on Transverse Joints</th>
<th>Distance between Longitudinal Joints</th>
<th>Instruction on Longitudinal Joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>JPCP</td>
<td>No</td>
<td>4.2</td>
<td>Skewed Joints</td>
<td>4.3</td>
<td>Tied</td>
</tr>
<tr>
<td></td>
<td>Yes</td>
<td>4.5</td>
<td>Square Joints</td>
<td></td>
<td></td>
</tr>
<tr>
<td>JRCP</td>
<td>Yes</td>
<td>8.0-12.0</td>
<td>Square dowel</td>
<td>4.3</td>
<td>Tied</td>
</tr>
<tr>
<td>CRCP</td>
<td>No</td>
<td>As long as construction</td>
<td>N/A</td>
<td>4.3</td>
<td>Tied</td>
</tr>
<tr>
<td>SFP</td>
<td>Yes</td>
<td>joints</td>
<td>N/A</td>
<td>4.3</td>
<td>Tied</td>
</tr>
</tbody>
</table>

A variety of inputs including design traffic, subgrade CBR, subbase thickness and type, project design reliability (PDR), concrete flexural strength, vehicular load spectra (axle group load distributions) and provision of dowels and shoulders are taken into account to calculate the required concrete slab thickness based on the cumulative damage due to fatigue of the concrete slab and erosion of subbase and subgrade materials. The severity of fatigue and erosion damage depends on the structural response of concrete pavements as affected by vehicular load configurations, environmental factors, and material/layer characteristics.

Design traffic is an estimation of heavy vehicle volumes on the road during the life of a pavement. The method of estimating the number of Heavy Vehicle Axle Groups (HVAGs) has been described in the Guide. Although the PCA method was developed based on three types of axle loading i.e. SADT, TADT and TRDT, the Guide extended the method to cover SAST, SADT, TAST, TADT, TRDT and QADT.
The California Bearing Ratio (CBR) is the only subgrade information used in the design procedure and represents the subgrade resistance to applied load. Subgrade CBR values are affected by topography, soil type, and drainage conditions. The Guide provides a number of methods to estimate the field CBR values under various conditions. Five types of subbase including 125 mm bound, 150 mm bound, 170 mm bound, 125 mm Lean-Mix Concrete (LMC), and 150 mm LMC have been recommended in the Guide. The choice of subbase depends upon the value of the design traffic (Table 2.2).

The 150 mm LMC is the only choice for design traffic greater than 1×10^7 HVAGs. Effective subgrade strength has been defined in the Guide to consider the effect of the subbase layer on concrete pavement behaviour. The Guide provides a graphical estimation of effective subgrade strength based on subgrade CBR and subbase types.

Table 2.2: Selection of subbase type

<table>
<thead>
<tr>
<th>Design Traffic (HVAG)</th>
<th>Subbase Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 10^6</td>
<td>125 mm bound</td>
</tr>
<tr>
<td>Up to 5 × 10^6</td>
<td>150 mm bound or 125 mm LMC</td>
</tr>
<tr>
<td>Up to 1 × 10^7</td>
<td>170 mm bound or 125 mm LMC</td>
</tr>
<tr>
<td>Greater than 1 × 10^7</td>
<td>150 mm LMC</td>
</tr>
</tbody>
</table>

Design traffic is also employed to estimate the expected load repetitions for a given axle group by using a typical traffic load spectrum during the pavement’s life. Presumptive traffic load distributions for urban and rural roads are provided in the Guide, which can
be employed if specific traffic load spectra are not available. Effects of dowels and shoulders provision on thickness of concrete slab have been considered through the use of design coefficients for different axle group types for both fatigue and erosion analyses.

2.6.2 AASHTO 2003

In 2003, AASHTO released its new revision of concrete pavement design procedure for JPCP and CRCP. The following factors have been taken into consideration in this version: pavement layers arrangement, joint spacing, provision of dowels, tied concrete shoulder, base layer type, drainage, subgrade properties and modulus of reaction, design traffic, climatic data, the initial smoothness, estimation construction period, estimation on pavement age at the time the pavement is opened to the traffic and estimated permanent curling or warping condition of the concrete slab.

AASHTO 2003 design Guide depends upon the accuracy of input data. Three levels of input parameters have been identified in the guide as follow:

(1) Level 1: Input data is the result of direct tests.

(2) Level 2: Required parameters are determined by the use of correlations such as determination of subgrade modulus of reaction based on its CBR value.

(3) Level 3: Input data is defined in accordance with national default values or local experiences.
AASHTO (2003) design guide has taken into consideration input data such as general information; traffic load; climate forces; drainage and surface properties; and pavement structure.

**General information**

General information includes the following factors:

1. Project design life;
2. End of construction period in a particular month. This information is used to estimate the ambient temperature and moisture during construction time and also used to estimate the permanent curling and warping condition in the concrete slabs after construction;
3. Time when the pavement is opened to the traffic. This information is used to estimate concrete strength at the time when the concrete slabs are subjected to the vehicular loading;
4. JPCP or CRCP pavement type;
5. Estimation of initial IRI which ranges from 789 to 1578 mm/km;
6. AASHTO 2003 design guide considers a variety of performance criteria for the design of concrete pavements. These are transverse cracking and joint faulting in JPCP, crack width, load transfer efficiency at the crack, and number of punch out cracks in CRCP. The allowable range for each individual factor is 10% to 45% for transverse cracking, 2.54 mm to 5.08 mm in joint faulting, less than 0.51 mm for
crack width, more than 90% of LTE at the crack and 6 to 12 punches out cracks per km of CRCP;

(7) Surface smoothness which can be controlled through the use of IRI. The allowable value of IRI ranges from 2366 mm/km to 3944 mm/km.

Traffic load

AASHTO (2003) design guide has considered different axle groups including single axles (SA), tandem axles (TA), tridem (TRA) and quad axles (QA). Axle loads vary between 13.3 to 182 kN in SA, 26.7 to 364 kN in TA and 53.4 to 453.7 kN in TRA and QA. The axle load distribution for each axle is presented in an interval value of 4.45 kN, 8.9 kN and 13.35 kN respectively. Other parameters such as standard deviation of traffic wander, width of traffic lane, means wheel location, number and type of axles per truck class, axle configuration and wheel base have been taken into consideration in the (AASHTO, 2003) design guide. Traffic volume is calculated based on hourly traffic distribution within the pavement and traffic growth factor. It is then adjusted as per each month of the pavement service life using adjustment factor. This data information is then used to estimate the performance criteria.

Drainage and surface properties

AASHTO (2003) design procedure, differential temperature gradient and moisture content within the depth of the concrete pavements are affected by properties of the
pavement surface layer. The Guide defines four parameters to specify the magnitude of climatic effects. These are pavement surface short wave absorptivity, potential for infiltration, pavement cross slope, and length of drainage path. The short wave absorptivity is the ratio of absorbed solar energy by the pavement to the total solar energy radiated to the pavement surface. This parameter depends on pavement composition, colour, and texture and can be used to determine the magnitude of temperature gradients in depth of the pavement. This parameter ranges from 0 to 1 and varies from 0.7 to 0.9 for different concrete surfaces. The recommended value for this parameter is 0.85.

The infiltration shows the amount of water entering the pavement structure due to a given rainfall event. Provision of shoulder can significantly affect the magnitude of this parameter. The following regimes have been considered in the AASHTO 2003 Guide:

1. Minor: Tied concrete shoulders are used or edge-drain is present under the shoulder;
2. Moderate: This is valid for all other shoulder types;
3. Extreme: Not used for new or reconstructed pavement.
4. Pavement cross-slope is the transverse slope of pavement surface and defines the required time to drain free water from the pavement subbase layer. Drainage path length defines the horizontal distance between the highest point of the pavement and the point where drainage occurs.
Pavement structure

This method considers a variety of layers within the concrete pavements. These include concrete slabs, unbonded course asphalt or cement treated base, unbonded stabilized subbase, compacted subgrade, natural subgrade and bedrock.

AASHTO (2003) developed a specific program for design procedure to present the following information:

1. Average hourly number of single, tandem, tridem, and quad axles in each axle load category for each month of the analysis period;
2. Minimum one year’s weather station data used to determine the value of temperature at 11 evenly spaced nodes in the concrete slab layer for every hour of the available climatic data;
3. Average monthly relative humidity for each month;
4. Estimation of the concrete strength and modulus of elasticity for each month of the analysis period;
5. Determination of the base strength in each month;
6. Monthly average of effective subgrade modulus of reaction.

JPCP design features

AASHTO (2003) design guide recommended a distance of 3658 mm between longitudinal joints and 4572 mm between transverse joints. Dowel diameter has a reverse
relationship with joint faulting. In fact, the adequate dowel diameter is calculated through the performance criteria developed for joint faulting. The recommended dowel size is 30.5 mm. AASHTO (2003) design guide has identified two types of shoulder, namely tied and widened. The value of LTE in tied shoulder needs to be between 50% and 70% for monolithically constructed shoulders and between 30% and 50% for separately constructed shoulders. The slab width is 4267 mm if a widened shoulder is used. Bonded and unbonded boundary conditions between concrete slabs and base layer can be considered through the use of a friction reducer layer. In terms of erosion of subbase and subgrade materials, five classes of base erodibility have been developed in the AASHTO 2003 Guide as follow:

1. Class 1: Extremely erosion resistant material;
2. Class 2: Very erosion resistant material;
3. Class 3: Erosion resistant material;
4. Class 4: Fairly erodible material;
5. Class 5: Very erodible material.

**Surface roughness**

The development of pavement roughness (as measured by IRI) depends upon the subsequent development of distresses including slab cracking, joint faulting and joint spalling for JPCP, and punch outs for CRCP over a period of the time. IRI is incrementally estimated over the entire design period on a monthly basis.


**Thickness of concrete slab**

Thickness of concrete slab is calculated to satisfy the above mentioned performance criteria such as joint faulting, slab cracking, punch outs, and IRI. It is determined by the use of cumulative damage percentage for relative performance criteria. The slab thickness is adequate provided that the magnitude of different damage processes satisfies the minimum relevant requirement described in the (AASHTO, 2003) design guide.

**2.7 Summary**

A comprehensive literature review on various factors affecting concrete pavement performance has been discussed in this chapter. Fatigue of the concrete slab can cause top-down or bottom-up cracking. Corner, longitudinal and transverse cracks are the most common fatigue failure modes in concrete pavements. The presented review also indicates that the research conducted to study the pavement joints and particularly the induced stresses around dowel bars were mainly addressing the identification of compressive bearing stresses above and under the dowels. There is a lack of sound approach to identify with any degree of accuracy the modulus of dowel support (k), which makes it difficult to rely on the analytically developed formulas that are sensitive to its value. It is also apparent from the review that the looseness of the dowels was believed to be mainly a result of initial misalignment and is gradually developed by crushing of concrete particles in the compression zones around the dowels, disregarding the contribution of the tensile stresses at the dowel/concrete contact. It is also noticed that
2DFE was unable to capture the full state of stresses around dowel bars. It is believed that the state of stresses at the dowel/concrete interface needs to be closely explored. Such a study could reveal important facts about the formation of different types of stresses that could have a significant effect on the behaviour of rigid pavement joints along with compressive bearing stresses.

Analysis of concrete pavements under vehicular loads and environmental effects depends on parameters that relate to characteristics of concrete slab, subbase and subgrade, type of debonding layer between concrete slab and subbase, the provision of shoulder and dowel bars, traffic loads and climatic effects. The methodology of concrete pavement design guides developed based on PCA method (Austroads, 2004), and mechanistic-empirical approach (AASHTO, 2003), were also presented in this chapter.
Finite Element Model Development with Parametric Studies

3.1 Introduction

Transverse joints in rigid pavements are the locations where most pavement distress appears, leading to deteriorating the riding quality and result in high maintenance cost. The state of stresses in the concrete surrounding dowel bars, in dowel jointed concrete pavements, is a major factor that contributes to transverse joint distress. As such, a three-dimensional finite element model was developed for analysing a dowel-jointed concrete pavement. The effect of different pavement and joint related parameters on the load transfer characteristics of a joint has been evaluated using the FE model. Group action of the dowel bar system has also been examined. Five loading cases are applied to replicate realistic vehicular loadings approaching and leaving the joint.

The structural behaviour of the pavement at the doweled joint is investigated for: (1) Behaviour of pavement with and without dowels; (2) Behaviour of doweled joint with and without lean concrete; (3) Behaviour of doweled joint with and without voids; (4) Behaviour of undowered and doweled joint under subgrade Strength Variation California Baring Ratio (CBR); (5) Behaviour of doweled Joint under load magnitude variation (tyre
(5) Pressure variation; (6) Dowel spacing variation; (7) Slab thickness change; (8) Single and dual wheel loads.

The amount of load transfer is obtained from the shear force in the beam elements that simulate dowels. Results show that the voids underneath the joint causes an increase in the vertical displacement of the concrete slab and vertical stress at concrete/dowel bar interface which may result in crushing of the concrete and dowel loosening. Wider dowel spacings result in increased shear forces and the size of the region containing engaged dowels does not change significantly with dowel spacing, only effecting the distribution of shear forces. Maximum Principle Stress (MPS) is about 6.7 times greater and steeper variation in the distribution pattern in the concrete pavement without Lean Concrete Base (LCB).

A thick concrete slab provides a significant benefit: higher load transfer and develops less curvature along the loaded side of the joint. The deformed shape explains why more dowels are engaged in the load transfer for the thicker concrete slab models. There were no significantly affects on load transfer ratio with the increase applied wheel load. This phenomenon is also evident in the dowel shear force distribution. However, it will increase the demand on a few inner dowels beneath the wheel load, which may cause more damage to the joints and eventually lead to pavement failure. The study shows that the dowel bars perform effectively as a load transfer device in the concrete pavement system even under severe conditions.
3.2 Methodology

3.2.1 Modelling strategy

The modelling and simulation are performed using the Strand7 (2004) FEA System. The FEM is used to provide a solution that is approximate because the theoretical representation is usually too complicated. However, the accuracy of the approximate solution can be improved by increasing the number of elements. Note that the number of elements are not always related to the accuracy of the solution and will effect the simulation time. A convergence study is undertaken to determine the least amount of elements which provide the most accurate solution. This is accomplished, by increasing the number of elements until the variation of von Mises stress measured at a single element becomes insignificant.

The number of elements is controlled by changing the maximum edge length ($MEL$) of each element within the auto mesh settings of Strand7 (2004). The $MEL$ is the maximum distance between the nodal points of an element along each edge. Note in this study that the $MEL$ is represented by a percentage of the maximum element edge length pre-programmed by Strand7 (2004) according to the specified brick element geometry.
3.2.2 Selection of element types and meshes refinement

8 nodes hexahedra brick elements are used to model the concrete pavement section. The basic representation of the dowel bars in the FE model is the beam element. To simulate the dowel bars, beam elements are connected at the nodes on the brick elements according to the dowel section encased in the slab and the dowel location within the structure. Beams with any arbitrary cross-section shape can be displayed in 3D as rendered solid beams. For each element is assigned with 3 degrees of freedom at each node. The pavement model is discretized into many small brick elements. The region under investigation is located at the vicinity of the joint; therefore finer mesh sizes are developed in that area to capture more accurate stress and strain behaviours. The total numbers of 8 node hexahedral brick elements are 2184 for the concrete slab, 4368 for the base course and 12600 for the subgrade. The total number of nodal points for the entire pavement model is 22964.
Figure 3.1: Finite element model of pavement system
3.2.3 Pavement material models

The material properties are provided in Table 3.1; 3.2 and are assumed to be linear, homogeneous and elastic in behaviour.

Table 3.1: Material properties and layer thicknesses

<table>
<thead>
<tr>
<th>Description</th>
<th>Concrete Slab</th>
<th>Base Course</th>
<th>Subgrade</th>
<th>Dowel Bar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus [MPa]</td>
<td>28,000</td>
<td>350</td>
<td>50</td>
<td>200,000</td>
</tr>
<tr>
<td>Layer thickness [mm]</td>
<td>250</td>
<td>150</td>
<td>1,900</td>
<td>-</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.18</td>
<td>0.4</td>
<td>0.4</td>
<td>0.3</td>
</tr>
<tr>
<td>Density [kg/m³]</td>
<td>2,400</td>
<td>2,000</td>
<td>1,800</td>
<td>7,830</td>
</tr>
</tbody>
</table>

Note: “—” information not required.

Table 3.2: Material properties and layer thicknesses for M1 (a) Case 1

<table>
<thead>
<tr>
<th>Description</th>
<th>Concrete Slab</th>
<th>Cement Treated Subbase</th>
<th>Subgrade</th>
<th>Dowel Bar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus [MPa]</td>
<td>28,000</td>
<td>5000</td>
<td>50</td>
<td>200,000</td>
</tr>
<tr>
<td>Layer thickness [mm]</td>
<td>200</td>
<td>100</td>
<td>1,900</td>
<td>-</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.18</td>
<td>0.4</td>
<td>0.4</td>
<td>0.3</td>
</tr>
<tr>
<td>Density [kg/m³]</td>
<td>2,400</td>
<td>2,000</td>
<td>1,800</td>
<td>7,830</td>
</tr>
</tbody>
</table>

Note: “—” information not required.

(b) Case 2

<table>
<thead>
<tr>
<th>Description</th>
<th>Concrete Slab</th>
<th>Lean Concrete Base</th>
<th>Subgrade</th>
<th>Dowel Bar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus [MPa]</td>
<td>28,000</td>
<td>15000</td>
<td>50</td>
<td>200,000</td>
</tr>
<tr>
<td>Layer thickness [mm]</td>
<td>250</td>
<td>150</td>
<td>1,900</td>
<td>-</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.18</td>
<td>0.4</td>
<td>0.4</td>
<td>0.3</td>
</tr>
<tr>
<td>Density [kg/m³]</td>
<td>2,400</td>
<td>2,000</td>
<td>1,800</td>
<td>7,830</td>
</tr>
</tbody>
</table>

Note: “—” information not required.
3.2.4 Plane of symmetry and boundary conditions

Three-dimensional (3D) representation of the three types of pavement layers, i.e. concrete slab, base course and subgrade, are defined and full friction is applied at the boundaries. Illustrated in Figure 3.1 are the vehicular loading and restraint conditions applied to the pavement. These loading and restraint conditions are commonly assumed in previous literature (Hadidy, 2009 and Ju, 2009). Dowel bars are modelled using beam elements with a defined diameter of 32mm, see Figure 3.1. The mesh size is reduced in the vicinity of the joint and dowel/concrete interface to aid the accuracy of displacement and stress measurements. As indicated in Figure 3.1, the symmetrical plane is restrained from rotating around the z-axis and translating along the x- and y-axes. The bottom surface of the pavement is restraint from deforming in all directions and roller restraints are applied to the sides of the pavement.

3.2.5 Loading conditions

Four wheel loads of 20 kN each representing an equivalent 80 kN (i.e. 707 kPa) single axle load is assumed. The effect of the air pressure within the tyre is neglected, hence the contact pressure of the tyre is assumed to be uniformly distributed over a rectangular area of $2.66 \times 10^{-2}$ m$^2$ (see Figure 3.1).

Five loading cases are applied to the model to replicate a vehicle approaching the joint as shown in Figure 3.2. The first ($LC1$), second ($LC2$), third ($LC3$), and forth load case
(LC4) are 370 mm, 270 mm, 165 mm and 55 mm to the centre of the joint, and the fifth load case (LC5) is at the centre of the joint.

![Figure 3.2: Loading scenarios](image-url)
3.2.6 Stress measurement method

Indicated in figure 3.3 are the locations of the lines along which the displacements, stresses and strains are measured at nodal points located along on the lines. The details of these lines are provided in table 3.3. From section AA, the locations for displacement (TT) and stress (DD) measurement in the concrete slab can be identified. Note that the displacement and stress values in the vertical direction (i.e. y-direction) are measured at the nodal points along lines TT (along the transverse direction) and DD (along the travel direction). Note also that to measure displacements and LTE, the line TT is selected along the top edge of the concrete slab at the joint. This line can therefore represent both edges on the approach and leave sides at the joint location. These are detailed in section AA of figure 3.3. To measure the stresses, a dowel bar directly in line with the inner wheel is selected. For the vertical stresses and strains along the longitudinal direction, the nodal points along the lines LL are measured.
Figure 3.3: Stress measurement within pavement system
### Table 3.3: Stress measurement lines

<table>
<thead>
<tr>
<th>Direction</th>
<th>Description</th>
<th>Location</th>
<th>Concrete Slab</th>
<th>LCB</th>
<th>Base Course</th>
<th>Measuring</th>
<th>Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse (along z-axis)</td>
<td>TT&lt;sub&gt;HH&lt;/sub&gt;</td>
<td>Top</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>Vertical Displacement</td>
<td>1800</td>
</tr>
<tr>
<td></td>
<td>TT&lt;sub&gt;CC&lt;/sub&gt;</td>
<td>Bottom</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>Vertical, Longitudinal and Transverse Strains</td>
<td>1800</td>
</tr>
<tr>
<td></td>
<td>TT&lt;sub&gt;LCB&lt;/sub&gt;</td>
<td>—</td>
<td>Bottom</td>
<td>—</td>
<td>—</td>
<td>Vertical Stress and Transverse Strain</td>
<td>4520</td>
</tr>
<tr>
<td></td>
<td>TT&lt;sub&gt;BC&lt;/sub&gt;</td>
<td>—</td>
<td>—</td>
<td>Bottom</td>
<td>—</td>
<td>Vertical Displacement</td>
<td>1800</td>
</tr>
<tr>
<td>Longitudinal (along x-axis)</td>
<td>LL&lt;sub&gt;CC&lt;/sub&gt;</td>
<td>Bottom</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>Vertical Stress</td>
<td>4520</td>
</tr>
<tr>
<td></td>
<td>LL&lt;sub&gt;LCB&lt;/sub&gt;</td>
<td>—</td>
<td>Bottom</td>
<td>—</td>
<td>—</td>
<td>Vertical Stress</td>
<td>4520</td>
</tr>
<tr>
<td></td>
<td>LL&lt;sub&gt;BC&lt;/sub&gt;</td>
<td>—</td>
<td>—</td>
<td>Bottom</td>
<td>—</td>
<td>Vertical Displacement</td>
<td>1800</td>
</tr>
<tr>
<td></td>
<td>DD</td>
<td>Dowel/Concrete Interface</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>Vertical Stress</td>
<td>460</td>
</tr>
</tbody>
</table>

#### 3.3 Case study on Pacific Highway, Reedy Creek, Gold Coast

#### 3.3.1 Behaviour of Case Study 1 concrete pavement with and without dowels

From the observation of pavement models Case Study 1 with and without dowels, one can see that the dowel had significantly reduced the maximum displacement at the transverse joint along the line \(TT_{HH}\) by 10%; 37.3% reduced in longitudinal tension stress on the top of the concrete slab. However, 34.5% reduction in vertical compression stress and 30.4% in longitudinal strain on the concrete slab interface with cement treated sub-base is also noted. Besides, there is 14.6% reduction in vertical compression stress; 15% in vertical strain; 19.5% in longitudinal strain and 13.6% in transverse compression stress on the cement treated sub-base interface with sub grade. Figures of these cases can be found in appendix A1.
### Table 3.4: Present percentages of reduction in magnitude of vertical displacement, stress and strain in X-X; Y-Y and Z-Z axis in concrete pavement with dowels

<table>
<thead>
<tr>
<th>Layers</th>
<th>X-X axis (%) (direction of traffic)</th>
<th>Y-Y axis (%) (vertical)</th>
<th>Z-Z axis (%) (perpendicular to direction of traffic)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Slab</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum vertical displacement</td>
<td></td>
<td>(10)</td>
<td></td>
</tr>
<tr>
<td>Tension stress</td>
<td></td>
<td>(7.3)</td>
<td></td>
</tr>
<tr>
<td>Longitudinal compression stress</td>
<td></td>
<td>(3.8)</td>
<td></td>
</tr>
<tr>
<td>Longitudinal tension stress</td>
<td></td>
<td>(37.3)</td>
<td></td>
</tr>
<tr>
<td>Transverse compression stress</td>
<td></td>
<td>(3)</td>
<td></td>
</tr>
<tr>
<td>Concrete slab interface with cement treated sub-base</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical compression stress</td>
<td></td>
<td>(34.5)</td>
<td></td>
</tr>
<tr>
<td>Vertical strain</td>
<td></td>
<td>(2.2)</td>
<td></td>
</tr>
<tr>
<td>Longitudinal strain</td>
<td></td>
<td>(30.4)</td>
<td></td>
</tr>
<tr>
<td>Transverse strain</td>
<td></td>
<td>(2.5)</td>
<td></td>
</tr>
<tr>
<td>Cement treated sub-base interface with subgrade</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical compression stress</td>
<td></td>
<td>(14.6)</td>
<td></td>
</tr>
<tr>
<td>Vertical strain</td>
<td></td>
<td>(15)</td>
<td></td>
</tr>
<tr>
<td>Longitudinal compression stress</td>
<td></td>
<td>(3)</td>
<td></td>
</tr>
<tr>
<td>Longitudinal strain</td>
<td></td>
<td>(19.5)</td>
<td></td>
</tr>
<tr>
<td>Transverse compression stress</td>
<td></td>
<td>(13.6)</td>
<td></td>
</tr>
<tr>
<td>Transverse strain</td>
<td></td>
<td>(6.5)</td>
<td></td>
</tr>
</tbody>
</table>

### Table 3.5: Case study 1 load transfer efficiency (a) with dowels

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Leave Side (Unloaded), ( \delta_U ) [mm]</td>
<td>-0.178</td>
<td>-0.178</td>
<td>-0.177</td>
<td>-0.187</td>
</tr>
<tr>
<td></td>
<td>Approach Side (Loaded), ( \delta_L ) [mm]</td>
<td>-0.184</td>
<td>-0.185</td>
<td>-0.188</td>
<td>-0.191</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>96.74</td>
<td>96.22</td>
<td>94.15</td>
<td>97.90</td>
<td>100.00</td>
</tr>
</tbody>
</table>
(b) Without dowels

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Leave Side (Unloaded), $\delta_U$ [mm]</td>
<td>-0.189</td>
<td>-0.192</td>
<td>-0.192</td>
<td>-0.202</td>
<td>-0.208</td>
</tr>
<tr>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.198</td>
<td>-0.201</td>
<td>-0.207</td>
<td>-0.209</td>
<td>-0.208</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>95.45</td>
<td>95.52</td>
<td>92.75</td>
<td>96.65</td>
<td>100.00</td>
</tr>
</tbody>
</table>

As expected LC5 has a LTE of 100% due to it being positioned evenly on both sides of the joint. Overall, the pavement indicates excellent LTE since the specified LTE for the majority of load cases exceeded approximately 94% for concrete pavement with dowels and 92% for concrete pavement without dowels. Further, an increased efficiency also shows that the dowel bars perform effectively as load transfer devices in the concrete pavement systems.
Figure 3.4: Maximum principal stress in cement treated sub-base

Figure 3.4(a) and (b) show the maximum principal stress on concrete slab interface with cement treated sub-base, and results from the FE models with and without dowels along the longitudinal pavement section, respectively. From the observation of pavement models with and without dowels, one can see that there was no significant change in the pavement MPS.
Figure 3.5: Maximum principal stress in sub grade

Figure 3.5(a) and (b) show the maximum principal stress on cement treated sub-base interface with sub grade, and results from the FE models with and without dowels along the longitudinal pavement section, respectively. From the observation of pavement models with and without dowels, one can see that there was no significant change in the magnitude of compressive MPS.

3.3.2 Behaviour of Case Study 2 concrete pavement with and without dowels

From the observation of pavement models Case Study 2 with and without dowels, one can see that the dowel had significantly reduced the maximum vertical displacement at the transverse joint along the line $TT_{HH}$ by 8.3%; 6.8% reduction in vertical tension stress and 8.4% in longitudinal compression stress on the top of the concrete slab. However,
29.4% reduction in vertical compression stress; 11.1% reduce in vertical strain; 49.5% reduction in longitudinal compression stress; 52.5% reduce in longitudinal strain; 64% reduction in transverse compression stress and 16.5% reduce in transverse strain on the concrete slab interface with lean concrete base was also observed. Besides, there was evidence of 12.2% reduction in vertical compression stress; 15.6% reduce in vertical strain; 25.5% in longitudinal strain and 10.4% reduction in transverse compression stress on the lean concrete base interface with sub grade. Figures of these cases can be found in appendix A1.

**Table 3.6: Present percentages of reduction in magnitude of vertical displacement, stress and strain in X-X; Y-Y and Z-Z axis in concrete pavement with dowels**

<table>
<thead>
<tr>
<th>Layers</th>
<th>X-X axis (%)</th>
<th>Y-Y axis (%)</th>
<th>Z-Z axis (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(direction of traffic)</td>
<td>(Vertical)</td>
<td>(perpendicular to direction of traffic)</td>
</tr>
<tr>
<td>Concrete Slab</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum vertical displacement</td>
<td></td>
<td>(8.3)</td>
<td></td>
</tr>
<tr>
<td>Tension stress</td>
<td></td>
<td>(6.8)</td>
<td></td>
</tr>
<tr>
<td>Longitudinal compression stress</td>
<td>(8.4)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse compression stress</td>
<td></td>
<td></td>
<td>(1.1)</td>
</tr>
<tr>
<td>Concrete Slab interface with lean concrete base</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical compression stress</td>
<td></td>
<td>(29.4)</td>
<td></td>
</tr>
<tr>
<td>Vertical strain</td>
<td></td>
<td>(11.1)</td>
<td></td>
</tr>
<tr>
<td>Longitudinal compression stress</td>
<td>(49.5)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal strain</td>
<td>(52.5)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse compression stress</td>
<td></td>
<td></td>
<td>(64)</td>
</tr>
<tr>
<td>Transverse strain</td>
<td></td>
<td></td>
<td>(16.5)</td>
</tr>
<tr>
<td>Lean concrete base interface with subgrade</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical compression stress</td>
<td></td>
<td>(12.2)</td>
<td></td>
</tr>
<tr>
<td>Vertical strain</td>
<td></td>
<td>(15.6)</td>
<td></td>
</tr>
<tr>
<td>Longitudinal compression stress</td>
<td>(1.8)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal strain</td>
<td>(25.5)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse compression stress</td>
<td></td>
<td></td>
<td>(10.4)</td>
</tr>
<tr>
<td>Transverse strain</td>
<td></td>
<td></td>
<td>(4.4)</td>
</tr>
</tbody>
</table>
### Chapter 3: Finite Element Model Development with Parametric Studies

#### Table 3.7: Case study 2 load transfer efficiency (a) with dowels

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Leave Side (Unloaded), $\delta_U$ [mm]</td>
<td>-0.145</td>
<td>-0.141</td>
<td>-0.136</td>
<td>-0.143</td>
</tr>
<tr>
<td></td>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.148</td>
<td>-0.144</td>
<td>-0.144</td>
<td>-0.145</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>97.97</td>
<td>97.92</td>
<td>94.44</td>
<td>98.62</td>
<td>100.00</td>
</tr>
</tbody>
</table>

#### (b) Without dowels

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Leave Side (Unloaded), $\delta_U$ [mm]</td>
<td>-0.154</td>
<td>-0.151</td>
<td>-0.148</td>
<td>-0.155</td>
</tr>
<tr>
<td></td>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.158</td>
<td>-0.156</td>
<td>-0.157</td>
<td>-0.158</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>97.47</td>
<td>96.79</td>
<td>94.27</td>
<td>98.10</td>
<td>100.00</td>
</tr>
</tbody>
</table>

As expected LC5 has a LTE of 100% due to it being positioned evenly on both sides of the joint. Overall, the pavement indicates excellent LTE since the specified LTE for the majority of load cases exceeded approximately 94%. Further, an increased efficiency also shows that the dowel bars perform effectively as load transfer devices in the concrete pavement systems.

![Distance along Longitudinal Pavement Section (mm)](image)

(a) Without dowels
Figure 3.6: Maximum principal stress in lean concrete

Figure 3.6(a) and (b) show the maximum principal stress on concrete slab interface with lean concrete, results from the FE models with and without dowels along the longitudinal pavement section, respectively. From the observation of pavement models with and without dowels, one can see that the pavement with dowel has reduced the magnitude of compressive MPS by 92.5%. However, the pavements with dowel have increased the tensile MPS by 33%.
Figure 3.7: Maximum principal stress in sub grade

Figure 3.7(a) and (b) show the maximum principal stress on lean concrete interface with sub grade, and results from the FE models with and without dowels along the longitudinal pavement section, respectively. From the observation of pavement models
with and without dowels, one can see that there was no significant change in the magnitude of compressive MPS. However, the pavements with dowel did reduce the compressive MPS by 3%.

### 3.4 Three and four-layer concrete pavement system

#### 3.4.1 Behaviour of pavement with and without dowels

#### 3.4.1.1 Three layer concrete pavement system

From the observation of three-layer concrete pavement system with and without dowels, one can conclude that the dowel has significantly reduced the maximum vertical displacement at transverse joint along line $TT_{HH}$ by 12%; 18.7% reduction in vertical compression stress on the top of the concrete slab. However, 50.7% reduction in vertical compression stress; 39.5% reduce in transverse strain on the concrete slab interface with sub-base was also observed. Besides, there was 15% reduction in vertical compression stress; 16.4% reduce in transverse strain on the sub-base interface with sub grade. Figures of these cases can be found in appendix A1.
Table 3.8: Maximum displacement; Maximum transverse strain and compressive bending stress for three layer concrete pavements with and without dowels

<table>
<thead>
<tr>
<th>Top of Concrete slab</th>
<th>Max. displacement (mm)</th>
<th>Max. compressive bending stress(MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>With dowels</td>
<td>-0.271</td>
<td>-0.887</td>
</tr>
<tr>
<td>Without dowels</td>
<td>-0.307</td>
<td>-1.091</td>
</tr>
<tr>
<td><strong>Bottom of Concrete slab</strong></td>
<td><strong>Max. Transverse Strain (mm)</strong></td>
<td><strong>Max. compressive bending stress(MPa)</strong></td>
</tr>
<tr>
<td>With dowels</td>
<td>-0.000023</td>
<td>-0.394</td>
</tr>
<tr>
<td>Without dowels</td>
<td>-0.000038</td>
<td>-0.799</td>
</tr>
<tr>
<td><strong>Bottom of Sub-base</strong></td>
<td><strong>Max. Transverse Strain (mm)</strong></td>
<td><strong>Max. compressive bending stress(MPa)</strong></td>
</tr>
<tr>
<td>With dowels</td>
<td>-0.00138</td>
<td>-0.0163</td>
</tr>
<tr>
<td>Without dowels</td>
<td>-0.000165</td>
<td>-0.0192</td>
</tr>
</tbody>
</table>

Table 3.9: Three layers load transfer efficiency without dowels (a) without cavity (voids)

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Leave Side (Unloaded), $\delta_U$ [mm]</td>
<td>-0.220</td>
<td>-0.227</td>
<td>-0.232</td>
<td>-0.262</td>
</tr>
<tr>
<td></td>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.281</td>
<td>-0.294</td>
<td>-0.307</td>
<td>-0.298</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>78.29</td>
<td>77.21</td>
<td>75.57</td>
<td>87.92</td>
<td>100.00</td>
</tr>
</tbody>
</table>

(b) With cavity (voids)

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Leave Side (Unloaded), $\delta_U$ [mm]</td>
<td>-0.080</td>
<td>-0.083</td>
<td>-0.085</td>
<td>-0.221</td>
</tr>
<tr>
<td></td>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.508</td>
<td>-0.540</td>
<td>-0.570</td>
<td>-0.462</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>15.74</td>
<td>15.37</td>
<td>14.91</td>
<td>47.83</td>
<td>100.00</td>
</tr>
</tbody>
</table>

As expected LC5 has a LTE of 100% due to it being positioned evenly on both sides of the joint. Overall, the pavement without cavity indicates excellent LTE since the specified LTE for the majority of load cases exceeded approximately 75%. Further, decreased load transfer efficiency as much as 62.55% for LC1, 61.84% for LC2, 60.66% for LC3 and 40.09% for LC4 is present in the concrete pavement with cavity. Maximum displacements were increased by 44.69% for LC1, 45.56% for LC2, 46.14% for LC3,
35.50% for LC4 and 18.21% for LC5 in the three layer concrete pavement without dowels with cavity.

### 3.4.1.2 Four layer concrete pavement system

From the observation of four-layer concrete pavement system with and without dowels, one can conclude that the dowel had significantly reduced the maximum vertical displacement at the transverse joint along line TT_{HH} by 8% on the top of the concrete slab. However, there are 21.2% reductions in vertical compression stress; 15.3% reduce in transverse strain on the concrete slab interface with lean concrete also observed. Besides, there are 31.6% reductions in vertical compression stress; 21.6% reduce in transverse strain on the lean concrete interface with sub-base. Finally, it has 10.3% reduction in vertical compression stress and 18.3% reduce in transverse strain on the sub-base interface with sub grade. Figures of these cases can be found in appendix A1.

**Table 3.10: Maximum displacement; Maximum transverse strain and compressive bending stress for four layer concrete pavements with and without dowels**

<table>
<thead>
<tr>
<th>Top of Concrete slab</th>
<th>Max. displacement (mm)</th>
<th>Max. compressive bending stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>With dowels</td>
<td>-0.143</td>
<td>-</td>
</tr>
<tr>
<td>Without dowels</td>
<td>-0.155</td>
<td>-</td>
</tr>
<tr>
<td>Bottom of Concrete slab</td>
<td>Max. Transverse Strain (mm)</td>
<td>Max. compressive bending stress (MPa)</td>
</tr>
<tr>
<td>With dowels</td>
<td>-0.0000298</td>
<td>-0.878</td>
</tr>
<tr>
<td>Without dowels</td>
<td>-0.0000352</td>
<td>-1.114</td>
</tr>
<tr>
<td>Bottom of lean concrete</td>
<td>Max. Transverse Strain (mm)</td>
<td>Max. compressive bending stress (MPa)</td>
</tr>
<tr>
<td>With dowels</td>
<td>-0.0000138</td>
<td>-0.0199</td>
</tr>
<tr>
<td>Without dowels</td>
<td>-0.0000176</td>
<td>-0.0291</td>
</tr>
<tr>
<td>Bottom of Sub-base</td>
<td>Max. Transverse Strain (mm)</td>
<td>Max. compressive bending stress (MPa)</td>
</tr>
<tr>
<td>With dowels</td>
<td>-0.0000397</td>
<td>-0.00796</td>
</tr>
<tr>
<td>Without dowels</td>
<td>-0.0000486</td>
<td>-0.00887</td>
</tr>
</tbody>
</table>
Table 3.11: Four layers load transfer efficiency without dowels (a) without cavity (voids)

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Leave Side (Unloaded), δ_U [mm]</td>
<td>-0.153</td>
<td>-0.150</td>
<td>-0.146</td>
<td>-0.153</td>
<td>-0.156</td>
</tr>
<tr>
<td>Approach Side (Loaded), δ_L [mm]</td>
<td>-0.156</td>
<td>-0.154</td>
<td>-0.155</td>
<td>-0.156</td>
<td>-0.156</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>98.08</td>
<td>97.40</td>
<td>94.19</td>
<td>98.08</td>
<td>100.00</td>
</tr>
</tbody>
</table>

(b) With cavity (voids)

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Leave Side (Unloaded), δ_U [mm]</td>
<td>-0.130</td>
<td>-0.128</td>
<td>-0.124</td>
<td>-0.151</td>
<td>-0.173</td>
</tr>
<tr>
<td>Approach Side (Loaded), δ_L [mm]</td>
<td>-0.196</td>
<td>-0.199</td>
<td>-0.207</td>
<td>-0.192</td>
<td>-0.173</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>66.32</td>
<td>64.32</td>
<td>59.90</td>
<td>78.64</td>
<td>100.00</td>
</tr>
</tbody>
</table>

As expected LC5 has a LTE of 100% due to it being positioned evenly on both sides of the joint. Overall, the pavement without cavity indicates excellent LTE since the specified LTE for the majority of load cases exceed approximately 94%. Further, there is evidence of decreased load transfer efficiency as much as 34.29% for LC3 in the concrete pavement with cavity.
3.4.2 Behaviour of dowelled joint with and without lean concrete

Figure 3.8: Maximum vertical displacement of concrete pavement at transverse joint along line $TT_{HH}$
Figure 3.8 (a) and (b) show the maximum displacement results from the FE models with and without lean concrete along the joint with all five load cases, respectively. From the observation of pavement models with dual wheel load and dowels spacing at 300mm centres, the maximum vertical displacement at transverse joint along the line $TT_{HH}$ for the load case 3 were reduced by 47% in dowel-jointed pavement with lean concrete.

Figure 3.9: Shear force diagram for concrete pavement
Figure 3.9 (a) and (b) show the shear force diagram for FE models with and without lean concrete. From the FE model analysis results show that the ratio of load transfer has been reduced from 15.45% to 4.365% in the dowel-jointed pavement with lean concrete for the load cases 3 where dowel was right beneath the wheel load. This observation can be
explain by the bending moment diagram shown in figure 3.10 (a) and (b) where in the dowel-jointed pavement without lean concrete at the moment peak at 110 kN.mm whereas for dowel-jointed pavement with lean concrete the moment peak at 28 kN.mm which means less stress has been incurred in transferring to the leave side.

Table 3.12: Load transfer efficiency (a) with lean concrete

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Leave Side (Unloaded), $\delta_U$ [mm]</td>
<td>-0.144</td>
<td>-0.140</td>
<td>-0.135</td>
<td>-0.142</td>
</tr>
<tr>
<td></td>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.147</td>
<td>-0.144</td>
<td>-0.143</td>
<td>-0.144</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>97.96</td>
<td>97.22</td>
<td>94.40</td>
<td>98.62</td>
<td>100.00</td>
</tr>
</tbody>
</table>

(b) Without lean concrete

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Leave Side (Unloaded), $\delta_U$ [mm]</td>
<td>-0.239</td>
<td>-0.247</td>
<td>-0.253</td>
<td>-0.269</td>
</tr>
<tr>
<td></td>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.252</td>
<td>-0.261</td>
<td>-0.272</td>
<td>-0.277</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>94.84</td>
<td>94.64</td>
<td>93.01</td>
<td>97.11</td>
<td>100.00</td>
</tr>
</tbody>
</table>

As expected LC5 has a LTE of 100% due to it being positioned evenly on both sides of the joint. Overall, the pavement indicates excellent LTE since the specified LTE for the majority of load cases exceeds approximately 94% for concrete pavement with lean concrete and 93% for concrete pavement without lean concrete. Further, an increased efficiency also shows that the pavement with lean concrete performs more effectively.
Figure 3.11: Maximum Principal Stress for concrete pavements along Longitudinal Pavement Section

Comparison of the MPS distribution across the granular base for concrete pavement with and without LCB is show in Figure 3.11(a) and (b). It can be observed that peak MPS is about 6.7 times greater and steeper variation in the distribution pattern in the concrete pavement without LCB than that with LCB. In the concrete pavement with LCB, MPS level at the bottom of the granular base, it stays constant and the difference in the magnitude at the top of the granular base between peak MPS and MPS at the symmetry
plane is 0.0045 MPa. Whereas in the concrete pavement without LCB, the MPS at the bottom of the granular base varies between positive and negative, and the MPS distribution curve is very steep this makes the centre area of the granular base (under wheel path) subject to very high tensile MPS up to 0.43 MPa. This indicates that without the provision of the LCB, the zone along the centre of the granular base (1800 mm from the symmetry plane) in the direction of traffic is critical to the impact load. Overall, MPS is critical at the mid and bottom of the concrete slab joint face. Loss of support at the joint, increase in the peak MPS at mid and bottom of the slab is expected. LCB reduces the MPS in the granular base. Without LCB, load transferred through the concrete slab over the granular base is much larger, and makes the centre of the granular base a very weak zone. This can be easily subjected to failure under more critical action of stress, shear and bending.

3.4.3 Behaviour of doweled joint with and without voids

3.4.3.1 Vertical displacement

Figure 3.12 (a) and (b) shows vertical displacement at transverse joint along the line $TT_{HH}$ for all loading cases for pavement with and without voids. Table 3.13 (a) and (b) summarises the maximum deflection and LTE of the five LC with and without voids.
Figure 3.12: Vertical displacement at transverse joint along line $TT_{HH}$

Figure 3.12 (a) indicates that the vertical displacement, on the approach side, increases as the load approaches the joint. The difference in displacement becomes less significant as the load approaches the joint. Note that the difference between $LC1$ and $LC2$ is less than
compared with LC2 and LC3, LC3 and LC4, and LC4 and LC5. This may be due to half of LC1 being positioned over the void and therefore the base course is reducing the displacement. The displacement distribution of LC3 to LC5 show peaks at 750 and 1050 mm along the line TTHH. At these locations the void begins and ends and presents geometrical discontinuities which cause displacement peaks. The displacement on the leave side for LC1 is lowers that on the approach side. This difference indicates that the load transfer efficiency (LTE) is reduced. Note that LC5 produces the same displacement on both the approach and leave sides because it is divided equally between both concrete slabs.

From the observation of figure 3.12(a) and (b), the maximum vertical displacement for the load case 1&2 were increased by 16.7%; 18.2% increased for load case 3&4 and 20.6% increased for load case 5 respectively in pavement with cavity (voids).

3.4.3.2 Load transfer efficiency at the top of the concrete layer.

‘Load transfer’ is a term used to describe the transfer of load across discontinuities such as joints (Austroads, 2008). When vehicular loading is applied near a pavement joint, both loaded and unloaded slabs deflect because a portion of the load applied to the loaded slab is transferred to the unloaded slab. The amount the unloaded slab deflects is directly related to joint performance. If a joint is performing perfectly, both the loaded and unloaded slabs deflect equally. During vehicular loading that is relatively close to the joint of the concrete slab, dowel bars immediately under the applied load assume a major portion of the load with other dowel bars assuming progressively lesser amounts (Huang,
2010). The LTE is defined as a parameter that measures the load transfer from the loaded side to the unloaded side of the joint, and it is given by (Austroads, 2008) as:

\[ LTE = \frac{\delta_U}{\delta_L} \times 100 \]

Where, \( \delta_U \) and \( \delta_L \) are respectively the unloaded and loaded vertical displacements.

**Table 3.13: Load transfer efficiency (a) with cavity (voids)**

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Leave Side (Unloaded), ( \delta_U ) [mm]</td>
<td>-0.28</td>
<td>-0.29</td>
<td>-0.31</td>
<td>-0.32</td>
</tr>
<tr>
<td></td>
<td>Approach Side (Loaded), ( \delta_L ) [mm]</td>
<td>-0.30</td>
<td>-0.31</td>
<td>-0.33</td>
<td>-0.34</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>93.33</td>
<td>94.90</td>
<td>93.93</td>
<td>95.59</td>
<td>100.00</td>
</tr>
</tbody>
</table>

**Table 3.13: Load transfer efficiency (b) without cavity (voids)**

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Leave Side (Unloaded), ( \delta_U ) [mm]</td>
<td>-0.24</td>
<td>-0.25</td>
<td>-0.25</td>
<td>-0.27</td>
</tr>
<tr>
<td></td>
<td>Approach Side (Loaded), ( \delta_L ) [mm]</td>
<td>-0.25</td>
<td>-0.26</td>
<td>-0.27</td>
<td>-0.28</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>96.0</td>
<td>96.15</td>
<td>92.59</td>
<td>96.43</td>
<td>100.00</td>
</tr>
</tbody>
</table>

\( LC1 \) has a LTE which is marginally increased when compared to \( LC2 \) because \( LC1 \) is only partly distributed over the void. \( LC3 \) presents the lowest LTE when compared to the other loading cases for both pavements with and without voids; see Table 3.13 (a) and (b), which could be due to its location on the edge of the joint. \( LC4 \) shows an increased LTE because it shares the load with the leave side of the concrete slab. As expected \( LC5 \) has a LTE of 100% due to it being positioned evenly on both sides of the joint. Overall, the pavement indicates excellent LTE since the specified LTE for the majority of load
cases exceeded 92%. Further, an increased efficiency also shows that the dowel bars perform effectively as load transfer device in the concrete pavement systems.

### 3.4.3.3 Vertical stress at dowel-concrete interface

In this subsection, the vertical stress along the line $DD$ at the dowel/concrete interface is evaluated for the five load cases. The dowel bars length is 460 mm with both ends of 220 mm encased in concrete and 20 mm at the joint opening. The vertical stress is evaluated to predict fatigue fracturing at the interface (Quintus, 1993). Figure 3.13 illustrates that most of the vertical stresses are acting in compression. It is also observed that the void may cause an increase in the vertical stress on the concrete slabs edge slab at the joint. This is because the larger deformation at the joints (see Figure 3.12: a, and b), will introduce higher vertical stress along the dowel/concrete interface. The side of approach shows vertical stresses that are increased when compared to the side of leave.

![Figure 3.13: Vertical Stress at Dowel/Concrete Interface.](image)
3.4.3.4 Load transfer ratio

Figure 3.14: Shear force diagram for pavement

Figure 3.14 (a) and (b) shows the shear force diagram in the pavement with and without dowels and table 3.14 shows the ratio of load transfer in the pavement with and without voids (the amount of transferred load by the dowels divided by the amount of applied wheel load). The load transfer ratios were 18.55% and 15.45% respectively in pavement with and without voids for the load case 3 where wheel load was applied directly beneath the dowel.
Table 3.14: Ratio of load transfer in pavement with and without cavity (voids)

<table>
<thead>
<tr>
<th>Concrete pavement</th>
<th>Applied Load (kN)</th>
<th>Transferred Load (kN)</th>
<th>Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without voids</td>
<td>20</td>
<td>3.71</td>
<td>18.55</td>
</tr>
<tr>
<td>With voids</td>
<td>20</td>
<td>3.09</td>
<td>15.45</td>
</tr>
</tbody>
</table>

Table 3.15: Load transfer efficiency (a) with dowels with cavity (voids)

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Leave Side (Unloaded), $\delta_U$ [mm]</th>
<th>Approach Side (Loaded), $\delta_L$ [mm]</th>
<th>Load Transfer Efficiency, LTE [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>LC1</td>
<td>-0.280</td>
<td>-0.300</td>
<td>93.33</td>
</tr>
<tr>
<td>LC2</td>
<td>-0.290</td>
<td>-0.310</td>
<td>94.90</td>
</tr>
<tr>
<td>LC3</td>
<td>-0.310</td>
<td>-0.330</td>
<td>93.93</td>
</tr>
<tr>
<td>LC4</td>
<td>-0.320</td>
<td>-0.340</td>
<td>95.59</td>
</tr>
<tr>
<td>LC5</td>
<td>-0.340</td>
<td>-0.340</td>
<td>100.00</td>
</tr>
</tbody>
</table>

(b) Without dowels with Cavity (voids)

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Leave Side (Unloaded), $\delta_U$ [mm]</th>
<th>Approach Side (Loaded), $\delta_L$ [mm]</th>
<th>Load Transfer Efficiency, LTE [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>LC1</td>
<td>-0.080</td>
<td>-0.508</td>
<td>15.74</td>
</tr>
<tr>
<td>LC2</td>
<td>-0.083</td>
<td>-0.540</td>
<td>15.37</td>
</tr>
<tr>
<td>LC3</td>
<td>-0.085</td>
<td>-0.570</td>
<td>14.91</td>
</tr>
<tr>
<td>LC4</td>
<td>-0.221</td>
<td>-0.462</td>
<td>47.83</td>
</tr>
<tr>
<td>LC5</td>
<td>-0.346</td>
<td>-0.346</td>
<td>100.00</td>
</tr>
</tbody>
</table>

As expected LC5 has a LTE of 100% due to it being positioned evenly on both sides of the joint. Overall, the pavement with dowels with cavity indicates excellent LTE since the specified LTE for the majority of load cases exceeded approximately 93%. Further, decreased load transfer efficiency as much as 77.59% for LC1, 79.53% for LC2, 79.02% for LC3 and 47.76% for LC4 was seen in the concrete pavement without dowels with cavity. Maximum displacements were increased by 40.94% for LC1, 42.59% for LC2, 42.10% for LC3, 26.41% for LC4 and 1.73% for LC5 in the concrete pavement without dowels with cavity.
3.4.4 Behaviour of undowered and doweled joint under subgrade Strength Variation of California Baring Ratio (CBR)

Figure 3.15: Vertical displacement at transverse joint along the line TT_{HH} for load case 3 for pavement with dual wheel loading
The maximum vertical displacements were reduced by 23.8%, 47.4% and 60.5% respectively with increased sub-grade strength from 30 to 50, 100 and 170 MPa; see Figure 3.15 (a). The maximum vertical displacements were reduced by 25.3%, 50.0% and 63.5% respectively with increased sub-grade strength from 30 to 50, 100 and 170 MPa, as illustrated in Figure 3.15 (b).

However, from figure 3.15 (a) and (b) one can concluded that the pavement with dowels can reduce the maximum displacement by around 9.7%, 11.4%, 14.2% and 16.4% respectively with sub-grade strength range from 30 to 50, 100 and 170 MPa and this also demonstrated that the dowel bars perform effectively as load transfer devices in the concrete pavement systems.

Table 3.16: Shear force distribution of the dowels spacing at 300 mm centre for dual wheel loading and load case 3 only with sub-grade varies from 30 to 170 MPa

<table>
<thead>
<tr>
<th>Dowel Number</th>
<th>Applied Load (kN)</th>
<th>Es-30 Mpa Transferred Load (kN)</th>
<th>Ratio (%)</th>
<th>Es-50 MPa</th>
<th>Es-100 MPa</th>
<th>Es-170 MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>1.7844</td>
<td>8.922</td>
<td>1.7772</td>
<td>1.7657</td>
<td>1.7556</td>
</tr>
<tr>
<td>2</td>
<td>20</td>
<td>1.7848</td>
<td>8.924</td>
<td>1.7774</td>
<td>1.7656</td>
<td>1.7552</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
<td>0.4915</td>
<td>2.488</td>
<td>0.4843</td>
<td>0.4730</td>
<td>0.4632</td>
</tr>
<tr>
<td>4</td>
<td>20</td>
<td>0.1566</td>
<td>0.783</td>
<td>0.1496</td>
<td>0.1131</td>
<td>0.1298</td>
</tr>
</tbody>
</table>

From the table, one can observes that the first eight engaged dowels carry more than 42.17% of the transferred shear force across the joints and there will be no significant effect on the shear force distribution with sub-grade varying from 30 to 170 MPa.
Table 3.17: Load Transfer Efficiency for Four layer concrete pavement with dowels spacing at 300mm centres (a) Es=30 MPa

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Leave Side (Unloaded), $\delta_U$ [mm]</th>
<th>Approach Side (Loaded), $\delta_L$ [mm]</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>-0.217</td>
<td>-0.208</td>
<td>-0.199</td>
<td>-0.205</td>
<td>-0.207</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>98.19</td>
<td>98.11</td>
<td>96.14</td>
<td>98.55</td>
<td>100.00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(b) Es=50 MPa

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Leave Side (Unloaded), $\delta_U$ [mm]</th>
<th>Approach Side (Loaded), $\delta_L$ [mm]</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>-0.144</td>
<td>-0.140</td>
<td>-0.135</td>
<td>-0.142</td>
<td>-0.144</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>97.30</td>
<td>97.22</td>
<td>94.40</td>
<td>98.62</td>
<td>100.00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(c) Es=100 MPa

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Leave Side (Unloaded), $\delta_U$ [mm]</th>
<th>Approach Side (Loaded), $\delta_L$ [mm]</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>-0.087</td>
<td>-0.086</td>
<td>-0.085</td>
<td>-0.091</td>
<td>-0.093</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>96.67</td>
<td>95.55</td>
<td>91.40</td>
<td>96.81</td>
<td>100.00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(d) Es=170 MPa

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Leave Side (Unloaded), $\delta_U$ [mm]</th>
<th>Approach Side (Loaded), $\delta_L$ [mm]</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>-0.061</td>
<td>-0.062</td>
<td>-0.062</td>
<td>-0.068</td>
<td>-0.070</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>95.31</td>
<td>95.38</td>
<td>89.86</td>
<td>97.14</td>
<td>100.00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

This illustrates vertical displacement at transverse joint along the line $TT_{HH}$ for load case 3 on four layer concrete pavements with dowels at 300mm spacing with dual wheel loading, the maximum vertical displacements were reduced by 32.16%, 57.29% and 68.84% respectively with increased sub-grade strength from 30 to 50, 100 and 170 MPa. However, there was no significant change in the load transfer efficiency with the sub-grade CBR varying from 3% to 17%.
3.4.5 Behaviour of doweled joint

3.4.5.1 Load magnitude variation

![Graph showing shear force variation for different load magnitudes and configurations of concrete pavements.]

(a) Three layer concrete pavements

(b) Four layer concrete pavements

Figure 3.16: Shear force diagram with 1500 kPa loading for
Table 3.18: Shear force transmitted by a dowel for three layer concrete pavement

<table>
<thead>
<tr>
<th>Dowel Number</th>
<th>Single-wheel Loading</th>
<th>Applied Load (kN)</th>
<th>Transferred Load (kN)</th>
<th>Ratio (%)</th>
<th>Dual-wheel Loading</th>
<th>Applied Load (kN)</th>
<th>Transferred Load (kN)</th>
<th>Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SWL</td>
<td>40</td>
<td>7.98</td>
<td>19.94</td>
<td>DWL</td>
<td>40</td>
<td>11.98</td>
<td>29.97</td>
</tr>
<tr>
<td>2</td>
<td>SWL</td>
<td>40</td>
<td>4.06</td>
<td>10.15</td>
<td>DWL</td>
<td>40</td>
<td>11.89</td>
<td>29.73</td>
</tr>
<tr>
<td>3</td>
<td>SWL</td>
<td>40</td>
<td>1.23</td>
<td>3.07</td>
<td>DWL</td>
<td>40</td>
<td>5.23</td>
<td>13.08</td>
</tr>
<tr>
<td>4</td>
<td>SWL</td>
<td>40</td>
<td>0.70</td>
<td>1.75</td>
<td>DWL</td>
<td>40</td>
<td>1.84</td>
<td>4.60</td>
</tr>
</tbody>
</table>

Figure 3.16 (a) and (b) demonstrates the dowel shear distribution (shear force diagram) for three and four layer pavement. Table 3.18 shows the amount of shear force transmitted by a dowel. Table 3.18, one can observe that eight engaged dowels carry more than 70% and up to 95% of the transferred shear force across the joints for model three layer pavements in single and dual-wheel loading cases respectively, for both 750 kPa and 1500 kPa loading. A dowel is considered “engaged” if the shear force carried by the dowel is larger than 1% of the total shear force transferred across the doweled joint. The number of engaged dowels appears to be independent of the magnitude of the applied wheel load pressure.

![Figure 3.17: Vertical displacement between approach side (loaded) and leave side (adjacent slab) segments at transverse joint along the line TT<sub>HH</sub>](image-url)
The largest positive relative displacement is observed at the origin, where the wheel load is applied. In fact, the loaded slab always has a larger curvature than the adjacent slab, maximum displacement always occurs beneath the wheel load as does the maximum relative displacement along the joint. Figure 3.17 shows that the maximum displacements have been increased by 100% for load case 3 if the applied load increased from 750 kPa to 1500 kPa. So it can be seen that the load has a significant influence on displacement.

Table 3.19: Ratio of load transfer for four layer concrete pavements (the amount of transferred load by the dowels divided by the amount of applied wheel load)

<table>
<thead>
<tr>
<th>Dowel Number</th>
<th>Dual-wheel Loading</th>
<th>Applied Load (kN)</th>
<th>Transferred Load (kN)</th>
<th>Ratio (%)</th>
<th>Dual-wheel Loading</th>
<th>Applied Load (kN)</th>
<th>Transferred Load (kN)</th>
<th>Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>DWL</td>
<td>20</td>
<td>1.78</td>
<td>8.89</td>
<td>DWL</td>
<td>40</td>
<td>3.55</td>
<td>8.89</td>
</tr>
<tr>
<td>2</td>
<td>DWL</td>
<td>20</td>
<td>1.78</td>
<td>8.89</td>
<td>DWL</td>
<td>40</td>
<td>3.55</td>
<td>8.89</td>
</tr>
<tr>
<td>3</td>
<td>DWL</td>
<td>20</td>
<td>0.48</td>
<td>2.42</td>
<td>DWL</td>
<td>40</td>
<td>0.97</td>
<td>2.42</td>
</tr>
<tr>
<td>4</td>
<td>DWL</td>
<td>20</td>
<td>0.15</td>
<td>0.75</td>
<td>DWL</td>
<td>40</td>
<td>0.30</td>
<td>0.75</td>
</tr>
</tbody>
</table>

This ratio increased with the increase of applied type pressure. That means more wheel load can be transferred to the adjacent concrete slab if wheel load pressure increases. This phenomenon is also evident in the dowel shear force distribution. The inner four dowels of the 1500 kPa case carried almost twice the shear force of those of the 750 kPa case. However, the high wheel load increases did not significantly affect a load transfer ratio. Meanwhile, it also increased the demand on a few inner dowels beneath the wheel load, which may cause more damage to the joints and eventually lead to pavement failure.
Table 3.20: Load transfer efficiency for four layer concrete pavement with dowels spacing at 300mm centres Es=50 MPa (a) 20 kN applied wheel load

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Leave Side (Unloaded), $\delta_U$ [mm]</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>-0.144</td>
<td>-0.140</td>
<td>-0.135</td>
<td>-0.142</td>
<td>-0.144</td>
<td></td>
</tr>
<tr>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.148</td>
<td>-0.144</td>
<td>-0.143</td>
<td>-0.144</td>
<td>-0.144</td>
<td></td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>97.30</td>
<td>97.22</td>
<td>94.40</td>
<td>98.62</td>
<td>100.00</td>
<td></td>
</tr>
</tbody>
</table>

(b) 40 kN applied wheel load

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Leave Side (Unloaded), $\delta_U$ [mm]</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>-0.289</td>
<td>-0.280</td>
<td>-0.270</td>
<td>-0.284</td>
<td>-0.287</td>
<td></td>
</tr>
<tr>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.295</td>
<td>-0.287</td>
<td>-0.287</td>
<td>-0.288</td>
<td>-0.287</td>
<td></td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>97.97</td>
<td>97.56</td>
<td>94.08</td>
<td>98.61</td>
<td>100.00</td>
<td></td>
</tr>
</tbody>
</table>

This illustrates vertical displacement at transverse joint along the line $TT_{HH}$ for all load case on four layer concrete pavements with dowels at 300mm spacing with dual wheel loading. The maximum vertical displacements were increased by 100% while double the wheel loading. However; there was no significant change in the load transfer efficiency when the wheel loading increased from 20 kN to 40 kN.

3.4.5.2 Dowel spacing variation

Four different dowels spacing, 225 mm, 300 mm, 450 mm and 600 mm, were evaluated under a single and dual wheel loading. Figure 3.18 shows the dowel shear force distribution. Table 3.21 represents the contribution of each dowel to the total amount of transferred load and number of dowels engaged in the transferred load. Figure 3.19 show the normalized maximum displacement of various dowels spacing of approach and leave side. Table 3.22 shows the value of maximum displacement of approach and leave side for various dowel spacing.
Figure 3.18: Dowel shear force distribution

Figure 3.18: Four lines of dowel shear force distribution, each dowel in the 300 mm, 450 mm and 600 mm spacing cases carries more shear force than the 225 mm spacing case due to the wider spacing. Nevertheless, the total amount of transferred shear for the 225 mm spacing case was about 6%, 14.3% and 26.2% larger than for the 300 mm, 450 mm and 600 mm spacing cases respectively in the first 1800 mm. The total amount of transferred load does not change much in terms of relative location between wheel load and dowel, if the spacing remains the same.

Table 3.21 shows that only three dowels were engaged for 600 mm spacing cases within 1800 mm symmetric region, while four dowels were engaged in 450 mm spacing case, six dowels were engaged for 300 mm spacing cases, and eight were engaged for 225 mm spacing case.
Table 3.21: Contribution of each dowel to the total amount of transferred load and number of dowels engaged in transferred load

<table>
<thead>
<tr>
<th>Dowel Number</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>Total (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applied Load (kN)</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Transferred Load (kN)</td>
<td>5.18</td>
<td>4.96</td>
<td>3.44</td>
<td>2.68</td>
<td>1.46</td>
<td>1.04</td>
<td>0.75</td>
<td>0.73</td>
<td></td>
</tr>
<tr>
<td><strong>225 mm spacing</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>100</strong></td>
</tr>
<tr>
<td>Ratio (%)</td>
<td>25.91</td>
<td>24.80</td>
<td>17.22</td>
<td>13.39</td>
<td>7.30</td>
<td>5.20</td>
<td>3.76</td>
<td>3.67</td>
<td></td>
</tr>
<tr>
<td>Transferred Load (kN)</td>
<td>5.99</td>
<td>5.95</td>
<td>2.66</td>
<td>2.61</td>
<td>0.92</td>
<td>0.70</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>300 mm spacing</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>94.17</strong></td>
</tr>
<tr>
<td>Ratio (%)</td>
<td>29.94</td>
<td>29.73</td>
<td>13.30</td>
<td>13.10</td>
<td>4.60</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transferred Load (kN)</td>
<td>7.64</td>
<td>5.76</td>
<td>2.23</td>
<td>1.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>450 mm spacing</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>85.70</strong></td>
</tr>
<tr>
<td>Ratio (%)</td>
<td>38.20</td>
<td>28.80</td>
<td>11.20</td>
<td>7.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transferred Load (kN)</td>
<td>8.81</td>
<td>5.20</td>
<td>0.74</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>600 mm spacing</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>73.80</strong></td>
</tr>
<tr>
<td>Ratio (%)</td>
<td>44.10</td>
<td>26.00</td>
<td>3.70</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

This result suggests that the size of the region containing engaged dowels does not change significantly with dowel spacing, only the distribution of shear forces varies. The distribution of shear force varies from almost 100% for the 225 mm spacing cases to approximately 73.8% for the 600 mm spacing cases.
Figure 3.19: Maximum vertical displacement of approach side (loaded) and leave side (adjacent slab) at transverse joint along the line TT\textsc{hh} for dowels spacing various from 225 mm to 600 mm

The difference suggests that the stress response of the concrete slab is more sensitive to the relative location between applied wheel load and dowels than the dowel spacing. As a consequence, this extra deformation causes more bending stress in the approach side (loaded slab) segment. In contrast, less deformation was observed on the leave side (adjacent slab) and correspondingly less stress was generated in the leave side (adjacent slab). Friberg anticipated such behaviour in his paper, and our numerical results support his observation.

Table 3.22: Maximum vertical displacement on the approach and leave side at transverse joint along the line TT\textsc{hh} for varies dowels spacing (225 mm to 600 mm)

<table>
<thead>
<tr>
<th>Dowels Spacing (mm)</th>
<th>Approach Side (mm)</th>
<th>Leave Side (mm)</th>
<th>Load Transfer Efficiency (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>225</td>
<td>0.269</td>
<td>0.253</td>
<td>94.05</td>
</tr>
<tr>
<td>300</td>
<td>0.272</td>
<td>0.253</td>
<td>93.01</td>
</tr>
<tr>
<td>450</td>
<td>0.277</td>
<td>0.253</td>
<td>91.34</td>
</tr>
<tr>
<td>600</td>
<td>0.282</td>
<td>0.251</td>
<td>89.00</td>
</tr>
</tbody>
</table>
Table 3.23: Load transfer efficiency for four layer concrete pavement \( E_s = 50 \text{ MPa} \) with dowels spacing at (a) 225 mm centres

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>( \text{LC1} )</th>
<th>( \text{LC2} )</th>
<th>( \text{LC3} )</th>
<th>( \text{LC4} )</th>
<th>( \text{LC5} )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Maximum Vertical Displacements</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Leave Side (Unloaded), ( \delta_U ) [mm]</td>
<td>-0.144</td>
<td>-0.139</td>
<td>-0.134</td>
<td>-0.140</td>
<td>-0.142</td>
</tr>
<tr>
<td>Approach Side ( Loaded), ( \delta_L ) [mm]</td>
<td>-0.146</td>
<td>-0.142</td>
<td>-0.141</td>
<td>-0.142</td>
<td>-0.142</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>98.63</td>
<td>97.89</td>
<td>95.04</td>
<td>98.59</td>
<td>100.00</td>
</tr>
</tbody>
</table>

(b) 300 mm centres

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>( \text{LC1} )</th>
<th>( \text{LC2} )</th>
<th>( \text{LC3} )</th>
<th>( \text{LC4} )</th>
<th>( \text{LC5} )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Maximum Vertical Displacements</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Leave Side (Unloaded), ( \delta_U ) [mm]</td>
<td>-0.144</td>
<td>-0.140</td>
<td>-0.135</td>
<td>-0.142</td>
<td>-0.144</td>
</tr>
<tr>
<td>Approach Side ( Loaded), ( \delta_L ) [mm]</td>
<td>-0.148</td>
<td>-0.144</td>
<td>-0.143</td>
<td>-0.144</td>
<td>-0.144</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>97.30</td>
<td>97.22</td>
<td>94.40</td>
<td>98.62</td>
<td>100.00</td>
</tr>
</tbody>
</table>

(c) 450 mm centres

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>( \text{LC1} )</th>
<th>( \text{LC2} )</th>
<th>( \text{LC3} )</th>
<th>( \text{LC4} )</th>
<th>( \text{LC5} )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Maximum Vertical Displacements</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Leave Side (Unloaded), ( \delta_U ) [mm]</td>
<td>-0.146</td>
<td>-0.142</td>
<td>-0.137</td>
<td>-0.144</td>
<td>-0.146</td>
</tr>
<tr>
<td>Approach Side ( Loaded), ( \delta_L ) [mm]</td>
<td>-0.149</td>
<td>-0.146</td>
<td>-0.146</td>
<td>-0.147</td>
<td>-0.146</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>97.98</td>
<td>97.26</td>
<td>93.83</td>
<td>97.96</td>
<td>100.00</td>
</tr>
</tbody>
</table>

(d) 600 mm centres

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>( \text{LC1} )</th>
<th>( \text{LC2} )</th>
<th>( \text{LC3} )</th>
<th>( \text{LC4} )</th>
<th>( \text{LC5} )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Maximum Vertical Displacements</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Leave Side (Unloaded), ( \delta_U ) [mm]</td>
<td>-0.147</td>
<td>-0.143</td>
<td>-0.139</td>
<td>-0.146</td>
<td>-0.148</td>
</tr>
<tr>
<td>Approach Side ( Loaded), ( \delta_L ) [mm]</td>
<td>-0.151</td>
<td>-0.148</td>
<td>-0.148</td>
<td>-0.149</td>
<td>-0.148</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>97.35</td>
<td>96.62</td>
<td>93.92</td>
<td>97.98</td>
<td>100.00</td>
</tr>
</tbody>
</table>

The maximum vertical displacements were reduced by 1.4%, 2.9% and 3.6% respectively with dowel spacing at 225 mm to 300 mm, 450 mm and 600 mm. However, there was no significant change in the load transfer efficiency with the dowel spacing at 225 mm to 600 mm (LTE only increased by 1.12% for LC3 compared with dowels spacing at 225 mm with 600 mm).
3.4.5.3 Slab thickness change

Three different FE models were constructed with three different thicknesses of the concrete slab (250 mm, 375 mm, and 500 mm) to evaluate the behaviour of the doweled joint under slab thickness changes. Figure 3.20 (a), (b) and (c) illustrate the dowel shear force distribution for concrete thickness of 250 mm, 375 mm and 500 mm. Table 3.24 shows the contribution of each dowel to the total amount of load transfer. Figure 3.21 (a) and (b) show the bending moment diagram.

(a) 250 mm concrete pavement
Figure 3.20: Shear force diagram

Figure 3.20 (a) (b) and (c) show that the amount of load transfer increases, as the slab thickness increases. Further, the number of engaged dowels increases with slab thickness.
Table 3.24: Ratio of total load transfer for varies concrete pavement thickness

<table>
<thead>
<tr>
<th>Dowel Number</th>
<th>Applied Load (kN)</th>
<th>250 mm Thickness Transferred Load (kN)</th>
<th>Ratio (%)</th>
<th>375 mm Thickness Transferred Load (kN)</th>
<th>Ratio (%)</th>
<th>500 mm Thickness Transferred Load (kN)</th>
<th>Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>3.99</td>
<td>19.94</td>
<td>6.25</td>
<td>31.26</td>
<td>5.47</td>
<td>27.36</td>
</tr>
<tr>
<td>2</td>
<td>20</td>
<td>2.03</td>
<td>10.14</td>
<td>3.99</td>
<td>19.95</td>
<td>3.82</td>
<td>19.10</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
<td>1.99</td>
<td>9.92</td>
<td>3.81</td>
<td>19.04</td>
<td>3.57</td>
<td>17.83</td>
</tr>
<tr>
<td>4</td>
<td>20</td>
<td>0.61</td>
<td>3.07</td>
<td>2.17</td>
<td>10.85</td>
<td>2.74</td>
<td>13.72</td>
</tr>
<tr>
<td>5</td>
<td>20</td>
<td>0.55</td>
<td>2.77</td>
<td>1.90</td>
<td>9.50</td>
<td>2.30</td>
<td>11.36</td>
</tr>
<tr>
<td>6</td>
<td>20</td>
<td>0.35</td>
<td>1.75</td>
<td>1.05</td>
<td>5.23</td>
<td>1.66</td>
<td>8.28</td>
</tr>
<tr>
<td>Total (%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td><strong>47.60</strong></td>
<td></td>
<td><strong>95.82</strong></td>
<td></td>
<td><strong>97.65</strong></td>
</tr>
</tbody>
</table>

The ratio of load transfer also increases with slab thickness as shown in the table 3.24. For single wheel loading case with dowel spacing at 300 mm, a 250 mm concrete slab shows the total transfer ratio in the six engaged dowels of 47.6% whereas for 375 mm and 500 mm concrete slab the first six engaged dowels transfer 95.8% and 97.6% respectively. In fact, in theory the thicker concrete slab is stiffer and, therefore, develops less curvature along the loaded side of the joint. The deformed shape explains why more dowels are engaged in the load transfer for the thicker concrete slab models.

![Graph](image)

(a) 375 mm thickness concrete pavement
(b) 250 mm thickness concrete pavement

**Figure 3.21: Bending Moment diagram**

As a consequence of the stiffness increase for a thick concrete slab, a much higher percentage of load transfer was observed for the 375 mm and 500 mm cases. In fact, a thick concrete slab provides a significant benefit: higher load transfer. This can be explaining in the bending moment diagram shown in figure 3.21(a) & (b) where the moment for the immediate engaged dowel in LC3 (dowel local beneath applied wheel load), for 375 mm moment peak is 89.08 kN.mm whereas for 250 mm case the moment peak is 66.45 kN.mm, which mean less stress had been transfered though dowels across the adjacent slab.

**Table 3.25: Load transfer efficiency for three layer (a) 375 mm concrete slab pavement with dowels spacing at 300 mm centres Es=50 MPa single wheel loading**

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Leave Side (Unloaded), $\delta_U$ [mm]</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.179</td>
<td>-0.179</td>
<td>-0.178</td>
<td>-0.179</td>
<td>-0.178</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>93.85</td>
<td>94.41</td>
<td>94.38</td>
<td>97.21</td>
<td>100.00</td>
<td></td>
</tr>
</tbody>
</table>
The maximum vertical displacements at the transverse joint along the line TT\textsubscript{H}H were reduced by 16.2\% for LC1, 17.9\% for LC2, 20.2\% for LC3, 20.7\% for LC4 and 21.3\% for LC5 respectively with an increase of the concrete slab thickness from 375 mm to 500 mm. However, there was no significant change in the load transfer efficiency.

### 3.4.5.4 Single and dual wheel loads

Two different loading configurations (single and dual) were applied to the FE model to investigate the behaviour of dowelled joints under different wheel loads. Various numerical results are presented in section 3.4.8, including the dowel shear force distribution, the deformed shape of the concrete slab, and the maximum displacement.

#### Table 3.26: Dowel shear force distribution and the ratio of load transfer with respect to loading configuration

<table>
<thead>
<tr>
<th>Dowel Number</th>
<th>Applied Load (kN)</th>
<th>Single-wheel Loading Transferred Load (kN)</th>
<th>Ratio (%)</th>
<th>Dual-wheel Loading Transferred Load (kN)</th>
<th>Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>3.99</td>
<td>19.94</td>
<td>5.99</td>
<td>29.94</td>
</tr>
<tr>
<td>2</td>
<td>20</td>
<td>2.03</td>
<td>10.14</td>
<td>5.95</td>
<td>29.73</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
<td>1.99</td>
<td>9.92</td>
<td>2.66</td>
<td>13.30</td>
</tr>
<tr>
<td>4</td>
<td>20</td>
<td>0.61</td>
<td>3.07</td>
<td>2.61</td>
<td>13.10</td>
</tr>
<tr>
<td>5</td>
<td>20</td>
<td>0.55</td>
<td>2.77</td>
<td>0.92</td>
<td>4.60</td>
</tr>
<tr>
<td>6</td>
<td>20</td>
<td>0.35</td>
<td>1.75</td>
<td>0.70</td>
<td>3.50</td>
</tr>
</tbody>
</table>
The amount of transferred load increased when more wheel loads were applied. For the first six engaged dowels, there were only 47.60% of load transfer for single wheel loading and 94.17% for dual wheel loading case in concrete pavement with lean concrete with dowels spacing at 300 mm.

Figure 3.22: Maximum vertical displacement at transverse joint along TT on four layer concrete pavement with
Figure 3.22 (a) and (b) shows the wheel load associated deformed shape of the concrete slab along the joint line TT\textsubscript{HH}. The maximum displacement occurred beneath wheel load. In fact, dowels provided partial continuity to the discrete concrete slab segments by transferring shear force. The maximum displacement reduced by 51.2\% for single wheel loading when compared with dual wheel loading in concrete pavement with lean concrete and with dowels spacing at 300 mm.

Table 3.27: Load transfer efficiency for four layer concrete pavement with dowels spacing at 300 mm centres Es=50 MPa (a) single wheel loading

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Leave Side (Unloaded), $\delta_U$ [mm]</td>
<td>-0.070</td>
<td>-0.068</td>
<td>-0.066</td>
<td>-0.066</td>
</tr>
<tr>
<td></td>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.072</td>
<td>-0.069</td>
<td>-0.067</td>
<td>-0.067</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>97.22</td>
<td>98.55</td>
<td>98.50</td>
<td>98.50</td>
<td>100.00</td>
</tr>
</tbody>
</table>

(b) Dual wheel loading

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>LC1</th>
<th>LC2</th>
<th>LC3</th>
<th>LC4</th>
<th>LC5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Vertical Displacements</td>
<td>Leave Side (Unloaded), $\delta_U$ [mm]</td>
<td>-0.144</td>
<td>-0.140</td>
<td>-0.135</td>
<td>-0.142</td>
</tr>
<tr>
<td></td>
<td>Approach Side (Loaded), $\delta_L$ [mm]</td>
<td>-0.148</td>
<td>-0.144</td>
<td>-0.143</td>
<td>-0.144</td>
</tr>
<tr>
<td>Load Transfer Efficiency, LTE [%]</td>
<td>97.30</td>
<td>97.22</td>
<td>94.40</td>
<td>98.62</td>
<td>100.00</td>
</tr>
</tbody>
</table>

The maximum vertical displacements at transverse joint along TT\textsubscript{HH} were reduced by 51.35\% for LC1, 52.08\% for LC2, 53.15\% for LC3, 53.47\% for both LC4 and LC5 respectively with single wheel loading when compared to dual wheel loading. However, there was no significant change in the load transfer efficiency (LTE increased by only 4.1\% for LC3).
3.4.6 Comparison with existing observation

(1) Dowels have been used in rigid pavement systems for a long time and as a result, a great deal of research has been done on the behaviour of doweled joints. Friberg (1940) found that the maximum positive moment of the concrete slab for the edge loading case occurs right beneath the wheel load and that maximum negative moment occurred at a point $1.8 \ell_r$ from the point of loading, where $\ell_r$ is the radius of relative stiffness defined by Westergaard. He observed that the magnitude of bending moment showed only minor changes after this $1.8 \ell_r$ distance. Friberg further stated that “effective dowel shear decreases inversely as the distance of the dowel from the point of loading, to zero at a distance of $1.8 \ell_r$. No dowels beyond that point influence the moment at the load point”. This observation implies that most of the load transfer should occur within $1.8 \ell_r$ of the loading. However, Friberg’s analysis was based on dowel bars having a diameter of 19 mm or 22 mm and with dowel bar spacing ranging from 300 mm to 500 mm. If a larger dowel bar is used or the bar spacing is less than 300 mm, then Friberg’s model no longer applies. (Tabatabaie, 1979) modelled a doweled joint using finite element and was able to show that an effective length of $1.0 \ell_r$ is more appropriate for today’s construction practices. (Tabatabaie, 1979) was also able to show that a linear approximation does exist with the maximum shear occurring beneath the applied load itself. This study results supports the observation of (Tabatabaie, 1979).
(2) The different deformation suggests that the stress response of the concrete slab is more sensitive to the relative location between applied wheel load and dowels than the dowel spacing. As a consequence, this extra deformation caused more bending stress in the approach side (loaded slab) segment. In contrast, less deformation was observed on the leave side (adjacent slab) and correspondingly less stress was generated in the leave side (adjacent slab). (Friberg, 1940); (Kim, 2003) and (Bhattacharya, 2000) anticipated such behaviour in their paper, and this study results supports their observation.

(3) A thick concrete slab provides a significant benefit: higher load transfer and it develops less curvature along the loaded side of the joint. (Kim, 2003) notes such behaviour in his paper, and these study results agree with his observation.

(4) Wider spacing cases carries more shear force than the narrow spacing case and results suggest that the size of the region containing engaged dowels does not change significantly with dowel spacing, only the distribution of shear forces varies. (Kim, 2003) documents such behaviour in his paper, and these study results support his observation.

(5) The largest positive relative displacement is observed where the wheel load is applied. In fact, the loaded slab always has a larger curvature than the adjacent slab, maximum displacement always occurs beneath the wheel load and larger curvature is observed when the applied load is increased. So it can be seen that the
load has a significant influence on displacement. (Bhattacharya, 2000) anticipated such behaviour in his paper, and the results of this study support his observation.

3.4.7 Discussion of analysis

Dowel can significantly reduced maximum vertical displacement, compression stress and stain at the transverse joint and MPS (maximum principal stress) at the longitudinal joint as a means to bridge vehicle loads across adjacent slab, particularly in circumstances where heavy traffic or weak foundation are present. The dowel bars also allow axial thermal expansion and contraction of the concrete slab along the axis of the dowel.

(1) From the observation of pavement models of Case 1 with and without dowels, it can be seen that the dowel had significantly reduced the maximum vertical displacement at the transverse joint along line TT_{HH} by 10%; 37.3% reduction in longitudinal tension stress on the top of the concrete slab. However, 34.5% reduction in vertical compression stress and 30.4% in longitudinal strain on the concrete slab interface with cement treated sub-base were also noted. In the meantime, 14.6% reduction in vertical compression stress; 15% in vertical strain; 19.5% in longitudinal strain and 13.6% was seen in transverse compression stress on the cement treated sub-base interface with sub grade. Overall, the pavement indicates excellent LTE since the specified LTE for the majority of load cases exceeded approximately 94% for concrete pavement with dowels and 92% for concrete pavement without dowels. Furthermore, an increased efficiency also shows that the dowel bars perform effectively as load transfer devices in the concrete pavement systems.
(2) From the observation of pavement models of Case 2 with and without dowels, one can concluded that the dowel significantly reduces the maximum vertical displacement at transverse joint along line TT\textsubscript{HH} by 8.3%; 6.8% reduction in vertical tension stress and 8.4% in longitudinal compression stress on the top of the concrete slab. However, 29.4% reduction in vertical compression stress; 11.1% reduce in vertical strain; 49.5% reduction in longitudinal compression stress; 52.5% reduce in longitudinal strain; 64% reduction in transverse compression stress and 16.5% reduce in transverse strain on the concrete slab interface with lean concrete base was also observed. At the meantime, 12.2% reduction in vertical compression stress; 15.6% reduce in vertical strain; 25.5% in longitudinal strain and 10.4% reduction in transverse compression stress was seen on the lean concrete base interface with sub grade. Overall, the pavement indicates excellent LTE since the specified LTE for the majority of load cases exceeded approximately 94%. Meanwhile, from the observation of pavement models with and without dowels, one can see that the pavement with dowel has reduced the magnitude of compressive MPS by 92.5%. However, the pavements with dowel have increased the tensile MPS by 33% (maximum principal stress on concrete slab interface with lean concrete base, results from the FE models with and without dowels along the longitudinal pavement section).

(3) From the observation of the three-layer concrete pavement system with and without dowels, one can conclude that the dowel had significantly reduced the maximum vertical displacement at the transverse joints along line TT\textsubscript{HH} by 12%; 18.7%...
reduction in the vertical compression stress on the top of the concrete slab. However, 50.7% reduction in vertical compression stress; 39.5% reduce in transverse strain on the concrete slab interface with sub-base were also observed. In the meantime, 15% reduction in vertical compression stress; equal to 16.4% reduced transverse strain on the sub-base interface with sub grade. The pavement without cavity indicates excellent LTE since the specified LTE for the majority of load cases exceeded approximately 75%. Further, decreased load transfer efficiency as much as 62.55% for LC1, 61.84% for LC2, 60.66% for LC3 and 40.09% for LC4 in the concrete pavement without dowels with cavity was seen. Maximum displacements were increased by 44.69% for LC1, 45.56% for LC2, 46.14% for LC3, 35.50% for LC4 and 18.21% for LC5 in the three layer concrete pavement without dowels with cavity.

(4) From the observation of a four-layer concrete pavement system with and without dowels, one can conclude that the dowel had significantly reduced the maximum vertical displacement at the transverse joint along line $TT_{HH}$ by 8% on the top of the concrete slab. However, 21.2% reduction in vertical compression stress; 15.3% reduced transverse strain on the concrete slab interface with lean concrete was also observed. In the meantime, 31.6% reductions in vertical compression stress; 21.6% reduced transverse strain on the lean concrete interface with sub-base. Finally, 10.3% reduction in vertical compression stress meant 18.3% reduced transverse strain on the sub-base interface with sub grade. The pavement without cavity indicates excellent LTE since the specified LTE for the majority of load cases exceeded...
approximately 94%. Further, decreased load transfer efficiency was as much as 34.29% for LC3 in the concrete pavement with cavity.

With the provision of lean concrete, maximum vertical displacement along transverse joint and MPS along longitudinal joints can reduced significantly. Lean concrete not only provide resistance to erosion and pumping between wearing surface and base course, as well as better foundation support in the pavements.

(5) From the observation of pavement models with dual wheel load and dowels spacing at 300 mm centres, the maximum vertical displacement at the transverse joint along line TT\textsubscript{HH} for the load case 3 was reduced by 47% in dowel-jointed pavement with lean concrete. From the FE model analysis results it shows that the ratio of load transfer has been reduced from 15.45% to 4.365% in the dowel-jointed pavement with lean concrete for the load cases 3 where dowel was right beneath the wheel load. Overall, the pavement indicates excellent LTE since the specified LTE for the majority of load cases exceeds approximately 94% for concrete pavement with lean concrete and 93% for concrete pavement without lean concrete. It can be observed that peak MPS is about 6.7 times greater and steeper variation is seen in the distribution pattern in the concrete pavement without LCB than with LCB.

Void underneath the concrete slab can cause significantly increase in maximum vertical displacement along transverse joint and vertical stress at dowel/concrete interface. When a load W is applied on one slab near the joint, part of the load will be transferred to the adjacent slab through the dowel group. If the dowels are 100% efficient, both slabs will deflect the same amount and relative forces under both slabs
will be the same, each equal to 0.5W, which is also the total shear force transferred
by the dowel group. If the dowel are less than 100% efficient, as in the case of old
pavements where some dowels become loose and erosion or pumping exits on the
base course due to repeated loading and subjected to high bearing stresses, the
reactive forces under the loaded slab will be greater than 0.5W, while those under the
unloaded slab will be smaller than 0.5W.

(6) The cavity (void) underneath the joint causes an increase in the vertical displacement
of the concrete slab and vertical stress at dowel bar/concrete interface. This will
increase the possibility of formation of cracks at the bottom of the slab. The loss of
sub base support at the joint through the formation of a void means that higher
stresses are applied on the dowel bars, which can lead to crushing of the concrete
resulting in dowel looseness and adversely affect the load transfer efficiency. From
this observation, the maximum displacement for the load case 1&2 were increased by
16.7%; 18.2% increased for load case 3&4 and 20.6% increased for load case 5
respectively in pavement with cavity (voids). The load transfer ratios were 18.55%
and 15.45% respectively in pavement with and without voids for the load case 3
where wheel load was applied directly beneath the dowel. Overall, the pavement
with dowels with cavity (voids) indicates excellent LTE since the specified LTE for
the majority of load cases exceeded approximately 93%. Further, decreased load
transfer efficiency was as much as 77.59% for LC1, 79.53% for LC2, 79.02% for
LC3 and 47.76% for LC4 in the concrete pavement without dowels with cavity
(voids). Maximum displacements were increased by 40.94% for LC1, 42.59% for
LC2, 42.10% for LC3, 26.41% for LC4 and 1.73% for LC5 in the concrete pavement without dowels with cavity (voids).

(7) The maximum vertical displacement at the transverse along the line $TT_{HH}$ for load case 3 for pavement without dowels with dual wheel loading was reduced by 23.8%, 47.4% and 60.5% respectively with increase sub-grade strength from 30 to 50, 100 and 170 MPa. However, the maximum vertical displacement at transverse joint along line $TT_{HH}$ for load case 3 for pavement with dowels with dual wheel loading were reduced by 25.3%, 50.0% and 63.5% respectively with increased sub-grade strength from 30 to 50, 100 and 170 MPa. One can conclude that the pavement with dowels can reduce the maximum vertical displacement by around 9.7%, 11.4%, 14.2% and 16.4% respectively with sub-grade strength a range from 30 to 50, 100 and 170 MPa and this also demonstrates that the dowel bars perform effectively as load transfer device in the concrete pavement systems. One can observe that the first eight engaged dowels carry more than 42.17% of the transferred shear force across the joints and there was no significant effect on the shear force distribution with sub-grade variants from 30 to 170 MPa. Finally, for load case 3 of four layer concrete pavements with dowels at 300 spacing with dual wheel loading, the maximum vertical displacements at the transverse joint along line $TT_{HH}$ were reduced by 32.16%, 57.29% and 68.84% respectively with increased sub-grade strength from 30 to 50, 100 and 170 MPa. However, there was no significant change in the load transfer efficiency with the sub-grade strength varying from 30 to 50, 100 and 170 MPa.
Regardless of the load magnitude, dowels are still capable of transferring applied vehicular loading from one segment to the adjacent segment. This phenomenon proves that dowel bars perform effectively as load transfer devices, particularly in circumstances where heavy traffic or weak foundation are present.

(8) Results demonstrate that eight engaged dowels carry more than 70% and 95% of the transferred shear force across the joints for model three layer pavements in single and dual-wheel loading cases respectively, for both 750 kPa and 1500 kPa loading. The maximum displacements have been increased by 100% for load case 3 if the applied load increased from 750 kPa to 1500 kPa. So it can be deduced that the load has a significant influence on displacement. The ratio of load transfer for a four-layer pavement system increased with the increase of applied type pressure. That means more wheel load can be transferred to the adjacent concrete slab if wheel load pressure increases. This phenomenon is also evident in the dowel shear force distribution. The inner four dowels of the 1500 kPa case carried almost two times more shear force than those of the 750 kPa case. However, the high wheel load increases did not significantly affect a load transfer ratio. Meanwhile, it also increased the demand on a few inner dowels beneath the wheel load, which may cause more damage to the joints and eventually lead to pavement failure. For all load case on four layer concrete pavements with dowels at 300 mm spacing with dual wheel loading, the maximum vertical displacements at transverse joint along line TT_{HH} were increased by 100% with double the wheel loading. However; there was no significant change in the load transfer efficiency (LTE) when the wheel loading increased from 20 kN to 40 kN.
In the design point of view, pavement with closer dowel spacing can perform better but depend on design criteria. Friberg (1940) found that the maximum positive moment of the concrete slab for the edge loading case occurs right beneath the wheel load and that maximum negative moment occurred at a point $1.8 \ell_r$ from the point of loading, where $\ell_r$ is the radius of relative stiffness defined by Westergaard. Friberg further stated that “effective dowel shear decreases inversely as the distance of the dowel from the point of loading, to zero at a distance of $1.8 \ell_r$. No dowels beyond that point influence the moment at the load point”. This observation implies that most of the load transfer should occur within $1.8 \ell_r$ of the loading. As results, the distribution of shear force fluctuates from almost 100% for the 225 mm spacing cases to approximately 73.8% for the 600 mm spacing cases.

(9) The results show that each dowel in the 300 mm, 450 mm and 600 mm spacing cases carries more shear force than the 225 mm spacing case due to the wider spacing. Nevertheless, the total amount of transferred shear for the 225 mm spacing case was about 6%, 14.3% and 26.2% larger than for the 300 mm, 450 mm and 600 mm spacing cases respectively in the first 1800 mm. Results show that only three dowels were engaged for 600 mm spacing cases within 1800 mm symmetric region, while four dowels were engaged in 450 mm spacing case, six dowels were engaged for 300 mm spacing cases, eight were engaged for 225 mm spacing case. This result suggests that the size of the region containing engaged dowels does not change significantly with dowel spacing, only the distribution of shear forces varies. The distribution of shear force fluctuates from almost 100% for the 225 mm spacing cases to approximately 73.8% for the 600 mm spacing cases. The maximum vertical
displacements at transverse along line $TT_{HH}$ were reduced by 1.4%, 2.9% and 3.6% respectively with dowels spacing at 225 mm to 300 mm, 450 mm and 600 mm. However, there was no significant change in the load transfer efficiency with the dowel spacing at 225 mm to 600 mm (LTE only increased by 1.12% for LC3 compared with dowel spacing at 225 mm with 600 mm).

Thicker concrete slab can significantly increase load transfer ratio and reduced maximum vertical displacement along transverse joint. In fact, in theory the thicker concrete slab is stiffer and, therefore, develops less curvature along the loaded side of the joint and also higher load transfer. (10) Results show that the amount of load transfer increases, as the slab thickness increases. Further, the number of engaged dowels increases with slab thickness. The ratio of load transfer also increases with slab thickness. For the single wheel loading case with dowel spacing at 300 mm, a 250 mm concrete slab shows the total transfer ratio in the six engaged dowels of 47.6% whereas for a 375 mm and 500 mm concrete slab the first six engaged dowels transfer 95.8% and 97.6% respectively. In fact, in theory the thicker concrete slab is stiffer and, therefore, develops less curvature along the loaded side of the joint. The deformed shape explains why more dowels are engaged in the load transfer for the thicker concrete slab models. With the stiffness increase for a thick concrete slab, a much higher percentage of load transfer was observed for the 375 mm and 500 mm cases. In fact, a thick concrete slab provides a significant benefit: higher load transfer. The maximum vertical displacements at transverse joint along line $TT_{HH}$ were reduced by 16.2% for LC1,
17.9% for LC2, 20.2% for LC3, 20.7% for LC4 and 21.3% for LC5 respectively with increased the concrete slab thickness from 375 mm to 500 mm. However, there was no significant change in the load transfer efficiency (LTE).

The amount of transferred load increased when more wheel loads were applied. For the first six engaged dowels, it was only 47.60% of load transfer for single wheel loading and 94.17% for dual wheel loading case in concrete pavement with lean concrete with dowels spacing at 300 mm. The maximum displacement occurred beneath the wheel load. In fact, dowels provided partial continuity to the discrete concrete slab segments by transferring shear force. The maximum vertical displacements at transverse joint along line TT_{HH} were reduced by 51.35% for LC1, 52.08% for LC2, 53.15% for LC3, 53.47% for both LC4 and LC5 respectively with single wheel loading when compare with dual wheel loading in concrete pavement with lean concrete and with dowel spacing at 300 mm. However, there was no significant change in the load transfer efficiency (LTE increased by only 4.1% for LC3).

### 3.5 Summary

(1) From the observations, dowel can significantly reduce the maximum vertical displacement at the transverse joint along line TT_{HH} by 12%. In the meantime, dowel has reduced the magnitude of compressive MPS by 92.5% along the longitudinal
pavement section. Further, an increased efficiency also shows that the dowel bars perform effectively as a load transfer device in the concrete pavement systems.

(2) Maximum vertical displacement at the transverse joint along line TT_{HH} for the load case 3 was reduced by 47% in dowel-jointed pavement with lean concrete. It can be observed that peak MPS is about 6.7 times greater and there is steeper variation in the distribution pattern in the concrete pavement without LCB than with LCB.

(3) Maximum displacement at the transverse joint along line TT_{HH} was increased by as much as 20.6% in pavement with cavity (voids). Furthermore, decreased load transfer efficiency was as much as 77.59% for LC1, 79.53% for LC2, 79.02% for LC3 and 47.76% for LC4 in the three layer concrete pavement without dowels with cavity (voids). Maximum displacements were increased by 40.94% for LC1, 42.59% for LC2, 42.10% for LC3, 26.41% for LC4 and 1.73% for LC5 in the three layer concrete pavement without dowels with cavity (voids).

(4) One can conclude that the pavement with dowels can reduce the maximum vertical displacement by around 9.7%, 11.4%, 14.2% and 16.4% respectively with a sub-grade strength range from 30 to 50, 100 and 170 MPa. In the meantime, for load case 3 of four layer concrete pavements with dowels at 300 mm spacing with dual wheel loading, the maximum vertical displacements at transverse joint along line TT_{HH} was reduced by 32.16%, 57.29% and 68.84% respectively with increased sub-grade strength from 30 to 50, 100 and 170 MPa. However, there was no significant change
in the load transfer efficiency with the sub grade strength varying from 30 to 170 MPa.

(5) Results demonstrate that eight engaged dowels carry more than 70\% and 95\% of the transferred shear force across the joints for model three layer pavements in single and dual-wheel loading cases respectively, for both 750 kPa and 1500 kPa loading. The maximum displacements have been increased by 100\% for load case 3 if the applied load increased from 750 kPa to 1500 kPa.

(6) The results show that the total amount of transferred shear for the 225 mm spacing case was about 6\%, 14.3\% and 26.2\% larger than for the 300 mm, 450 mm and 600 mm spacing cases respectively in the first 1800 mm. Only three dowels were engaged for 600 mm spacing cases within 1800 mm symmetric region, while four dowels were engaged in 450 mm spacing case, six dowels were engaged for 300 mm spacing cases, eight were engaged for 225 mm spacing case. The distribution of shear force varies from almost 100\% for the 225 mm spacing cases to approximately 73.8\% for the 600 mm spacing cases.

(7) Results show that the amount of load transfer increases, as the slab thickness increases. For a single wheel loading case with dowel spacing at 300 mm, 250 mm concrete slab shows the total load transfer ratio in the six engaged dowels of 47.6\% whereas for 375 mm and 500 mm concrete slab the first six engaged dowels transfer 95.8\% and 97.6\% respectively. The maximum vertical displacements at the
transverse joint along line $TT_HH$ were reduced by 16.2% for LC1, 17.9% for LC2, 20.2% for LC3, 20.7% for LC4 and 21.3% for LC5 respectively with increased concrete slab thickness from 375 mm to 500 mm. However, there was no significant change in the load transfer efficiency (LTE).

(8) The amount of transferred load increased while more wheel loads were applied. For the first six engaged dowels, it was only 47.60% of the load transfer for single wheel loading and 94.17% for dual wheel loading case in concrete pavement with lean concrete with dowels spacing at 300 mm. In fact, dowels provided partial continuity to the discrete concrete slab segments by transferring shear force. The maximum vertical displacements at the transverse joint along line $TT_HH$ were reduced by 51.35% for LC1, 52.08% for LC2, 53.15% for LC3, 53.47% for both LC4 and LC5 respectively with single wheel loading when compared with dual wheel loading in concrete pavement with lean concrete and with dowel spacing at 300 mm. However, there was no significant change in the load transfer efficiency (LTE increased by only 4.1% for LC3).

(9) These research results not only provide a new thought and theoretical basis to evaluate the behaviour of pavement with and without dowels; the benefit of lean concrete base; effect of voids beneath rigid pavement slab; behaviour of undowered and doweled joint under variation of sub-grade strength; effect of dowels under load level variation; variation dowel spacing; slab thickness change; single and dual wheel loading; but also
offer the technical support for making scientific and reasonable maintenance measures, which have theoretical significance and engineering value.
Dowel Looseness

4.1 Introduction

This chapter evaluates the effect of dowel looseness on the response of jointed concrete pavement using 3D finite-element analyses of rigid pavement systems. These studies indicate that significant reduction in load transfer efficiency and increase in both slab and base course stresses can be expected due to small gaps varying from 0.25 to 1.25 mm between the dowels and the slabs. In the worst case the LTE was reduced to 11.3% and 11.6% respectively for single wheel loading and the odd dual wheel loading case while there were voids present at the base course layer for 1.25 mm gaps between dowel and slabs.

The realistic modelling of dowel load transfer across rigid pavement joints is necessary for accurate prediction of pavement responses to applied wheel loadings. Dowels bars transfer the load across a pavement joint primarily by shear action. The load transfer mechanism between the dowel and the concrete is a complex phenomenon. This mechanism depends mainly on a parameter known as the modulus of dowel support (K), the value of which can be determine by load testing (Yoder, 1975). A high modulus of dowel support value indicates a good contact between the concrete and the steel dowel. However, with repeated application of wheel loads, the contact between the surrounding concrete and the dowel deteriorates, particularly where the bar is embedded and near the face of the joint. At these locations, the concrete may be crush over the time under repeated loading and subjected to high bearing stresses. As the crushed concrete particles are displacing, voids are created around the dowels causing dowel looseness (DL). The size of looseness may vary along the length of the dowel bars. However, near the joint face, the looseness is generally more severe than other locations along the dowel bar. DL is generally composed of two parts-initial looseness and
looseness from enlargement of the socket under repetitive loading (Buch, 1996). In experimental work reported by (Guo, 1995) on the performance of dowels under repetitive loading, it was observed that DL, produced by an imperfect fit or void between the dowel and the surrounding concrete (initial looseness), greatly affects load transfer efficiency (LTE), maximum deflection, critical stresses, and the rate of pavement deterioration. A deteriorated joint under repetitive loading ultimately leads to erosion under the dowel. In another experimental study performed by (Buch, 1996) it was found that DL is affected by the type, texture, and shape of the aggregate particles used in the concrete slab, the bearing stress of concrete, and by the load magnitude and the number of load cycles.

The first analysis of dowel looseness for concrete pavement joints was performed by (Teller, 1936). They plotted the ratios of apparent vertical stiffness of loaded and unloaded slab edges (p/k). If the ratio was constant over a range of loads, this indicated no looseness was present. The first theoretical joint considerations were presented by (Friberg, 1938). Friberg developed a dowel response model based on a structural analysis of laterally loaded elastic steel dowels of infinite length embedded in an elastic half-space. Friberg used the modulus of dowel support, $K_0$, in his equation. The modulus of dowel support is the reaction per unit area causing a deflection equal to one. Friberg used the expression $K_0b$ to replace the modulus of foundation, $K$, from Timoshenko’s model. Teller (1958) performed some of the first full scale repeated load tests of doweled joints in a controlled laboratory environment, and it was designed to evaluate some of the theory presented by Friberg. These were perhaps the first studies that demonstrated load transfer deterioration and joint looseness development for doweled joints.

Skarlatos (1949) developed an extension of Westergaard’s edge stress equations to quantify load transfer at joints. A parameter called joint stiffness ($q_0$) was introduced, a constant value in units of (Ib/in)/in, meaning Ib of load transferred in vertical shear per inch along the joint line, per inch of
relative vertical slab displacement (deflection different) along the joint (Skarlatos, 1998; Skarlatos, 1949 and Ioannides, 1996). The joint stiffness variable has also been called AGG (Ioannides, 1996), or $K_{\text{Joint}}$ (Brill, 2010), or $k_l$ (Byrum, 2010) in past research. This linear-elastic constant joint stiffness assumption is the most simplified relation that can be use to numerically simulate joint load transfer. It is important to understand how joint looseness is affecting joint load transfer in order to be able to accurately predicting slab-bending stresses for use in pavement thickness design.

4.2 Methodology

4.2.1 Modelling strategy

The major assumptions considered for the dowel looseness model are, (1) load transfer is only through dowel bar interaction and no aggregate interlocking is modelled; (2) the dowels are in full contact with the concrete slab on the loaded side (approach side) and looseness around the dowels is considered only on the unloaded side (leave side); (3) the looseness is assumed to be the same for all dowels; (4) the slab is fully supported by the subgrade.

4.2.2 Selection of element types and meshes refinement

8 nodes hexahedra brick elements are used to model the concrete pavement section. A technique for modelling dowels in three dimensional finite element analyses of concrete pavement systems is presented that relies on an embedded formulation of a beam element. This embedded element permits the efficient modelling of dowel looseness using a nodal contact approach and allows the dowels to be exactly located irrespective of the slab mesh lines. For each element is assigned with 3 degrees of freedom at each node. The pavement model is discretized into many small brick elements. The region under investigation is located in the vicinity of the joint; therefore finer mesh sizes are developed in that area to capture more accurate stress and strain behaviours.
4.2.3 Pavement material models

The material properties are provided in Table 4.1 and are assumed to be linear, homogeneous and elastic in behaviour.

<table>
<thead>
<tr>
<th>Description</th>
<th>Concrete Slab</th>
<th>Base Course</th>
<th>Subgrade</th>
<th>Dowel Bar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus [MPa]</td>
<td>28,000</td>
<td>350</td>
<td>50</td>
<td>200,000</td>
</tr>
<tr>
<td>Layer thickness [mm]</td>
<td>250</td>
<td>150</td>
<td>1,900</td>
<td>-</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.18</td>
<td>0.4</td>
<td>0.4</td>
<td>0.3</td>
</tr>
<tr>
<td>Density [kg/m$^3$]</td>
<td>2,400</td>
<td>2,000</td>
<td>1,800</td>
<td>7,830</td>
</tr>
</tbody>
</table>

Note: “—” information not required.

4.2.4 Plane of symmetry and boundary conditions

Three-dimensional (3D) representation of the three types of pavement layers, i.e. concrete slab, base course and subgrade, are defined and full friction is applied at the boundaries. Similar vehicular loading and restraint conditions are applied to the dowel looseness model (refer to figure 3.1). As indicated in Figure 3.1, the symmetrical plane is restrained from rotating around the z-axis and translating along the x- and y-axes. The bottom surface of the pavement is restrained from deforming in all directions and roller restraints are applied to the sides of the pavement.
4.2.5 Loading conditions

Four wheel loads of 20 kN each representing an equivalent 80 kN (i.e. 707 kPa) single axle load is assumed. The effect of the air pressure within the tyre is neglected, hence the contact pressure of the tyre is assumed to be uniformly distributed over a rectangular area of $2.66 \times 10^{-2}$ m$^2$. Three different loading scenarios are applied to the dowel looseness model (see Figure 4.1).

(a) Single wheel loading                                            (b) Odd dual wheel loading

(c) Even dual wheel loading

Figure 4.1: Loading scenarios
4.2.6 Dowel looseness modelling cases

At unloaded side, the joint between the concrete slabs is not in contact and as such has no specified stiffness. The gaps between the dowel and surrounding concrete were modelled by releasing the nodal contact. Hence, there was no contact and the gap was created. No bond-slip law have been adopted or considered in the model. The dowel bar (or beam element) shared nodes with the concrete (brick element). Four cases with gaps vary from 0 to 1.25 mm between the dowel and slabs are investigated in this study (refer to Figure 4.2).

![Dowel looseness modelling](image)

(a) Case 1-the gap is present only half of embedded portion of the dowel for downward direction (uni-directional contact);

(b) Case 2-the gap is present only whole embedded portion of the dowel for downward direction (uni-directional contact);

(c) Case 3-the gap is present only half of embedded portion of the dowel for both downward and upward direction;

(d) Case 4-the gaps are present on whole embedded portion of the dowel;

Figure 4.2: Dowel looseness modelling Cases: Four cases with gaps varying from 0 to 1.25 mm between the dowel and slabs
4.3 Results of analysis

![Graph showing vertical displacement vs TT_HH (mm) with different cases of dowel looseness.]

Figure 4.3: Even dual wheel loading; vertical displacement at transverse joint along line TT_HH without void

4.3.1 Vertical displacement

From observation, the maximum displacement was increased by 10% for Even Dual Loading case on concrete pavement without void, while the gap between slab and dowel vary from 0 to 1.25 mm; 10.8% and 70% increased for Odd Dual Loading case on concrete pavement without and with void respectively, while the gap between slab and dowel vary from 0 to 1.25mm; 10.5% and 67.2% increased for Single Wheel Loading case on concrete pavement without and with void respectively, while the gap between slab and dowel vary from 0 to 1.25 mm.
Dowel looseness can significantly reduce the load transfer capacity. The contact between the dowel and surrounding concrete, particularly near the face of the joint may deteriorates over the time under repeated loading and subjected to high bearing stresses. As the crushed concrete particles are displacing, voids are created around the dowels causing dowel looseness (DL). As a result, dowels have lost its function as a means to bridge vehicle loads across adjacent slab. Figure 4.4 shows the shear force diagram for case 1-case 4 with 1.25mm gaps between slab and dowels. The transferred shear force for the 1.25 case 1, 1.25 case 2, 1.25 case 3 and 1.25 case 4 are 9.4%, 9.6%, 72.3% and 96.5% lower respectively while comparing with no dowel looseness case.

4.3.2 Load transfer efficiency (LTE) at the top of concrete layer

‘Load transfer’ is a term used to describe the transfer of load across discontinuities such as joints (Austroads, 2008). When vehicular loading is applied near a pavement joint, both loaded and unloaded slabs deflect because a portion of the load applied to the loaded slab is transferred to the unloaded slab. The amount the unloaded slab deflects is directly related to joint performance. If a
joint is performing perfectly, both the loaded and unloaded slabs deflect equally. During vehicular loading it is relatively close to the joint of the concrete slab, dowel bars immediately under the applied load assume a major portion of the load with other dowel bars assuming progressively lesser amounts (Huang, 2010). The LTE is defined as a parameter that measures the load transfer from the loaded side to the unloaded side of the joint, and it is given by (Austroads, 2008) as:

\[ LTE = \frac{\delta_U}{\delta_L} \times 100 \]  

(4.1)

Where, \( \delta_U \) and \( \delta_L \) are respectively the unloaded and loaded vertical displacements in millimetres.

Table 4.2: Single wheel loading case: summarises the reduction in load transfer and LTE of all four cases of dowel looseness, with the gap varying from 0.25-1.25 mm on concrete pavement with and without voids

<table>
<thead>
<tr>
<th>Gap (mm)</th>
<th>Without void</th>
<th>With void</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Reduction in Load Transfer (%)</td>
<td>LTE (%)</td>
</tr>
<tr>
<td>0.25 case 1</td>
<td>6.7</td>
<td>81.4</td>
</tr>
<tr>
<td>0.25 case 2</td>
<td>6.8</td>
<td>81.4</td>
</tr>
<tr>
<td>0.25 case 3</td>
<td>72.7</td>
<td>69.3</td>
</tr>
<tr>
<td>0.25 case 4</td>
<td>96.7</td>
<td>65.1</td>
</tr>
<tr>
<td>0.5 case 1</td>
<td>8.5</td>
<td>81.1</td>
</tr>
<tr>
<td>0.5 case 2</td>
<td>8.6</td>
<td>81.1</td>
</tr>
<tr>
<td>0.5 case 3</td>
<td>72.9</td>
<td>69.3</td>
</tr>
<tr>
<td>0.5 case 4</td>
<td>96.7</td>
<td>65.1</td>
</tr>
<tr>
<td>0.75 case 1</td>
<td>9.3</td>
<td>81.0</td>
</tr>
<tr>
<td>0.75 case 2</td>
<td>9.4</td>
<td>80.9</td>
</tr>
<tr>
<td>0.75 case 3</td>
<td>73.1</td>
<td>69.2</td>
</tr>
<tr>
<td>0.75 case 4</td>
<td>96.7</td>
<td>65.1</td>
</tr>
<tr>
<td>1.0 case 1</td>
<td>9.7</td>
<td>80.9</td>
</tr>
<tr>
<td>1.0 case 2</td>
<td>9.9</td>
<td>80.9</td>
</tr>
<tr>
<td>1.0 case 3</td>
<td>73.3</td>
<td>69.2</td>
</tr>
<tr>
<td>1.0 case 4</td>
<td>96.7</td>
<td>65.1</td>
</tr>
<tr>
<td>1.25 case 1</td>
<td>10.0</td>
<td>80.9</td>
</tr>
<tr>
<td>1.25 case 2</td>
<td>10.1</td>
<td>80.8</td>
</tr>
<tr>
<td>1.25 case 3</td>
<td>73.3</td>
<td>69.2</td>
</tr>
<tr>
<td>1.25 case 4</td>
<td>96.7</td>
<td>65.1</td>
</tr>
</tbody>
</table>
Table 4.3: Odd dual wheel loading case: summarises the reduction in load transfer and LTE of all four cases of dowel looseness, with the gap varying from 0.25-1.25 mm on concrete pavement with and without voids

<table>
<thead>
<tr>
<th>Gap (mm)</th>
<th>Without void Reduction in Load Transfer (%)</th>
<th>LTE (%)</th>
<th>With void Reduction in Load Transfer (%)</th>
<th>LTE (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25 case 1</td>
<td>6.0</td>
<td>81.4</td>
<td>2.1</td>
<td>78.3</td>
</tr>
<tr>
<td>0.25 case 2</td>
<td>6.2</td>
<td>81.4</td>
<td>2.1</td>
<td>78.2</td>
</tr>
<tr>
<td>0.25 case 3</td>
<td>71.5</td>
<td>69.3</td>
<td>45.6</td>
<td>38.0</td>
</tr>
<tr>
<td>0.25 case 4</td>
<td>96.6</td>
<td>65.1</td>
<td>91.1</td>
<td>11.7</td>
</tr>
<tr>
<td>0.5 case 1</td>
<td>7.7</td>
<td>81.1</td>
<td>2.7</td>
<td>77.6</td>
</tr>
<tr>
<td>0.5 case 2</td>
<td>7.8</td>
<td>81.1</td>
<td>2.8</td>
<td>77.5</td>
</tr>
<tr>
<td>0.5 case 3</td>
<td>71.8</td>
<td>69.3</td>
<td>46.0</td>
<td>37.8</td>
</tr>
<tr>
<td>0.5 case 4</td>
<td>96.6</td>
<td>65.1</td>
<td>91.2</td>
<td>11.7</td>
</tr>
<tr>
<td>0.75 case 1</td>
<td>8.4</td>
<td>81</td>
<td>3.0</td>
<td>77.3</td>
</tr>
<tr>
<td>0.75 case 2</td>
<td>8.6</td>
<td>80.9</td>
<td>3.0</td>
<td>77.2</td>
</tr>
<tr>
<td>0.75 case 3</td>
<td>72.0</td>
<td>69.2</td>
<td>46.2</td>
<td>37.7</td>
</tr>
<tr>
<td>0.75 case 4</td>
<td>96.6</td>
<td>65.1</td>
<td>91.2</td>
<td>11.6</td>
</tr>
<tr>
<td>1.0 case 1</td>
<td>8.8</td>
<td>80.9</td>
<td>3.1</td>
<td>77.2</td>
</tr>
<tr>
<td>1.0 case 2</td>
<td>9.0</td>
<td>80.9</td>
<td>3.2</td>
<td>77.0</td>
</tr>
<tr>
<td>1.0 case 3</td>
<td>72.0</td>
<td>69.2</td>
<td>46.3</td>
<td>37.6</td>
</tr>
<tr>
<td>1.0 case 4</td>
<td>96.6</td>
<td>65.1</td>
<td>91.2</td>
<td>11.6</td>
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<td>9.0</td>
<td>80.9</td>
<td>3.2</td>
<td>77.0</td>
</tr>
<tr>
<td>1.25 case 2</td>
<td>9.2</td>
<td>80.8</td>
<td>3.3</td>
<td>77.0</td>
</tr>
<tr>
<td>1.25 case 3</td>
<td>72.1</td>
<td>69.2</td>
<td>46.4</td>
<td>37.5</td>
</tr>
<tr>
<td>1.25 case 4</td>
<td>96.6</td>
<td>65.1</td>
<td>91.2</td>
<td>11.6</td>
</tr>
</tbody>
</table>

Table 4.4: Even dual wheel loading case: summarises the reduction in load transfer and LTE of all four cases of dowel looseness, with the gap varying from 0.25-1.25 mm on concrete pavement without voids

<table>
<thead>
<tr>
<th>Gap (mm)</th>
<th>Without void Reduction in Load Transfer (%)</th>
<th>LTE (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25 case 1</td>
<td>6.3</td>
<td>83.0</td>
</tr>
<tr>
<td>0.25 case 2</td>
<td>6.5</td>
<td>83.0</td>
</tr>
<tr>
<td>0.25 case 3</td>
<td>71.6</td>
<td>70.8</td>
</tr>
<tr>
<td>0.25 case 4</td>
<td>96.4</td>
<td>66.5</td>
</tr>
<tr>
<td>0.5 case 1</td>
<td>8.0</td>
<td>82.8</td>
</tr>
<tr>
<td>0.5 case 2</td>
<td>8.2</td>
<td>82.8</td>
</tr>
<tr>
<td>0.5 case 3</td>
<td>71.9</td>
<td>70.8</td>
</tr>
<tr>
<td>0.5 case 4</td>
<td>96.4</td>
<td>66.5</td>
</tr>
<tr>
<td>0.75 case 1</td>
<td>8.8</td>
<td>82.6</td>
</tr>
<tr>
<td>0.75 case 2</td>
<td>9.0</td>
<td>82.6</td>
</tr>
<tr>
<td>0.75 case 3</td>
<td>72.1</td>
<td>70.7</td>
</tr>
<tr>
<td>0.75 case 4</td>
<td>96.5</td>
<td>66.5</td>
</tr>
<tr>
<td>1.0 case 1</td>
<td>9.2</td>
<td>82.5</td>
</tr>
<tr>
<td>1.0 case 2</td>
<td>9.4</td>
<td>82.4</td>
</tr>
<tr>
<td>1.0 case 3</td>
<td>72.2</td>
<td>70.7</td>
</tr>
<tr>
<td>1.0 case 4</td>
<td>96.5</td>
<td>66.5</td>
</tr>
<tr>
<td>1.25 case 1</td>
<td>9.4</td>
<td>82.5</td>
</tr>
<tr>
<td>1.25 case 2</td>
<td>9.6</td>
<td>82.4</td>
</tr>
<tr>
<td>1.25 case 3</td>
<td>72.3</td>
<td>70.7</td>
</tr>
<tr>
<td>1.25 case 4</td>
<td>96.5</td>
<td>66.5</td>
</tr>
</tbody>
</table>
4.3.3 Dowels shear and load transfer efficiency (LTE)

The largest gap considered in the analyses is only 1.25 mm, because larger values produce no significant change in the model response. The variation of gap size makes significant changes on the behaviour of load transfer. Wider gaps reduce the amount of transferred load over the joint. The result of dowel looseness studies for single wheel loading case; odd dual wheel loading case and even dual wheel loading case, the amount of load transfer reduced to 96.7%; 96.6% and 96.5% respectively for 1.25 cases 4. From the observation on joints, traditional LTE varied from 94.4% (intact joint) to 65.1% (1.25 cases 4) for single wheel loading case; 92.6% (intact joint) to 66.5% (1.25 cases 4) for an even dual wheel loading case. These confirm the localized nature of dowel load transfer noted by other researchers (Guo, 1995; Ioannides, 1992; Davids, 2000; Kim, 2003). However, the LTE were reduced to 11.3% and 11.6% respectively for single wheel loading and odd dual wheel loading case while there were voids present at the base course layer for 1.25 cases 4.

4.3.4 Concrete slab and base course layer stresses;

Given that increasing gaps between slab and dowel significantly decrease the dowel shears, an increase in stresses applied to the base course layer under loaded slab adjacent to the joint with a concomitant decrease in applied stresses under unloaded slab. In fact, the maximum vertical stress applied to the base course layer by the loaded slab varies from 259.4 to 468.4 kPa as the gap varies from 0-1.25 mm, an increase of 80.6%. A similar drop in stresses applied to the base course layer under an unloaded slab also occurs. However, the fact that the stresses increase dramatically with the presence of small gaps is significant: loss of base support and strength due to pumping action and base deterioration under high stresses, are suspected of being significant component of many joint failures (Ioannides, 1992). Meanwhile, the principal tensile stresses in the bottom of the slab vary from 342.9 to 426.9 kPa with gaps of between 0 and 1.25 mm, an increase of 25%. In the
meantime, with the presence of voids at the base course layer, and the principal tensile stresses at the bottom of the slab increases of 52% have been observed.

4.4 Discussion of analysis

The largest gap considered in the analyses is only 1.25 mm, because larger values produce no significant change in the model response. The variation of gap size makes significant changes on behaviour of load transfer. Wider gaps reduce the amount of transferred load over the joint. The result of the dowel looseness study for single wheel loading case; odd dual wheel loading case and even dual wheel loading case, showed that the amount of load transfer reduced to 96.7%; 96.6% and 96.5% respectively for 1.25 cases 4. From the observation on joints, traditional LTE varied from 94.4% (intact joint) to 65.1% (1.25 cases 4) for single wheel loading case; 92.6% (intact joint) to 66.5% (1.25 cases 4) for even dual wheel loading case. These confirm the localized nature of dowel load transfer noted by other researchers (Guo, 1995, Ioannides, 1992, Davids, 2000 and Kim, 2003). However, the LTE were reduced to 11.3% and 11.6% respectively for single wheel loading and odd dual wheel loading case while there were voids present at the base course layer for 1.25 cases 4.

4.5 Summary

This study examines the effect of dowel looseness on the response of jointed concrete pavement. Small gap between the dowels and slabs were shown to significantly decrease the dowel shear action and in contrast increase principal tensile stresses in concrete pavement slabs subjected to wheel loading, as well as to increase the vertical stresses applied to the base course layer.
The effects of dowel looseness on LTE are quite significant. It is observed that LTE decreases with the increase in dowel looseness. LTE varied from 94.4% (intact joint) to 65.1% (1.25 cases 4) for single wheel loading case; 92.6% (intact joint) to 66.5% (1.25 cases 4) for even dual wheel loading case. However, in the worst case scenario the LTE were reduced to 11.3% and 11.6% respectively for single wheel loading and odd dual wheel loading case while there were voids present at the base course layer for 1.25 cases 4.

Concrete slab and base course layer stresses; given that increasing gaps between slab and dowel significantly decreases the dowel shears, show an increase in stresses applied to the base course layer under loaded slab adjacent to the joint with a concomitant decrease in applied stresses under unloaded slab. In fact, the maximum vertical stress applied to the base course layer by the loaded slab varies from 259.4 to 468.4 kPa as the gap varies from 0-1.25 mm, an increase of 80.6%. A similar drop in stresses applied to the base course layer under unloaded slab also occurs. Meanwhile, the principal tensile stresses in the bottom of the slab vary from 342.9 to 426.9 kPa with gaps of between 0 and 1.25 mm, an increase of 25%. In the meantime, with the presence of voids at the base course layer, the principal tensile stresses at the bottom of the slab, increases of 52% have been observed.

Through the FE analysis, small looseness gaps between the concrete slab and dowel makes a significant change in the behaviour of concrete pavement.
Prediction Models

5.1 Analytical theory of dowel bars system

5.1.1 Analytical model

Timoshenko (1925) worked on the first model of beam on an elastic foundation that could be applied to a dowel bar system. According to (Timoshenko, 1925) the deflection of a beam on an elastic foundation is found as follow:

\[ EI \frac{d^4y}{dx^4} = -ky \]  (5.1)

Where \( k \) is a constant, usually called the modulus of foundation and \( y \) is the deflection. The modulus of foundation denotes the reaction per unit length when the deflection is set equal to one.

The solution to Timoshenko’s differential equation is found as follow:

\[ Y = e^{\beta x} (A \cos\beta x + B \sin\beta x) + e^{-\beta x} (C \cos\beta x + D \sin\beta x) \]  (5.2)
Where, \( \beta = \sqrt[4]{\frac{Kb}{4EI}} \) = relative stiffness of the beam on elastic foundation;

\[ k = \text{modulus of foundation}; \]

\[ E = \text{modulus of elasticity of the dowel}; \]

\[ I = \text{moment of inertia of the dowel}. \]

By applying the appropriate boundary conditions for the problem the constants A, B, C, and D can be solved. For a semi-infinite beam with a moment, \( M_0 \), and a point load, \( P \), equation 5.2 is equivalent to equation 5.3.

\[
y = \frac{e^{-\beta x}}{2\beta^2 EI} [P \cos \beta x - \beta M_0 (\cos \beta x - \sin \beta x)] \quad (5.3)
\]

**Figure 5.1: Semi-infinite beam on an elastic foundation**

Friberg (1938) applied Timoshenko’s elastic foundation theory to a beam of semi-infinite length. When trying to calculate the deflection at the face of the joint, it can be determined by setting \( x=0 \) in equation 5.3. This equation then becomes equation 5.4 as follow:
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\[ Y = \frac{P_t}{4\beta^3EI}(2 + \beta z) \]  

(5.4)

Where, \( \beta = \frac{4\sqrt{K_o b}}{4EI} \) = relative stiffness of the dowel bar encased in concrete;

\( K_o = \) modulus of dowel support;
\( b = \) dowel bars width;
\( E = \) modulus of elasticity of the dowel;
\( I = \) moment of inertia of the dowel;
\( P_t = \) load transferred through the dowel bar;
\( Z = \) joint width.

Figure 5.2: Slope and deflection of dowel at joint face

Friberg used the modulus of dowel support, \( K_o \), in his equation. The modulus of dowel support is the reaction per unit area causing a deflection equal to one. Friberg used the expression \( K_o b \) to replace the modulus of foundation, \( K \), from Timoshenko’s model.
Friberg’s equation was developed using semi-infinite dowel length. Dowel bars have a finite length so this equation could not apply to dowel bars used in practice today. However, (Porter, 1992) has shown that Friberg’s equation can be used with little or no error if the $\beta L$ value is greater than two. Where the length, $L$, is taken to be the length of the dowel bar embedded in concrete, or approximately one-half the dowel bar length.

### 5.1.2 Relatively deflection across a pavement joint

The relative deflection across a pavement joint consists of four separate components. These components, as shown in figure 5.3, consist of deflection of the dowel at each joint face, the deflection due to the slope of dowel bar, shear deflection, and flexural deflection. When considering all possible components for relative deflection the following is expressed in equation 5.5.

\[
\Delta = 2y_o + Z \left( \frac{dy_o}{dx} \right) + \delta + \frac{P_t Z^3}{12EI}
\]

(5.5)

Where,

- $Y_o =$ deflection at the face of the joint;
- $\Delta =$ shear deflection;
- $P_t =$ load transferred by dowel bars;
- $\lambda =$ form factor, equal to 10/9 for solid circular sections and ellipses;
- $A =$ cross-sectional area of the dowel bar;
- $G =$ shear modulus.
Due to the small joint width, deflection due to the slope of the dowel bar is approximately equal to zero, this also means that the flexural deflection is approximately equal to zero since the joint width term is cubed. After removing both the slope and flexural deflections from equation 5.5, equation 5.6 remains.

\[
\Delta = 2y_0 + \delta
\]  

(5.6)

Solving equation 5.6 for \( y_0 \) yields equation 5.7

\[
y_0 = \frac{\Delta - \delta}{2}
\]

(5.7)

![Figure 5.3: Relative deflection between adjacent pavement slabs](image)
5.1.3 Bearing stress of dowel bars in concrete

The bearing stress on the concrete at the face of the joint is critical for proper function of the dowel bar in the concrete. If the bearing stress on the concrete becomes too large the concrete will begin to break away where it contacts the dowel bar. Repetitive high-stress loadings of the dowel bar-concrete interface will create a void. This void creates an additional amount of deflection in the system before the dowel bar will begin to take on the applied load. This additional deflection creates a loss in the efficiency of the dowel bar to transfer load across the joint. The loss in efficiency must now be carried by the subgrade, which puts additional stress on the subgrade and creates the possibility for differential settlement of adjacent slabs.

If the dowel behaves as a beam on an elastic foundation, the bearing stress at the face of the joint, $\sigma_b$, is proportional to deflection at the face of the joint. This relationship is expressed using equation 5.8,

$$\sigma_b = K_0 y_0$$  \hspace{1cm} (5.8)

The bearing stress on the concrete needs to remain low to make sure no crushing of the concrete occurs. According to American Concrete Institute’s (ACI) Committee 325, the allowable bearing stress on the concrete is equivalent to equation 5.9,

$$\sigma_a = \left(\frac{4 - b}{3}\right) f_c$$  \hspace{1cm} (5.9)
Where, \( \sigma_a \) = allowable bearing stress;
\[ b = \text{dowel bar width}; \]
\[ f'_c = \text{compressive strength of concrete}. \]

This equation provides a factor of safety of approximately three.

### 5.2 Bar spacing

When slabs are of the same dimensions and are subjected to equal loads, the spacing of the dowel bars will determine how much load each dowel bar is subjected to. The larger the spacing between the dowel bars, the greater the loads applied to the dowel bars will be, due to the distribution of loads through the concrete and subgrade.

### 5.3 Dowel bar load distribution

In an ideal situation, when a load is placed near a joint, the dowel bars would assume half the load and the remaining load transfers to subgrade. However, no joint will behave in this ideal manner because of the repeated loadings seen by a pavement joint. This repetitive loading will create a small void and some load transfer efficiency of the dowel bar will be lost.

According to (Ioannides, 1992) this efficiency can be determined by calculating the transferred load efficiency (TLE) in equation 5.10,
TLE = \frac{P_t}{P_w} \times 100 \quad (5.10)

Where, 
TLE = transferred load efficiency (%);
P_t = load transferred across the joint;
P_w = applied wheel load.

The maximum value for the transferred load efficiency is 50 percent. (Brown, 1993) stated that for heavy truck traffic, a TLE ranging from 30 to 40 percent is considered acceptable.

Yoder (1975) suggested a 5-10 percent decrease in load transfer across a joint due to the void that appears after repetitive loadings. Allowing a conservative five percent decrease in load transfer yields is illustrated in equation 5.11,

\[ P_t = 0.45P_w \quad (5.11) \]

Where, 
P_t = load transferred across the joint;
P_w = applied wheel load.

When a wheel load is applied near a joint, not all dowel bars at the joint aid in transferring the load. The dowel bars closest to the applied wheel load transfer more of the load than the dowel bars furthest away from the applied load. Friberg (1940) was the first to investigate the load distribution to the dowel bars across a joint. Based on an analysis by (Westergaard, 1925) Friberg proposed that dowel bars contained outside
1.8 $\ell_r$ from the applied load were ineffective in transferring any additional load, where

$\ell_r$ is the radius of relative stiffness as shown in equation 5.12,

$$\ell_r = \frac{4E_c h^3}{12(1-\mu^2)k}$$  

(5.12)

Where, $\ell_r =$ Radius of relative stiffness (mm);

$E_c =$ elastic modulus of concrete pavement in MPa;

$h =$ thickness of concrete pavement in mm;

$\mu =$ Poisson’s ratio of concrete pavement;

$K =$ modulus of subgrade reaction in MPa/mm.

Friberg also believed that a linear distribution of load occurred inside the radius of relative stiffness as shown in figures 5.4. Friberg’s analysis was based on dowel bars having a diameter of 19 mm or 22 mm and with dowel bar spacing ranging from 300 mm to 500 mm. If a larger dowel bar is used or the bar spacing is less than 304.8 mm, then Friberg’s model no longer applies.

Since Friberg’s model is not useful for the construction practices used today, another model was required. Tabatabaie (1979) modelled a doweled joint using finite element and was able to show that an effective length of $1.0 \ell_r$ is more appropriate for today’s construction practices. Tabatabaie was also able to show that a linear approximation does exist with the maximum shear occurring beneath the applied load itself. For dowel bar
design, a calculation of the maximum shear load is useful and will be referred to as the load seen by the critical dowel bar, Pc.

Since Tabatabaie was able to show a linear relationship with the load distribution along the joint, the load seen by the critical dowel bar, Pc, can now be calculated. The first step is to draw a triangle with a base that extends out a distance 1.0 \( \ell_r \) in both directions from where the load is applied and has a peak height directly below the applied load, as shown in figure 5.4. For simplicity, set the height of the triangle below the load equal to one Pc. using the linear relationship of the load distribution along the joint, calculate the height of the triangle below each dowel bar. Sum up the heights below all the dowel bars to determine the number of effective dowels, \( N_{\text{eff}} \), that are used in transferring the applied load. The shear force that is directly beneath the applied load can now be calculated using equation 5.13,

\[
P_c = \frac{P_t}{N_{\text{eff}}} \tag{5.13}
\]
Chapter 5: Prediction Model

5.4 Prediction model for shear force

Dowel group action

Considering that the dowel group action and relative contribution of individual dowels in a dowel group has been modelled in different ways in the past, the group action of the
dowel bar system has been examined in the present study using the finite element model developed for the analysis of dowel-jointed concrete pavement. Group action of a dowel bar system in a typical jointed concrete pavement was evaluated by performing a parametric study. The group action of the dowel bar system was examined and useful relationships have been developed for estimation of the relative load shared by individual dowel bars. These relationships have been used to develop a prediction model to predict the shear force in group action of a dowel bar system and deflection at the loading nodal point.

When a load W is applied on one slab near the joint, as shown in Figure 5.5, part of the load will be transferred to the adjacent slab through the dowel group. If the dowels are 100% efficient, both slabs will deflect the same amount and relative forces under both slabs will be the same, each equal to 0.5W, which is also the total shear force transferred by the dowel group. If the dowels are less than 100% efficient, as in the case of old pavements where some dowels become loose due to repeated loading and subjected to high bearing stresses, the reactive forces under the loaded slab will be greater than 0.5W, while those under the unloaded slab will be smaller than 0.5W.

![Figure 5.5: Load transfer through dowel group](image)

Three-Dimensional Finite Element Analysis of Concrete Pavement on Weak Foundation
The dowel directly below the load carries the maximum shear force and the shear force carried by other dowels gradually decreases to zero following a parabolic shape. The distance (D) at which the shear becomes zero was found to depend on radius of relative stiffness \( (\ell_r) \). Where \( \ell_r \) is expressed in mm

\[
\ell_r = \sqrt[4]{\frac{E_c h^3}{12(1-\mu^2)k}}
\]  \hspace{1cm} (5.14)

Where, \( \ell_r \) = Radius of relative stiffness (mm);
\( E_c \) = elastic modulus of concrete pavement in MPa;
\( h \) = thickness of concrete pavement in mm;
\( \mu \) = Poisson’s ratio of concrete pavement;
\( K \) = modulus of subgrade reaction in MPa/mm.

However, the thickness of concrete pavement should be replaced by effective slab thickness \( (h_e) \), which was considered to take care of the effect of bonded or unbonded layers of the pavement (Zollinger, 2005)

Where

\[
\ell_r = \sqrt[4]{\frac{E_c h_e^3}{12(1-\mu^2)k}}
\]  \hspace{1cm} (5.15)

= Radius of relative stiffness in mm
K = modulus of subgrade reaction in MPa/mm;

$E_c = \text{elastic modulus of concrete pavement layer in Mpa;}

\mu = \text{Poisson’s ratio of concrete pavement layer;}

h_e = \text{effective thickness of concrete pavement layer in mm.}

The expressions for the effective thickness of concrete pavement layers are given by Eq. below (Ioannides, 1992). For fully-bonded layers, $h_e$ is expressed as below

$$h_e = h_{e-b}$$

$$= \left\{ h_1^3 + \frac{E_2}{E_1} h_2^3 + 12 \left[ \left( x_{na} - \frac{h_1}{2} \right) h_1 + \frac{E_2}{E_1} \left( h_1 - x_{na} + \frac{h_2}{2} \right)^2 h_2 \right] \right\}^{1/3}$$

$X_{na} = \text{neutral axis distance from top of concrete pavement layer in mm}$

$$X_{na} = \frac{E_1 h_1 \frac{h_1}{2} + E_2 h_2 \left( h_1 + \frac{h_2}{2} \right)}{E_1 h_1 + E_2 h_2}$$

For unbonded layers, $h_e$ is express as below

$$h_e = h_{e-u} = \left[ \left( h_1^3 + \frac{E_2}{E_1} h_2^3 \right) \right]^{1/3} \quad (5.16)$$

$E_1$ and $E_2 = \text{elastic modulus of concrete pavement and lean concrete layers, respectively, in MPa;}$
\( h_1 \) and \( h_2 \) = thickness of concrete pavement and lean concrete layers, respectively, in mm.

\[ y = 1.0623e^{-0.0033x} \]

\[ R^2 = 0.9815 \]

**Figure 5.6: variation of factor factor ‘n’ with the radius of relative stiffness \( \ell_r \).**

However, the shear force taken by different dowel bars and the distance (D) up to which the dowels participate in load transfer depend also on the dowel parameters and the applied wheel load. All the dowels are assumed to have zero looseness and perfectly aligned. The shear forces in the participating dowels decrease gradually with distance from the applied load position. The distance (D) at which the shear in the dowel becomes zero is found to depend not only on the radius of relatives stiffness \( (\ell_r) \), but also on the spacing of the dowel bar system. The shear in the first dowel \( (P_1) \) can be estimated from the equation developed in the present study as below

\[
P_1 = e^{0.0138s} \left[ \left( 0.0138 \frac{P \ell}{2} - P_1 \right) \sin 0.0138s + P_1 \cos 0.0138s \right]
\]

(5.17)

Where, \( P_i = \frac{P}{\Sigma nP_r} \);
\[ P = \frac{1}{2} \text{(applied wheel load in N)}; \]

\[ nP \ell_r = \text{number of dowels participate in load transfer over the radius of relative stiffness (} \ell_r); \]

\[ s = \text{spacing of dowel bars in mm}; \]

\[ z = \text{joints width in mm}. \]

Once the shear in the first dowel is known, the remainder of the dowels along the joint can be estimated using the following equation

\[
P_n = e^{0.0138S} \left[ \frac{P_{dn}z}{2} - P_{dn} \right] \sin 0.0138s + P_{dn} \cos 0.0138s \tag{5.18}\]

Where, \( P_{dn} = 1.0623e^{-0.0033S} (P_1) \) and other parameters were previously defined.

Equations 5.17 and 5.18 were proposed based on the group action of the dowel bar system and have been examined in the present study using the finite element model developed for the analysis of dowel-jointed concrete pavement. Group action of a dowel bar system in a typical jointed concrete pavement was evaluated by performing a parametric study. The group action of the dowel bar system was examined and useful relationships have been developed for estimation of the relative load shared by individual dowel bars. These relationships have been used to develop prediction model to predict the shear force in group action of a dowel bar system and deflection at the loading nodal point. Factor ‘n’ in the Figure 5.6 is the factor of each individual dowels (in term of transfer shear force) within the radius of relative stiffness (} \ell_r). Table 5.1 represented comparison between currently developed prediction model with the finite element results and estimate model widely used by pavement engineer Friberg model (1940) and
Tabatabaie model (1979) to estimate load distribution along the joint within the radius of relative stiffness ($\ell_r$).

### Table 5.1: Comparison between currently developed prediction model with F.E.model results and Friberg model (1940) and Tabatabaie model (1979) load distribution model

<table>
<thead>
<tr>
<th>Distance from Applied Wheel Load (mm)</th>
<th>600</th>
<th>300</th>
<th>0</th>
<th>0</th>
<th>300</th>
<th>600</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Shear Force</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F.E Model (kN)</td>
<td>0.92</td>
<td>2.66</td>
<td>5.99</td>
<td>5.95</td>
<td>2.61</td>
<td>0.89</td>
</tr>
<tr>
<td>Prediction Model (kN)</td>
<td>0.94</td>
<td>2.53</td>
<td>6.4</td>
<td>6.4</td>
<td>2.53</td>
<td>0.94</td>
</tr>
<tr>
<td>Different (%)</td>
<td>2.2</td>
<td>4.9</td>
<td>6.4</td>
<td>7</td>
<td>3</td>
<td>5.6</td>
</tr>
<tr>
<td>Friberg model 1940 (kN)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Different (%)</td>
<td>79.3</td>
<td>20.7</td>
<td>56.9</td>
<td>56.9</td>
<td>20.7</td>
<td>79.3</td>
</tr>
<tr>
<td>Tabatabaie model 1979 (kN)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Different (%)</td>
<td>65</td>
<td>19.2</td>
<td>19</td>
<td>19</td>
<td>19.2</td>
<td>65</td>
</tr>
</tbody>
</table>

#### 5.5 Prediction model for deflection

The deflection of the dowel will consist of the shear deflection and the flexural deflection. Because the joint opening distance ‘$Z$’ is small, the shear deflection will be the predominate deflection of the dowel and will be added to the small flexural deflection of the dowel.
Figure 5.7: Deflection across the joint

From the figure above, it can be seen that the deflection across the joint will be:

\[ \Delta_{\text{slab}} = 2(\Delta_{\text{conc}} + \Delta_{\text{dowel}}) \]

\[ \Delta_{\text{dowel}} = \Delta_{\text{flex}} + \Delta_{\text{shear}} = \frac{P\left(\frac{Z}{2}\right)^3}{3EI} + \frac{P\left(\frac{Z}{2}\right)F}{GA} \]

\[ = \frac{Pz^3}{24EI} + \frac{PZF}{2GA} \]

(5.19)

Where,

\( E \) = Modulus of elasticity for the steel dowel;
\( I \) = Moment of inertia for the steel dowel;
\( G \) = Shear modulus of elasticity for the steel dowel;
\( A \) = Cross-sectional area for the steel dowel;
\( F \) = Shear shape factor for the steel dowel.
The deflection of the concrete ($\Delta_{\text{conc}}$) can be estimated by assuming the dowel to be a beam on an elastic foundation.

$$\Delta_{\text{conc}} = \frac{P(2 + \beta Z)}{4\beta^3 EI}$$  \hspace{1cm} (5.20)

Where, $\beta = 4\sqrt{\frac{K_0 b}{4EI}}$ = relative stiffness of the dowel bar encased in concrete;

$K_0 =$ modulus of dowel support; $b =$ dowel bars width;

$E =$ modulus of elasticity of the dowel;

$I =$ moment of inertia of the dowel;

$P_t =$ load transferred through the dowel bar;

$Z =$ joint width.
Therefore, total slab deflection is equal to:

$$\Delta_{\text{slab}} = 2(\Delta_{\text{conc}} + \Delta_{\text{dowel}})$$

$$\Delta_{\text{slab}} = 2 \left( \frac{P(2 + \beta Z)}{4 \beta^3 EI} + \frac{Pz^3}{24EI} + \frac{PZF}{2GA} \right)$$  \hspace{1cm} (5.21)

Table 5.2: Comparison between currently developed deflection prediction models with F.E. model results 300 mm dowel spacing cases.

<table>
<thead>
<tr>
<th>Deflection (mm)</th>
<th>F.E. Model</th>
<th>Prediction Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>300 mm Spacing Case</td>
<td>0.272</td>
<td>0.283</td>
</tr>
<tr>
<td>Different(%)</td>
<td>4.2</td>
<td></td>
</tr>
</tbody>
</table>
5.6 Summary

The group action of the dowel bar system is examined and useful relationships have been developed for estimation of the relative load shared by individual dowel bars. These relationships have been used to develop a prediction model for the shear force in group action of dowel bar system and deflection at the loading nodal point. The outcomes of the prediction model agree reasonably well with the finite element results.
3D FE Model Validation with Analytical Solution

6.1 Background

Dowel bar stresses result from shear, bending, and bearing under applied vehicular loading. These stresses can be analysed to determine factors which affect load-transfer characteristics. The use of rigorous analysis in design, however, is open to some question, since several simplifying assumptions need to be made. Firstly, it is assumed that the dowel bars are perfectly aligned and free to move. Improper alignment or lubrication may cause the dowels to “freeze”, with the result that joint spalls may occur. In addition, the effectiveness of dowel bars to transfer the load across a joint depends to some extent upon the characteristics of the subgrade. As a result, most agencies have resorted to standard dowel sizes, wherein the diameter and length of dowel is dependent only upon pavement thickness.

In spite of the uncertainties involved, in the analysis of stresses in dowels, it is important for the design engineer to understand the causes and relative magnitudes of the stresses. The stress analysis of dowels is based upon work presented by (Timoshenko, 1925). According to Timoshenko, a dowel bar encased in concrete will deflect as shown in (Figure 2.4). When a load is applied at the end of the dowel bar, it will deflect downward.
exerting pressure at the lower face of the dowel for a distance designated as A to B in (Figure 2.4). At this point, a point of counter flexure exists with resulting bearing on the top of the dowel, and then, at some distance beyond this, bearing again in the bottom of the dowel bar. For the purpose of theoretical analysis, it is necessary to assume that the dowel bar is infinite in length, extending into an elastic body. This assumption can, however, be simplified by neglecting the small pressure exerted on the bar at some distance into the elastic body.

Bradbury (1938) and Friberg (1940) have presented mathematical analyses of dowel design, which are based upon the principles first presented by (Timoshenko, 1925). The relative stiffness of a bar embedded in concrete is given in equation 6.1.

$$\beta = 4\sqrt{\frac{Kb}{4Ei}} \quad (6.1)$$

Where, 
- \(K\) = modulus of dowel support;
- \(b\) = diameter of the dowel;
- \(E\) = modulus of elasticity of the dowel;
- \(I\) = moment of inertia of the dowel.

According to Timoshenko (1925), the deflection of the dowel bar resulting from the load \(P_t\) is:
\[ \Delta = \frac{e^{-\beta x}}{2\beta^2EI} [P_1 \cos \beta x - \beta M_0 (\cos \beta x \sin \beta x)] \quad (6.2) \]

Where,  
\( e \) = natural logarithm base;  
\( x \) = distance along dowel from face of concrete;  
\( M_0 \) = bending moment on dowel at face of concrete;  
\( P_t \) = transferred load.

Adoption of equation 6.2 to dowels has been expanded by (Friberg, 1940) for design purposes as follows. Bending moment and shear in the dowel can be expressed as in equation 6.3 and 6.4.

\[ -EI \frac{d^2y}{dx^2} = M = -\frac{e^{-\beta x}}{\beta} [P_1 \sin \beta x - \beta M_0 (\sin \beta x \cos \beta x)] \quad (6.3) \]

\[ \frac{dM}{dx} = V = -e^{\beta x} [(2\beta M_0 - P_t) \sin \beta x + P_t \cos \beta x] \quad (6.4) \]

If the joint-width opening is designated \( Z \) and since the concrete is very stiff compared with the steel bar, the moment at the dowel concrete interface is as follow:

\[ M_0 = -\frac{P_t Z}{2} \quad (6.5) \]

From equation 6.2, for \( x=0 \) and \( M_0 = -\frac{P_t Z}{2} \) the deflection of the dowel at the joint is:
\[ \Delta = \frac{P_i}{4\beta^3 EI} (2 + \beta z) \quad (6.6) \]

The bearing pressure on the concrete at the joint face is:

\[ \sigma = Ky_0 \]
\[ = \frac{KP_i}{4\beta^3 EI} (2 + \beta z) \quad (6.7) \]

Maximum moment occurs where the shear is equal to zero \( \frac{dM}{dx} = 0 \) and can be written as

\[ M = -\frac{P_i e^{-\beta x}}{2\beta} \sqrt{1 + (1 + \beta z)^2} \quad (6.8) \]

In each case \( P_i \) is the transferred load on the dowel and is less than the design load since a portion of the load is transferred by pavement to the subgrade.

### 6.2 Validation results

The 3D FE Model was validated with analytical solutions for shear force and moment along the dowel bar presented by (Timoshenko, 1925) and (Friberg, 1938). The general equation for the deflection of a dowel structure extending infinitely into an elastic mass can be written as follows (Timoshenko, 1925):
\[ y(x) = \frac{e^{-\beta x}}{2\beta^2 EI} [P \cos \beta x - \beta M_0 (\cos \beta x - \sin \beta x)] \quad (6.9) \]

Where \( \beta \) is the relative stiffness of the dowel structure and is defined as follow:

\[ \beta = 4 \sqrt{\frac{Kb}{4EI}} \quad (6.10) \]

Where \( b \) is diameter of the dowel; \( E \) is modulus of elasticity of the dowel; \( I \) is moment of inertia of the dowel. \( K \) is the modulus of dowel-support, representing the pressure intensity (MPa) required inducing a 1mm settlement, and \( P \) is the downward shear acting on the dowel, and transferred from one segment to the next.

In order to validate the numerical solution with the classical formulation (Timoshenko, 1925), a proper value of \( \beta \) should be used in the analytical expressions. The value of \( \beta \) can be obtained by calculating as follow: once the deflection of the dowel at the face of the joint \( y_o \) and the maximum shear acting on the dowel (\( P \)) is obtained from numerical results, the following equation can be solved for \( \beta \):

\[ y_o = \frac{P}{4\beta^3 EI} (2 + \beta z) \quad (6.11) \]
V(x) = -e^{-\beta x} \left[(\beta M_0 - P) \sin \beta x + P \cos \beta x\right]

M(x) = -\frac{e^{-\beta x}}{\beta} \left[P \sin \beta x - \beta M_0 (\sin \beta x - \cos \beta x)\right]

Figure 6.1: Validation of 3D FE Model with classical solutions at the dowel directly under the applied load for (a) shear force, and (b) moment along the embedded section of the dowel for $\beta = 0.0136 \text{ mm}^{-1}$
The modulus of dowel support $K$ can be known once the relative stiffness of the dowel ($\beta$) is determined. Knowing $\beta$, the shear $V(x)$ and moment $M(x)$ in the dowel bar can be solved, as shown in figure 6.1 (a) and (b) respectively. Friberg (1938) assumed a point of counter-flexure exists at the dowel midpoint, hence expressing the moment $M_0$ at the face of the joint as a function of $P$ and the joint width of $Z$ as follow:

$$M_0 = -\frac{PZ}{2}$$  \hspace{1cm} (6.12)

### 6.3 Summary

Figure 6.1 shows a comparison of the analytical solution of (Timoshenko, 1925) and (Friberg, 1938) with the obtained numerical solutions for shear force and moment along the dowel directly under the applied load. The 3D FE results for shear force and moment along the dowel were relatively close to the analytical solution. The solutions did not match exactly due to, the limitations involved in the assumptions behind the analytical solutions presented by (Timoshenko, 1925) and (Friberg, 1938). The reasons being (1) the assumption of semi-infinite dowel length as opposed to actual dowels of finite length; (2) the assumption of semi-infinite elastic mass in which the dowel bar is embedded, as opposed to an actual layered pavement structure with finite thickness for each layer; and (3) the assumption of uniform modulus of dowel support along the dowel, as opposed to a varying $K$ found from the back calculation from numerical results. However, (Porter, 1992) and (Albertson, 1992) have shown that Friberg’s equation can be used with little or
no error if the $\beta L$ value is greater than two. Where $L$ is taken to be the length of the dowel bar embedded in concrete, or approximately one-half the dowel bar length.

### 6.4 Validation of the FE Model with existing experimental Data

The 3D finite-element model of concrete pavement developed in the present study was validated using the results from an experimental study conducted by the U.S. Naval Civil Engineering Research and Evaluation Laboratory (Keeton and Bishop 1957) on load transfer characteristics of dowels used in airfield pavement expansion joints. The experimental setup is briefly discussed here for the sake of completeness.

The concrete pavement considered for the experiment was 15.24m long, 4.572m wide and 254mm in thickness. The pavement consisted of two 7.62m long slabs separated by a 19mm wide expansion joint. A transverse weakened plane joint was provided at the centre of each 7.62m long section without any dowels. The test pavement was constructed on a compacted subgrade having a modulus of subgrade reaction ($k$) of 54.3 mN/m$^3$. Fifteen 28.6mm diameter steel dowel bars of length 508mm were placed across the expansion joint at a spacing of 304.8mm. The dowels used in the test slab were greased and capped at one end and bonded or uncapped at the other end. Capping was provided alternately at the end of the dowels for the loaded and the unloaded slabs. Experimental results were reported for a static load of 222.4 kN and a tire pressure of 1.38 N/mm$^2$ having rectangular tire imprint of size 396.24mm x 406.4mm. The properties of concrete and steel, instrumentation used and the nature of load application device can be found in the paper by Keeton (1956).
Table 6.1: Comparison between F.E. Model results with Experimental Data (Keeton and Bishop, 1957) and Swati (2009) Model.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2</td>
<td>1.94</td>
<td>2.26</td>
<td>1.93</td>
<td>1.79</td>
<td>1.85</td>
</tr>
<tr>
<td>304.8</td>
<td>1.97</td>
<td>1.89</td>
<td>2.21</td>
<td>1.92</td>
<td>1.69</td>
<td>1.83</td>
</tr>
<tr>
<td>609.6</td>
<td>1.88</td>
<td>1.85</td>
<td>2.08</td>
<td>1.85</td>
<td>1.57</td>
<td>1.78</td>
</tr>
</tbody>
</table>
Conclusion and Recommendation for Further Research

7.1 Conclusion

A three-dimensional finite element model is developed in this study for analysing dowel-jointed concrete pavements. The effects of different pavement and joint related parameters on the load transfer efficiency (LTE) of a joint have been evaluated using the newly developed finite element model. The numerical results are validated with classical analytical solutions of shear and moment along the dowel. The group action of the dowel bar system is examined and useful relationships have been developed for estimation of the relative load shared by individual dowel bars. These relationships have been used to develop two prediction models to predict the shear force in group action of a dowel bar system and deflection at the loading nodal point. The results of the prediction models are relatively close and consistent with the finite element results. This study has reached the following conclusions:

(1) From observation, the dowel can significantly reduce the maximum vertical displacement at the transverse joint along line TT\textsubscript{HH} by 12%. Correspondingly, dowel has reduced the magnitude of Maximum Principle Stress (MPS) by 92.5% along the longitudinal pavement section. The increase in efficiency demonstrates that the
dowel bars perform effectively as load transfer device in the concrete pavement systems.

(2) For dowel-jointed pavement with lean concrete base (LCB), the maximum vertical displacement at transverse joint along line TT<sub>HH</sub> for the load case 3 were reduced by 47%. It can be observed that peak MPS is about 6.70 times greater and shows steeper variation in the distribution pattern in the concrete pavement without LCB than that with LCB.

(3) In the case of pavement with cavity (voids), the maximum vertical displacement at transverse joint along line TT<sub>HH</sub> was increased by as much as 20.6%. Furthermore, evidence of decreased LTE was observed to be as much as 77.59%, 79.53%, 79.02% and 47.76% for LC1, LC2, LC3 and LC4 respectively in the three-layer concrete pavement without dowels and with cavity (voids). Maximum displacements were increased by 40.94%, 42.59%, 42.10%, 26.41% and 1.73% for LC1, LC2, LC3, LC4 and LC5 respectively in the three-layer concrete pavement without dowels and with cavity (voids).

(4) For varying subgrade strength, it can be concluded that the pavement with dowels reduces the maximum vertical displacement by around 9.7%, 11.4%, 14.2% and 16.4% respectively with subgrade strength range from 30 to 50, 100 and 170 MPa. In the meantime, for load case 3 of four-layer concrete pavements with dowels at 300 mm spacing under dual wheel loading, the maximum vertical displacements at the transverse joint along line TT<sub>HH</sub> are reduced by 32.16%, 57.29% and 68.84%
respectively with increasing subgrade strength from 30 to 50, 100 and 170 MPa. However, there is no significant change in the load transfer efficiency with the subgrade strength varying from 30 to 170 MPa.

(5) Results demonstrate that eight engaged dowels carry more than 70% and up to 95% of the transfer shear force across the joints for three-layer pavement models in single and dual-wheel loading cases respectively, for both 750 kPa and 1500 kPa loading. The maximum displacements have been increased by 100% for load case 3 if the applied loads increased from 750 kPa to 1500 kPa.

(6) The total amount of transfer shear for the 225 mm spacing case was computed to be about 6%, 14.3% and 26.2% larger than for the 300 mm, 450 mm and 600 mm spacing cases respectively in the first 1800 mm along the joint. Only three dowels were engaged for 600 mm spacing cases within 1800 mm symmetric region, while four dowels were engaged in 450 mm spacing case, six dowels were engaged for 300 mm spacing cases and eight were engaged for 225 mm spacing case. The distribution of shear force varies from almost 100% for the 225 mm spacing cases to approximately 73.8% for the 600 mm spacing cases.

(7) It can be observed that the amount of load transfer increases as the slab thickness increases. For single wheel loading case with dowel spacing at 300 mm in the 250 mm thick concrete slab shows the total load transfer ratio in the six engaged dowel of 47.6% whereas for 375 mm and 500 mm concrete slab the first six engaged dowels transfer 95.8% and 97.6% respectively. The maximum vertical displacements at the
transverse joint along line $TT_{HH}$ were reduced by 16.2\% for LC1, 17.9\% for LC2, 20.2\% for LC3, 20.7\% for LC4 and 21.3\% for LC5 respectively with increased concrete slab thickness from 375 mm to 500 mm. However, there was no significant change in the load transfer efficiency (LTE).

(8) The amount of load transfer was observed to increase when more wheel loads were applied. For the first six engaged dowels, there were only 47.60\% of load transfer for single wheel loading and 94.17\% for dual wheel loading case in concrete pavement with lean concrete with dowels spacing at 300 mm. In fact, dowels provided partial continuity to the discrete concrete slab segments by transferring shear force. The maximum vertical displacements at transverse joint along line $TT_{HH}$ were reduced by 51.35\% for LC1, 52.08\% for LC2, 53.15\% for LC3, 53.47\% for both LC4 and LC5 respectively with single wheel loading while compared to dual wheel loading in concrete pavement with lean concrete and with dowel spacing at 300 mm. However, there was no significant change in the load transfer efficiency (LTE increased by only 4.1\% for LC3).

(9) The effects of dowel looseness on LTE are quite significant. It is observed that LTE decreases with the increase in dowel looseness. LTE varied from 94.4\% (intact joint) to 65.1\% (1.25mm gap for case 4) for single wheel loading case; 92.6\% (intact joint) to 66.5\% (1.25mm gap for case 4) for even dual wheel loading case. However, in the worst cases the LTE was reduced to 11.3\% and 11.6\% respectively for single wheel loading and odd dual wheel loading case while there were voids present at the base course layer for 1.25 mm gap between dowel and slabs.
(10) Increasing gaps between slab and dowel significantly decreases the dowel shears. The increase in the gap also increases the stresses applied to the base course layer under loaded slab adjacent to the joint with a concomitant decrease in applied stresses under unloaded slab. In fact, the maximum vertical stress applied to the base course layer by the loaded slab varies from 259.4 to 468.4 kPa as the gap increases from 0-1.25 mm, an increase of 80.6%. A similar decrease in stresses applied to the base course layer under unloaded slab also occurs. Meanwhile, the principal tensile stresses in the bottom of the slab vary from 342.9 to 426.9 kPa with gaps of between 0 and 1.25 mm, an increase of 25%. In the meantime, with the presence of voids at the base course layer, the principal tensile stresses at the bottom of the slab increase by 52% have been observed.

(11) The 3D FE results for shear force and moment along the dowel were observed to be relatively close to the analytical solution. The solutions did not exactly match due to the limitations involved in the assumptions behind the analytical solutions presented by (Timoshenko, 1925) and (Friberg, 1938). The reasons were (1) the assumption of semi-infinite dowel length as opposed to actual dowels of finite length; (2) the assumption of semi-infinite elastic mass in which the dowel bar is embedded, as opposed to an actual layered pavement structure with finite thickness for each layer; and (3) the assumption of uniform modulus of dowel support along the dowel, as opposed to a varying K found from the back calculation from numerical results. However, (Porter, 1992) and (Albertson, 1992) have shown that Friberg’s equation can be used with little or no error if the $\beta L$ value is greater than two. Where $L$ is
taken to be the length of the dowel bar embedded in concrete, or approximately one-half the dowel bar length.

(12) The prediction model results for shear force in dowel group action of dowel bar system and deflection at the loading nodal point were relatively close to the F.E. model results, with the percentage difference varying between 2.2% to 7%.

7.2 Recommendation for further research

Based on the findings, shown herein, the following research is proposed:

(1) The analysis of pavement structural response in static linear elastic models may lead to inaccurate structural response analysis of pavements with nonlinear behaviour. Therefore further research is recommended for developing a 3D FE pavements model to consider the effects of dynamic loading and pavement nonlinearities. Appropriate material property inputs are essential for obtaining accurate and meaningful advancement of 3D FE dynamic analysis.

(2) The current study focused on the stresses around the dowel bars due to the application of axle loads. Further studies are needed to explore the effect of temperature variations through the slab depth on the induced stresses. This research is relevant to the study of the impacts of climate change on the structural deterioration of concrete pavements.
(3) During the course of this research, it was found that there is no data on the coefficient of friction between regular dowels and concrete is available in current literature. There is a need to establish the coefficient of friction parameter.

(4) More research is needed to identify the effect of concrete curing at early stages of pavement constructions on the joints characteristics. This study could provide useful information on the residual stresses developed around the dowel bars due to shrinkage of concrete as well as the variation of moisture content and temperature in the slab at early stages.

(5) The numerical solutions from F.E modelling for shear force and moment along the dowel directly under the applied load have been validated with the analytical solutions of Timoshenko (1925) and Friberg (1938). In future research, the FE results should be verified with the field data to increase confidence in using the FE program. This is because 3D FE modelling delivers more detailed analysis and accurate results of a complete structure rather than any other approaches.

(6) Perform field tests to obtain void dimensions and locations for different pavement distresses. Develop finite element models for different void details and evaluate pavement performance.

(7) Further studies are also required to determine effects of surface roughness, traffic wander, and length to width ratio of the concrete slab panel, width of longitudinal
and transverse joints and vertical position of reinforcement on dynamic structural response of different concrete pavements under diverse transient axle group loads.

(8) Modelling a nodal contact between the slab and the subgrade is essential as it is critical when considering the wheel loading at the transverse joint.


Alexander, W.K., M.T., N.M.J., B.C., M.B., 2005, “Concrete Slab Instrumentation and Comparison of Field Measured Response to Computed Responses”, 5th International Conference on Korea Airfield pavement technology, Seoul Korea, 10-12 May.


References


Vorobieff, G., 2001, “Recent Development in Australia in Concrete Pavement Thickness Design”, 7th International Conference on Concrete pavements, Orlando, Florida, USA.


Wesevich, J.W., McCullough, B.F., Burns, N.H., 1987, “Stabilised Subbase Friction Study for Concrete Pavements”, Research Report 459-1, Centre for Transportation Research, University of Texas, Austin, USA.


