Enhancement of Backcalculation Techniques for Assessing Flexible Pavement Layer Moduli Using Genetic Algorithms

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Vorwort des Herausgebers

Zur Bewertung des Zustands von Straßen werden vielfach Falling Weight Deflectometer (FWD)-Tests durchgeführt. Hierbei wird über eine definierte Fläche eine dynamische Last auf den Straßenkörper ausgeübt und die sich dadurch ergebende Deformation der Straßenoberfläche in verschiedenen Abständen über Geophone gemessen. Die (zerstörungsfrei) gemessene Verformungsmulde erlaubt dann Rückschlüsse auf den Zustand und insbesondere auf die Steifigkeiten der einzelnen Tragschichten. Es handelt sich hierbei um ein klassisches Problem der inversen Parameteridentifikation.

Als Modell zur Beschreibung des Verhaltens des Straßenkörpers wird in der Praxis fast ausschließlich die elastische Mehrschichttheorie verwendet. Sofern der generelle Straßenaufbau bekannt ist, d. h. insbesondere die Schichtdicken der einzelnen Lagen, besteht die Aufgabe in der Suche der Kombination von E-Moduln der einzelnen Schichten, für welche die berechnete Deformationsmulde (bzw. die diskreten Verschiebungen) optimal mit den gemessenen Verschiebungen übereinstimmt. Schwierig gestaltet sich dieses Optimierungsproblem vor allem deshalb, weil die Zielfunktion zahlreiche lokale Minima aufweist. Forschungen in den letzten Jahren haben gezeigt, dass die Verwendung „genetischer“ mathematischer Algorithmen (GA) diesbezüglich sehr erfolgversprechend ist.

Herr Thongindam befasst sich vor diesem Hintergrund mit der Anwendung und Weiterentwicklung genetischer Algorithmen auf das beschriebene Optimierungs-problem.

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Prof. Dr.-Ing. habil. J. Hothan

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Puttapon Thongindam

Abstract

Backcalculation of pavement layer moduli based on surface deflection measurements using falling weight deflectometer (FWD) has been widely used for structural evaluation of in-service flexible pavement structures. Most practical iterative backcalculation programs use multi-layered elastic theory (MLET) as forward model and arrive at their solutions by minimizing the differences between computed and measured deflections. It has been found that the solution obtained from the traditional backcalculation programs may not always be appropriate from an engineering point of view. The reason is the local minima in solution.

Genetic algorithms (GAs) have been used successfully in the recent past for backcalculation program by showing the capability to overcome the local minima problem. Two main limitations of such GA-based programs are the relatively long computing time and the indicating of the optimal set of GA parameters. On the other hand, several methods for determining depth to bedrock (DTB) from deflection basins have been proposed to improve the accuracy of setting up pavement models. It has also been found that adding artificial bedrock into pavement model can improve the convergence behavior of iterative backcalculation programs since this technique can deal with the behavior of subgrade stiffening with depth. Two existing procedures have been investigated in this work. Unfortunately, both procedures have yielded unacceptable results.

In order to overcome the mentioned problems, a new GA-based backcalculation program GAMLET has been developed in this work for assessing flexible pavement layer moduli. This program also uses MLET as forward model to keep the approach still practical. A new proposed method for determining depth to artificial bedrock (DTAB) coined as Consistent Slope Changing Method (CSCM) has also been proposed and verified. The CSCM has been added into GAMLET as an option to help in making decision about setting up the pavement model. Performance of GAMLET which contain several new GA operators and techniques has been evaluated using deflection basins obtained from both pavement models and in situ data. The results show that the algorithms used in GAMLET can improve the robustness of search process and have potential to overcome the limitations encountered in the existing GA-based backcalculation programs.

Key words: backcalculation, genetic algorithms, falling weight deflectometer, flexible pavement, depth to bedrock

Kurzfassung

Die Rückrechnung von Schichtmoduln einer Verkehrsbefestigung auf der Grundlage von Messungen der Oberflächendeflexionen mit dem Falling Weight Defelectometer (FWD) wird weltweit angewendet. Die meisten Rückrechenprogramme basieren dabei auf der Mehrschichtentheorie. Der Algorithmus dieser Programme versucht in einer Vorwärtsrechnung die Differenzen zwischen den gemessenen und den berechneten Einsenkungen zu minimieren. Wird das Abbruchkriterium, eine frei wählbare Abweichung der Deflexionen, erreicht, stoppt das Programm. Diese Vorgehensweise führt häufig zu Schichtmoduln, die allein aus der ingenieurmäßigen Anschauung heraus falsch sein müssen. Die Begründung dafür liegt darin, dass häufig lediglich ein lokales Minimum gefunden wird.

Genetische Algorithmen (GA) wurden in der Vergangenheit bereits erfolgreich eingesetzt, um das Problem lokaler Minima zu lösen. Allerdings gibt es zwei wesentliche Einschränkungen bei der Anwendung von Programmen auf der Grundlage von GA, die lange Rechenzeit und die Wahl der besten Parameter für den Algorithmus. Andere Versuche die Konvergenzen der iterativen Rückrechnungen zu erhöhen, wie die Annahme eines Bedrocks, wurden an Hand von zwei bekannten Verfahren untersucht und führten zu nicht akzeptablen Ergebnissen.

Um die zuvor genannten Probleme zu lösen, wurde in dieser Arbeit ein neues Rückrechenprogramm (GAMLET) für die Bestimmung von Schichtmoduln flexibler Befestigungen entwickelt. Das Programm basiert ebenfalls auf der Mehrschichtentheorie, damit es praktikabel und anwenderfreundlich bleibt. Zusätzlich wurde ein weiteres Modul entwickelt und implementiert, welches dazu dient die Tiefe eines fiktiven Bedrocks zu bestimmen (Consistent Slope Changing Method (CSCM)). Außerdem wurden neue und komplexere genetische Algorithmen entwickelt und in verschiedene Module der Software übernommen. Abschließend wurden zahlreiche Tests an berechneten und gemessenen Deflexionsmulden mit verschiedenen Moduln durchgeführt, die zeigen, dass die entwickelten Algorithmen sowohl stabiler laufen als auch die Probleme der bekannten Algorithmen behoben werden.

Schlagwörter: Rückrechnung, genetische Algorithmen, Falling Weight Deflectometer, flexible Befestigungen, Bedrock

บทคัดย่อ

การคำนวณย้อนกลับ (Backcalculation) เพื่อหาค่าโมดูลัสยืดหยุ่นของชั้นโครงสร้างทางโดยอาศัยค่าจากการทดสอบการยุบตัวของผิวทางโดยเครื่องมือ Falling Weight Deflectometer (FWD) ถูกใช้อย่างแพร่หลายในการประเมินสภาพโครงสร้างผิวทางลาดยางขณะใช้งาน โปรแกรมคำนวณย้อนกลับแบบวนซ้ำส่วนใหญ่ใช้ทฤษฎียืดหยุ่นสำหรับชั้นทางหลายชั้น (Multi-Layered Elastic Theory, MLET) เป็นแบบจำลองในการคำนวณแบบเดินหน้าเพื่อหาค่าการยุบตัวบนผิวทางของแบบจำลองนั้น และหาค่าคำตอบโดยวิธีการลดค่าความแตกต่างระหว่างค่ายุบตัวที่ได้จากแบบจำลองและค่าวัดจริง ปัญหามักเกิดขึ้นเมื่อค่าคำตอบที่ได้ขัดแย้งกับค่าความเป็นไปได้จากมุมมองทางวัสดุวิศวกรรม สาเหตุนี้เกิดขึ้นได้เนื่องจากการมีอยู่ของหลายจุดต่ำสุดในพื้นผิวคำตอบ

ในช่วงหลายปีที่ผ่านมาขั้นตอนวิธีเชิงพันธุกรรม (Genetic Algorithm, GA) ได้ถูกนำมาประยุกต์ใช้อย่างประสพความสำเร็จในการแก้ไขปัญหาการมีอยู่ของหลายจุดต่ำสุดในพื้นผิวคำตอบดังกล่าว แต่ข้อจำกัดในการใช้งานจริงของวิธีการนี้ก็เกิดขึ้นเนื่องจากระยะเวลาการคำนวณที่นานขึ้น และปัญหาในการเลือกค่าตัวแปรที่เหมาะสม

ในอีกด้านหนึ่งก็ได้มีการนำเสนอวิธีการในการหาค่าความลึกของชั้นหินแข็ง (bedrock) ใต้โครงสร้างทางจากค่าการยุบตัวของผิวทางเพื่อปรับปรุงความถูกต้องของแบบจำลอง นอกจากนั้นยังมีข้อบ่งชี้ว่าการเพิ่มชั้นหินแข็งสมมุติในแบบจำลองสามารถช่วยพัฒนาลักษณะการลู่เข้าหาคำตอบ ซึ่งอธิบายได้ว่าชั้นหินแข็งสมมุติดังกล่าวสามารถจำลองลักษณะความแข็งแรงที่เพิ่มขึ้นตามความลึกของดินในชั้นคันทาง โดยในงานวิจัยนี้ได้มีการทดสอบค่าความถูกต้องของสองวิธีการที่ใช้หาความลึกชั้นหินแข็ง ซึ่งพบว่าทั้งสองวิธีการไม่สามารถให้คำตอบที่น่าพอใจ

เพื่อแก้ไขปัญหาดังกล่าวทั้งหมดข้างต้นโปรแกรมการคำนวณย้อนกลับชื่อ GAMLET ได้ถูกพัฒนาขึ้นในงานวิจัยนี้เพื่อใช้ในการคำนวณหาค่าโมดูลัสยืดหยุ่นของชั้นโครงสร้างผิวทางลาดยางโดยใช้ขั้นตอนวิธีเชิงพันธุกรรม โดยยังคงใช้ MLET เป็นพื้นฐานของแบบจำลองในการคำนวณแบบเดินหน้าเพื่อให้เหมาะสมกับสภาพการใช้งานในภาคปฏิบัติ นอกจากนั้นวิธีการประเมินค่าความลึกของชั้นหินแข็งสมมุติแบบใหม่ชื่อ Consistent Slope Changing Method (CSCM) ได้ถูกนำเสนอพร้อมทั้งได้แสดงการตรวจสอบความน่าเชื่อถือในงานชิ้นนี้ วิธี CSCM นี้ได้ถูกจัดรวมเข้าไว้ในโปรแกรม GAMLET เพื่อช่วยในการตัดสินใจเลือกใช้แบบจำลอง ในช่วงท้ายของงานวิจัยชิ้นนี้ได้มีการทดสอบประสิทธิภาพของโปรแกรม GAMLET ซึ่งเป็นโปรแกรมที่ประกอบด้วยตัวปฏิบัติการและเทคนิคใหม่จำนวนมากของขั้นตอนวิธีเชิงพันธุกรรม โดยใช้ข้อมูลค่าการยุบตัวของโครงสร้างทางที่ได้จากแบบจำลองคอมพิวเตอร์และค่าที่วัดได้จริงจากภาคสนาม ผลการทดสอบชี้ให้เห็นว่าระบบขั้นตอนวิธีที่ใช้ใน GAMLET สามารถพัฒนาเสถียรภาพของกระบวนการค้นหาคำตอบ พร้อมทั้งยังมีศักยภาพในการแก้ไขปัญหาและข้อจำกัดที่พบในโปรแกรมคำนวนย้อนกลับที่ใช้ขั้นตอนวิธีเชิงพันธุกรรมที่มีอยู่ได้

คำสำคัญ: การคำนวณย้อนกลับ ขั้นตอนวิธีเชิงพันธุกรรม เครื่อง Falling weight deflectometer ผิวทางลาดยาง ความลึกของชั้นหินแข็ง

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1 Introduction

In recent years, it seems that the nature of road building has evolved toward preserving and rehabilitating existing roads, more than building new one. It is therefore very useful to test the pavement in place, nondestructively, and to analyze the data to determine desired pavement layer properties. Thus, nondestructive testing (NDT) has evidently become an important part of the structural evaluation of pavement structures. Research concerning the development of new techniques and methods to measure the pavement layer properties needed for the accurate evaluation, design and management of pavement receives great recognition among pavement engineers. The properties needed for making realistic predictions of the remaining life, designing overlays, and recycling layers properties of flexible pavement can be summarized as the following [57]: the elastic stiffness of each layer, meaning either the elastic modulus or the stress-strain curve properties of stress-independent materials, layer thickness, water content in unbound layers, binder content in asphalt bound layers, fatigue properties for both load and thermal fatigue processes, permanent deformation properties of each layer, residual stress in situ, and other properties.

Among these properties, the elastic moduli of pavement layers can be considered as the most common properties of pavement materials. For instance, pavement rehabilitation projects involve in many cases the retention of most of the layers in the existing pavement. The in situ layer elastic moduli are hence needed to be known for using as an input data in almost further analysis. Thus, it is worthwhile to be able to determine the in situ layer moduli. The procedure to determine Young's modulus of elasticity for pavement materials using measured surface deflections by working elastic layer theory "backward" is generally called "backcalculation" [46].

The NDT equipment used in the pavement surface deflection measurements includes a variety of modes for applying loads to a pavement and a number of sensors for measuring the pavement response. Taken overall of the well-known equipments, theses can probably be divided into six categories as the following [20]: 1) Static deflection measurement equipment, e.g. Benkelman Beam; 2) Automated beam deflection measurement equipment, e.g. Lacroix Deflectograph; 3) Dynamic deflection measurement equipment, e.g. Curviameter; 4) Deflection instrument with a harmonic load, e.g. Dynaflect, Road Rater; 5) Wave propagation measurement; and 6) Deflection measuring equipment with an impulse load, e.g. Falling Weight Deflectometer (FWD).

Among these NDT equipments, the FWD has evolved as the favorite and most suitable device for pavement structural evaluation because of its ability to simulate traffic loading, both in term of magnitude and loading time. Moreover, the accuracy of reproducibility and repeatability of FWD has shown a superior performance over the others [24]. Not surprisingly, the use of FWD in both project and network level extends quickly to pavement management systems work worldwide. The endeavor to enhance backcalculation techniques in this work is therefore mainly based on the surface deflection data obtained from the FWDs.

1.1 Definition of Problem Statement

Although the FWD equipments, the pavement mechanistic theories and the computer technologies have been evidently developed in the last few decades, backcalculation are still not an easy task. It requires a good engineering background and extensive experience in order to get the reliable results. Manifold problems in the field of backcalculation of pavement layer moduli from FWD testing data need to be solved solemnly in order to get more practical and more trustworthy results. Some of these significant problems can be listed below:

- Optimization of computational algorithms,
- Method for determining depth to related stiff layer called “bedrock”,
- Dealing with nonlinearity in subgrade material.

1.1.1 Optimization of computational algorithms

One of the biggest problems in the backcalculation work is optimization of computational algorithms. Numerous computer programs have been developed for backcalculating layer moduli. Most practical backcalculation programs for flexible pavement layer moduli are based on iteration method and arrive at their solutions by minimizing a function related to the differences between computed and measured surface deflections. The manner in which this takes place differs in each program.

Unfortunately, it has been found that the backcalculated modulus values and their accuracy are procedure-dependent. Harichandran et al. have examined this dependency by making a comparison of several popular backcalculation algorithms based on their computed modulus values for selected problems. It was evident that different backcalculated layer modulus values for a given pavement structure may be obtained from different procedures [36]. Moreover, most of these traditional backcalculation procedures require seed moduli to initiate the backcalculation process. It has been reported that different seed moduli often lead to different backcalculated moduli which, in turn, lead to different pavement designs and evaluations. All these cause the solution obtained from those backcalculation programs may not always be appropriate from an engineering point of view especially when the in situ data is analyzed. The reason for this is probably due to model parameter errors such as layer thicknesses or Poisson’s ratio values or any measurement errors. Another possible reason, perhaps the main reason, is due to the computational algorithms used in those backcalculation procedures. Since the solution of the backcalculation problem of pavement-layer moduli is known to contain many local minima [[57], [82], [51], [28] and [2]], algorithms lacking for a good potential in global search ability (determining global minimum) can have some difficulties in solving such problems.

On the other hand, the maximum number of layers in pavement model allowed to be analyzed in many of the traditional backcalculation programs are usually limited to only 3 or 4 layers in order to reduce the error associated with the backcalculation process. This can be a very significant limitation when the pavement structure is complicated. It requires that distinctly different layers be combined, and the resultant moduli do not model the pavement very accurately [46]. Furthermore, this limitation makes increasing the layer number of input model, in some cases, to get more representative variation of the subgrade moduli with depth troublesome or impossible.

In the last few decades, an optimization algorithm called "genetic algorithm" (GA) has been successfully applied in many scientific and engineering fields. Pavement engineering is one of those. GAs have been provided themselves as a robust optimization procedure and capable of solving many large complex problems where other traditional methods have experienced difficulties. The most prominent nature of GAs is that they have a very good potential in global search ability. Recently, many of new GA operators and techniques have been developed. These features make GAs appealing to be used in enhancement of backcalculation analysis.

1.1.2 Method for determining depth to bedrock

Another independent problem in doing backcalculation is in setting up the pavement model correctly. Generally, the thicknesses of each pavement layer are needed as model input parameters for backcalculation process. The thicknesses of layers in upper zone could be determined by borings, coring or using new equipment such as Ground Penetrating Radar (GPR). On the other hand, the subgrade thickness is usually unknown and not clearly defined. At first glance, it seems to be no problem since most of practical backcalculation schemes are based on elastic layer theory where the bottom layer is assumed to be semi-infinite in depth with a constant elastic modulus. However, in a real pavement subgrade layer, an apparent stiff layer or "bedrock" can exist everywhere and at any depth which may lead to an unaccounted for high-modulus layer. Practically, the only way to determine the depth to bedrock (DTB) under pavements has been through coring, borings or penetration but all of these seem impractical to use at every point tested with FWD. Ideally, the DTB should be inferred from the measured deflection data since this is the easiest and least expensive way. Several methods have been proposed to estimate the DTB using maximum surface deflections and/or deflection-time history data. Unfortunately, the field verifications of these procedures are rarely reported. Thereby the reliability of the results obtained from these procedures is often dubitable.

Since the backcalculation procedure based on multi-layered elastic theory (MLET) is the most practical for routine analysis based on its accuracy and required computing time, the practical method for determining DTB from in situ deflection data should also be compatible with the MLET. For this reason, the reliabilities and limitations of the existing methods for determining DTB qualified for this category will be investigated in this work. A new method that is able to overcome those limitations should be developed.

1.1.3 Dealing with nonlinearity in subgrade materials

As discussed, the backcalculation methods based on equivalent thickness and MLET are more practical for routine deflection analysis because their inherent assumptions are a linear elastic and infinitely thick subgrade. Nevertheless, most subgrade soils in the real world are well-known as stress sensitive or “non-linear” in stiffness behavior. Its stiffness is influenced by the prevailing stress state. The stress condition changes vertically and horizontally and so does the stiffness of the subgrade. Additionally, it also depends on the gradation of moisture content, the modulus can either increase (stress hardening), or decrease (stress softening), as the load stress increases. However, due to overburden pressure, it is likely that in subgrade layer the confining pressure increases with depth, as does the density of material. Thus, the modulus in most subgrade materials often increases with depth. Considering all of these facts, the results obtained from backcalculation methods with constant subgrade modulus are always doubtful.

It is clear that, more reliable backcalculated results can possibly be obtained from the use of other more complicated approaches, such as nonlinear elastic approach, finite element method (FEM), dynamic analysis. Although a number of computer programs based on such approaches have been developed in the field of backcalculation for many years and there has been also a growing interest in the use of these methods among pavement engineers. These programs involve, however, a large number of input parameters and hence require substantial computing capacity. As a result, their use has mainly been limited to the research community and there appears to be no general use of such methods by practitioners in routine backcalculation analysis.

In the past, there have been some attempts to deal with subgrade stiffening with depth in backcalculation analysis based on MLET by adding artificial bedrock at some depth under the subgrade layer. It has been found that such technique can deal in some degree with subgrade stiffening with depth by yielding more reliable results. Moreover, this technique can somehow improve the convergence behavior of most iterative backcalculation procedures. However, most of such techniques reported in literature are referred to generated database or developed from empirical data which are valid only under the environment for which they developed. There are no generalized methods that can indicate the appropriate depth to artificial bedrock (DTAB) at which this fictive layer should be assigned into pavement model. The limitations of the existing methods should be investigated. A new method for determining the DTAB based on these limitations should be developed.

1.2 Research Objectives

As the title expressed, the overall objective of this research is to enhance the backcalculation techniques for assessing flexible pavement layer moduli. In the view of the definition of problem statement discussed earlier, the following secondary objectives can be drawn:

1. Study the difficulties of backcalculation problem using FWD data: accuracy of deflection data, limitations and accuracy of input model, objective functions to backcalculate the moduli, complexity of the solution space, result criterions, etc.
2. Develop a new backcalculation computer program using genetic algorithms (GAs). New recently developed GA operators and techniques should be considered in the development to overcome the mentioned difficulties in backcalculation problem. This new developing backcalculation program should show the improvement of robustness in search process. Furthermore, it should be practical for routine backcalculation analysis, i.e. user-friendly and requiring less computing time.
3. Study the effect of bedrock (depth from surface, stiffness, etc.) on the backcalculated moduli based on MLET and investigate the advantages and drawbacks of some existing procedures used to determine DTB.
4. From the limitations of the investigated procedures, develop a new method to determine the depth to artificial bedrock (DTAB) from FWD testing data. The depth result from this new procedure should indicate the depth to real bedrock (DTB), if exists, or the appropriate depth at which a fictive bedrock can represent the increasing stiffness with depth in most subgrade materials.
5. Illustrate the accuracy and reliability of the proposed method by performing verification using the deflection basins obtained from both pavement models in computer program and real flexible pavement structures underlain by real bedrock.
6. Integrate the new proposed method for determining DTAB into the new developed backcalculation program as an alternative function for setting up pavement model in backcalculation process.
7. Evaluate the accuracy and reliability of the new developing GA-based backcalculation program by performing verification with several cases of backcalculation problem using deflection data obtained from both pavement model in computer program and from real pavement structures.

2 An Overview of FWD Test and Backcalculation Analysis

2.1 FWD versus Flexible Pavement

Falling Weight Deflectometers (FWDs) have been in use since the 1980s. Even though these devices can be used to evaluate the structural capacity of both rigid and flexible pavements, the testing method with FWD and the analysis of data obtained from testing on both pavement types are usually performed in different manner because the physical structure and the in service behavior of both pavement types are significantly different. As the title pronounced, this work has been carried out for enhancing the backcalculation techniques for assessing the layer moduli using the FWD data obtained from flexible pavement. Some important technical information about FWD equipments and related issues about the test with this device on flexible pavement structures are briefly discussed below.

2.1.1 Mechanism of FWD

The main mechanical unit of FWD equipment is shown in Figure 2-1. This equipment is designed to impart a load pulse to the pavement surface which simulates the load produced by a rolling vehicle wheel such as moving truck or aircraft. The load is produced by dropping a weight package on a dampening system, and transmitted to the pavement through a circular load plate (typically 300mm diameter).

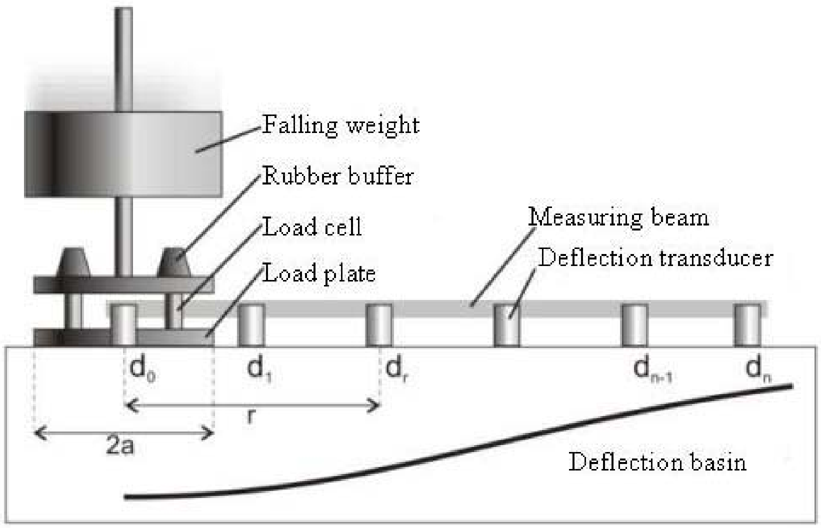


Figure 2-1. Working System of Falling Weight Deflectometer (FWD) equipment

By varying the size of the falling weight and/or the drop height, the size of the load pulse can be varied between 20 kN and 120 kN. The heavier FWD types can generate a load pulse of 250 kN. Depending upon the make of FWD, the pulse duration can vary between 25 ms and 60ms. A load cell mounted on top of the load plate measures the load imparted to the pavement surface. Usually, a series of 6 to 9 deflection sensors (most FWDs use geophones, force-balance seismometers are also used) mounted radially from the center of the load plate measure the deformation of the pavement in response to the load. The typical sensor locations used for testing flexible pavement in this work are 0, 200, 300, 450, 600, 900, 1200, 1500 and 1800 mm away from the load center.

Signal from the load cell and the deflection sensors are fed into the system processor which selects peak values and transfers this information to an onboard computer. Normally, a computerized system in the tow vehicle monitors and controls the testing cycle. A typical test sequence is approximately one minute long, so testing proceeds very rapidly down a street, highway or airfield.

2.1.2 FWD test on flexible pavement

To evaluate the bearing capacity or to assess the layer moduli of flexible pavement, one should comprehend the factors that can have significant effect on these values. These influences can be roughly divided into two groups: external and internal factor as shown in Figure 2-2.

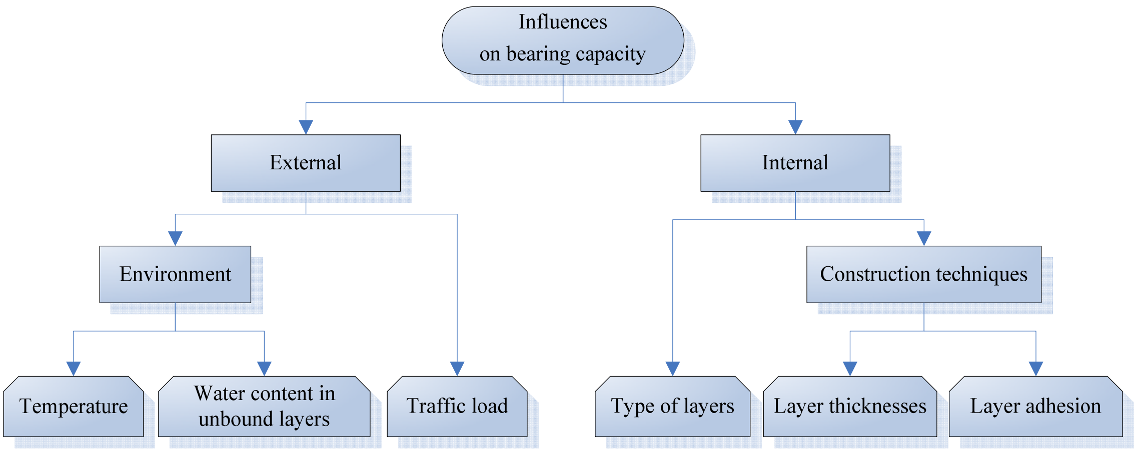


Figure 2-2. Influences on bearing capacity of pavement structure [71]

2.1.2.1 Temperature effect

Not surprisingly, temperature is one of the factors that have a large influence on bearing capacity and layer moduli of flexible pavements. It is clear that the upper layers (asphalt layers) of this pavement type consist of bituminous materials and the physical behavior of these materials is known to be viscoelastic. This indicates that the deformation behavior under loading of such material is dependent on temperature and loading time. The bituminous materials perform as viscous fluid at high temperature and long loading time. On the other hand, the elastic or quasi-elastic behavior can be observed at very low temperature and short loading time. Moreover, the asphalt layers are subjected to temperature gradients effect because the temperature inside these layers usually varies with depth. There was a report about the behavior of elastic modulus of bituminous material that this value can vary with the factor up to 7 in the range of practical temperature (-5 °C to 30 °C) [49].

From the above reasons, two important actions should be differentiated when carrying out deflection measurements on flexible pavements. These are measuring the deflections and measuring the asphalt temperatures. The temperature should be measured at the surface and at the middle depth of this layer. These parameters are important for assessing the reliability of the backcalculated moduli.

There have been some attempts to eliminate or reduce the influence of temperature on deflection values. One example of those attempts is converting the deflection value at the load center (D_0) from the middle depth temperature at measuring time to a certain standard middle depth temperature as shown in the equation below:

$$D_{0,\text{ standard temp.}} = K \cdot D_{0,\text{ measuring temp.}} \tag{2-1}$$

where the converting factor "K" is generally function of temperature of asphalt layer at middle depth and the asphalt layer thickness.

According to the annual average temperature of many countries in Europe, included Germany, the value of 20 °C has been chosen as the standard middle depth temperature of asphalt layer. Using the eq(2-1) as principal concept different converting factors, K, were proposed in some research works [[38], [26], [40]].

As mentioned earlier, using the factor "K" to convert the deflection values at measuring temperature to standard one is only one example of this issue. There are actually more methods and procedures, which struggle to reduce the influences of temperature on deflection values in bituminous layers [3]. An investigation to find out the advantages and drawbacks of each of these converting factors and procedures is beyond the objectives of this research. To avoid the concealed erroneousness that can possibly be suffered from using these different procedures, all the backcalculations of the in situ deflection data in this work are performed without any temperature correction procedure. The backcalculated results can be therefore considered as the layer moduli at the measuring temperatures.

Considering the influence of temperature on unbound materials, it has been found that this influence is trivial and negligible if the measuring temperatures are in the range of positive values. However, the influence of temperature on elastic modulus values of unbound materials can be significant for the range of negative measurement temperatures.

Von Becker [84] has pointed out that the elastic modulus value of unbound pavement material at 0 °C can increase up to 4 and 8 times if the temperature decreases to –5 °C and –10 °C respectively. This can be attributed to the effect of frost situation in such materials. By these reasons, surface temperature of higher than 5 °C has been recommended for deflection measurement in the "Information sheet of deflection measurement with Benkelman Beam" (in German: Merkblatt über Einsenkungsmessungen mit dem Benkelman-Balken) [5]. This recommendation has been considered in practical FWD testing as well.

2.1.2.2 Moisture content effect

The next influence factor on bearing capacity of pavement structure in Figure 2-2 is water content in unbound material layers. Even though the different moisture contents in unbound materials generally yield to the different Poisson's ratio parameter [72] which is one of the common input parameter for backcalculation process, it has been found that the effect of Poisson's ratio is relatively small for most values in backcalculation process. While it is important to make a good estimate of Poisson's ratio, the consequences of being incorrect are not very significant [46]. Typical values of the Poisson's ratio of conventional pavement materials given by AASHTO are illustrated in Table 2-1.

Table 2-1. Typical Poisson's ratios of pavement materials after AASHTO 1993[2]

Material	Range	Remarks	Typical Value
Portland Cement Concrete	0.10-0.20	-	0.15
Hot Mixed Asphalt/ Asphalt Treated Base	0.15-0.45	For temperatures < 30°C use 0.15; For temperature > 50 °C use 0.45.	0.35
Cement Stabilized Base	0.15-0.30	When sound free of cracks use 0.15; With crack use 0.30.	0.20
Granular Base/ Subbase	0.30-0.40	Crushed material use low value; Unprocessed rounded gravel/sand use high values	0.35
Subgrade Soils	0.30-0.50	For cohesionless soils use value near 0.30; For very plastic/cohesive clays use 0.50.	0.40

On the other hand, it is well known that the existence of much water in unbound material layers can lead to decreasing of bearing capacity. The worst situation could be observed in the thawing situation during and after the drastic winter season in many countries. Jessberger [50] has indicated that the overall bearing capacity can drop off up to 40% in thawing period if the materials used in unbound layers are frost susceptible.

Hence, the unbound materials which have a good freeze-thaw resistance should be used and the effective drainage system should be designed in freeze-thaw zoning to avoid this unfavorable circumstance.

It is obvious that temperatures and moisture contents in the pavement vary over time. Seasonal and diurnal changes in temperature will have a major effect on the modulus of an asphalt concrete layer. Changes in moisture content will affect the modulus of the upper subgrade and perhaps also that of the base course. A way to take the effect of long-term seasonal changes on pavement structures into account may be performed by applying Miner's hypothesis of cumulative fatigue damage [43] in the analysis.

2.1.2.3 Other effects

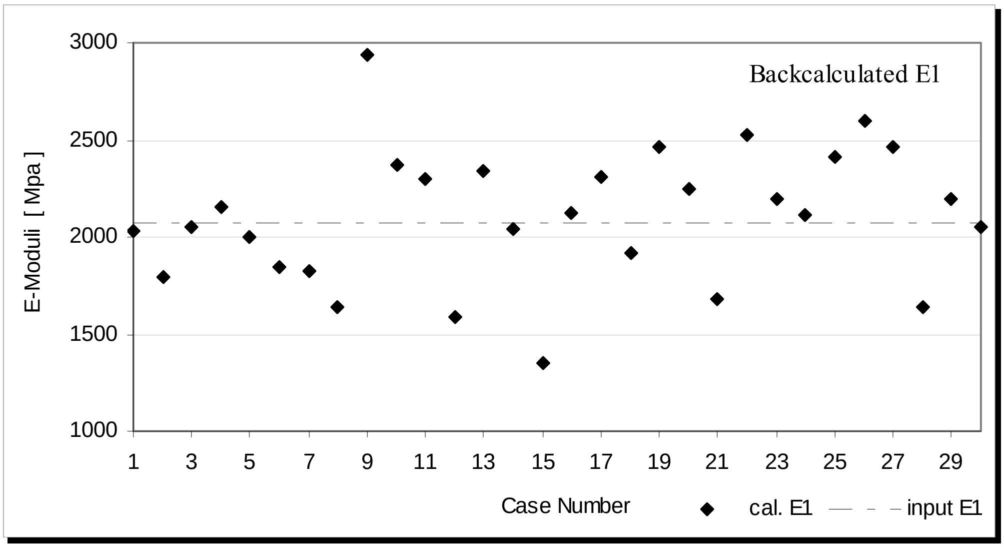
Considering other remaining influences in Figure 2-2 in the view of FWD test, it is found that only one influence, i.e. loading (magnitude and time), can be exactly obtained from FWD equipment. Since knowledge of the course of the pavement structure is indispensable for backcalculating process, supplement investigations must be carried out to get the required information of each layer (type, thickness, adhesion quality) at testing time. While determination of layer thickness has traditionally relied on either construction records or on destructive (coring) calibration, in recent years a nondestructive technique such as ground penetrating radar (GPR) has shown a great improvement in this area.

2.1.3 Accuracy of FWD deflection data

It is clear that accurate deflection readings must be obtained when using FWD device in order to obtain reliable backcalculation layer moduli. Most commercially manufactured FWDs are trying to offer a high level of accuracy in the deflection readings. The inevitable errors occur, however, in the practical testing. It should be noted that not only the error in deflection reading but also the inaccuracy of the sensor position can yield to a misleading pavement analysis [73]. Generally, the errors in FWD test originate from three main sources: seating errors, random errors and systematic errors [46].

Seating errors occur due to the rough texture and loose debris on pavement surfaces. Usually all that is necessary to eliminate these errors is to apply one or two drops at each new test point and discard data. Systematic errors, e.g. the error of sensor position, can be reduced, and possibly eliminated, through calibration. The last source of errors in FWD test is random errors, *e.g.* repeatability errors. It is comprehensible that such errors cannot be totally eliminated but they can be reduced by taking multiple readings and average the results. The error of the mean is reduced by the square root of the number of observations used in computing mean. For example, if four replicate drops are averaged, the random error would be reduced by half.

Some previous research works have shown the effect of these errors on backcalculation process. Irwin et al. [48] have used a normally distributed random number generator technique to generate 30 cases of deflection basins by adding or subtracting a random portion of the standard deviation to the theoretically calculated deflections of the simulated deflection basin (four-layer system). This might be thought of as 30 tests conducted at 30 different points on a perfectly homogeneous pavement having the constant values of materials properties and layer thicknesses. Using the backcalculation program MODCOM 2, the backcalculation has been performed and the results are shown in Figure 2-3. The results serve to illustrate how a seemingly inconsequential random measurement error with a standard deviation of only ±1.95 µm can have a major effect on the backcalculated layer moduli. The random deflection measurement error leaves the impression that there was substantial point-to-point variability of the moduli of the surface and base course layers, where in fact there should have been one.



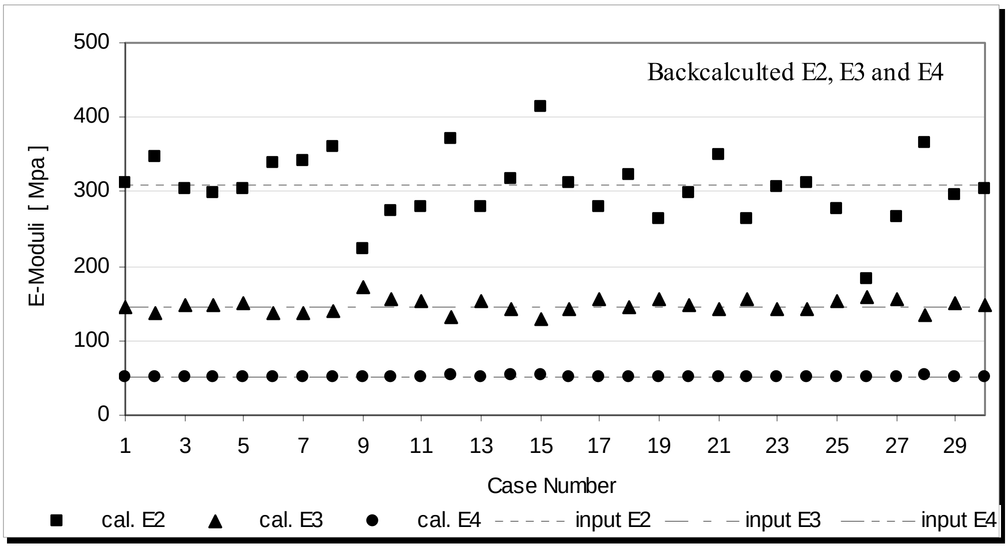


Figure 2-3. Effect of random deflection measurement error on backcalculated moduli-single drop per case (standard deviation ±1.95 µm, deflection tolerance 0.01%)

It is obvious in Figure 2-3 that the consequence of errors is most pronounced in the backcalculated modulus of surface course (*E1*). The moduli of the base (*E2*) and subbase (*E3*) layers are also affected, and the subgrade (*E4*) moduli are only slightly affected by these errors. Irwin et al. have also mentioned that to some degree this outcome may be due to the bottom-to-top approach (also works inward toward the center of the deflection basin) used by the backcalculation program MODCOM 2.

As mentioned earlier, even though most commercially manufactured falling weight deflectometers try to offer a high level of accuracy in deflection readings, they are specified to have an accuracy of ±2 percent ±2 microns. The ±2 percent represents the systematic errors whereas the ±2 microns represent the random errors. This means that whenever the deflections are larger than 100 microns, the systematic error may be larger than 2 microns. Since pavement deflection larger than 100 microns are quite common, it is important to calibrate the FWDs to reduce the effect of the systematic deflection errors on the backcalculated results. It has been reported that periodic calibration of the FWD can reduce such error to ±0.5 percent or better [73].

It can be concluded from this information that if a high degree of accuracy is desired from the backcalculation process, the deflections from a minimum of three replicate drops at one testing point should be averaged. Moreover, the backcalculation algorithms that take all the deflections of a whole basin into account are preferable.

2.2 General Process of Backcalculation Analysis

Most techniques of analysis of FWD deflection data obtained from flexible pavement fall into two categories. The first one can be called "direct" or "primary method" (e.g. Jendia-, Graetz-, KStB-method) in which the deflection parameters, e.g. D_0, curvature index, are used directly to evaluate pavement structural stability based on the empirical data regardless the mechanical parameter. The evident advantage of this method is its simplicity. On the other hand, the results from these methods cannot be well used in further pavement rehabilitation process since they neglect the mechanical parameters. Although some overlay design manners used deflection basin parameters have been developed such as in Virginia [83], Louisiana [54] and more. These empirical relationships are only valid for the environment and type of pavement structures for which they were developed [68].

The second category of analysis techniques is the "backcalculation" of layer moduli which is the main focus of this work. Unlike the direct method, backcalculation takes pavement mechanical parameters (thicknesses, Poisson's ratios, layer adhesion, etc.) into account. The goal of this analysis is also the mechanical parameters and mostly the elastic modulus. As described earlier, the elastic modulus of pavement materials is a fundamental property. This property is usually required as an input parameter in many pavement engineering calculations. This makes the elastic modulus parameter very important in evaluation of the performance of pavement structure especially for pavement management purposes.

2.2.1 Forward mechanistic models

To make backcalculation analysis possible, a forward mechanistic model for pavement structure is indispensable. This model is used in computing deflection basin of any input pavement model in order to compare with those from measurement. Some of the well-known forward models that have been used in backcalculation analysis are listed below:

- Boussinesq's equations and method of equivalent thickness (MET)
- Multi-layered Elastic Theory (MLET)
- Nonlinear Elastic Approach
- Finite Element Method (FEM)
- Discrete Element Method
- Visco-Elasticity Method
- Dynamic Analysis

2.2.1.1 Boussinesq's equation and method of equivalent thickness (MET)

As early as 1885 Boussinesq formulated a set of equations for calculating the stresses, strains, and deflections of a homogeneous, isotropic, linear elastic semi-infinite space under a point load. These equations are closed-form. The relationship between the surface deflection on the half-space, d_z, and elastic modulus, E, is described by the following equations.

For a uniformly distributed load on the surface at r = 0:

$$d_z = \frac{(1-\mu^2)*2P}{\pi * E * a} \tag{2-2}$$

For a point load on the surface:

$$d_z = \frac{(1-\mu^2)*P}{\pi * E * r} \tag{2-3}$$

where P = surface load, r = radial distance from center of load, a = radius of load area, and μ = Poisson's ratio.

It can be concluded that if the assumptions of Boussinesq are satisfied, then it is possible to calculate the elastic modulus for the half-space by measuring the surface deflection due to known load. Since the modulus is determined from a surface deflection, it is called "surface modulus". Using this surface modulus the simplest form of backcalculation can be performed.

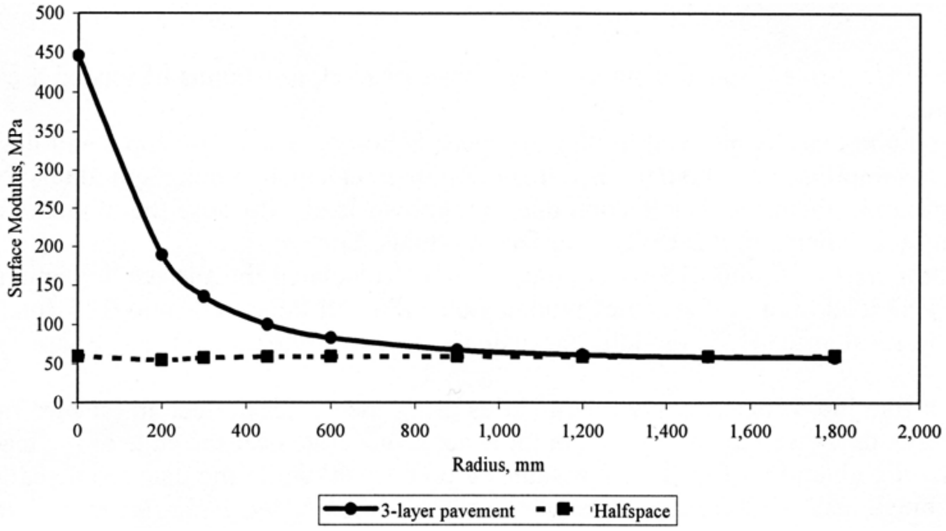


Figure 2-4. Surface modulus for a three- layer pavement and for a half-space [46]

Unfortunately, the concept of surface modulus is less straight forward when it is applied to a layered pavement system. Irwin [46] has calculated deflections from a typical flexible pavement using the multi-layered elastic program BISAR. This pavement consists of an asphaltic concrete surface layer (3600 MPa, 250 mm thick), granular base (200 MPa, 460 mm thick), and infinite-dept subgrade (60 MPa). All three layers had a Poisson's ratio of 0.35. The results are illustrated in Figure 2-4. It is evident that the half-space equations do not work very well at all for a multi-layered pavement. The surface modulus at the center underestimates the modulus of the surface layer, and overestimates the modulus of the subgrade. It is, however, interesting to notice that the surface modulus comes close to the subgrade modulus for the deflections at larger radii. This indicates that the outer deflections can be used to determine the moduli of the deep layers.

Another attempt to employ Boussinesq's equations in backcalculation process is combining these equations with method of equivalent thickness (MET) developed by Odemark. General speaking, the MET transforms a system with different moduli into a modulus system. The equivalent thickness of the transformed layer based on the original stiffness of the layer. However, the MET is an approximate method and stresses and strains obtained from this method must be corrected to improve the agreement with multi-layered elastic theory. The correction factor depends on the number of layers, layer thicknesses, Poisson's ratios, and modular ratios in the structure. Additionally, there are some limitations with respect to the use of the MET [78]. One is that the moduli should be decreasing with depth, preferably by a factor of at least two between consecutive layers. Another one is that the equivalent thickness of a layer should rather be larger than the radius of the loaded area. It is worthwhile to notice that the concept of combining Boussinesq's equations with MET neglect the interface behavior between any two consecutive layers in pavement structure. All these features make this combining concept less appealing to be used in backcalculation analysis.

2.2.1.2 Multi-layered Elastic Theory

In 1943 Burmister first presented a method for determining stresses and displacements in a two-layer system and then extended them to a three-layer system in 1945. With the advent of computers, the theory can be applied to a system with any number of layers as shown in Figure 2-5. This method was proposed as an effort to tackle the limitations of Boussinesq's equations. As discussed earlier, flexible pavements are layered system with better materials on top and cannot be represented by a homogeneous mass, so the use of Burmister multi-layered system is more appropriate than combining Bossinesq equations and MET.

The basic assumptions to be satisfied of the multi-layered elastic theory are [45]:

1. Each layer is homogeneous, isotropic, and linearly elastic with an elastic modulus E and a Poisson's ratio μ.
2. The material is weightless and infinite in areal extent.
3. Each layer has a finite thickness h, except that the lowest layer is infinite in thickness.
4. A uniform pressure q is applied on the surface over a circular area of radius a.
5. Continuity conditions are satisfied at the layer interfaces, as indicated by the same vertical stress, shear stress, vertical displacement, and radial displacement. For frictionless interface, the continuity of shear stress and radial displacement is replaced by zero shear stress at each side of the interface.

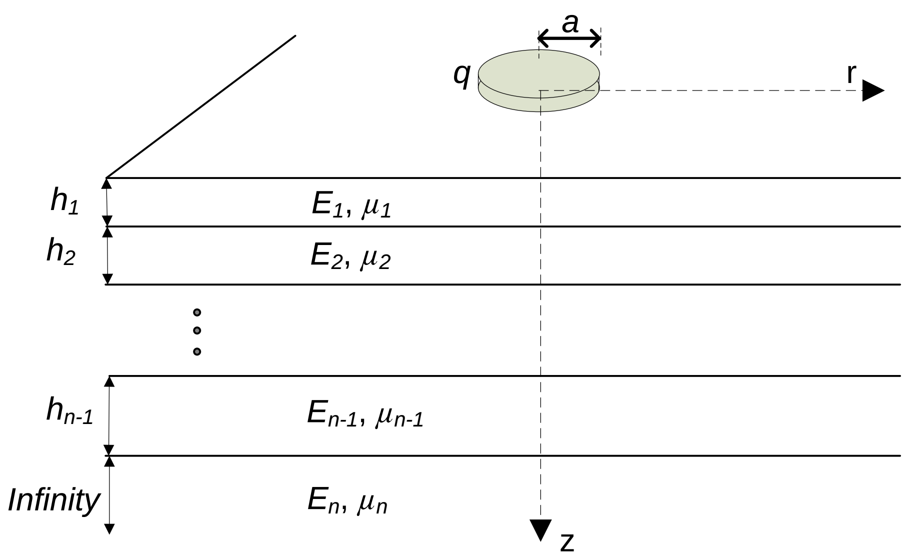


Figure 2-5. Scheme of elastic multilayer system

Based on this multi-layered elastic theory many automated computer programs have been developed for calculating stresses, strains, and deflections of layered elastic systems. One of the most widely used programs is BISAR developed by Shell [21].

2.2.1.3 Other more complex approaches

It is reasonable to believe that more reliable backcalculated results can possibly be obtained from the use of other more complex approaches as forward model: nonlinear elastic approach, visco-elasticity method, finite element method, discrete method, visco-elastic method, dynamic analysis and others. Unfortunately, these approaches involve a large number of input parameters, some of which are even not readily known for pavement materials [46]. Hence their use has mainly been limited to the research community and not practical for daily use. Not surprisingly, there appears to be no general use of backcalculation programs based on these complex methods by practitioners.

It is highly likely that the backcalculation procedure based on multi-layered elastic theory adding with some techniques in which the nonlinearity properties of unbound materials can be taken into account in the analysis process is the most appropriate procedure for practical backcalculation problem. This procedure can lead to the reliable solutions and still very practical for routine use. From this conclusion the techniques to be enhanced in this work are based mainly on the multi-layered elastic theory.

2.2.2 Difficulties of Inverse Algorithms

In recent years, a large number of computer programs for doing automated backcalculation have been developed based on different algorithms. These backcalculation programs can be divided based on their inverse approaches into 3 types as the following [2].

1. The equivalent thickness method (e.g. ELMOD, BOUSDEF)
2. The iterative method (e.g. MODCOMP, EVERCALC, VAHREN1)
3. The optimization method (e.g. MODULUS, WESDEF)

As discussed in section 2.2.1 that the concept of equivalent thickness leads to many doubtful conclusions.

Considering the backcalculation programs based on iterative method, most of such programs have in common that they attempt to match the measured deflection profile as well as possible. Iterative process is required, where an initial set of layer moduli (seed) are assumed or supplied by the user, the seed moduli are then used to compute the surface deflections, and these are compared to the measured deflections. Based on the differences, the seed moduli are adjusted, and the process is repeated until the calculated deflections match the measured deflections within some specified tolerance. Unfortunately, it has been found that, in many iterative backcalculation programs, different seed moduli often lead to different backcalculated results which, in turn, lead to different pavement designs and evaluations [29].

For research purposes a series of iterative backcalculation programs (DREIFFM, VAHREN1, etc.) has been developed in Pavement Engineering Section, Leibniz

University of Hannover. They rely on multi-layered elastic theory by employing the elastic layer computer program BISAR [21] as forward mechanistic model. Before any analysis in these programs, a number of set of sensors is predefined in permutation series. The number of sensors in each set is equal to the number of unknown layer moduli in the backcalculation problem. Like other iterative backcalculation programs, the iterative process associated with seed moduli is performed as described above till the calculated deflections match the measured deflections within the specified tolerance.

By doing this, one can investigate the influence of deflections from various set of sensors on the backcalculated moduli. Considering only the backcalculated moduli obtained from sets of sensors which success in convergence, a significant problem evidently arises. Even though the deflections differences between the calculated and measured are less than the predefined tolerance (convergence), the backcalculated moduli of each pavement layer obtained from different set of sensors have shown a large variation. Many set of these moduli results are considered to be impossible from an engineering point of view. Additionally, there are no specific trend exists between the moduli values and sets of sensors. This fact indicates that the uniqueness solution of such problems cannot be guaranteed. In other words, the backcalculation problem has a high degree of multimodality.

However, some iterative backcalculation programs (e.g. MODCOM [48]) use the number of deflection values equal to the number of unknown layers to calculate deflection basin in theirs iteration. Although the appropriate backcalculated layer moduli could be obtained, those moduli may be considered as too sensitive to measurement random errors because each of them is derived from only certain deflection data points, not a whole basin. It is clear that when the number of sensors equals that of unknowns, a perfect fit will be achieved in the collocation sense. However, possible high random errors are included in the backcalculated moduli. In other words, unless random errors are eliminated, a perfect fit of the deflection bowl may induce errors in the backcalculated moduli. Since random errors from each sensor are unavoidable as discussed in section 2.1.3, the number of measuring deflection sensors should be greater than the number of unknowns to be backcalculated. By these reasons, it would be more appropriate to have a redundant system of equations (more equations than unknowns) and solve the system by using the optimization method to minimize the error [82].

Moreover, most of backcalculation programs are restricted to only three- or four-layer system, so the user need engineering judgment to combine judiciously any adjacent layers of similar stiffness to come up with a total of the required layers including the infinite subgrade. This makes increasing the layer number of input model, in some cases, to get more representative variation of the moduli with depth troublesome. Hence, the alternative computational algorithms which analyzes the entire set of deflection basin and can be applied to pavement model with more layers should be developed to improve the accuracy and reliability of the backcalculated results.

The Backcalculation process based on the optimization method has shown capability in that case. Usually, the ultimate goal of conventional backcalculation algorithms based on optimization method is to minimize the function of root mean square error (RMSE) which can be used to quantify the goodness of fit of the entire set of deflections. The function of RMSE is defined as:

$$\text{RMSE} = \sqrt{\frac{1}{N} * \sum_{i=1}^{N} \left[\frac{d_i - D_i}{D_i} \right]^2} \tag{2-4}$$

where N is the number of geophones, D_i is the measured surface deflections at geophones i, and d_i is the calculated surface deflections at geophones i.

Various optimization methods are used to determine the minimum value of RMSE, such as the Gauss-Newton method, Kalman filter, Bayesian method [51] and Hooke-Jeeves's pattern search algorithm [82]. It has been found that, the solution obtained from backcalculation programs based on optimization method may not be appropriate from an engineering point of view, particularly, when inappropriate seed moduli are used. One possible reason may be due to the optimum method used in those programs.

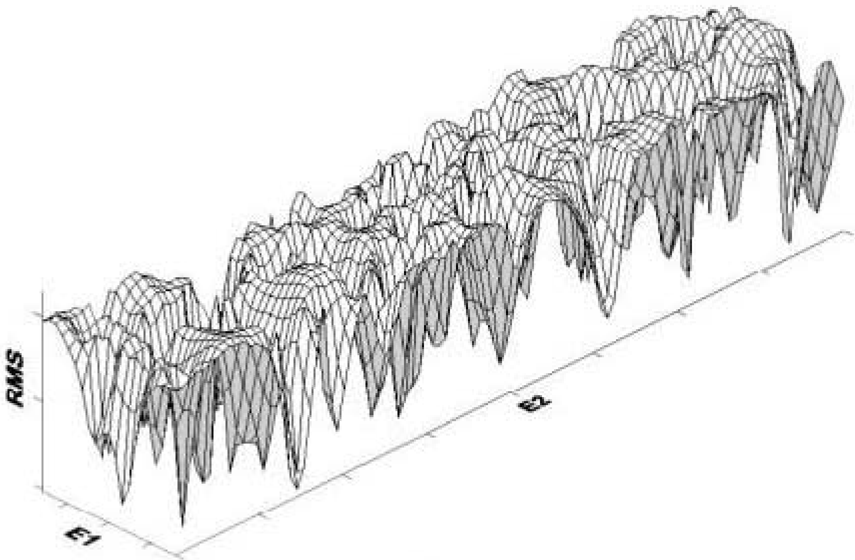


Figure 2-6. Solution surface of a two-layer system [2]

Many researchers [[57], [82], [51], [28] and [2]] have reported that in inverse problems such as those in backcalculation analysis, there are many local minimum values of the evaluation function (e.g. RMSE) including the global minimum. This kind of problem usually called "multimodal problem". The solution surface of a two-layer flexible pavement system shown in Figure 2-6 demonstrates the high degree of multimodality of the backcalculation problem. Therefore, when executing backcalculation by one of the above mentioned optimization methods with inappropriate seed moduli, it is possible that the obtained solution may only be a local minimum not the global one. One might be concluded that the algorithms used in backcalculation based on optimization method which do not have a good potential in global search ability can have some difficulties in determining the result because of the local minima problem.

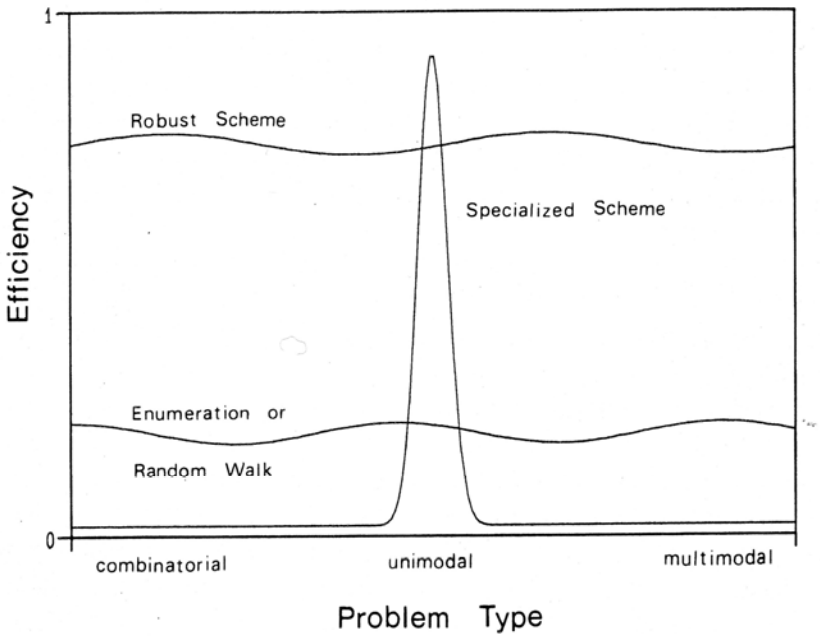


Figure 2-7. Efficiency of various schemes versus spectrum of problems [30]

Unlike the above mentioned optimization method, genetic algorithms (GAs) have proved themselves capable of solving many large complex multimodal problems in many scientific and engineering fields. The most prominent nature of GAs may be that they have a very good potential in global search ability which is desired in solving of multimodal problems. To get more perspective, inspect the problem spectrum of Figure 2-7. It is obvious that many specialized schemes, e.g. traditional calculus-based method, work very well in a narrow problem domain but it becomes highly inefficiently (if useful at all) elsewhere. On the other hand, enumerative schemes and random walks work equally inefficiently across a broad spectrum. Only idealized robust scheme work well in across a broad spectrum of problems.

GAs have been known as a robust optimization procedure. Moreover, many of new GA operators and techniques have been developed recently. These features make GAs appealing to be used in enhancement of backcalculation analysis. A principal concept of GAs, many interesting new GA operators and techniques, and the existing backcalculation programs based on GAs are discussed in chapter 3.

3 Genetic Algorithms in Backcalculation Analysis

3.1 Introduction to Genetic Algorithms

Genetic Algorithms (GAs) are search algorithms based on both mechanics of natural selection and natural genetics. In other words, GAs are inspired by biological evolution and are based on the Darwin's principle: "survival of the fittest". These algorithms are a family of adaptive techniques, developed by John Holland, his colleagues, and his students at the University of Michigan [39]. A GA comprises a set of individual elements (the population) and a set of biologically inspired operators defined over the population itself. According to evolutionary theories, only the most suited elements in a population are likely to survive and generate offspring, thus transmitting their biological heredity to new generations.

Because GAs are rooted in both natural genetics and computer science, the terminology used in the GA literature is a mix of the natural and the artificial. Hence the correspondence between both of these terms should be reviewed. Some significant terminologies in this field are summarized in Table 3-1.

Table 3-1. Comparison o f Natural and GA Terminology [30]

Natural	Genetic Algorithms
chromosome	string
gene	feature, character, or detector
allele	feature value
locus	string position
genotype	structure
phenotype	parameter set, alternative solution, a decoded structure
epistasis	nonlinearity

Goldberg [30] has admirably explained the relation of these terminologies as following. The strings of artificial genetic systems are equivalent to chromosomes in biological world. In natural systems, one or more chromosomes combine to form the total genetic prescription for the construction and operation of some organism. In natural terminology the chromosomes are described to be composed of genes, which may take on some number of values called alleles. In genetics, the position of genes (its locus) is identified separately from the gene's function. In artificial genetic search the strings are described to be composed of features or detectors, which take on different values. Features may be located at different positions on the string. In natural system the total genetic package of strings is called the genotype. In artificial genetic systems the total package of string is called a structure. In natural system, the organism formed by the interaction of the total genetic package with its environment is called the phenotype. In artificial genetic systems, the structures decode to form a particular parameter set, solution alternative, or point (in the solution space).

3.2 Basic Mechanics of GA-Process

In computing terms, simple genetic algorithm (SGA) consists of the following key operations: creation of a population, evaluation, selection, crossover, and mutation. The process shown in Figure 3-1 is the representation of evolutionary theory.

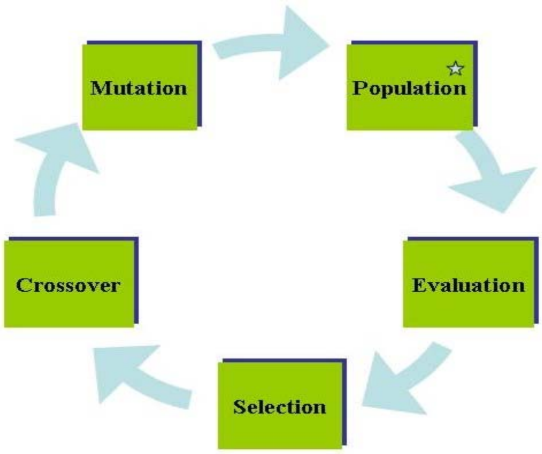


Figure 3-1. Genetic algorithm cycle

To begin with, an initial population of size n is created from a random selection of the parameters in the parameter spaces. Each parameter set represents the individual's chromosomes. Each of the individuals is assigned a fitness value based on how well each individual's chromosomes allow it to perform in its environment. There are then three operations that occur in GAs to create the next generation of population: 1) selection, 2) crossover, and 3) mutation. Fitter individuals are likely to be selected for mating, whereas weaker individuals die off. Mated parents create children with chromosome sets that are some mix of the parent's chromosomes. Then there is a small probability that one or more of child's chromosome will be mutated. The process of mating and child creation is continued until an entirely new population of size n is generated with the hope that strong parents will create a fitter generation of children; in practice, the average fitness of the population tends to increase with each new generation. The process of selection/ crossover/ mutation is repeated. Successive generations are created until very fit individuals are obtained [11]. It is very important for GA users to understand how these basic GA operators work in the computing process. Each of these GA-internal processes should be discussed more in detail in the view of backcalculating work.

3.2.1 Creation, en- and decoding of population

As mentioned earlier, the conventional GA begins with an initial population of size *n*. This population is created from a random selection of the parameters in the predefined parameter spaces (possible solutions). Each set of these solutions (individual), e.g. a set of elastic layer moduli in case of backcalculation problem, is then evaluated based on its performance under the defined environment (fitness function). The possible fitness functions which can be used in backcalculation problem are discussed in the next section. Afterwards the parameters in each set are encoded into strings (chromosome). Generally, there are two parameter coding schemes used in GA world: binary coding and floating point (continuous) coding:

1) In floating point coding, the parameter is descritized into a number of possibilities, and there is one chromosome for each parameter. The value of the chromosome is stored as a floating point number.

2) In binary coding, the parameter is also descritized into a number of possibilities, but the chromosome length is based on the total number of possibilities in a binary format. For problems where many parameters are involved such as backcalculation problem of layer moduli, the concatenated, multiparameter, mapped, fixed-point coding can be well employed [30]. Using this coding scheme the developing of backcalculation program for analyzing a pavement model with any number of layer systems is possible. Considering a backcalculation problem of three-layer system in which the set of correct elastic moduli for a three-layer pavement system is as following: E_1-surface course = 3072 MPa, E_2-base course = 192 MPa, and E_3-subgrade = 48 MPa. If the possible solution range of each layer moduli lies between zero and any maximum values ($0 < E_{i,correct} < E_{i,max}$). A representative binary string (chromosome) can be demonstrated in Figure 3-2. The same concept, but inverse, will be used in decoding process in order to transfer the chromosome strings back to the desired parameters.

$$\text{chromosome } i = \underbrace{110000000000}_{\substack{\text{gene}_1 \\ (E_1=3072)}}\ \underbrace{\mathbf{0011000000}}_{\substack{\mathbf{gene_2} \\ (\mathbf{E_2=192})}}\ \underbrace{\mathit{000110000}}_{\substack{gene_3 \\ (E_3=48)}}\ \ldots\ \underbrace{*********}_{\substack{gene_n \\ (E_n)}}$$

Figure 3-2. Variables encoding and decoding in binary coding system

The issue of which coding scheme can performs better for GA mechanic process is yet controversial in GA world. However, if the quantization limitation has been taken into account, one can conclude that floating-point coding is appropriate for solving a problem where the values of the variables are continuous and the full machine precision is desired. In such a problem each variable is large, and so is the size of the chromosome. It is therefore more reasonable to represent them by floating-point number [37]. It is evident that the optimization of backcalculation problem is not the case described. Both of these coding concepts are therefore applicable for backcalculating work.

3.2.2 Evaluation of the fitness value

To evaluate the fitness value of the individuals in population, the fitness function is, of course, indispensable. It has been found that the fitness function has in some degree influence on the GA-performance. Selection of the fitness function in optimization problem should be, therefore, performed carefully. The following functions are some examples of fitness function reported in literature that can be used to explore the solution surface of backcalculation problem.

Case1: Maximize
$$f = \frac{1}{1+OBJ} \tag{3-1}$$

Case2: Maximize
$$f = \frac{1}{100(1-OBJ)+1} \tag{3-2}$$

Case3: Maximize
$$f = \frac{1}{1000(1-OBJ)+1} \tag{3-3}$$

Case4: Maximize
$$f = \frac{1}{0.1(1-OBJ)+1} \tag{3-4}$$

Case5: Maximize
$$f = \frac{1}{0.01(1-OBJ)+1} \tag{3-5}$$

Case6: Maximize
$$f = \frac{1}{0.001(1-OBJ)+1} \tag{3-6}$$

Case7: Minimize
$$f = \sqrt{\frac{OBJ}{N}} \tag{3-7}$$

Case8: Minimize
$$f = r^2 = 1 - \frac{OBJ}{\sum_{i=1}^{N}(D_i - \bar{D})^2} \tag{3-8}$$

Case9: Minimize
$$f = \sum_{i=1}^{N}\left|d_i - D_i\right| \tag{3-9}$$

Case10: Minimize
$$f = \sum_{i=1}^{N} (d_{\max} - D_i)^2 \tag{3-10}$$

Case11: Minimize
$$f = \sum_{i=1}^{N} (d_i - D_{\max})^2 \tag{3-11}$$

Case12: Minimize
$$f = \frac{1}{2D_0}\left[D_0 r_1 + \left(\sum_{i=1}^{N-1} D_i (r_{i+1} - r_i) \right) + D_N (r_N - r_{N-1}) \right] \tag{3-12}$$

Case13: Minimize
$$f = RMSE = \sqrt{\frac{1}{N} \cdot \sum_{i=1}^{N} \left(\frac{d_i - D_i}{D_i} \right)^2} \tag{3-13}$$

Case14: Minimize
$$f = \sqrt{w_i \cdot (d_{i\_}D_i)^2} \tag{3-14}$$

Case15: Maximize
$$f = \frac{1}{1 + \left(\frac{OBJ}{10^6} \right)} \tag{3-15}$$

where *OBJ* is the objective function and defined as:

$$OBJ = \sum_{i=1}^{N} (d_i - D_i)^2 \tag{3-16}$$

N is the number of sensors, d_i is the backcalculated deflection at point *i*, D_i and $\bar{D}$ are the measured deflections at sensors *i* and the average deflection of all sensors, respectively. d_{max} and D_{max} are the maximum calculated and measured deflections of all sensors respectively. The variable r_i is the distance between the center of the loading plate and sensor *i*, and w_i is the weight factor on the measurement points.

Among different types (mini-, maximization) and forms of fitness functions shown above, the function which found to be the most sensitive for backcalculation problem should be chosen. Case 1 is the simplest form of using objective function. Cases 2 through 6 represent obviously nonlinear fitness amplifications that have been used to increase the sensitivity of the selection process to any variation in gradient of the fitness surface in cases where the surface is flat. Case 7 and 8 use the correlation coefficient as the objective function. Case 9 through 11 are different forms based on the least error. Case 12 uses the area parameter as the objective function [2]. Case 13 is the simplest form of the root mean square error function used by most of backcalculation programs based on optimization method. Case 14 and 15 are other forms of capable fitness function, once again, based on objective function.

Alkasawneh [2] has investigated the effect of the fitness function cases 1 through 12 on the backcalculated moduli of a three-layer system and concluded that the fitness functions are, as expected, not equal in their effect even if the least square error of modified form of it has been used in almost case (except case 12). The result is illustrated in Figure 3-3. He pointed out that while almost all cases showed some stochastic behavior of error along with increase of generation, case 12 has shown the best performance through a quick convergence with a constant behavior.

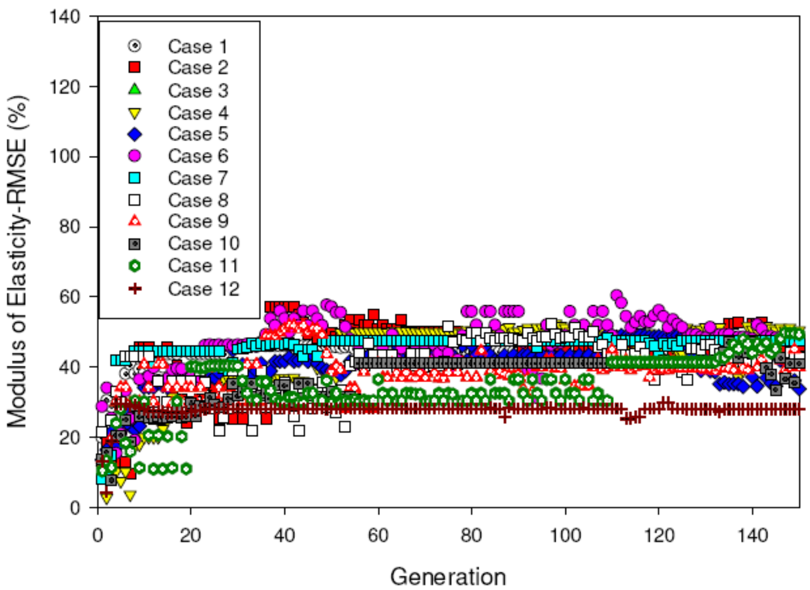


Figure 3-3. Generation's RMSE of the backcalculated moduli based on various fitness function [2]

Even though Alkasawneh [2] has recommended the fitness function in case 12 over the other tested functions for using in backcalculating work, the fitness function in case 1 and case 13 have been used in his later investigations instead.

3.2.3 Selection process

The basic idea of selection is that it should be related to fitness value. The most common scheme for its implementation is known as the roulette-wheel method. Roulette-wheel selection uses a probability distribution in which the selection probability of a given string is directly proportional to its fitness. In computing term, the fitness of all the individuals in the population is summed, and then the expected probability of being selected of each individual is equal to the fitness of individual divided by the total fitness of the population as defined in the following equation.

$$p_{s,k} = \frac{f_k}{\sum_{i=1}^{n} f_i} \tag{3-17}$$

where $p_{s,\,i}$ is the probability of selection of individual i, f_i is fitness value of individual i, and n is number of all individual (size of population).

Random number is then used to select an individual in the population. The eq(3-17) implies that the individuals with higher fitness values are more likely to be selected into the reproduction process than those with lower fitness. The expected number of parents with chromosome set i is simply $n*p_i$. This procedure will fill most of the parent's slots, but there will be a fractional remainder of slots that are filled using the stochastic remainder sampling without replacement method [30]. The selected individuals are gathered in mating pool preparing for parenthood. Random pairs of mates are then chosen from this population of fit parents. Then each pair of mates undergoes crossing over process in order to create offspring.

3.2.4 Crossover operator

Crossover is simply a matter of replacing some of the alleles in one parent by alleles of the corresponding genes of the other. Suppose that we have 2 strings (individuals), A and B, as shown in Figure 3-4. Each consists of 10 variables which represent a possible solution to the problem. In case of binary coding scheme each of these variables can contain any values of 0 and 1. To implement the simplest form of crossover: single-point crossover, a crossover point is chosen at random from the integer numbers 1 to 9. The offspring (new solutions) are produced by combining pieces of the original parents. For instance, assuming that the crossover point is 2, and then the offspring solutions can be demonstrated in Figure 3-4.

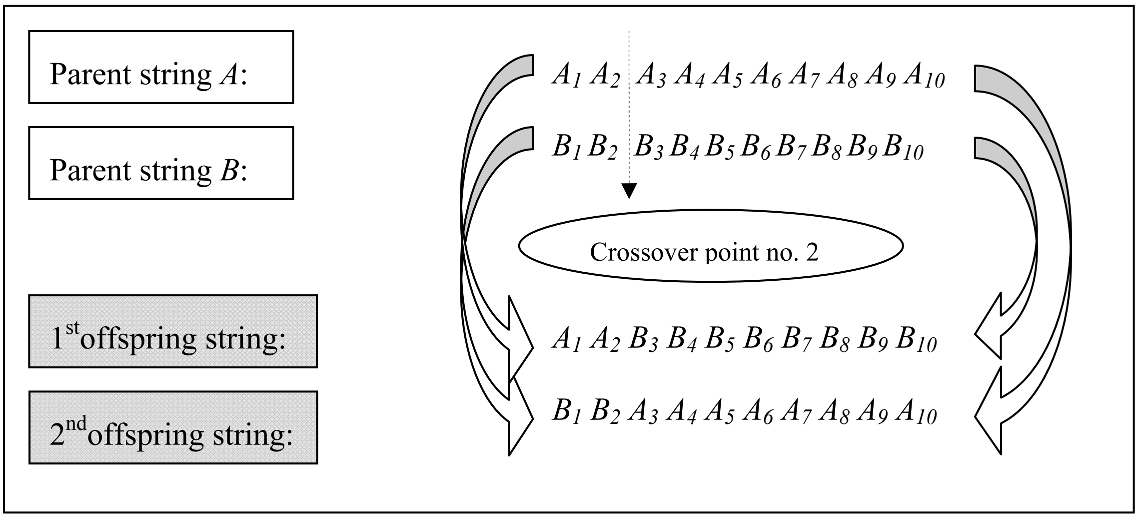


Figure 3-4. Crossover process based on simple single-point crossover method

It can be concluded that the action of crossover with previous reproduction in GAs speculates on new information from the survived strings (mostly high fitness string). These processes are continued until an entirely new population of size n is generated. In other words, a whole new population of possible solutions is produced by selecting the fitter individuals from the current generation, and mating them to produce new sets of chromosomes. This new generation contains a higher proportion of the characteristics possessed by the good member of the previous generation. In this manner, over many generations, good characteristics are spread throughout the population. By favoring the mating of the fitter individuals, the most promising areas of the search space are explored. Keeping this way, one can hope that the population will converge to the optimal solution of the problem.

3.2.5 Mutation operator

Even though reproduction and crossover of good strings effectively search and recombine extant information, mutation is usually needed in the GA process. Leave reproduction and crossover alone in the process, they may become overzealous and lose some potentially useful genetic material (1s or 0s at particular locations in case of binary coding scheme). This can lead to a premature convergence which is undesirable in solving optimization problem. In artificial genetic systems, the mutation operator protects against such irrecoverable loss. Hence, there is a small probability in GA-process that one or more of the offspring's chromosome will be mutated.

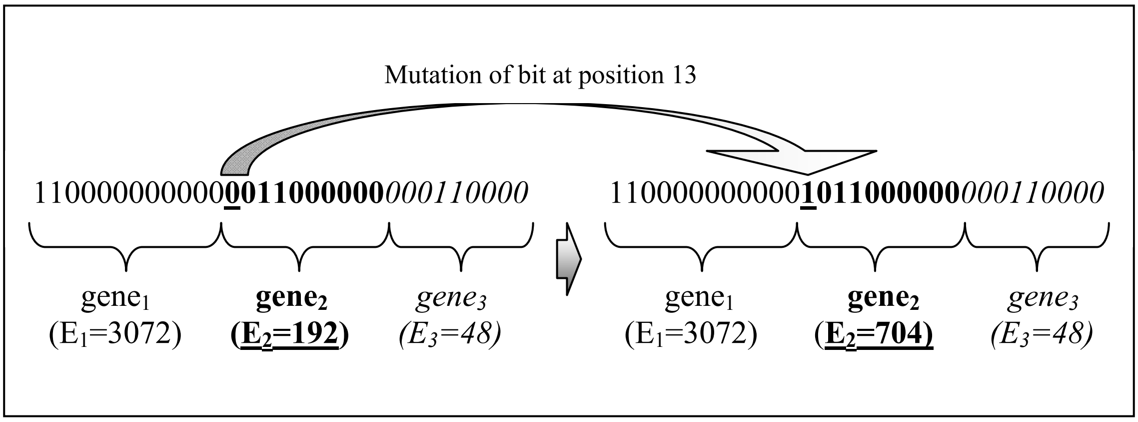


Figure 3-5. Demonstration of mutation process in binary coding scheme

The mechanic concept of the mutation is simpler than crossover. A gene (or a subset of gene) is chosen randomly and the allele value of the chosen genes is changed. In the binary coding scheme, this simply means changing a 1 to a 0 and vice versa. Recall the chromosome *i* of a three-layer pavement problem shown in Figure 3-2 to demonstrate this mutation process. Assuming that this chromosome must be mutated at the bit number 13 which locates in gene no.2 (E_2), Figure 3-5 shows the result of this mutation. As can be seen, the mutation produced a new chromosome that was randomly picked to be in the range of appropriate parameter. As a result, the parameter jumped from one side of the range to another side (e.g. from 192 to 704 MPa). By this reason, this simple mutation method is known as "jump mutation". Obviously, the mutation process protects GAs from premature convergence and makes GAs capable to search for the global maximum (or minimum). On the other hand, the mutation itself is a random walk through the string space. Therefore, it is always recommended that the mutation probability in GAs should be kept at low rate.

3.3 Principle Theorem of GAs

The first contribution to principle theorem of GAs is credited to Holland, who published the book "*Adaptation in Natural and Artificial Systems*" [39] in 1975. The Schema Theorem in Holland's book has always been used to describe the mechanics of GAs in sense of mathematic. Understanding this principle, one can answer the question that why the very simple mechanics of GA-operators can be powerful in solving many complex problems. Some interesting points of this theorem in the view of backcalculation problem based on binary coding scheme are discussed in this section. Most of the following content about the schema theorem is summarized from one of the most excellent GA text book written by Goldberg [30].

The schema theorem can be used to describe why the important similarities among highly fit strings can help guide a search. A schema can be explained as a similarity template describing a subset of strings with similarities at certain string positions. By appending a special symbol, *, (called "*do not care*" or "*wild card*" symbol), to the binary alphabet (0 and1), one can create strings (schemata) over the ternary alphabet (0, 1, *). This schema can be thought of as a pattern matching device. For instance, a schema H is defined as follow:

$$\text{Schema } H \quad = \quad *\,01*\,0$$

This schema H matches any of the four strings of length 5 that has a 0, 1, and 0 in the second, third and fifth position, respectively. In other words, this schema describes a subset with these four members {00100, 00110, 10100, and 10110}. As a result, there are 3^l schemata or similarities defined over a binary string of length l.

In the schema theorem, two important schema properties are "order" (o) and "defining length" (δ). These properties of a schema S_i denoted by:

$$o(S_i) = \text{the number of fixed positions present in the template}$$

$$\delta(S_i) = \text{distance between the first and the last specific string position}$$

Recall the schema H mentioned above, the order and the defining length of this schema are $o(H) = 3$ and $\delta(H) = 5 - 2 = 3$, respectively. Schema and their properties provide the basic means for analyzing the effect of reproduction and genetic operators. The effect of reproduction on a particular schema is easy to determine. Since more highly fit strings have higher probabilities of selection, on average we give an ever increasing number of samples to the observed best similarity patterns. However, reproduction alone samples no new points in the solution space. On the other hand, crossover leaves a schema unscathed if it does not cut the schema, but it may disrupt a schema when it does. As a result, schemata of short defining length are left alone by crossover and reproduced at a good sampling rate by reproduction operator. As mentioned earlier, the mutation possibility in GAs is usually kept at low rate, the possibility of any particular schema to be destroyed by mutation is, therefore, small. The effect of each operator in simple genetic algorithm in mathematical sense is discussed below using the schema theorem.

3.3.1 Effect of reproduction

In mathematical terms, the effect of reproduction on the expected number of schemata can be investigated by supposing that at a given time step t there are m examples of a particular schema S contained within the population $A(t)$ or written as $m = m(S, t)$. During reproduction, a string is copied according to its fitness. After picking a nonoverlapping population of size n with replacement from the population $A(t)$, it can be expected to have $m = m(S, t+1)$ representatives of the schema S in the population at time $t+1$ as given by the following equation:

$$m(S,t+1) = m(S,t) \cdot n \cdot \frac{f(S)}{\sum f_i} \tag{3-18}$$

or rewritten as:

$$m(S,t+1) = m(S,t) \cdot \frac{f(S)}{\bar{f}} \tag{3-19}$$

where $f(S)$ = the average fitness of the strings representing schema S at time t

$\bar{f}$ = average fitness of the entire population

It can be concluded from the eq(3-19) that schemata with fitness values above the population average will receive an increasing number of samples in the next generation, while schemata with fitness values below the population average will receive a decreasing number of samples. Further assume that a particular schema S remains above average an amount $c \cdot \bar{f}$ with c a constant. Under this assumption the schema difference equation can be rewritten as follows:

$$m(S,t+1) = m(S,t) \cdot \frac{\left(\bar{f} + c\bar{f}\right)}{\bar{f}} = (1+c) \cdot m(S,t) \tag{3-20}$$

Starting at $t = 0$ and assuming a stationary value of c, the following equation can be written.

$$m(S,t) = m(S,0) \cdot (1+c)^t \tag{3-21}$$

The effect of reproduction can be clearly seen from the eq(3-21). Reproduction allocates exponentially increasing (decreasing) numbers of trials to above- (below-) average schemata.

3.3.2 Effect of crossover

Unfortunately, reproduction alone does nothing to promote exploration of new regions of the solution space, since no new points are searched. To investigate the effect of crossover on schemata, consider the string of chromosome i represented a set of layer moduli of a three-layer pavement system in Figure 3-2. Additionally, two representative schemata within this string, S_1 and S_2, are also illustrated in Figure 3-6.

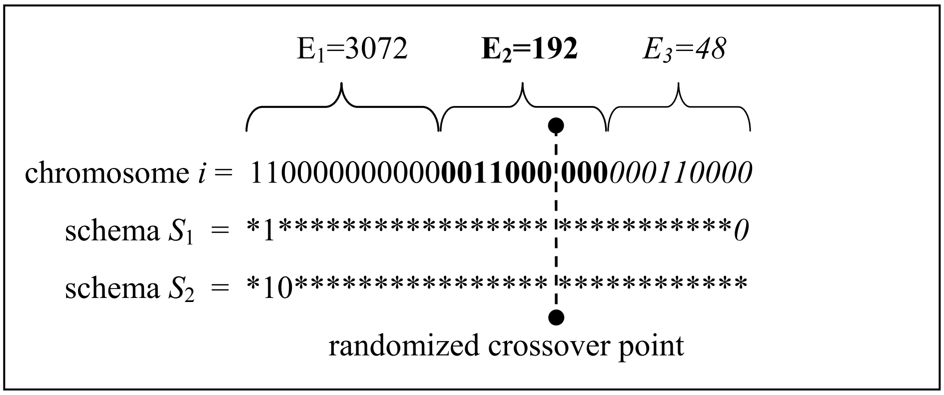


Figure 3-6. Crossover site on string and its schemata

It can be seen that the string and its schemata have length of string l = 31. Supposing that among the l-1 possible crossing sites the random number generator has selected the location between bit position 19 and 20 as the crossing site shown in the Figure 3-6. The effect of this cross cut on the two schemata S_1 and S_2 can be evidently seen from the figure. The schema S_1 will be destroyed because the 1 at position 2 and the 0 at position 31 will be placed in different offspring. Unlike the first schema, schema S_2 will survive because the 1 at position 2 and 0 at position 3 will be carried intact to a single offspring. In computing terms, the possibility of schema S_i to be destroyed by single-crossover is:

$$p_d = \delta(S_i) / (l\text{-}1) \tag{3-22}$$

Noting that schema S_1 has defining length $\delta(S_1)$ = 29, so the schema S_1 has the possibility to be destroyed with probability $p_d = \delta(S_1) / (l\text{-}1) = 0.967$. On the other hand, since $\delta(S_2) = 1$ the possibility of schema S_2 to be destroyed by single crossover is $p_d = \delta(S_2) / (l\text{-}1) = 0.033$ only. Put another way, if crossover is itself performed by random choice with probability p_c, the survival probability, p_s, of a schema S can be written as follow:

$$p_s \geq 1 - p_c \cdot \frac{\delta(S)}{l-1} \tag{3-23}$$

The combined effect of reproduction and crossover can be obtained by multiplying the expected number of schemata for reproduction alone by the survival probability under crossover, p_s, as shown in the following expression.

$$m(S,t+1) \geq m(S,t) \cdot \frac{f(S)}{\bar{f}} \cdot \left[1 - p_c \cdot \frac{\delta(S)}{l-1}\right] \tag{3-24}$$

It can be clearly seen in eq(3-24) that whether schema S grows or decays depending upon the fitness value and defining length. In other words, schemata with both above-average fitness and short defining lengths are going to be sampled at exponentially increasing rate.

3.3.3 Effect of jump mutation

In binary coding scheme, jump mutation is the random alteration of a single position with probability p_m. It is clear that a schema S survives when each of $o(S)$ fixed positions within the schema survives. Since a single allele survives with probability ($1-p_m$), the probability of surviving under mutation is:

$$p_s = (1 - p_m)^{o(S)} \tag{3-25}$$

Since the p_m value is usually very small ($p_m \ll 1$), eq(3-25) can be approximated by the following expression:

$$p_s = 1 - (o(S) \cdot p_m) \tag{3-26}$$

Combining the effect of reproduction, crossover and jump mutation on the schema S, it can be expressed as the following equation:

$$m(S,t+1) \geq m(S,t) \cdot \frac{f(S)}{\bar{f}} \left[1 - p_c \frac{\delta(S)}{l-1} - o(S) p_m \right] \tag{3-27}$$

Consideration of the eq(3-27) shows that short, low-order schemata will increase their representation in the next population provided their fitness ratio is slightly more than 1, while long and/or high-order schemata need much higher ratios. In other words, short, low-order, above-average schemata receive exponentially increasing trials in subsequent generations. This conclusion is what being called “the Schema Theorem”. The ideal situations for a GA would therefore seem to be those where short, low-order schemata or “building blocks” are able to combine with each other to form better solutions.

3.4 Existing Backcalculation Programs Based on GAs

As discussed earlier, the main problem the automated backcalculation programs based on traditional optimization methods face is premature convergence (local minima) which leads to inaccuracy, sometimes unreasonable, solutions. This can be attributed to the optimization method itself or the seed moduli needed as input values in most of such programs. Unlike the traditional programs, backcalculation based on GAs does not require the input of seed moduli, but only the lower and upper domain bounds of the layer moduli, i.e. the range of moduli considered appropriate for an engineering judgment. It is clear that the wider the range is defined, the larger the searching space becomes and the longer the convergence probably takes. On the contrary, if a small range is used, the solution could be restricted at any local minima. Table 3-2 presents typical values of the modulus of pavement materials given by AASHTO [1].

Table 3-2. Typical values of modulus of elasticity for pavement materials (after AASHTO, 1993)

Pavement Material	Range [MPa]	Typical Value [MPa]
Hot-mix asphalt	1,500 – 3,500	3,000
Portland cement concrete	20,000 – 50,000	30,000
Asphalt-treated base	500 – 3,000	1,000
Cement-treated base	3,500 – 7,000	5,000
Lean concrete	7,000 – 20,000	10,000
Granular base	100 – 350	200
Granular subgrade soil	50 – 150	100
Fine-grained subgrade soil	20 – 50	30

It should be emphasized that the values in Table 3-2 should only be used as guidelines because there are many factors that have effect on elastic modulus value of pavement materials as discussed after Figure 2-1.

At this stage, existing backcalculation programs based on GAs should be summarized and discussed more in detail. In 1997 Fwa et al. [28] developed the backcalculation program, NUS-GABACK. This was the first GA-based backcalculation program reported in literature. The simple genetic algorithm (SGA), i.e. roulette wheel selection, single-point crossover, jump mutation, discussed in the section 3.2 was employed in this program. Moreover, two different forward programs based on different mechanistic model, Odermark equivalent-layer method and multi-layered elastic theory (BISAR), were used in that study. About a year later, Kameyama et al. [51] presented another backcalculation program using GAs for backcalculation of layer moduli of both flexible and rigid pavements. This program uses floating-point as coding scheme. Roullett-wheel selection, heuristic crossover, and dynamic mutation are used in GA process. In 2002 BACKGA program was developed by Reddy et al. [64] keeping in view the specific features of the FWD developed by Reddy et al. In 2004 Tsai et al. [77] demonstrated the applicability of GAs based on floating-point coding scheme in backcalculating and other routine works in the field of asphalt pavement design. In 2006 Park et al. [62] developed another SGA-based backcalculation program, GAPAVE, using finite element method as forward model for deflections calculation. Table 3-3 summarizes the important features of the above mentioned programs.

Table 3-3.Important features of existing GA-based backcalculation programs

Program	GA-process	Coding scheme	Evaluation function	Deflection model	Max no. of pavement layers
Fwa et al. (NUS-GABACK)	SGA	Binary	*Minimize* $f = RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(\frac{d_i - D_i}{D_i}\right)^2}$	Boussinesq+ MET, MLET (BISAR)	4-layer system
Kameyama et al.	*	Real number	*Minimize* $f = \sqrt{w_i(d_{i-}D_i)^2}$ $F_i = U - f$:U=constant	MLET (ELSA)	4-layer system
Reddy et al. (BACKGA)	SGA	Binary	*Maximize* $f = \frac{1}{1+OBJ}$	MLET (ELAYER)	4-layer system
Tsai et al.	*	Real number	*Minimize* $f = OBJ = \sum_{i=1}^{N}(d_i - D_i)^2$	MLET (ELSYM5)	3-layer system
Park et al. (GAPAVE)	SGA	Binary	*Maximize* $f = \frac{1}{1+OBJ}$	FEM (n. s.)	3-layer system

**As discussed in context.*

It can be seen from the Table 3-3 that there are both similar and different features in each of the existing GA-based backcalculation programs. Obviously, all the programs based on binary-coding scheme employ simple genetic algorithm (SGA) as the searching driver. Moreover, the maximum number of layers of input pavement model in each of the existing GA-based backcalculation is not greater than 4 layers. The advantages and drawbacks of these will be discussed more in the next section.

Nearly in the same time of this work, Alkasawneh [2] was developing another GA-based backcalculation program, BACKGENETIC3D. This program is also based on the binary coding scheme and uses the program MultiSmart3D for calculating deflection basin in forward model. Alkasawneh has reported that the program BACKGENETIC3D can be used to backcalculate the layer moduli of any pavement system with any arbitrarily of layers, loading conditions and configurations. At this stage, it may be interesting to illustrate that all these existing GA-based programs have in common the operating process as shown in Figure 3-7.

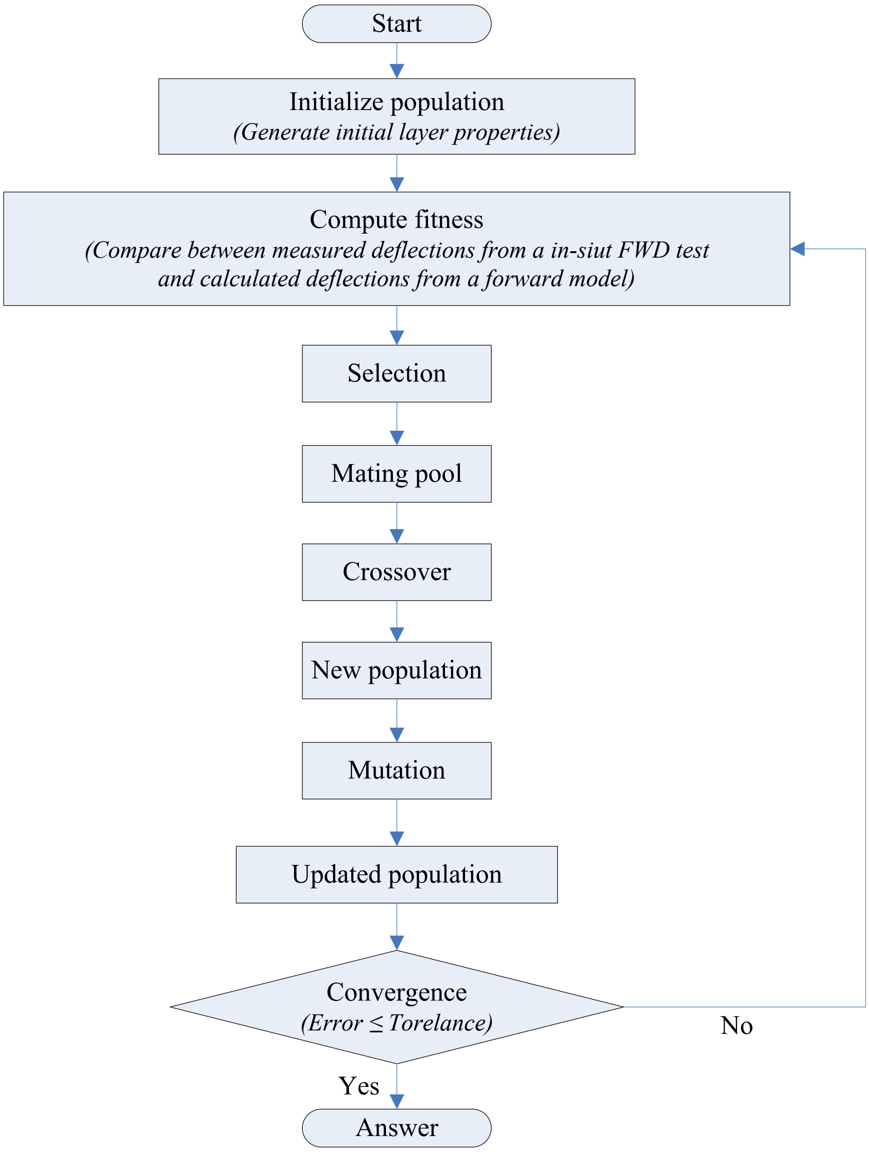


Figure 3-7. Common process of GA-based backcalculation program

3.5 Difficulties of Existing GA-Based Backcalculation Programs

As mentioned that the advantages and drawbacks of the GA-based backcalculation programs listed in Table 3-3 should be discussed. For the backcalculation methods based on floating-point coding scheme (Kameyama et al. and Tsai et al.), it has been described in 3.2.1 that this coding scheme is specially suitable for solving a problem in which the values of the variables are continuous and the full machine precision is desired. Actually, this is unnecessary for backcalculating work. One of the advantages of this scheme may be that it can save the computer memory during the calculation process. On the other hand, during crossover with binary coding, the crossover point may occur in the middle of one of the parameter strings. This allows the child to have a parameter string that is a mix of the parent parameter strings and, consequently, the child may have an allele (parameter value) between two alleles of the parents.

In floating-point coding, the child must have a mix of the parent's alleles but cannot have alleles which are not presents in the parent's chromosome strings. It can be reckoned that in binary coding, more alleles (possible values of the parameters) are preserved as new generations are created [11]. Moreover, it has been explained in the literature [31] that floating-point coded GAs can be blocked from finding the global minimum because important alleles can be lost as separated local minimum are found. Therefore, the binary coding scheme should be recommended for using in backcalculation of layer elastic moduli of pavement structure.

As can be noted that most of the existing GA-based backcalculation programs with binary coding scheme use simple genetic algorithm (SGA) as the searching driver, it is important to note that some drawbacks of this SGA have been later reported by several researchers. For example, Goldberg and Deb [34] have shown that the tournament selection method is highly efficient and requires less population sizes to converge compared to other selection methods. Furthermore, many new GA operators and techniques have been developed and shown the superior over SGA in the view of both generality and backcalculation problem. For instance, Syswerda has presented a different crossover concept called "uniform crossover". This crossover operator has been theoretically and empirically compared with single-point and two-point crossover. It has been found that this uniform crossover has shown to be superior in most cases he studied [74]. Such new GA operators and techniques will be discussed more in detail in the next chapter.

The main handicap of the existing GA-based backcalculation programs listed in the Table 3-3 has been reported even by most of the developers themselves is the relatively long computation time required [[28], [65], and [77]]. Moreover, the selection of GA parameters is crucial for the performance of GA models. If the GA has been designed well, the population will converge to the optimal solution. On the contrary, improper selection of parameters can result in local minima. By this reason, the selection of optimal GA parameter can be a further problem of the GA-based backcalculation programs. In order to relieve this problem, Reddy et al. [65] have presented in their later work the selection of GA parameters for backcalculation of pavement layer moduli using BACKGA. The method used in that study was based on level of desired accuracy and the computational effort. However, it is reasonable to believe that the results of sets of GA parameter shown in that study work well only under the environment of BACKGA and hence cannot be used in general. A new procedure struggling to overcome these two main drawbacks of GA-based backcalculation programs using new GA operators and techniques will be proposed and discussed in the next chapter.

4 A New Backcalculation Program "GAMLET"

In the view of the limitations and problems of the existing backcalculation programs, a new GA-based computer program, GAMLET, for backcalculation flexible pavement layer moduli has been developed by the author at Pavement Engineering Section (PES), Leibniz University of Hanover. This program embodies the multi-layered elastic theory (MLET) computer program BISAR as forward mechanistic model. Thus, the name "GAMLET" stands clearly for the two main algorithms used in this program: **G**enetic **A**lgorithms & **M**ulti-**L**ayered **E**lastic **T**heory.

GAMLET itself is written with DIGITAL FORTRAN computer language with approximately 5,800 lines of source code to run on desktop computers and works as a "stand alone" program. For en- and decoding the parameters, it uses binary coding scheme with the concatenated, multiparameter, mapped, fixed-point coding method as shown in Figure 3-2. Using this coding method, the algorithm used in backcalculation program GAMLET is capable for analyzing pavement models with any number of layer systems. However, since high accuracy of the solution is desired, the redundant system of equations (no. of deflections > no. of unknown layers to be backcalculated) is considered to be very useful to reduce the effect of random error from FWD measurement. As a result, even though the deflection computing program BISAR embodied in GAMLET can calculate the pavement model with number of layers up to 30 layers, the backcalculation program GAMLET is recommended for analyzing pavement structure with maximum of eight-layer system since conventional FWD equipments have mostly eight or nine sensors. The important components and features of GAMLET are discussed thoroughly in this chapter.

4.1 Computer Program BISAR

BISAR (BItumen Stress Analysis in Roads) is a FORTRAN computer program developed by De Jong et al. [21] at Shell-Laboratorium. This program is based on Burmister's multi-layered elastic theory discussed in section 2.2.1.2. BISAR has been devised as a general-purpose program for the calculation of stresses, strains, and displacements in systems, induced by one or more uniform circular loads. The program is a logical extension of the earlier developed program BISTRO. In the BISAR program, the loads can be combinations of unidirectional tangential and normal stresses. Moreover, the layers can be allowed to slip over each other. This feature of BISAR is important for backcalculation of pavement layer moduli especially at the test point where the adhesion between layer interfaces is absent. The effect of lacking of layer adhesion on backcalculated layer moduli cannot be taken into account in the backcalculation programs based on method of equivalent thickness (MET).

The stresses, strains and displacements resulting from the action of more than one load are simply found by concept of superposition. After many years of using BIASR in pavement analysis works at the Pavement Engineering Section (PES), the program does a fine job in calculating the accurate results with a good calculation speed. These properties are suitable for use as forward calculating in backcalculation program. By these reasons, BISAR program has been chosen as forward calculating model for the new backcalculation program GAMLET.

4.2 Classical Pavement Model and CSCM

Basically, pavement models based on the assumptions of the MLET discussed in section 2.2.1.2 are used as input model for the backcalculation programs used MLET as forward model. Hence, it is clear that the existing backcalculation programs based on MLET regardless their backward algorithms cannot take the natural variation of subgrade into account since the bottom layer of the model in this theory is assumed to be semi-infinite in depth, with a constant elastic modulus. As discussed earlier, most of real subgrade materials are stress sensitive. Moreover, "bedrock" can also exist everywhere and at any depth which may lead to an enormous high modulus layer. By this reason, a new method for setting up pavement model which is able to deal in some degree with the effect of bedrock and/or the stiffening behavior of most subgrade materials is proposed to improve the reliability of backcalculation results. The proposed method coined as "Consistent Slope Changing Method" (CSCM) is added as an option in GAMLET and will be discussed thoroughly in chapter 6.

Nevertheless, the classical pavement model for backcalculation based on MLET (pavement system with constant elastic modulus and semi-infinite in depth) can still be used without any difficulties as input pavement model for GAMLET. This classical model will be used throughout this chapter in order to compare the performance of GAMLET with other MLET-based backcalculation programs.

4.3 Evaluation Functions and Constraints

4.3.1 Evaluation Functions

Unlike the existing GA-based backcalculation programs, GAMLET uses three evaluation functions in parallel to control the goodness of fit between calculated and measured deflection basins in backcalculation process. These three equations are illustrated in following equations:

Max. of % error: Minimize $$f_1 = \max.of\left|\frac{(d_i - D_i)}{D_i} * 100\right| \quad (4\text{-}1)$$

RMSE function (in %): Minimize $$f_2 = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(\frac{d_i - D_i}{D_i}\right)^2} * 100\% \quad (4\text{-}2)$$

Fitness function: Maximize $$f_3 = \frac{1}{1 + \left(\frac{OBJ}{10^6}\right)} \quad (4\text{-}3)$$

where

Objective function: $$OBJ = \sum_{i=1}^{N}(d_i - D_i)^2 \quad (4\text{-}4)$$

As given by eq(3-16), N is the number of sensors, d_i and D_i are the calculated and the measured deflections, respectively, at sensors i.

While the evaluation function, f_1, in eq(4-1) controls the maximum error at each point of sensor locations, the conventional root mean square error, in percent, f_2, controls the overall goodness of fit of the backcalculated basin. In addition, using this RMSE function makes comparing results obtained from GAMLET with other backcalculation programs and with some recommendations about backcalculation easier. For example, an RMSE of 3% or less are used as an acceptable error in the Long Term Pavement Performance (LTPP) test section in USA and many backcalculation programs use RMSE values between 1% and 3% as an acceptable error [2]. However, among different forms of evaluation functions examined (see section 3.2.2), it has been found that backcalculated moduli from the program GAMLET were the most sensitive to the evaluation function, f_3, given by the eq(4-3). Moreover, the results of fitness values obtained from this function were found to be more representative than the eq(3-1) used by other existing GA-based backcalculation programs [[2], [62], and [64]]. Considering the goodness of fit of a backcalculated basin which yields the maximum percent error according to eq(4-1) = 4% and %RMSE = 3.85%, this can be generally considered as poor fitting and the obtained backcalculated moduli should not be further used in any rehabilitation project. Evaluating the goodness of fit of this poor basin with eq(4-3), the fitness value of 0.0138 is obtained, which intuitively indicates the poor performance. On the other hand, based on the identical poor basin, the fitness value of 0.9999289 will be obtained from eq(3-1), this value does obviously not show the good scale for fitness indication.

4.3.2 Constraints

Thus far the existing GA-based backcalculation programs using binary coding scheme search for the optimal solution with unconstrained objective functions. Many practical problems contain one or more constraints that must also be satisfied. Even though there is no need of addition constraints to be satisfied in solving the backcalculation of pavement layer moduli, a constraint designed for discarding the very poor individuals in the population has been combined in GAMLET for research purposes. Using this constraint, user can assign the threshold of maximum percent error after the eq(4-1). The fitness value of any individual which has the maximum percent greater than the assigned value will be automatically set to zero. This deprives the survival possibility of these individuals in the next generation.

4.4 Versatile GAs Search Techniques

It has been presented in the last chapter that most of the existing GA-based backcalculation programs have been developed based on simple genetic algorithm (SGA). This SGA uses binary coding scheme, roulette wheel selection and jump mutation. As the usage of GAs has grown, objections to their performance on specific problems have arisen, and when this happen, natural remedies are often tried and proposed as new GA operators and techniques. In order to overcome the problems suffered from using SGA, some new versatile GA operators and techniques have been added in the backcalculation program GAMLET.

4.4.1 Selection and Reproduction method

As mentioned in the last chapter, SGA uses roulette wheel selection method. Since Goldberg and Deb [34] have shown that tournament selection method is highly efficient and requires less population sizes to converge compared to many other selection methods, the tournament selection method is, therefore, used in GAMLET in selection and reproduction process. The mechanic of this method is very common. Random pairs are selected from the population and the stronger (most fit) of each pair is allowed to mate. Each pair of mates creates offspring, which have some mix of the two parent's chromosomes according to the method of crossover. The process of selecting random pairs and mating the stronger individuals continues until a new generation of size *n* is repopulated.

4.4.2 Uniform and Single-point crossover operators

The single-point crossover is still available in GAMLET for analyzing the problems. However, a different crossover concept, uniform crossover, has been added as an option in GAMLET. Syswerda [74] has proposed this GA operator and compared it theoretically and empirically with single-point and two-point crossover. It has been found that uniform crossover has shown the superior in most cases he studied.

Crossover Mask:	**1 0 0 1 0 1 1 1 0 0**
Parent 1:	1 0 1 0 0 0 1 1 1 0
	↓ ↓ ↓ ↓ ↓
Offspring:	1 1 0 0 0 0 1 1 1 1
	↑ ↑ ↑ ↑ ↑
Parent 2:	0 1 0 1 0 1 0 0 1 1

Figure 4-1. Mechanism of uniform crossover

Uniform crossover is radically different to single-point crossover. Each gene in the offspring is created by copying the corresponding gene from one or the other parent, chosen according to a randomly generated crossover mask. Where there is a 1 in the crossover mask, the gene is copied from the first parent, and where there is a 0 in the mask, the gene is copied from the second parent, as shown in Figure 4-1. The process is repeated with the parents exchanged to produce the second offspring. A new crossover mask is randomly generated for each pair of parents. Therefore, offspring contain a mixture of genes from each parent. The number of effective crossing points is not fixed, but will average $l/2$ (where l is the chromosome length) [7]. Before calculation process in GAMLET begins, user can specify which crossover operators (uniform or single-point) should be applied to the problem analyzed.

4.4.3 Creep and Jump mutation operators

GAMLET still uses the simple jump mutation explained earlier. Moreover, an alternative operator, creep mutation, has been added as an option for user. The creep mutation is another small probability that one or more of offspring's parameters will be mutated by a single increment away from their original value. In other words, the creep mutation produces a parameter value that is randomly picked to be larger or smaller, so long as it remains in the range of the appropriate parameter. Creep mutations can be useful in the sense that they can slide the gene pool toward the optimal solution rather than just having to jump towards it as jump mutation.

However, both jump and creep mutations can be used in parallel in the same backcalculation process. It should be noted that the major difference between these two mutation operators is that the creep mutation acts on the decoded parameters (phenotype) while the jump mutation acts on the encoded chromosome (genotype). The probability value of each mutation will be discussed later. It should be also noted here that increasing the possibility of any mutation will increase the algorithm's freedom to search outside the current region of variable space.

4.4.4 Niche method

Usually, solution surface of backcalculation problem is multimodal as shown in Figure 2-6. It is, sometimes, useful in analyzing the fitness function which is known to be multimodal by locating all the peaks. Unfortunately, SGA will not do this. It has been found that in dealing with multimodal, SGA will converge to a single peak, even though multiple peaks of equality are exist as illustrated in Figure 4-2 (a). This is due to genetic drift [35]. Several modifications to the SGA have been proposed to solve this problem, all with some basis in natural ecosystems.

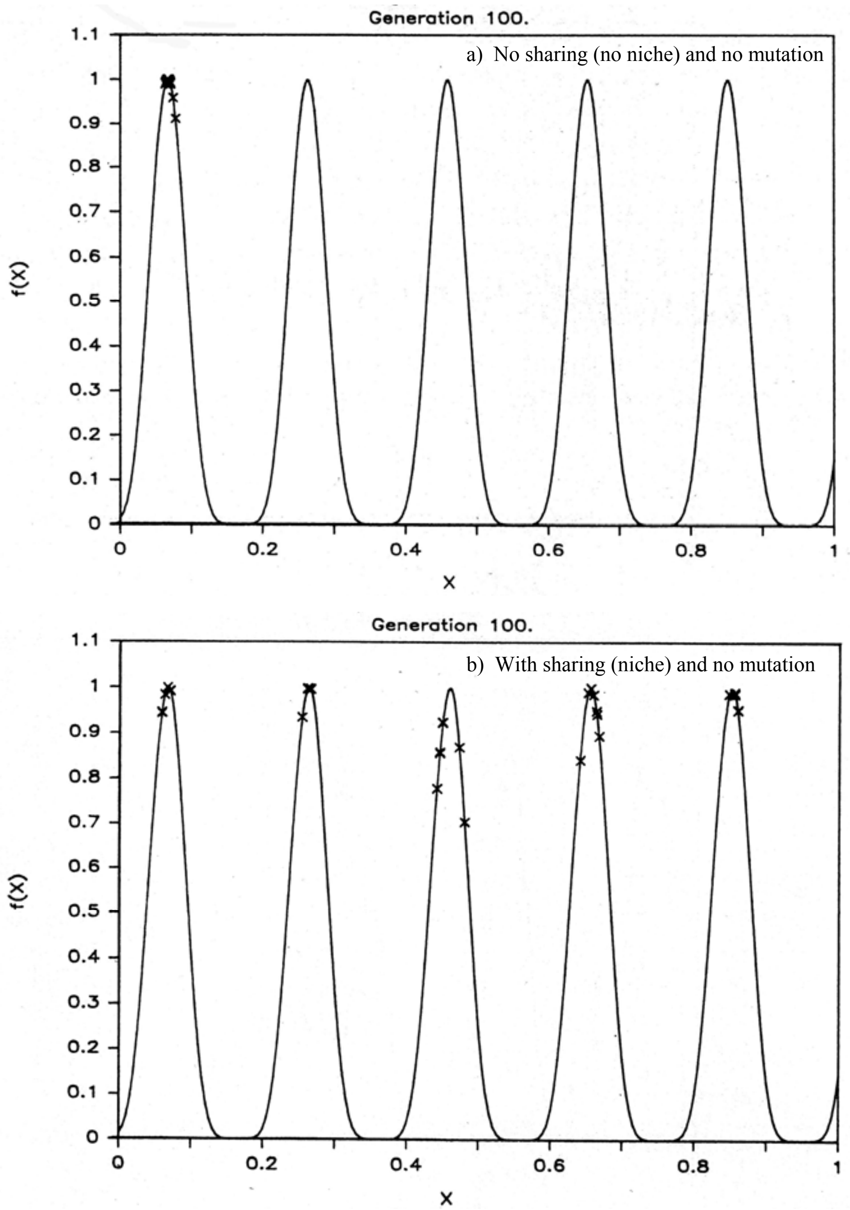


Figure 4-2. Simple Genetic Algorithm performance without and with sharing (Niche) [33]

In natural ecosystems, there are many different ways in which animals may survive and different species evolve to fill each ecological niche. Speciation is the process whereby a single species differentiates into two (or more) different species occupying different niches. In artificial genetic search, multimodal functions can be optimized by inducing the natural concepts of niche (or sharing) and species into a population of strings. Since Carroll [12] has successfully used the Goldberg's multidimensional phenotypic sharing scheme with a triangular sharing function in solving some multimodal problems in chemical field, the basic concept of this method is briefly discussed.

Goldberg & Richardson [35] have described the advantages of sharing scheme that several individuals which occupy the same niche are made to share the fitness payoff among them. Once a niche has reached its "carrying capacity", it no longer appears rewarding in comparison with other unfilled niches. However, the difficulty with sharing payoff within a niche may be that the boundaries of the niche are not easily identified. Goldberg uses a sharing function to define how the sharing is to be done. Essentially, the payoff given to an individual is reduced according to a function of the "distance" of each neighbour. The distance may be measured in different ways, for example in terms of genotype Hamming distance, or parameter differences in the phenotype.

Furthermore, Deb and Goldberg [22] have shown in a later continuation of the work that sharing is superior to crowding method proposed by De Jong. Genotypic sharing (based on some distance measure between chromosome strings) and phenotypic sharing (based on the distance between the decoded parameters) are analyzed. Phenotypic sharing is shown to have advantages. A sharing function based on Euclidian distance between neighbours implements niches which are hyperspherical in shape. The effect of niching on a function with equal peaks is illustrated in Figure 4-2 (b). It can be seen that GA with sharing has been able to converge and distribute trials at all the peaks of the function. The Goldberg and Richardson's multidimensional phenotypic sharing scheme with a triangular sharing function [35] has been added as an option for the user in the backcalculation program GAMLET.

4.4.5 Elitism concept

One way in which one could help a genetic algorithm converge is to keep a record at each time step of the best individual seen so far. This concept is called "elitism". Any GAs can easily be made elitist by the following adaptation:

1) Record the current best individual, e.g. *k*, in the population
2) Generate the next population as usual
3) If there is nothing in the new population better than *k*, add *k* to the new population, replacing some other individual randomly.

Although this elitism technique is not necessary since the mechanism of SGA produces naturally the better population in the view of computing time, it has been recommended by several researchers [[66], [12], and [37]] to turn on this technique in GA-running process. The concept of elitism has also been added as the user's option in GAMLET.

4.4.6 FWDLine module

In practical, FWDs are not used to evaluate only one point but a series of testing points on a long section of road or runway. Consequently, it is likely that the consecutive points could have similar structural condition since they are suffered under similar environment. Thereby, the evolved population which is usually obtained from the last generation of solving any problems may be (but not always) useful in helping the backcalculation of the consecutive testing point converges faster. In order to improve the computing time of GAMLET, the FWDLine function has been combined as another user's option. As described, this function uses the evolved population from the previous analysis as the first population of the analysis of the consecutive FWD testing point.

4.4.7 Micro Genetic Algorithm (µGA)

As pointed out by most of the developers of the existing backcalculation programs based on GA [[28], [65], [2] and [77]], the main limitations of such programs are the time penalty involved in evaluating the fitness functions generation after generation and the difficulties in indicating the optimal set of GA parameters for each problem. These limitations can be easier observed when GA model is applied to the large scale of problem such as backcalculation of pavement layer moduli from a number of FWD deflection basins. As a result, these limitations make the GA-based backcalculation programs less appealing or even impractical for routine work.

Micro Genetic Algorithm (µGA) has been added as a main module in GAMLET in order to overcome these limitations by improving the required computing time (or reducing the computational effort, CE) and eliminating some GA operators in the process. These will make GAMLET program more user-friendly and more appealing for analyzing the FWD data in network level where a number of deflection basins are involved. The detail of micro GA will be discussed later in section 4.6.2.

4.5 GAMLET in Backcalculation of Layer Moduli

In this section, the important guidelines about the input parameters needed for beginning the backcalculation using GAMLET are summarized and the convergence criterions of backcalculated results are discussed. In order to evaluate the performance of GAMLET, backcalculations based on some typical cases of pavement model using GAMLET have been performed. The results have been compared with those obtained from some popular backcalculation programs which are not based on GA.

4.5.1 Input Parameters

There are three types of parameter needed for a backcalculation of pavement layer moduli using GAMLET: pavement model parameters, GA parameters, and convergence criterion parameters.

4.5.1.1 Pavement model parameters

Despite the CSCM option which user can specify whether it should be applied to the model or not, the pavement model parameters needed for backcalculation a deflection basin using GAMLET are almost similar to those used in the traditional MLET-based backcalculation programs. These parameters include: 1) load radius, 2) load pressure, 3) number of sensors and their locations referred to the load center, 4) number of layers to be backcalculated, 5) layer thicknesses, 6) Poisson's ratio of each layer, 7) interface shear spring compliance of each layer (layer adhesions), 8) measured deflection values at all specified sensors, 9) range moduli of each layer, and 10) number of permissible value of each layer (specifying the precision level).

The important difference is that backcalculation programs based on GAs do not require the input seed moduli but only the lower and upper domain bounds of the layer moduli, i.e. the range of possible moduli considered by engineering judgment.

4.5.1.2 GA parameters

Basically, the four main GA parameters which are population size, probability of jump mutation, probability of crossover, and maximum number of generations must be assigned before any backcalculation begins. Even though finding the optimal set of these parameters is not an easy task, there are some guidelines and thumb rules in GA literature about setting these GA parameters and GAs are usually robust enough to handle some variations in the parameters. Other remaining GA parameters described in section 4.4, uniform crossover, creep mutation, elitism, niche, micro GA, and FWDLine are optional operators and techniques which user can independently turn on or off. The selection of these parameters and some related issues will be discussed later in this chapter. The variable names of each parameter and their units used in the GAMLET are shown in appendix A.

4.5.1.3 Convergence criterion parameters

Backcalculation process in GAMLET will be stopped based on either the convergence criterions are met or the number of generations is greater than the predefined value. For convergence criterions, user can assign any desired value of f_1, f_2, and f_3 according to the equations (4-1), (4-2), and (4-3), respectively. For example, if high accuracy of results is desired, the process is said to be convergence when the percent of maximum error (f_1) is less than 0.1% or the fitness value (f_2) is greater than 0.995. As discussed earlier, the fitness value and %RMSE value (f_3) control the goodness of fit of the whole basin, the increase of fitness function leads usually to the decrease of RMSE function in the same fashion.

4.5.2 Selection of GAMLET parameters

The performance of GA-based model depends on many factors, e.g. solution surface and set of GA parameters. Though GAs are usually robust enough to handle some variations in the parameters, it is obvious that the poor choice of parameter set can result in poor performance. Experienced user may be able to select the appropriate parameter set without any great exertion but it can be a tough task for the user who is new in GA world. Normally, these parameters are selected from a number of trials or based on empirical relationships or thumb rules. For instance, Goldberg et al. [33] have proposed a general relation for determining an appropriate population size by using the following equation:

$$n_{pop} = O\left(m\chi^{k}\right) \tag{4-5}$$

where $m = l\ /\ k$, χ is the cardinality of chromosomes (the number of possibilities for each chromosome, e.g. for binary χ =2), k is the size of the schema of interest, and l is the length of chromosome string.

For estimating purpose, it can be assumed that each parameter string would represent one important schema. Therefore, the schema length can be assumed to be equal to the average parameter length [12]. However, the population sizes used in the existing GA-based backcalculation programs reveal that the appropriate population size for backcalculating work should be less than the result obtained from eq(4-5) by many folds. Table 4-1 summarizes the population size and other basic GA parameters used in the existing GA-based backcalculation programs.

Table 4-1. Set of basic parameters used in existing GA-based backcalculation programs

Program	Population Size (n_{pop})	No. of Generations (G)	Crossover Probability (p_c)	Mutation Probability (p_{jump})	Computational Effort* (CE)
NUS-GABACK [28]	60	120	0.85	0.15	7200
(Kameyama et al.) [51]	50	42-593	Variable	Variable	NA
(Tsai et al.) [77]	500	50	NA	NA	25000
GAPAVE [62]	100-140	70-120	0.64-0.86	0.01-0.05	7000-16800
BACKGENETIC3D [2]	512	150	0.50	0.00	76800
BACKGA. [65]	60	60	0.74	0.10	3600

**After Reddy et al. [65]*

The parameters used in NUS-GABACK have been selected on the basis of some trial runs. Kameyama et al. have used a dynamic mutation and crossover suggested by Wright [65]. The information about selection of parameters used by Tsai et al. has been not available in the study. For GAPAVE, six sets of parameters have been selected based on the backcalculation result from six different pavement structures. Detailed information regarding the selection of parameters used in BACKGENETIC3D has also been not available in that study.

Reddy et al. [65] have presented a study conducted for selection of optimal GA parameters to be adopted for BACKGA using a typical three-layer pavement system. The parameters were selected on the basis of the level of accuracy desired and the corresponding computational effort. The total number of function evaluations was considered to be the CE in that study. The CE values computed after Reddy et al. [65] are also shown in Table 4-1 according to the parameters used in each program.

It is evident in Table 4-1 that there are large variations in the GA parameter values used in the existing GA-based backcalculation programs. This emphasizes the fact that selection of the GA parameters cannot be simply generalized in the form of rules or guidelines. The optimal set of parameters depends not only on the techniques and operators used in GAs but also on the nature of the particular problem to be solved. It is clear that the dimension of the solution surface is proportional to the number of unknown layers in pavement model and the precision level assigned to the process.

As explained, GAMLET consists of many new GA operators and techniques (uniform crossover, creep mutation, niching, elitism, CSCM, FWDLine, and micro GA). Each of these specifies features considered to be able to improve the backcalculation process compared with GA models used in the existing programs. Combination of basic GA operators and the following operators: uniform crossover, creep mutation, niching, and elitism, is referred in GAMLET and in this work as the “loaded GA module”. Since GAMLET is designed to be able to backcalculate deflection basin of pavement model with many layers (recommended no. of layers < no. of sensors), a unique optimal set of GA parameters for all pavement systems is not expected.

However, a good GA-based model must be robust enough to handle some variations in parameters and environment. After a series of computer runs using loaded GA module in GAMLET for solving a number of backcalculation problems in Pavement Engineering Section, two sets of basic GA parameters in Table 4-2 have shown the high performances by yielding accurate results for wide range of backcalculation problems. These two sets of basic GA parameters are also employed in combination with other GA operators and techniques in the loaded GA module in solving all the problems which will be analyzed in this work.

Table 4-2. Sets of basic parameters used in GAMLET-Loaded GA module for wide range of problems

Program	Population Size	No. of Generations	Crossover Probability	Mutation Probability	Computational Effort (CE)
GAMLET (set 1)	120	200	0.80	0.02	24000
GAMLET (set 2)	240	140	0.90	0.005	33600

Considering the creep mutation probability, Carroll [12] has recommended that this mutation operator should be used together with jump mutation in the same search process. He has set the jump mutation probability using the eq(4-6) based on the success in his former work.

$$p_{jump} = 1/n_{pop} \tag{4-6}$$

Moreover, he has also recommended that the number of jump and creep mutations in each generation should be approximately the same. Using basic probabilistic arguments, the overall probability of a jump mutation occurring for an individual i is:

$$p_{jump,i} = 1 - \left(1 - p_{jump}\right)^{n_c} \tag{4-7}$$

where n_c is the number of bits in an individual binary string. The overall probability of a creep mutation occurring for an individual i is:

$$p_{creep,i} = 1 - \left(1 - p_{jump}\right)^{n_p} \tag{4-8}$$

where n_p is the number of parameters in an individual binary string. Setting eq(4-7) equal to eq(4-8) yields:

$$p_{creep} = 1 - \left(1 - p_{jump}\right)^{n_c / n_p} \tag{4-9}$$

Taking a binomial expansion of eq(4-9) and neglecting lower order terms since p_{jump} <<1 gives:

$$p_{creep} \cong \frac{n_c}{n_p} \cdot p_{jump} = \frac{n_c}{n_p}\left(\frac{1}{n_{pop}}\right) \tag{4-10}$$

Eq(4-10) has been used as guideline for setting the probability of creep and jump mutations in this work as well.

4.6 Performance Evaluation of GAMLET

The easiest way to evaluate performance of a backcalculation program is generating deflection basin by using the forward mechanistic computer program, backcalculate that basin and then compare the backcalculated results with the entered moduli. However, it should be better to evaluate the performance of a new program by comparing the results with those obtained from other well-known programs. Since Harichandran et al. [36] have examined the dependency of backcalculation procedure by making a comparison of several popular backcalculation programs based on their computed modulus values for selected problems. The information available in that work makes a comparison of performance between a new backcalculation program and those popular programs possible. Therefore, the performance evaluation of the backcalculation program GAMLET is performed here by solving the same problems analyzed by Harichandran et al. [36] and, then, comparing the obtained results with those obtained from other traditional programs.

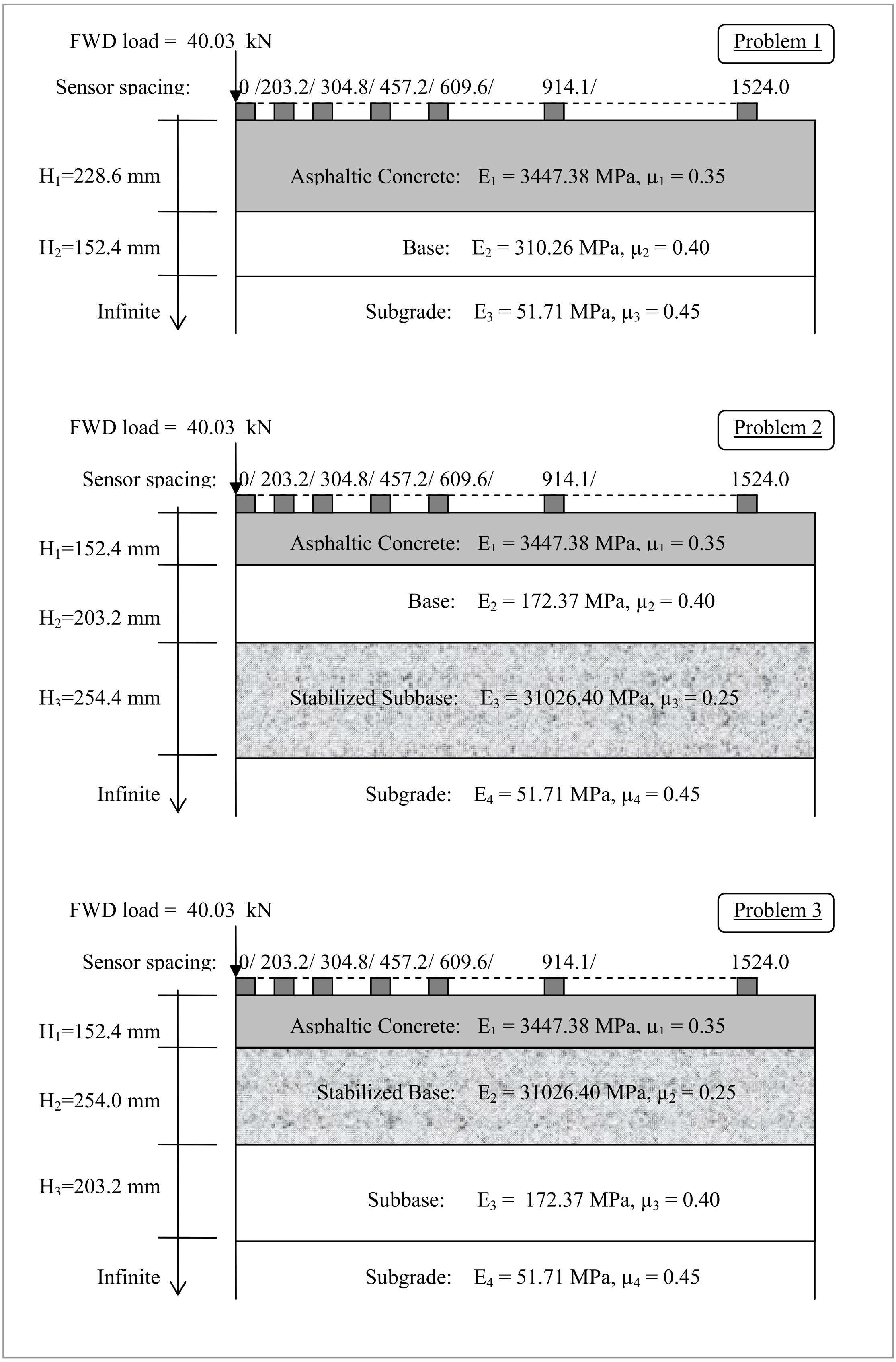


Figure 4-3. Properties of the pavement models used in the performance evaluation

Three different systems from all five problems analyzed by Harichandran et al. are used in this comparison. These problems are 1) a simple three-layer flexible pavement structure, 2) a four-layer flexible composite pavement with a stabilized subbase layer, and 3) a four-layer flexible composite pavement with stabilized base. The material properties of these three pavement structures and the configurations of simulated FWD equipment are shown in Figure 4-3.

Including GAMLET, the backcalculation programs involved in this comparison are EVERCAL, EVERCAL-Alt, MICHBACK, and MODULUS. Very briefly, EVERCAL is a backcalculation program using nonlinear least-squares optimization technique with CHEVRONX as the forward program. EVERCAL-Alt. is similar to EVERCAL, except that CHEVRON is used to compute the deflection basin. Both require a set of seed modulus values to start and adopt Newton's method to search for the set of deflections that best match the measured deflections. MICHBACK uses CHEVRONX for forward calculation. It is essentially similar to EVERCALC and EVERCAL-Alt, except that is uses a modified Newton's method to improve the speed convergence. MODULUS is developed by Uzan et al. [82]. The main different in this program is that it matches the measured deflection basin with a data base of deflection basins computed in advance for a variety of layer moduli [28].

4.6.1 Backcalculation using GAMLET-Loaded GA module

As mentioned, unlike the other traditional backcalculation procedures, no seed modulus values are needed to perform the backcalculation using GAMLET. Instead, only the range of modulus values is required as input for each unknown layer moduli. Other remaining input parameters used in GAMLET are chosen based on the guidelines, thumb rules discussed in section 4.5.1.2, and the experiences from solving backcalculation problems using GAMLET in Pavement Engineering Section. A same set of parameters has been used in solving of all three cases to examine the robustness of GAMLET. Table 4-3 gives the detail of these parameters.

It is worth to note that even though the good ranges of moduli have been employed to solve the problems, using GA-based program with the precision level of 0.01 MPa for all layers as done in these analyses produces enormous solution surface. For example, in case of the four-layer systems, each individual contains as many as 71 bits. In other words, the desired optimal solution must be searched by GA among approximately $2^{71} = 2.36 \cdot 10^{21}$ possible solutions in the space. This precision level is employed in this analyzing for the purpose of testing the performance of the GAMLET program. For routine backcalculation work in which a very high precision level may be not necessarily, other coarser precision levels can be used in order to reduce the computing time. In addition, in GAMLET program user can independently set the desired precision level for each layer.

Table 4-3. Parameters used in GAMLET-Loaded GA module

Parameters	Problem 1, 3-layer system	Problem 2, 4-layer system,	Problem 3, 4-layer system,
No. of backcalculated layer moduli	3	4	4
Size of population	240	240	240
Maximum generations	140	140	140
Probability of crossover	0.90	0.90	0.90
Single-point crossover	on	on	on
Uniform crossover	off	off	off
Probability of jump mutation	0.005	0.005	0.005
Probability of creep mutation	0.12	0.12	0.12
Elitism	on	on	on
Niching	off	off	off
Micro GA	off	off	off
FWDLine	*	*	*
CSCM	**	**	**
Range of possible moduli E_1 [MPa]	1000 to 8000	1000 to 8000	1000 to 8000
Range of possible moduli E_2 [MPa]	10 to 400	10 to 400	1000 to 50000
Range of possible moduli E_3 [MPa]	5 to 150	10000 to 50000	10 to 400
Range of possible moduli E_4 [MPa]	-	10 to 150	10 to 150
Precision level [MPa]	0.01/0.01/0.01	0.01/0.01/0.01/0.01	0.01/0.01/0.01/0.01

** For FWD Network level, **For stiffening subgrade*

The results of the backcalculated pavement layer moduli obtained from all mentioned programs for all cases are illustrated in Table 4-4, Table 4-5 and Table 4-6 respectively. The root mean square error in surface-deflection matching and the maximum percentage error (ΔM) in the backcalculated moduli for each pavement are also shown in these tables.

For the case of typical three-pavement layer system, Table 4-4 shows that GAMLET and MICHBACK produced the best root mean square error with the value of 0.014%, followed by other programs. Although MODULUS showed the approximately same least square error level as EVERCAL-Alt, it overestimated the base modulus with an error of 4%. EVERCALC showed the most inaccuracy results by overestimating the base modulus value with the ΔM error of 28.87 %.

Table 4-4. Comparing the Backcalculated Layer moduli of Problem 1, three-layer system

Computer Program	Backcalculated Modulus [MPa]			max. ΔM* (%)	RMSE (%)
	AC	Base	Subgrade		
Input Modulus values	3447.38	310.26	51.71	-	-
MICHBACK	3455.93	307.67*	51.74	0.84	0.014
MODULUS	3346.71	322.67*	51.71	4.00	0.139
EVERCALC	3030.65	399.81*	51.66	28.87	0.126
EVERCAL-Alt.	3450.86	315.52*	51.72	1.70	0.148
GAMLET	**3457.62**	**307.35***	**51.72**	**0.94**	**0.014**

Table 4-5. Comparing the Backcalculated Layer moduli of Problem 2, four-layer system

Computer Program	Backcalculated Modulus [MPa]				max. ΔM* (%)	RMSE (%)
	AC	Base	Stabilized Subbase	Subgrade		
Input Modulus values	3447.38	172.37	31026.40	51.71	-	-
MICHBACK	3444.31	172.29	30839.30*	51.67	0.60	0.030
MODULUS	3392.90	175.13	30354.10*	51.71	2.17	0.078
EVERCALC	4293.42*	190.71	27300.90	51.39	24.54	0.753
EVERCAL-Alt.	3388.46	176.00*	30423.60	51.79	2.10	0.105
GAMLET	**3447.90**	**172.13**	**31145.99***	**51.68**	**0.38**	**0.009**

Table 4-6. Comparing the Backcalculated Layer moduli of Problem 3, four-layer system

Computer Program	Backcalculated Modulus [MPa]				max. ΔM* (%)	RMSE (%)
	AC	Stabilized Base	Subbase	Subgrade		
Input Modulus values	3447.38	31026.40	172.37	51.71	-	-
MICHBACK	3443.03	31131.90	158.59*	51.75	7.99	0.007
MODULUS	3639.74	30827.10	67.57*	52.40	60.80	0.068
EVERCALC	10908.93*	15838.00	90.95	51.83	216.44	1.526
EVERCAL-Alt.	3410.81	29075.90	398.29*	51.52	131.07	0.064
GAMLET	**3456.85**	**30970.36**	**166.43***	**51.76**	**3.44**	**0.019**

For the case of problem 2, the four-layer flexible composite pavement with a stabilized subbase layer, Table 4-5 shows that MICHBACK, MODULUS, and EVERCAL-Alt could yield good estimates of results for which can be considered as equally satisfactory. EVERCAL, again, produced the worst ΔM errors of 24.54% and 10.64% by overestimating the AC and base moduli, respectively and underestimated the subbase modulus by 12.01 %. On the contrary, GAMLET was not only able to yield good estimates of pavement layer moduli, but also offered the best results compared with other programs.

For problem 3, it is obvious at the first glance in Table 4-6 that the maximum percentage error (ΔM) values obtained from all programs are higher than those obtained from the first two problems. EVERCALC yielded the highest ΔM error with value of 216.44% by overestimating the asphalt layer. For MODULUS and EVERCALC-Alt., it is very interesting to note that the obtained RMSE from both programs are all less than 1% (i.e. 0.068% and 0.064% respectively), so by considering these RMSE alone the associated backcalculated modulus values should be considered as acceptable results according to the criterions used in general practice. However, it is evident that with these so-called acceptable RMSE values MODULUS and EVERCAL-Alt. have under- and overestimated the subbase layer with high ΔM error values of 60.80% and 131.01% respectively. This implies that considering the overall matching between the measured and the backcalculated deflection basins alone using parameters such as RMSE may be not well enough to indicate the reliable backcalculated results. Thereby, GAMLET controls not only the overall matching of the entire basin (eq(4-2) and (4-3)) but also the maximum error at each point of sensor locations (eq(4-1)) to relieve such deceptive problem. The backcalculated moduli obtained from GAMLET shown in Table 4-6 are associated with the gratified RMSE value of 0.019% and yielded the ΔM value of only 3.44% which is the least ΔM error compared with other programs.

Figure 4-4 A and B illustrate convergence of solutions in the problem 3, typical three-layer pavement system, according to eq(4-1) to (4-4) used in GAMLET. The plots in these figures show that average and best fitness values keep increasing along with the generations. This indicates that GA mechanism in GAMLET did a fine job in searching for the optimal solution. Even though jump and creep mutations caused some oscillation in average fitness values, these occurrences provided the diversity of solutions and protected the search process against the premature convergence. At the same time, elitism process always kept the best (fittest) individual of each generation and transferred it to the next generation regardless the variation of average fitness values. The maximum percent error and root mean square error, also in percent, associated with the best individual are plotted in Figure 4-4B. It is evident in this analyzing that the elitism technique makes the backcalculation process more robust and helps the search process converge faster. The similar trends can be observed from the convergence behavior of the results obtained from the four-layer pavement systems. The same plot manner of the four-layer systems (problem 2 and 3) are illustrated in appendix B.

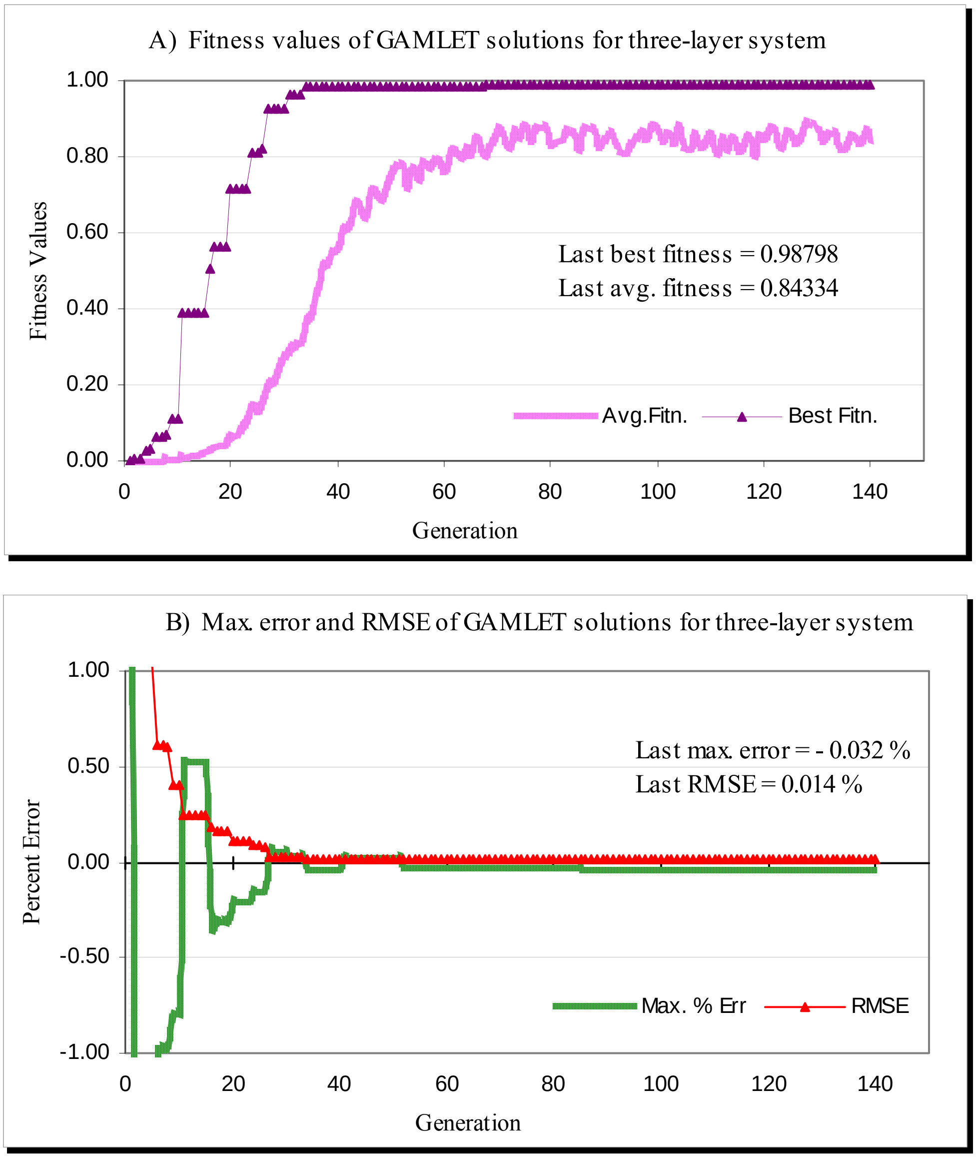


Figure 4-4. Convergence of GAMLET solutions of the Loaded GA module (Prob.1)

4.6.2 Backcalculation using GAMLET-Micro GA module

Thus far, the loaded GA module in GAMLET has shown a good capability in solving the backcalculation problems compared with other traditional programs. However, the two main limitations of applying the GA model in backcalculation analysis, the relatively long computation time and the difficulties in finding the optimal GA parameters, are left unsolved. These limitations are emphasized in Table 4-1 by the high computational efforts (CE = total number of function evaluations) and the difference of the selected GA parameters set used in each GA-based backcalculation program. For example, the CE associated with parameters used in BACKGENETIC3D can be as many as 76,800 times of function evaluations if all generations are needed to reach a result. It is highly likely that more CE would be needed to reach the same result accuracy when the program is used to analyze the in situ deflection data where the pavement structure is more complicated and much more variations can be appeared in deflection basins. These two limitations simply make GA-based programs less appealing for routine backcalculation analysis compared with other conventional procedures and restrict the use of such programs only in research area or in analyzing deflection data at project level. As mentioned earlier, micro genetic algorithm (µGA) has been added to GAMLET in order to overcome both of these limitations. The following discusses some important issues about this µGA technique.

Micro GA (µGA) was proposed by Krishnakumar [56] in 1989 for solving the optimization problem of stationary and non-stationary functions. He found that µGA could avoid premature convergence and demonstrated faster convergence to the near-optimal region than did a SGA for the multimodal problem he studied. Carroll [12] and Yang et al. [86] have successfully used micro GA in optimizing chemical oxygen-iodine lasers and optimization of permanent prostate implant, respectively. Hence, Krishnakumar's µGA scheme has been combined to the backcalculation program GAMLET in order to improve the effective performance of the program. The mechanism of micro GA is described below.

Just as in the SGA, the µGA works with binary coded populations and are implemented serially. The major differences between SGA and the µGA come in the population choice and the GA operators. In the µGA structure, the population size is fixed at any very small number (Krishnakumar has used n=5 in his study) and only crossover operator is involved in the process. It is a known fact that GAs generally do poor with very small population due to insufficient information processing and early convergence to non-optimal results, especially when performed without mutation operator. In the µGA this small population evolves in normal GA fashion and converges in a few generations (possibly local optima). At this point, a new random population is chosen while keeping the best individual (elitism) from the previously converged generation and the evolution process restarts until the desired best fitness value is found.

Note that the use of µGA has two main addition benefits. The first is users are not required anymore to fiddle with many GA operators such as jump and creep mutations, niching, or even population size (fixed at very small) which, in turn, makes the µGA-based computer program more user-friendly. The second is improvement in required computing time associated with the very small size of population. This makes the µGA-based computer program more appealing for analyzing the large scale problem such as backcalculation of pavement layer moduli in network level.

To demonstrate backcalculation of pavement layer moduli using µGA, all three problems shown in Figure 4-3 are recalled to be solved by the µGA module and compared the obtained results with those obtained from the loaded GA module. The backcalculation process of problem 1, the simple three-layer flexible pavement structure, using the µGA module in GAMLET has been thoroughly explained here. The four-layer systems of problem 2 and 3 have also been backcalculated by the µGA module. The results are illustrated in appendix B.

Table 4-7 shows the set of parameters used in the µGA module. The same ranges of possible moduli and the same precision level of each layer used in the loaded GA module are employed again to keep the condition of search surface unchanged. Although Krishnakumar's µGA scheme used elitism, single-point crossover with p_c=1.0, and a restart mechanism to reinfuse new genetic information into population when it converges (nominal), Carroll [12] founded that the use of Syswerda's uniform crossover with p_c=0.5 is able to enhance the µGA technique. Moreover, Coverstone et al. described that for certain complex problem with the number of parameters on the order of 100 or more, the µGA technique could work better if the population size is increased from 5 to 15 [86]. This effect is also found in the optimization of backcalculation problem where the large solution surface is usually involved. Therefore, the uniform crossover with p_c=0.5 and the population size of 15 are used in this backcalculation process as shown in Table 4-7.

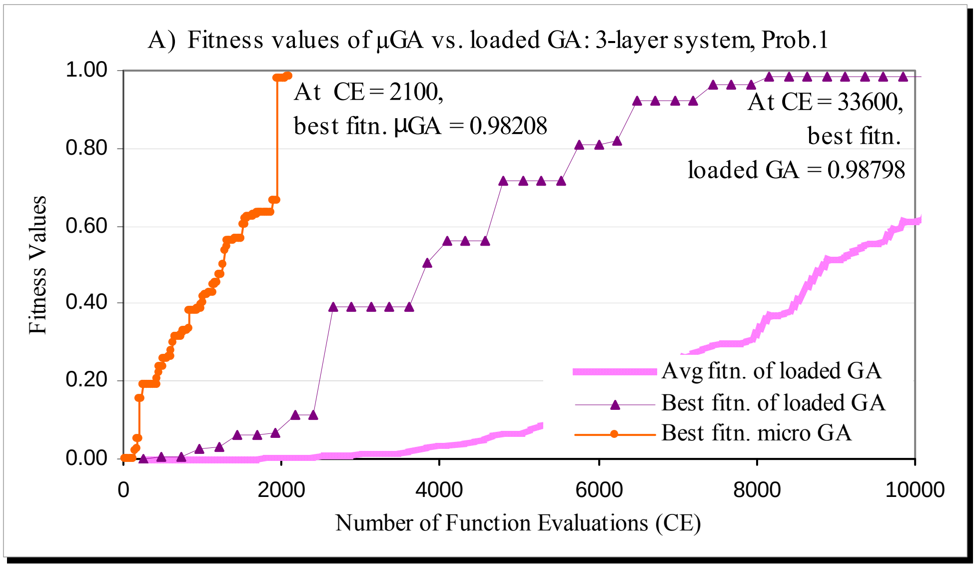
Table 4-7. Parameters used in GAMLET-µGA module

Parameters	Three-layer problem
No. of backcalculated layer moduli	3
Size of population	15
Maximum generations	140
Probability of crossover	0.50
Single-point crossover	off
Uniform crossover	on
Elitism	on (fixed)
Range of possible moduli E_1 [MPa]	1000 to 8000
Range of possible moduli $E2$ [MPa]	10 to 400
Range of possible moduli $E3$ [MPa]	5 to 150
Precision level [MPa]	0.01/0.01/0.01

The obtained results from the µGA module are compared with those from the loaded GA module in Table 4-8 to evaluate their performance. It can be seen in the Table 4-8 that the loaded GA module offers slightly better results by producing less root-mean-square error indicating the better overall matching of basins and also yield less maximum percentage error (ΔM) of the backcalculated moduli. However, the results obtained from the µGA can be considered as equally satisfactory and they are still better than most of the results obtained form the other conventional programs used in the former comparison (see Table 4-4).

Table 4-8. Comparing the backcalculated layer moduli of problem 1, loaded GA vs. micro GA

GAMLET module	Backcalculated Modulus [MPa]			ΔM (%)	RMSE (%)
	AC	Base	Subgrade		
Input Modulus values	3447.38	310.26	51.71	-	-
Loaded GA	3457.62	307.35	51.72	0.94	0.014
Micro GA	3457.59	307.01	51.72	1.05	0.016



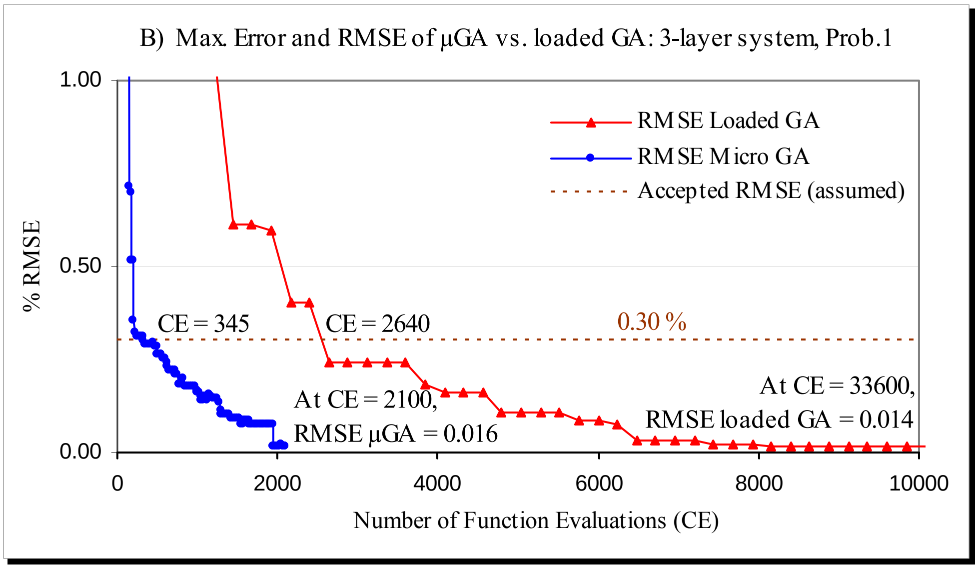


Figure 4-5. Comparing the convergence behavior of the Loaded- and Micro GA (Prob.1)

The more interesting is the number of function evaluations (or computational effort, CE) used in both GA modules. Figure 4-5 A and B show the comparison of fitness values and convergence behavior of RMSE values obtained from both models. Note that the same values of the loaded GA from Figure 4-4 are replotted in Figure 4-5 but this time not along the number of generations but number of function evaluations or CE (CE = population size × generation number) and the plot area is restricted at only 10,000 times of evaluation in order to make the comparison more evident. With the exception of the average fitness value, all the convergence values obtained from μGA are plotted in these figures. It should be noted that the average population fitness values are not meaningful with the concept of μGA because of the start-restart nature in evolution process of the μGA. Hence, only the best fitness values and RMSE values of μGA are plotted and discussed.

In Figure 4-5, it is evident that the μGA module demonstrated obviously faster convergence to the near-optimal region than did the loaded GA module. From all 140 generations, the loaded GA module archived the best fitness value of 0.98798 associated with RMSE=0.014% by using the computational effort as many as 33,600 times. From the same maximum number of generations, the μGA module achieved the best fitness value of 0.98208 associated with RMSE=0.016% by using the computational effort only 2,100 times. The same trend of this faster convergence of the μGA can also be found in the cases of four-layer pavement systems as shown in the appendix B.

To make this comparison more clearly, assuming that RMSE = 0.30% is used as an acceptable error. This threshold value is illustrated as horizontal dotted line in Figure 4-5 B. The loaded GA module yielded the first RMSE which is less than the assumed acceptable error with the value of 0.24% with the CE value of 2,640 times while the μGA module yielded the first RMSE which is also less than the assumed acceptable error with the value of 0.29% with the CE value of only 345 times or approximately eight times less than those obtained from the loaded GA module. For the cases of four-layer flexible pavement systems in problem 2 and 3 (see appendix B), the μGA converged roughly four times and nine times, respectively, faster than did the loaded GA module.

4.7 Discussions

GAMLET has shown thus far the capability in exploring the entire search domain of backcalculation problem to locate the optimal solution. The combination of many new GA operators and techniques in the loaded GA module has provided the robustness of the GA search process by illustrating the good increase in average fitness values along the generations. Moreover, this module has demonstrated the consistency in the accuracies of the solutions compare with other traditional computer programs.

However, just as in the other existing GA-based backcalculation programs, the number of required function evaluations in the loaded GA module is relatively high and hence long computation time is required which, in turn, makes such GA model less appealing for routine backcalculation analysis. Additionally, although the new added GA operators could make the search process more robust, it induces at the same time more difficulties for the practitioner to find the optimal set of GA parameters. The loaded GA module is therefore suitable for backcalculation by experienced users and for analyzing the deflection basin data at project level where only few basins involved and high result accuracy is desired.

The GAMLET-μGA module has been used in order to overcome the problems encountered in the existing GA-based backcalculation programs. It can be clearly seen from the backcalculation problems analyzed in this chapter that μGA accomplished the first goal by eliminating many GA operators in process which, in turn, makes itself more user-friendly. In addition, μGA was still able to yield good estimates of pavement layer moduli by using the number of function evaluations less than those used in the loaded GA and the existing GA-based backcalculation programs by many times. The computing time required in μGA is hence automatically reduced with the same proportion. It can be concluded that the GAMLET-μGA module has improved the efficiency of the GA-based backcalculation program and make GAMLET computer program practical for routine backcalculation analysis.

It is in this stage interesting to note that most of the performance evaluations of the existing GA-based backcalculation programs reported in literature [[2], [28], [51], and [77]] have been performed by doing backcalculation only with the deflection basins obtained from pavement models in forward computer programs. Obtaining backcalculated results with high accuracy from this evaluation manner does not make any surprise since the input pavement models match the assumptions of the forward theory perfectly. It is very possible that the performance of all backcalculation programs will be reduced in backcalculating the in situ deflection data where many sources of errors and variations of pavement model are involved. By this reason, the performance and efficiency of GAMLET should also be evaluated with backcalculating the in situ FWD deflection data.

Before that, the problem of setting up the input pavement model which is one of the main problems in the field of backcalculation using in situ data should be investigated in order to improve the input pavement model parameters by setting it more realistic. Problems in setting up pavement model include effect of layer thicknesses which are an independent major sources of errors and variations, bedrock depth and bedrock stiffness (if exists), and the characterization of nonlinearity in subgrade materials. Many issues related to these problems are discussed in chapter 5 and 6. A new method (CSCM) to deal with these problems which has already been added as one optional function in GAMLET are proposed and described in chapter 6.

5 Methods for Determining Depth to Bedrock

From the discussion in the section 1.1.2, it has been found that one issue needed to be investigated in the area of backcalculation is the methods to determine the depth to bedrock (DTB) under the pavement structure from in situ FWD deflection data.

As reported by several researchers, the existence of bedrock at shallow depths can have a profound effect on the backcalculation analysis. Rohde and Smith [68] have stated that failure to consider the DTB in the backcalculation process can result in non-conservative designs for new and rehabilitative pavement projects or overlay thickness calculations. It has also been found in the theoretical analysis by Briggs and Nazarian [8] that the assumed subgrade thickness does influence the backcalculation results. For example, if a DTB is assumed to be twice or more its actual depth in an analysis, the backcalculated moduli for the base and subgrade would in no way resemble their actual values. In addition, it has been documented by Chang et al. [15] that when shallow bedrock (less than 10 feet) is encountered, the backcalculated subgrade modulus can have a significant error if the DTB is incorrectly entered. Furthermore, Irwin [46] has described that when bedrock occurs at a depth less than 12 m or so, its presence will be need to be considered, otherwise the backcalculated moduli may be greatly in error. The worst situation probably occurs if bedrock is very shallow but it is not included at all in setting up the pavement model. This situation leads to a deceptive result and it can have a substantial effect on the structural evaluation of the pavement, the rehabilitation strategy and overlay thickness.

5.1 Existing Methods for Determining DTB

Referring the definition of bedrock in the field of backcalculation work used by Chen [17] that bedrock is a rigid layer (high modulus) beneath the pavement which is thicker than 0.3 m, the existing methods for determining DTB are summarized and some of them are investigated in this chapter.

5.1.1 Review of existing procedures

Based on different principles some procedures for predicting the DTB using in situ FWD deflection data have been developed. Chou [18] has suggested that the DTB could be estimated by performing several backcalculations assuming various bedrock depths. The depth that minimizes the deflection matching error or RMSE (see eq(2-4)) can be associated with the actual bedrock depth. In other words, the result is determined by finding the bedrock depth and set of moduli associated with the smallest RMSE.

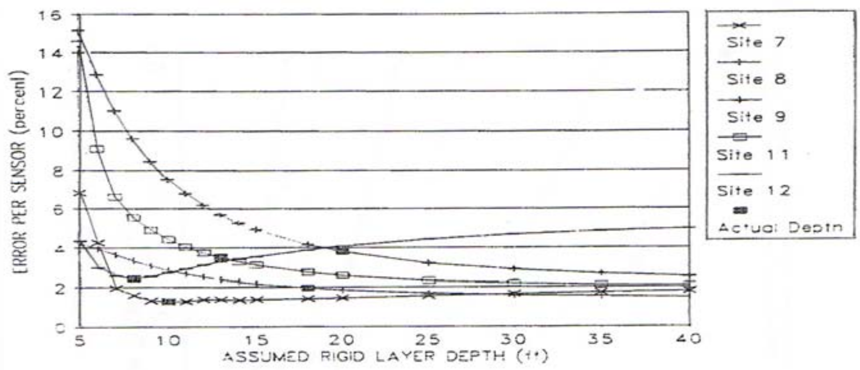
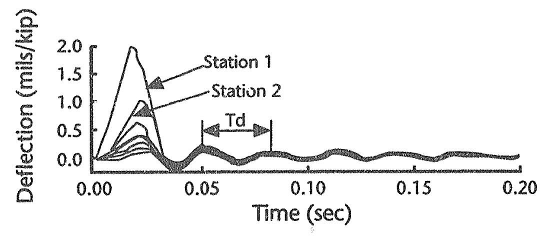
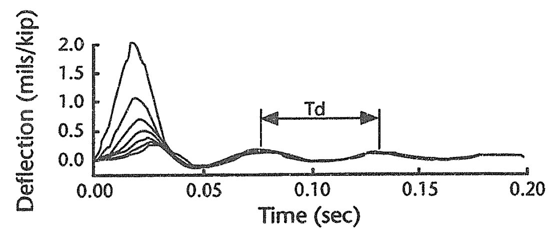


Figure 5-1. Estimating bedrock depths by minimizing the RMSE proposed by Chou [68]

This approach has been investigated by Rohde and Smith [68] with in situ FWD deflection data collected on 5 sites in Texas, USA. Using backcalculation program MODULUS 3.0, the deflection basins were analyzed using several subgrade thicknesses. The actual DTB for each site have been determined from seismic and penetration test. It can be seen from the results shown in Figure 5-1 that the minimum error per sensor does not always correspond with the actual bedrock depth.



(a) Depth to bedrock = 1.5 m



(a) Depth to bedrock = 3.1 m

Figure 5-2. Deflection-time histories in FWD testing and damped natural periods [72]

In 1993, Chang et al. [15] proposed a procedure for predicting the DTB based on damped natural period of free vibrations (*Td*) of the pavement system immediately after the FWD load application. Deflection basins in that study were generated from computer program UTFWD for both simulated dynamic (8 Hz) and static (0.75 Hz) loadings. This computer program is based on linear elastic approach and uses a Green's flexibility influence function to simulate dynamic response corresponding to a vertical disk load applied on a simplified pavement system. Figure 5-2 illustrates the damped natural periods in the time-deflection records obtained from simulated FWD test with various shallow DTBs.

This method has been later examined by Seng et al. [71] with various subgrade stiffness values and saturation conditions under four selected typical in-service Texas highways. The degrees of subgrade saturation condition were simulated by varying value of Poisson's ratio. Two simplified equations for estimating DTB with different subgrade saturation conditions for the flexible pavement are shown below.

For unsaturated subgrade:

$$D_b = \frac{Td}{(8.21 - 5.86\mu)}\sqrt{\frac{E}{(1+\mu)}} \tag{5-1}$$

For saturated subgrade:

$$D_b = \frac{\sqrt{E}}{7.0}Td \tag{5-2}$$

where D_b is depth to bedrock, defined as the total depth from the top of the pavement to the top of the bedrock in [ft], Td is damped natural period of free vibration in [sec], E is modulus of elasticity in [psf], and μ is Poisson's ratio [-].

Similar investigation was done by Roesset in 1998 [67]. Using FWD deflection data obtained from different pavement models which are simulated in a computer program, a general equation for predicting the DTB was proposed as following:

$$D_b = \frac{Vs * Td}{2.22} \tag{5-3}$$

where Vs is shear wave velocity of subgrade in [fps].

Unfortunately, it has been later found that such approaches are not practicable for real FWD testing. To attain the damped natural period, the measuring time period in FWD testing must be extended until the second and third peak of deflection value appear. At this period of time the rebound of weight mass on the rubber pad will quickly occur. Hence, the deflection data from in situ FWD test in this time period are not free from vibration of pavement structure anymore [67]. Therefore, such approaches have not been applied to FWD deflection data analysis.

5.1.2 Deflection and Inverse Offset method

Another approach for predicting DTB has been proposed by Ullidtz et al.[80]. Figure 5-3 shows a fundamental schematic diagram of a typical three-layer flexible pavement system deflected under a FWD load. The conical zone in the figure represents the stress zone generated by FWD load. The slope of this stress zone varies from layer to layer. It can be noted that the stiffer the layer, the wider the stress distribution.

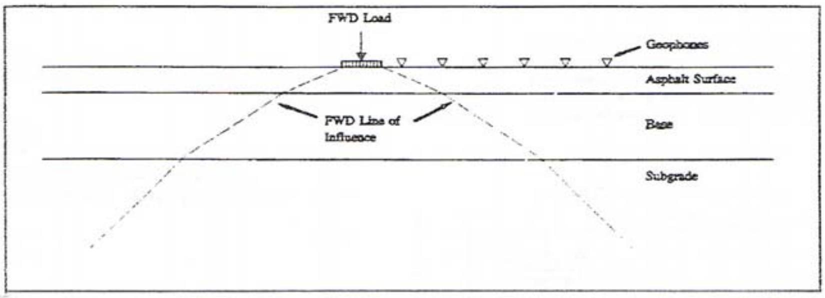


Figure 5-3. A schematic of the stress distribution below a FWD load [68]

Since it is assumed that the area above the stress zone is not affected by the load, the measured surface deflection is purely a result of the deformation of the material in the stress zone. The method to predict the DTB is based on the hypothesis that the position of zero surface deflection should be strongly related to the depth in the pavement at which no deflection occurs, i.e. an apparent stiff layer or bedrock. In addition, Ullidtz has compared the computed deflection basins of a semi-infinite halfspace deflected under a point load and uniform circular load (d=300 mm) as shown in Figure 5-4.

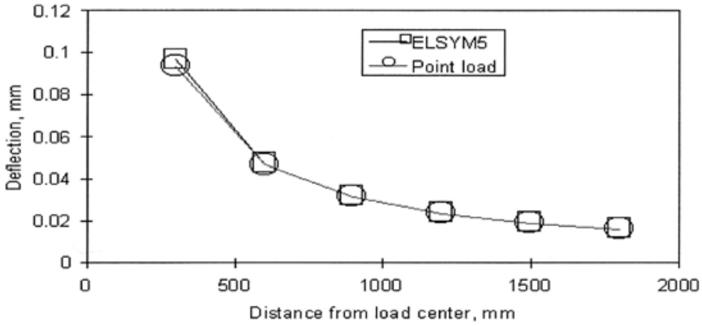


Figure 5-4. Comparison of uniformly distributed load to point load [78]

It can be seen that at distance of just one diameter of uniform load, the deflections produced by a point load are quite close to those from uniform load. It was therefore suggested to employ the Boussinesq's equation for surface deflection from a point load on a semi-infinite halfspace as shown in the eq (2-3) for prediction the position of zero surface deflection. The eq (2-3) is rewritten below.

$$d_r = \frac{(1-\mu^2) * P}{\pi * E * r} \tag{5-4}$$

As discussed in the section 2.2.1.1 that there is in eq(5-4) a relationship between surface deflection, (d_r), and reciprocal of radius at which the deflection occurs ($1/r$). Ullidtz has described that on a semi-infinite half space the deflection at a depth equal to the distance from the load is almost equal to the deflection at the surface. Within 45 degrees there may be a slight compression or extension, depending on Poisson's ratio, but it should be small compared to the compression of the material below 45 degrees.

Therefore if the bedrock is present, the approximated DTB may be found by plotting the deflections against the inverse of distance from the load center (inverse offset). The tangent to the plotted values intersects the abscissa should indicate the reciprocal of the approximated DTB. The concept of this graphical method illustrated in Figure 5-5 has been used in the backcalculation program ELMOD [25]. It may be interesting to note that the concept of straight line with 45 degrees from Ullidtz is not agreed with the "two-third" rule proposed by Irwin [47]. Irwin has pointed out that the measured surface deflections are attributable to compression occurring in layers below a line that can be approximated by a straight line with a 34 degrees angle from surface.

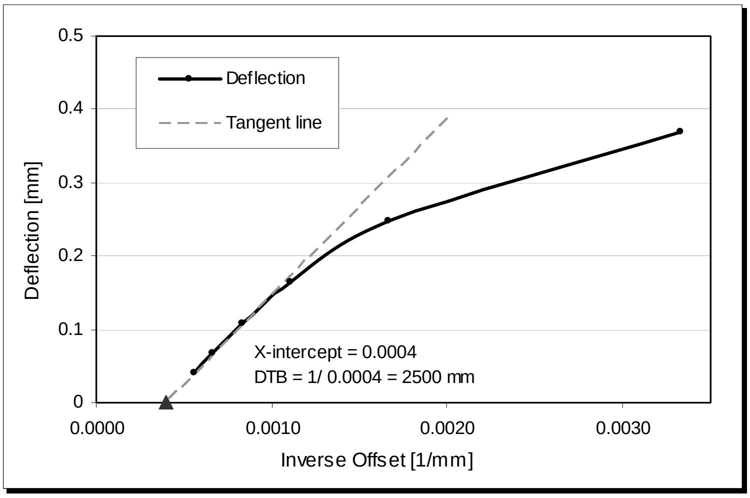


Figure 5-5. Deflection vs.1/r, replotted afte r Ullidtz [78]

The deflection and inverse offset method has been examined by Rohde and Smith [68] using deflection basins calculated from computer program BISAR. Deflections for a number of pavement models have been calculated and plotted against the inverse offset in Figure 5-6. The load level, pavement structures, and material properties used are also shown in this figure.

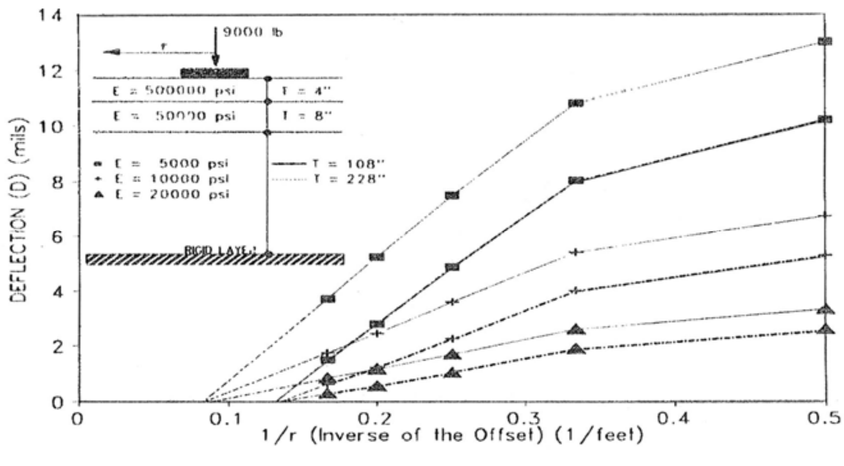


Figure 5-6. Deflection vs. 1/r for a number of Hypothetical Pavement Structures [68]

It can be seen that when the subgrade modulus is changed, the slope of the line changes but the intercept with the 1/r axis remains relatively constant. It can also be noted that the deeper the bedrock, the smaller the intercept. Although it can be concluded from Figure 5-5 and Figure 5-6 that this graphical method performs well with artificial deflection basins from pavement models, Rohde and Smith [68] have found that the intercept obtained from this method is also influenced by the stiffnesses and thicknesses of the upper layers.

5.1.3 Regression Equations

Based on the deflection and inverse offset method, Rohde and Smith have developed a method to approximate the DTB. Deflection basins and 1/r intercepts have been generated for 1008 pavement models under a 9000 lb (40 kN) and the bedrock is assumed as 100 times stiffer than the subgrade. In the analysis of relationship between the DTB and the 1/r intercept, a set of regression equations has been completed and also improved by taking the stiffnesses and thicknesses of the upper layers into account. The basin shape factors *SCI*, *BCI* and *BDI* have been used for that purpose. These factors represent in global terms of the thickness and stiffness of the upper layer [82]. Four separate equations based on the asphalt layer thickness have been proposed for determining DTB. These equations are shown below:

For pavement with asphalt surface layers less than 2 inches (≈ 50 mm.);

$$\frac{1}{B} = 0.0362 - 0.3242 r_0 + 10.2717 r_0^2 - 23.6609 r_0^3 - 0.0037 BCI \tag{5-5}$$

For pavement with asphalt surface layers between 2 and 4 inches (≈ 50 and 100mm.);

$$\frac{1}{B} = 0.0065 + 0.1652 r_0 + 5.4289 r_0^2 - 11.0026 r_0^3 + 0.0004 BDI \tag{5-6}$$

For pavement with asphalt surface layers between 4 and 6 inches (≈ 100 and 150mm.);

$$\frac{1}{B} = 0.0413 + 0.9929 r_0 - 0.0012 SCI + 0.0063 BDI - 0.0778 \log(BCI) \tag{5-7}$$

For pavement with asphalt surface layers greater than 6 inches (≈ 150mm.);

$$\frac{1}{B} = 0.0409 + 0.5669 r_0 + 3.0137 r_0^2 + 0.0033 BDI - 0.0665 \log(BCI) \tag{5-8}$$

where B is depth to bedrock [ft.], r_0 is $1/r$ intercept by extrapolating the steepest section of the deflection vs. $1/r$ curve [1/ft.], *SCI* (Surface Curvature Index) = d_0-$d_{12\ inches}$, *BDI* (Base Damage Index) = d_{12}-$d_{24\ inches}$, *BCI* (Base Curvature Index) = d_{24}-$d_{36\ \mathrm{inches}}$, and d_i is surface deflection at sensor i, normalized to a 9000 lb. load [inches*10^{-3}]

Since the modulus value of the subgrade has been modeled as linear elastic in program BISAR but in the reality it is often stress sensitive, Rohde and Smith have described that the curves plotted from actual surface deflections are not linear at the outer sensors but have an S-shape as shown in Figure 5-7 (a). It has been postulated that the deflections near the point of the steepest slope reflect the weakest modulus, normally found near the top of the unmodified subgrade. The $1/r$ intercept of a line drawn through the point of steepest curvature should be used in the eq(5-5) to (5-8). These regression equations have been employed for estimating the DTB in the backcalculation program MODULUS 4.0 till 4.2.

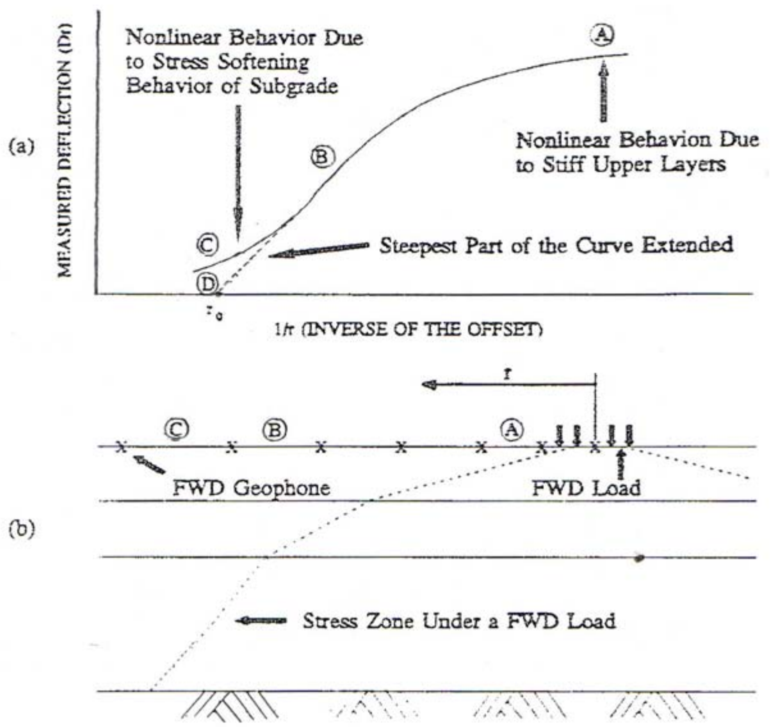


Figure 5-7. The Illustration of the Method to Determine the Effective Depth to a Rigid Layer

Later on, the regression equations shown above have been examined by Uzan [82] and Chen [17]. Some major drawbacks of this procedure have been stated. First, it requires that four sensors have to be placed at 0, 0.3, 0.6 and 0.9 m. Second, the intercept is sensitive to measurement errors, material segregation and cracking which greatly affects the slope of the segments d-$1/r$. Third, the calculated DTB changes substantially when two of the four equations are used, i.e. using the AC thickness corresponding to the transition from one equation to another. This indicates that the reliability of the regression equations may not be accurate enough to predict the DTB. Using generated deflection basins, Uzan [82] has shown a comparison between the predicted DTBs obtained from the regression equations and the input DTBs in Figure 5-8. It is seen that the regression equations performed not well enough in predicting the DTB (for artificial basin).

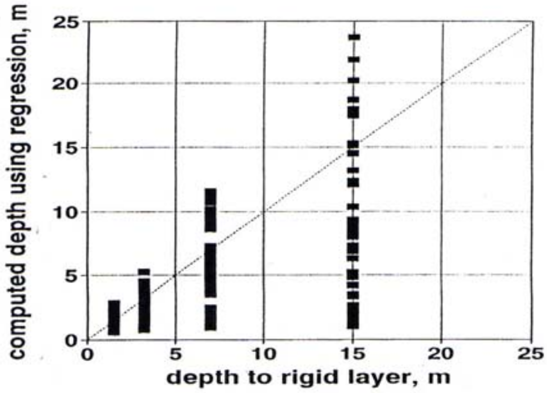


Figure 5-8. Predicted using computed DTB vs.the regression equations [82]

5.1.4 Method based on database

Uzan [82] has proposed another procedure for estimating thickness of subgrade which in turn indicates DTB using a-priori-knowledge of relative stiffness from user. Modular ratio between upper and lower "stiff" infinite subgrade has to be assigned with one of these values: 5, 10, 100 and 1000 by the user. A database using these four different modular ratios include surface deflections at nine radial distances for 21840 cases for four layer systems (two upper pavement layers, a finite subgrade and an infinite subgrade layers) has been made available. A computer program generates a working database for the specific upper-lower subgrade modular ratio and pavement layer thicknesses. The database is then used in search pattern procedure with an error minimization method to determine, for each deflection bowl, four unknowns: the thickness and modulus of the subgrade and the moduli of the two pavement layers.

In order to evaluate accuracy of this procedure, 140 runs in forward program WESLEA have been conducted using four different values of depth to rigid layer. Figure 5-9 presents a comparison between the DTBs used in generating the bowls and the back calculated one using the method based on database.

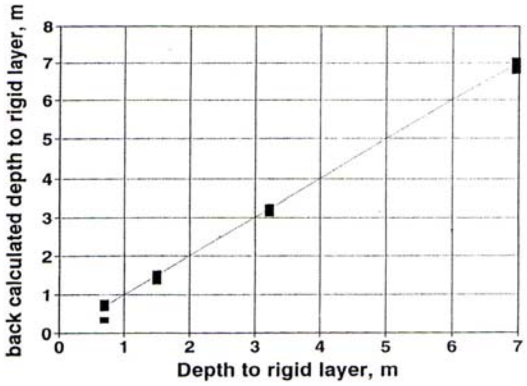


Figure 5-9. Predicted using computed depth to bedrock vs. method based on database [82]

It is obvious that the procedure proposed by Uzan was capable of predicting the DTB from artificial deflection basins and yielded better accuracy than the regression equations. Another comparison between these two methods has also been performed using the same in situ FWD data reported in Rohde and Smith's work. It was found that the results are in some cases different. The comparison between the results from borings and cone penetrations seemed to be in a favor of the predictions using the method proposed by Uzan. This method has been employed for predicting DTB in backcalculation program MODULUS 4.3.

Yet there are several drawbacks of this method based on database. First, it requires the broad knowledge of the modular ratio of the upper to the lower subgrade layer from the user which may be difficult, if possible, to know. Second, the value of only 7.62 m set in the program for the maximum DTB leads to some doubt of reliability of the obtained results. Third, the system is restricted to only three pavement layer systems, the user need engineering judgment to combine judiciously any adjacent layers of similar stiffness to come up with a total of only three layers including the finite subgrade. Fourth, this method cannot be used in general since the whole process needed the pre-generated database.

5.1.5 Other methods

In the last decade many researchers have given attempt to develop new approaches to predict DTB beneath the pavement structure. These new developed approaches are based on disparate principles, e.g. using combination of FWD deflections with other nondestructive test equipments such as seismic refraction data [61] or spectral analysis of surface waves measurements [4]. The numerical method such as artificial neural network is also applied in this field [14]. All of these approaches will be not discussed in this work since, as mentioned earlier, the purpose of this work is indicating or developing the most appropriate method for determining DTB which is practical and compatible with MLET-based backcalculation programs. The iterative backcalculation program developed in the Pavement Engineering Section VAHREN1 and the new developed GA-based program GAMLET are, of course, included in this category.

5.2 Investigation of Selected Existing Methods

Among the procedures for determining DTB from FWD deflection data discussed above, it is obvious that the deflection and inverse offset method and the regression equations are practical procedures. The DTB can be determined directly from the FWD deflection data without any broad knowledge and additional test. This feature is suitable for analysis deflection data at every testing point. These two procedures are therefore selected here to be examined with the deflection data obtained from both pavement models in forward computer program and from the in situ FWD data.

5.2.1 Examine selected methods with pavement models

To evaluate the selected methods, a number of three-layer pavement models are used. These models consist of an upper asphalt layer, an intermediate unbound aggregate layer (combined base, subbase and subgrade) and bedrock at various depths as illustrated in Figure 5-10.

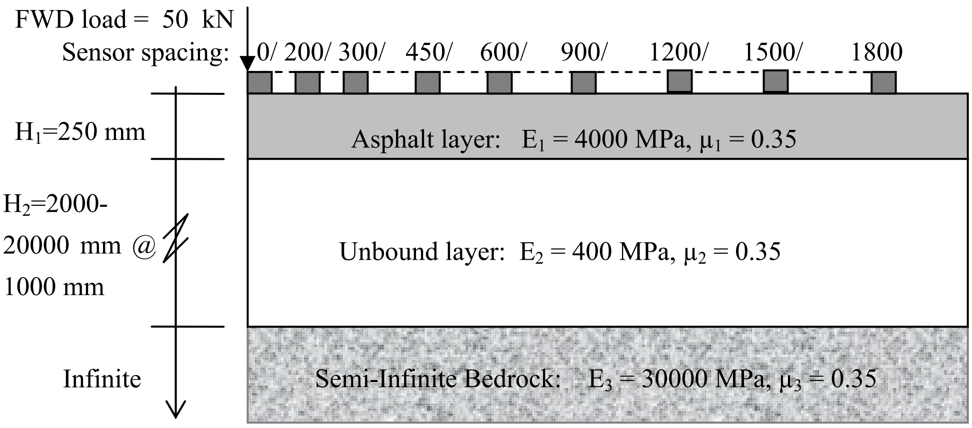


Figure 5-10. Model of Pavement systems with bedrock at various depths

Figure 5-10 shows that the simplified constant value of 0.35 is assigned for Poisson's ratio of all layers. The bedrock is generated by assigning a high modulus value equivalent to a typical concrete slab, ca. 30,000 MPa. This bedrock lies under pavement surface at dept varies from 2.25 m to 20.25 m.

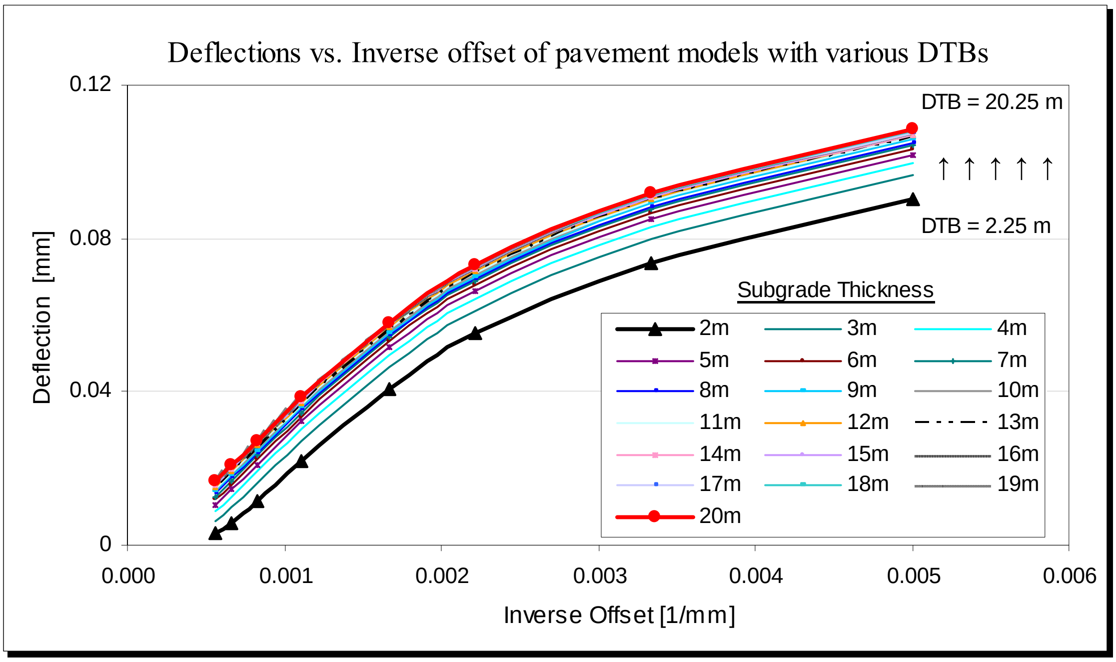


Figure 5-11. Influence of bedrock on calculated surface deflections

The surface deflections are then calculated by computer program BISAR and plotted against the inverse of offset distance as shown in Figure 5-11. At the first glance the graph shows a good trend of this procedure by delamination of the curves. The principle “the deeper the bedrock, the smaller the intercept” can be recognized from this graph. It is evident that the deflections generated in BISAR are highly influenced by the DTB or the thickness of subgrade layer. This emphasizes that the deflections analysis may be greatly in error if bedrock is present under the testing point but it is not included in the pavement model.

5.2.1.1 Examine the deflection and inverse offset method

According to the principle of the deflection and inverse offset method, DTB can be approximated by extending the curve with a tangent line of the deflections at the most outer sensors until the horizontal axis intercept is found. The desired DTB can be then computed from an inverse of intercept value as explained in section 5.1.2. It is worthy to remind that it has to be interpreted as no bedrock is present if the intercept yields zero or any negative values.

The DTBs associated with each curves illustrated in Figure 5-11 according to this method are determined. Percent errors of the results are calculated and illustrated in Figure 5-12 with the black bar. Additionally, results obtained from tangent lines at other pairs of adjacent sensors based on suggestions of some researchers are also shown in the same figure. Considering the DTBs obtained from the outmost tangent line, i.e. sensors at 1500&1800 mm, it can be seen that the results yield good estimates only when the bedrock lies at shallow depths (about 2 or 3 m.). After that the accuracies gradually decreases with the increase of DTB. The magnitude of error is already greater than 30% when bedrock lies at the depth of 6 m.

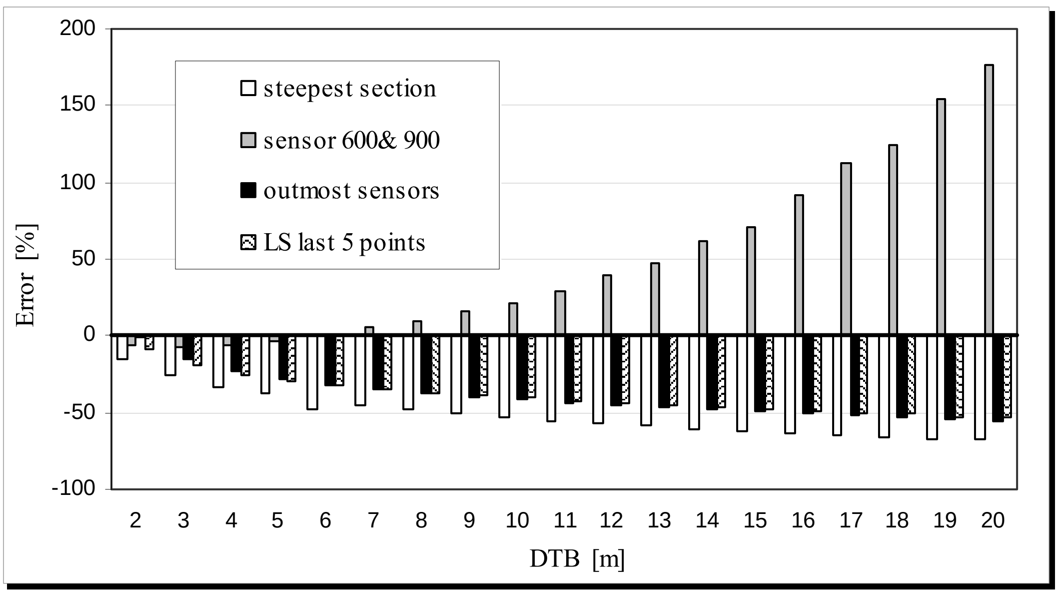


Figure 5-12. Accuracy of calculated DTB from D vs 1/r with different tangent lines

Rohde and Smith [68] have recommended that the tangent line at the steepest section of such curve should be used to approximate the DTB if the of subgrade is stress-sensitive. Although this is not the case because the subgrade moduli are assigned with constant values, the results from the tangent lines at the steepest section of each curve are determined to compare with the others. As is apparent, they delivered the same trend as the results obtained from the outmost tangent line but deplorably more inaccurate.

Astoundingly, the results obtained from the tangent line at sensor pair 600&900 mm delivers better accuracies than the others. Unfortunately, this occurrence appears only when bedrock lies at shallow depths (up to 7 m). Beyond this value, the error value increases very fast and achieves the greatest magnitude of more than 150% when bedrock lies at depth 19 m.

Evidently, the DTB obtained from tangent lines of a deflection vs. 1/offset curve are very sensitive. It is strongly dependent on the selected part of curve section or specified adjacent sensor pair. In order to avoid this sensitivity, only deflections at the last 5 sensors, which most further from the load center where the Boussinesq's equation is the most applicable, are considered. Least square technique is applied to determine the most matching linear of each curve. The intercepts and DTBs are determined and the result accuracies are plotted in the same graph. Once again, the results show the same trend as those from the outmost tangent line and the accuracies are not better than the others.

It can be concluded that the deflection and inverse offset method does not work well in approximating the DTB based on the deflection basins generated in computer program BISAR. This is probably due to that the intercept value in this method is influenced by the stiffness and thickness of the upper layers as described by Rohde and Smith.

5.2.1.2 Examine the regression equations

The performance of the regression equations in determining DTB had already been examined by Uzan as shown in Figure 5-8 and found that it performed not so well in predicting the DTB from artificial deflection basins. The examination was performed by comparing the predicted DTB obtained from the regression equations with the input DTB in the pavement model. These models had the values of thicknesses and modular ratio within the range for which the regression equations were developed. Since the three-pavement models used in examination of the deflection and inverse offset method have the parameters out of the range defined for the regression equations, it may be interesting to investigate the performance of the equations with these models.

Note that the modulus value assigned for bedrock in the three-layer pavement systems is 30000 MPa which is still realistic for natural rock. This bedrock is not 100 times stiffer than subgrade modulus as defined for the regression equations but only 75 times. The deflection basins must be first normalized to a 9000 lb (ca. 40 kN) as initial assumption. The DTBs are then calculated by using the intercept values obtained by extrapolating the steepest section of each curve. Although it is clear that eq(5-8) should be used since the thickness of asphalt layer is greater than 150 mm, all the four regression equations are evaluated in order to compare with each others. The predicted DTBs using regression equations versus assigning DTBs in the pavement models are illustrated in Figure 5-13.

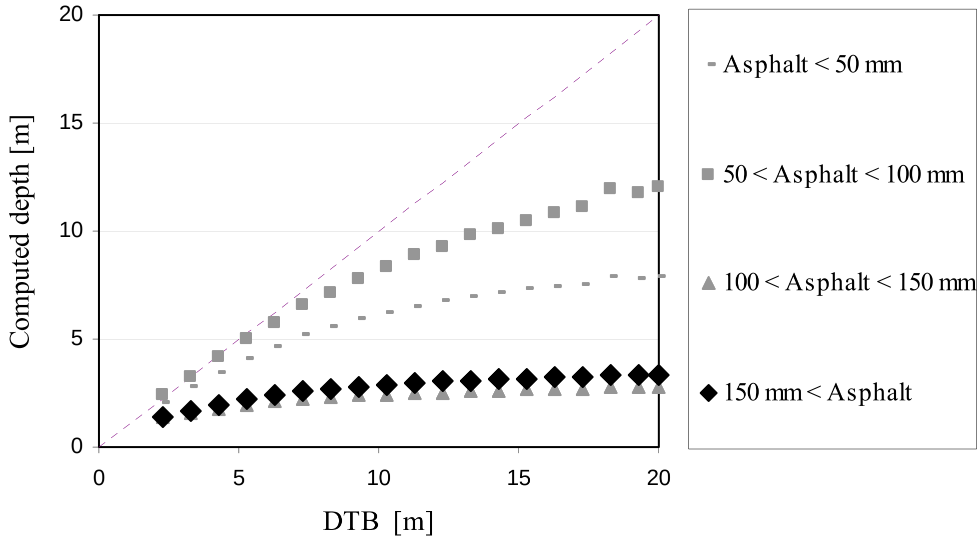


Figure 5-13. Accuracy of DTBs obtained from regression equations

The results of all equations show the same trend that most of the computed depths are less than the assigning depths. In addition, the magnitude of error always increases with increasing of DTB or subgrade thickness. Astoundingly, the results from regression equation for pavement with asphalt layer greater than 150 mm delivered almost the worst accuracy comparing with the others. The eq(5-6) which is developed for pavement with asphalt surfaces between 50 and 100 mm delivered instead the best accuracy in this case. Apparently, approximating the DTB with these regression equations is not reliable enough to analyze the deflection basins from artificial pavement systems. Moreover, the computed depths can possibly show more inaccurate if this procedure is applied with pavement systems which have parameters out of ranges for which they were developed.

5.2.2 Examine selected methods with in situ FWD data

It is clear that the best way to examine the accuracy of any procedures for determining DTB is performing the procedure by using the in situ deflection data obtained from pavement sites which is constructed above the real bedrock at exactly known depth and, then, compare the results from the procedure with the real DTB. Since beginning of 2004 the Federal Highway Research Institute (in German: Bundesanstalt für Straßenwesen, BASt) opened a new test track in Bergisch Gladbach, Germany. The aim of this test track is to investigate and monitor the behavior in short and long term of the typical standard road in Germany using the road model which has the most similarity of mechanical parameters and environmental system as the real one. Some important detail of this test track is summarized here.

The test track at BASt was built in a large dimension of concrete tank as is apparent in Figure 5-14. The detailed dimensions of the concrete tank both in longitudinal- and cross-section are also illustrated in the figure. It can be seen that the tank has dimensions of 38.0 m. long and 7.5 m width. Moreover, it is obvious that the depth from the surface to the bottom concrete slab varies between 3.0 m at the edges of the tank and the deepest point 3.5 m since the floor level is declined to the drainage point. At this point the inflow and outflow of water into the pavement can be managed in order to control the moisture content in the pavement structure. Doing this the behavior of pavement material under different moisture contents can be investigated.

This tank was equally divided in area to build up 8 different flexible pavement construction classes. All of these pavement prototypes illustrated in Figure 5-15 are selected from the German guidelines for the standardization of superstructure of road surfaces 2001 (in German: Richtlinien für die Standardisierung des Oberbaues von Verkehrsflächen, RStO´01) [6]. The whole test track is fully equipped with thermocouples, strain gages and load cells to monitor the behavior of pavement in short and long term. The layer thicknesses and materials used in each of these selected pavement construction classes are different. Figure 5-15 shows some important detail of each prototype. More information of each pavement construction class can be found either in RStO'01 [6] or in the report from BASt [9]. Some perspectives of this test track during the construction process are illustrated in appendix C.

It is very important to note that the thickness of the concrete tank floor is 1.5 m thick as illustrated in Figure 5-14 (b). According to the definition of bedrock in the backcalculation work, this value is much thicker than the bedrock thickness defined by Chen (0.3 m) [17]. Comparing with the stiffness of pavement material, the concrete slab has absolutely a very high stiffness. In case of this test track the silt soil was used as filled subgrade as can be seen in Figure 5-14 (a). The stiffness ratio between the silt subgrade to the concrete slab is considered as ideal for demonstrating the behavior of bedrock under pavement.

As a consequence of the above discussion, one might conclude that deflection behavior under FWD load of all the different eight pavement structures built at this test track should be closed to those of the real pavement underlain with bedrock at the same depth. With respect to the average value, the depth of 3.25 m is used as referential DTB in calculation process for this test track. The values between 3.0 to 3.5 m are considered anyway as the correct values of DTB.

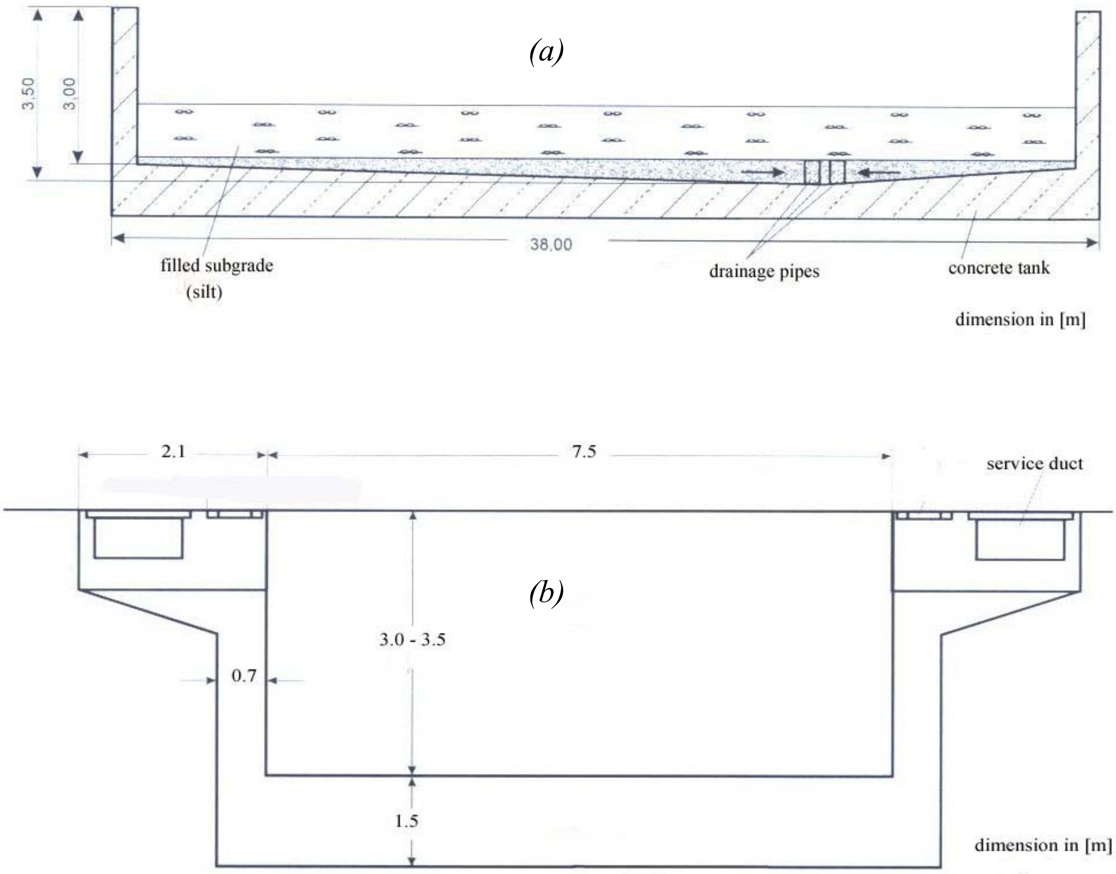


Figure 5-14. Dimensions of concrete tank in (a) longitudinal- and (b) cross-section

On February 2004, nondestructive deflection tests on the test track at BASt were implemented using FWD equipment from Pavement Engineering Section. These tests were conducted on the selected three different pavement construction classes and four different fields with the frequency of two testing points on each of field, altogether eight testing points. These eight selected testing points are illustrated with the circle symbols in Figure 5-15.

For research purposes, each testing point was tested in one day with 40 falling weight drops to minimize the random error and environmental effect. The asphalt surface and air temperatures are also measured. The obtained deflections are first normalized to FWD standard load (50 kN) for flexible pavement structure. The average deflections are then calculated and utilized in this investigation. Figure 5-16 shows the plot of deflections against the inverse offset. Apparently, the curves of deflection basin from test track show much less linear behavior at the outer part than those obtained from pavement models. The difference of deflection values close to the load center can be clearly seen due to the different stiffnesses of upper layers of each construction class.

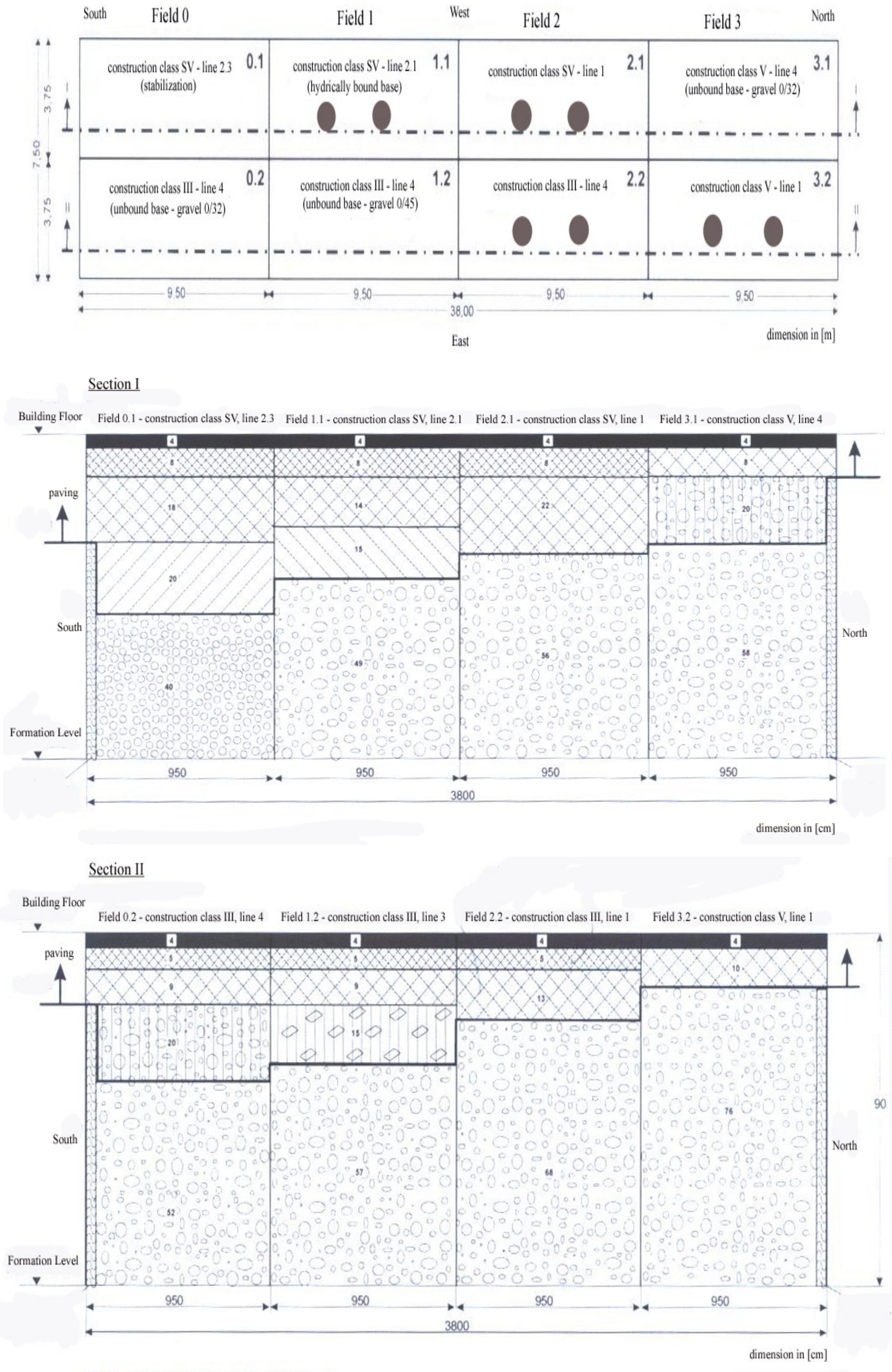


Figure 5-15. Top and cross-section view of test track at BASt and FWD testing points

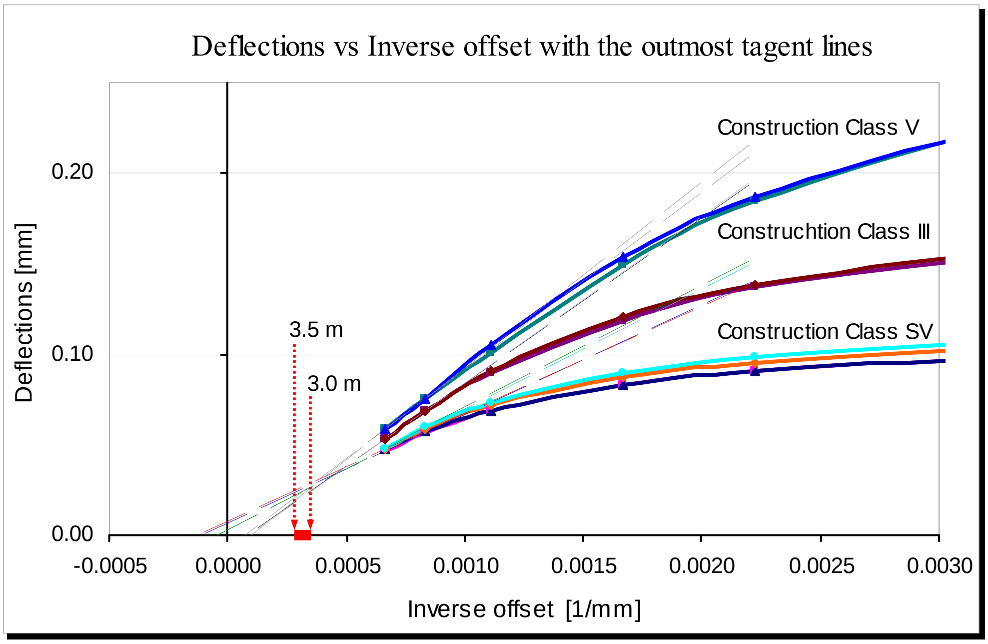


Figure 5-16. Results of DTBs obtained from test track at BASt using deflection and inverse offset method

As explained earlier, since all the pavement construction classes are constructed over the same concrete slab, the result of DTB acquired from each of these deflection curves should be close to each others. According to graphical method, all tangent lines should yield to x axis-intercept between the points of value 0.00033 or 0.0029 mm^{-1} corresponding to the depths of 3.0 and 3.5 m, respectively. Two dotted red vertical lines have been marked in the Figure 5-16 in order to show the desired portion on the horizontal axis. In other words, the horizontal-axis intercepts obtained from the tangent lines should be found somewhere within the red portion.

Unfortunately, the results obtained from sensors at distances 600 and 900 mm from the load which yielded the best accuracy using pavement models with shallow bedrock show in this case only the negative intercept values. These results pointed out so badly that there were no present of bedrock (concrete slab) at all. By using the least square technique through the five outer points, the results were not much better than the former one. Both of them are, therefore, not illustrated in the figure.

The tangent lines shown in the Figure 5-16 were generated from the outermost sensor pair. They delivered the best results of DTB in analyzing deflections from this test track among all tangent lines mentioned. However, it is evident that there was no intercept found between two vertical straight lines which are the expected portion. The acquired DTBs were much deeper than the depth to concrete slab under the test track. Moreover, the results obtained from FWD data tested at the pavement structure of class SV have yielded only negative intercepts which mean no present of bedrock. All the results computed by the deflection and inverse offset method are summarized in Table 5-1. In can be seen that the best result obtained from pavement class V with DTB = 10.2 m or about 214 % of error with respect to the average actual depth to concrete slab.

Table 5-1. Results of DTB using the deflection and inverse offset method based on FWD data at BASt

Construction Class	Testing Point	average DTB	best cal. DTB	Error [%]
SV Line 1	2.1.1	3.25 m	no bedrock	-
SV Line 1	2.1.2	3.25 m	no bedrock	-
SV Line 2.1	1.1.1	3.25 m	no bedrock	-
SV Line 2.1	1.1.2	3.25 m	no bedrock	-
III Line 1	2.2.1	3.25 m	11.82 m	264 %
III Line 1	2.2.2	3.25 m	11.48 m	253 %
V Line 1	3.2.1	3.25 m	13.69 m	321 %
V Line 1	3.2.2	3.25 m	10.19 m	214 %

Considering the DTB results in Table 5-1 with respect to their error magnitudes, they are easily judged as very poor for backcalculation work as stated in the work of Briggs and Nazarian [8] that if a bedrock is set to be twice or more its actual depth in an analysis, the backcalculated moduli for the base and subgrade would in no way resemble their actual values.

Table 5-2. Results of DTB using the regression equations based on FWD data at BASt

Construction Class	Testing Point	real DTB	cal. DTB	Error [%]
SV Line 1	2.1.1	3.25 m	no bedrock	-
SV Line 1	2.1.2	3.25 m	no bedrock	-
SV Line 2.1	1.1.1	3.25 m	no bedrock	-
SV Line 2.1	1.1.2	3.25 m	no bedrock	-
III Line 1	2.2.1	3.25 m	4.80 m	47.69%
III Line 1	2.2.2	3.25 m	4.82 m	48.31%
V Line 1	3.2.1	3.25 m	4.99 m	53.54%
V Line 1	3.2.2	3.25 m	4.44 m	36.62%

Since the regression equations use the horizontal axis-intercept from the deflection and inverse offset method as the basis value, these equations are suddenly inapplicable when the intercept is found at zero or any negative value. However, the results from the testing points that are still applicable (positive intercept values) for these regression equations are calculated and summarized in Table 5-2. A good trend of developing in accuracy can be seen from the results obtained from this procedure. Except the results obtained from the case of pavement construction class SV which yielded as no present of bedrock, the results are overestimated the real DTB with average of 46%. The best predicted DTB was found at the test point 3.2.2 of pavement construction class V with the error of 36.62%. However, the worst result was also found in this construction class at the point 3.2.1 with the error of 53.54% by indicating the depth of ca. 5 m from pavement surface to concrete slab.

5.3 Discussion

Thus far, two procedures for determining DTB from FWD deflection data have been selected based on their simplification and compatibility with multi-layered elastic theory. The accuracies of each procedure have been examined with the defection data obtained from computer program BISAR and from in situ FWD data. Both procedures did not show a good performance in predicting the DTB from artificial basins. In the case of predicting DTB using FWD data from test track, both procedures yielded not only inaccuracy results but they also yielded the results in many cases that bedrock (the concrete slab) were not existent at all under the test track.

The inaccuracies of these two procedures in case of artificial basin can be attributed to the influence of the stiffness and thickness of the upper layers since the subgrade material in the computer model matches the principle assumptions perfectly. For the case of analyzing in situ deflection data, the inaccuracies may not only be due to the influences of the asphalt layers but also to the influence of variations of subgrade materials in pavement structure.

Apparently, the procedures for determining DTB using FWD deflection data have a significant relationship with behavior of subgrade soil. Consider subgrade soil mechanistic properties under condition of real pavement, it can be concluded that type of soil, elastic modulus, Poisson’s ratio, water content, etc., can change along the road alignment in every directions and even all year round. By this reason, a new method for predicting DTB from in situ FWD deflection data which can take the behavior of subgrade materials into account and still keep the backcalculation procedure based on MLET practical for routine work should be developed.

6 A New Method for Determining DTAB

6.1 General Concept

It is well known that the stiffness of subgrade soil is seldom linear elastic but rather stress sensitive. Thus, using the layered elastic backcalculating program to analyze the pavement system with an assumed constant elastic modulus and semi-infinite in depth of subgrade to backcalculate the layer moduli ignores all the knowledge of those variations in subgrade soils. This can easily lead to the unreliable results such as an inverse pavement structure, e.g. the backcalculated base modulus is lower than the subgrade modulus. On the other hand, backcalculation using nonlinearity analysis is appropriate only when the subgrade soil properties are correctly explored. Since it is very difficult and also very expensive to explore all needed subgrade soil properties at every point and time when FWD test is conducted, the nonlinear analysis is not so practical for routine backcalculation analysis.

On the other hand, a technique of setting up pavement model for backcalculation based on MLET by adding artificial bedrock at some depth beneath the subgrade layer to deal with nonlinearity in subgrade materials has been used in some procedures. It has been found that the result obtained from backcalculation with such a technique can in most cases eliminate the problem of inverse pavement structures. Moreover, backcalculation using pavement model with artificial bedrock can improve the iteration convergence and deliver less deviated results in most iterative backcalculation programs [76]. These can be attributed to the fact that the stiffness of most subgrade soils increases with depth. In other words, this is due to a decrease in the load related deviatoric stress with increased distance from the load and an increase in confining stress caused by the weight of the materials with depth. The USCE [10] has recommended the use of value of 20 feet as depth to the artificial bedrock (DTAB) in all backcalculations. It is obvious that this recommendation seems not reliable since the depth of such layer (bedrock) is usually not only unknown, but also highly variable along the length of pavement section.

By the reasons discussed above, a new method should focus on analyzing the in situ deflection data to determine the depth at which the artificial bedrock can represent the effective behavior of both subgrade stiffening with depth and the real bedrock, if exists, rather than only determining the depth to real bedrock. The depth to artificial bedrock (DTAB) from this new method will be used to assign the artificial bedrock into the pavement model for backcalculation procedure based on MLET to deal with the nonlinearity of subgrade materials and real bedrock under the pavement structure. Some important issues related to the influence of bedrock (real and artificial) on the backcalculated moduli should be first investigated.

6.2 Influence of Bedrock Stiffness on Backcalculated Moduli

The important issues related to the influence of the stiffness of bedrock on the backcalculated moduli needed to be investigated are how stiff the artificial bedrock should be assigned to the pavement model and what the effect looks like if different bedrock stiffnesses are entered.

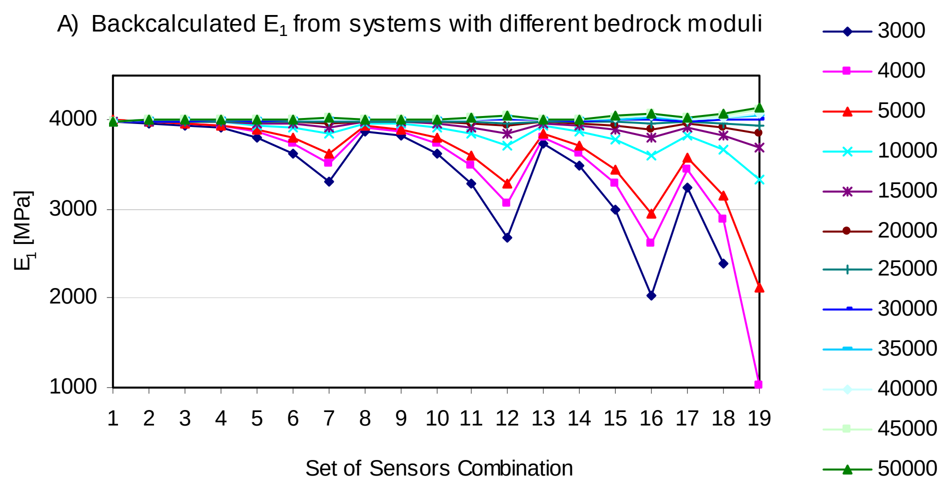
Of interest in this research are the backcalculation programs based on MLET. As mentioned earlier, a series of iterative backcalculation programs have been developed in Pavement Engineering Section. These programs are also based on MLET and will be used to evaluate the influence of the stiffness of artificial bedrock on backcalculated moduli because of two reasons. First, these programs have some appropriate features for examining the issues related to bedrock. Second, by the time of this investigation, the GA-based backcalculation program GAMLET was in the developing phase. Some important features about these iterative backcalculation programs are briefly explained.

For research purposes, computer programs DREIFFM and VAHREN1 have been developed [41] for backcalculating pavement layer moduli using FWD data. Both of them have been written in FORTRAN language to run on desktop computers and work as a “stand alone” program. Utilizing the computer program BISAR as forward model, stresses, strains, and deformations in the pavement are determined. The backcalculation process involves an iterative approach in which the layer moduli are systematically varied until the desired fit of specified points on the surface deflection data is achieved.

DREIFFM, the former version, was developed to handle a pavement system with 3 layers. In each backcalculation process, 3 systematically deflection values (from specified 3 sensors) are set as 1 combination and used to backcalculate 3 layer moduli. Consequently, if the FWD test is conducted with 8 sensors, there are 55 sets of layer moduli results for one deflection basin data. Unfortunately, these 55 sets of results always show a large variation of layer moduli especially the backcalculated uppermost layer moduli obtained from the combination of outer adjacent sensors. This can simply be attributed to the loading zone theory as illustrated in Figure 5-3. In other words, outer deflections are not the appropriate data for backcalculating the upper layers since they are originated from the lower layers. Using deflections from such combinations leads oft to failure of convergence.

In order to improve the performance of DREIFFM by reducing the variation of results and to study the influence of bedrock on backcalculated moduli, program VAHREN1 has been developed. It is generally based on the same principle as DREIFFM. The major difference between these two programs is that the artificial bedrock is always applied to pavement model using in VAHREN1. This means that the modulus of the third (lowest) layer in VAHREN1 is always set to relatively high value to serve as semi-infinite artificial bedrock and fixed in all iterations. Thus, it needs only 2 deflection values to set as 1 combination in each backcalculation process to find out the moduli of the two upper layers. In the same case of FWD test with 8 sensors, there are only 19 selected sets of sensors combination. It has been found that VAHREN1 show a significant reducing of variation in backcalculated results. Therefore, VAHREN1 is used in this investigation to evaluate the influences of bedrock stiffness on backcalculated layer moduli.

Two pavement systems with bedrock at shallow depth are selected from Figure 5-10 as the basis models in this investigation. They have the same upper layers as shown in the figure (E_1=4,000 MPa and E_2= 400 MPa) and bedrock at depth of 3.25 and 5.25 m respectively. By varying the modulus value of bedrock in the basis models, a number of deflection basins under FWD load from different models can be obtained from BISAR. The bedrock moduli are varied from 3000 MPa which recommended by Irwin [47] to a conceivable value of 50,000 MPa. Figure 6-1(a) and (b) present the backcalculated moduli of asphalt and base layer, respectively, using VAHREN1. The same trend of results from systems with DTB at 5.25 m can be seen in appendix D.



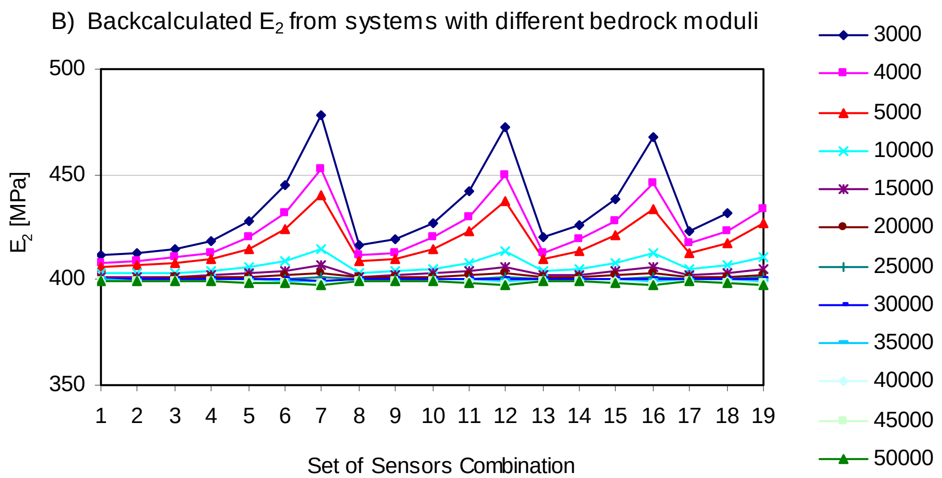


Figure 6-1. Influences of bedrock stiffness on backcalculated moduli using VAHREN1

As expected, the backcalculated moduli of both asphalt layer (E_1) and base (E_2) layers show a largest variation when the deflections from outermost sensor are used to backcalculate the upper layer moduli. It can be concluded that the more the sensor close to the load center, the more suitable it is for using in backcalculation of the upper layer moduli.

Considering the influence of bedrock stiffness shown in Figure 6-1, it is obvious that the results obtained from the pavement model with bedrock modulus of 3,000 MPa shows the largest variation. Furthermore, the result from this system at the 19th (the last) combination has so poor performance that the convergence was failed. This modulus value is therefore not appropriate to use in setting the stiffness of bedrock in backcalculation program VAHREN1. The results from the other pavement systems with stiffer bedrock moduli show gradually reducing of variation. Another conclusion which can be made from Figure 6-1 is that the best 5 combinations of sensors are the combination number 1 (d_1, d_2), 2 (d_1, d_3), 3 (d_1, d_4), 4 (d_1, d_5), and 8 (d_2, d_4) based on their least variations. To find out the most suitable bedrock stiffness for VAHREN1, the statistical parameter, standard deviation, defined in the following equation is used.

$$\sigma = \sqrt{\frac{1}{N} * \sum_{i}^{N} (x_i - \bar{x})^2} \tag{6-1}$$

where σ is the standard deviation, N is the number of data, and $\bar{x}$ is mean of the data.

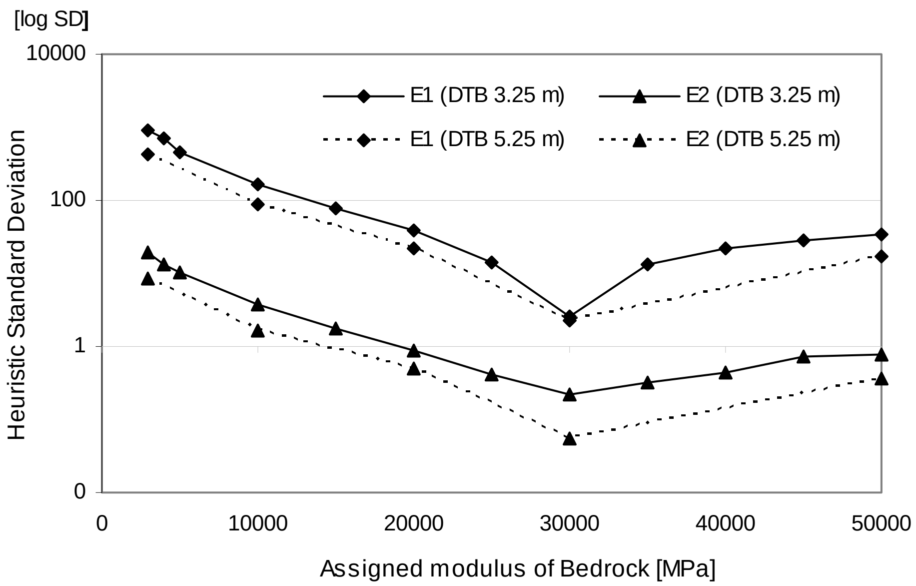


Figure 6-2. Standard deviation of backcalculated moduli from systems with different bedrock stiffness

The standard deviations of each set of backcalculated moduli are computed and plotted in Figure 6-2. Since the deviations of uppermost layer are quite substantial relative to the intermediate one, the values of the vertical axis in the Figure 6-2 are plotted in logarithmic scale against the assigned DTB. Apparently, the results obtained from the system with bedrock modulus of 30,000 MPa show the best performance with the deviation value of less than 3.0 for upper layer and less than 0.25 for intermediate layer. This can be deduced that if there is no supplement information about bedrock stiffness from the testing site, the modulus value of 30,000 MPa is recommendable to assign to the artificial bedrock in backcalculation program VAHREN1 or other iterative backcalculation programs to improve the result.

6.3 Limitation of Bedrock Influences on Backcalculated Moduli

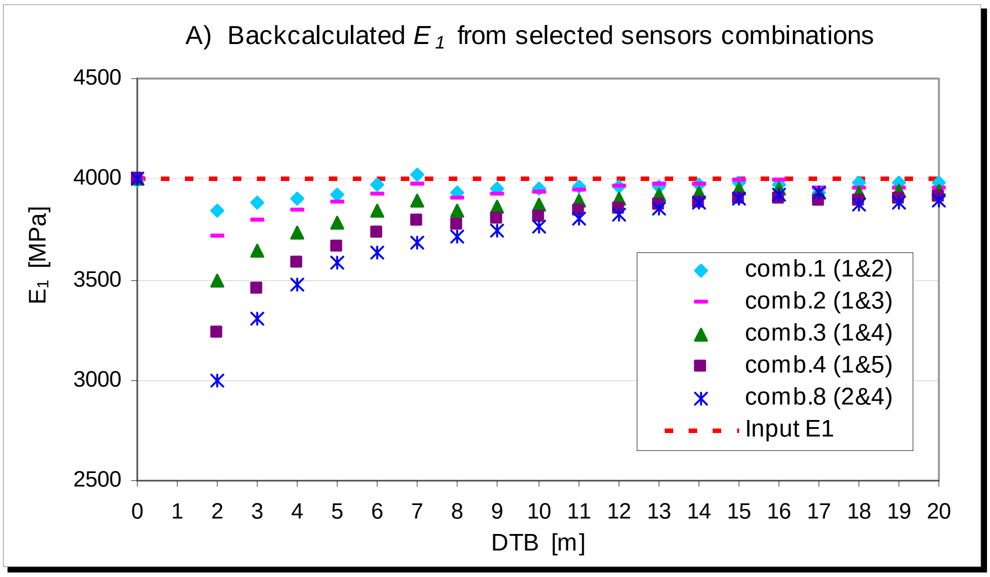
It is also very important to know that at which depth the existence of bedrock has little or no influence on the backcalculated moduli based on MLET. This issue can be addressed by using a MLET-based deflection calculation program such as BISAR to compute FWD deflection basins from pavement models with various DTABs. Then use the MLET-based backcalculation program, in this case VAHREN1, to backcalculate moduli from deflection basins obtained from the same set of pavement models but without bedrock. Comparing the backcalculated moduli with the correct (input) moduli should be capable to show that at which depth the presence of bedrock has little or no influence on the backcalculated moduli.

In this work, the deflection basins obtained from pavement models with bedrock at various DTBs in Figure 5-10 have been used as input data for backcalculation. The absence of bedrock in the backcalculation models was made by setting the thickness of base layer with a high value, e.g. 5,000 m, and fixing the modulus of the third layer with the value of 45 MPa. The backcalculated asphalt and base layer moduli from the best 5 sets of sensors combination found in Figure 6-1 are plotted against the assigned DTB in Figure 6-3 (A) and (B), respectively.

Obviously, the variances of results and their errors compared with the assigned moduli, E_1 = 4,000 MPa and E_2 = 400 MPa (see Figure 5-10), are large when bedrock is present at shallow depths. These variances and errors reduce gradually when DTB increases. The least error can be clearly found at the pavement model with bedrock at depth of 20 m or the deepest DTB in this investigation. To specify the depth at which the influences of bedrock on backcalculated moduli are insignificant for backcalculation analysis, the percentage deviation of the result at each DTB with respect to the assigned moduli is calculated using the following equation:

$$\%D_k = \left[\frac{1}{E_k} \left(\sqrt{\frac{1}{N} * \sum_{i}^{N} (x_i - E_k)^2} \right) \right] * 100 \qquad (6\text{-}2)$$

where $\%D_k$ is the percentage deviation at layer k, E_k is the assigned modulus at layer k, N is the number of set of sensor combinations, and x_i is the backcalculated modulus associated with set i.



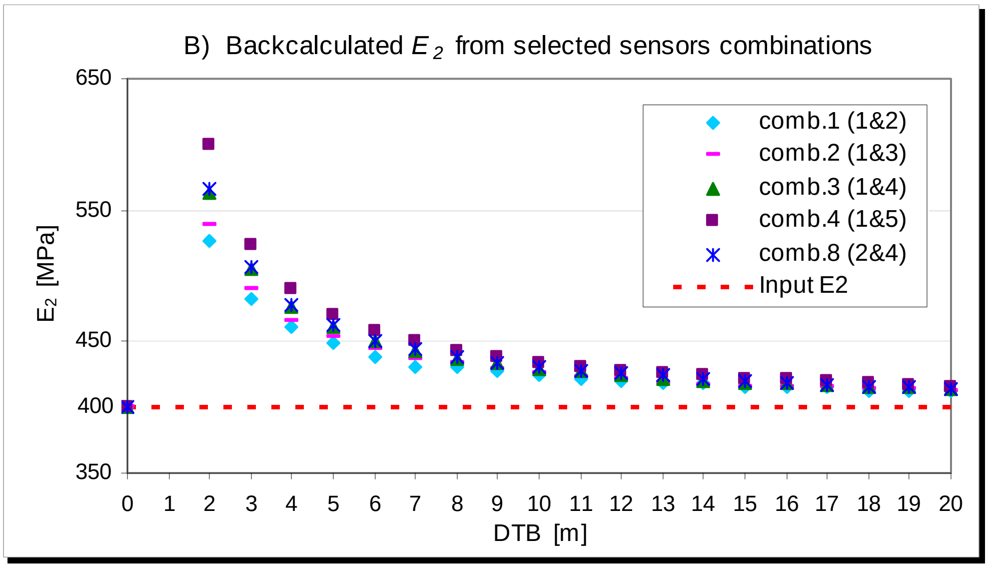


Figure 6-3. Variances of backcalculated moduli from system with various DTBs

The computed percentage deviation values obtained from eq(6-2) are plotted in Figure 6-4. These values present the dispersion of the backcalculated modulus of each layer from the correct moduli. In other words, the more spread apart the data in Figure 6-3, the higher the deviation value in Figure 6-4. It can be seen in the figure that the presence of bedrock has more impact on the modulus of the lower layer than the upper one. The ratio of the percentage deviation of the backcalculated upper layer moduli to the lower one is approximately 2 times. If the percent deviation of the backcalculated moduli from these 5 selected sets of sensor combination of less than 5% should be expected, it can be seen that all the pavement models with bedrock lying deeper than 14.0 m yield the deviation less than the expected value.

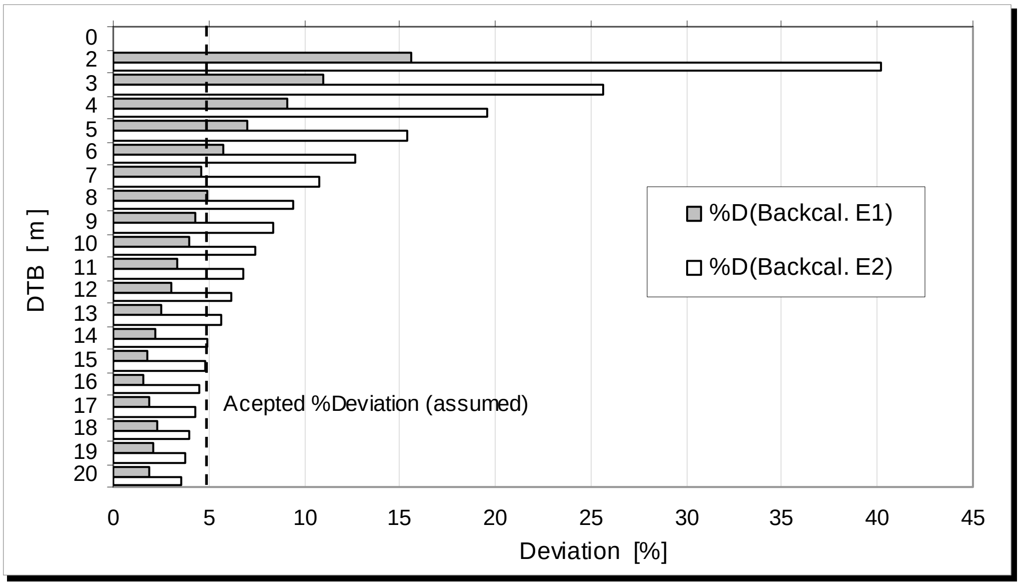


Figure 6-4. Percentage Deviation of backcalculated E_1 and E_2 with respect to assigned moduli

This finding also implies that even bedrock is no present under the pavement structure and even the subgrade material is perfectly linear elastic, assigning the artificial bedrock to the pavement model at depth of 14.0 m or deeper from the surface does not induce any large influence on the moduli obtained from backcalculation based on multi-layered elastic theory such as VAHREN1.

6.4 Developing a New Method

Based on the classical pavement mechanistic theory and the results from the related investigations, a new method for determine depth to artificial bedrock (DTAB) should be developed. Since the Boussinesq's equations of semi-infinite halfspace under point load and the conical stress distribution behavior of flexible pavement system under the FWD load (see Figure 2-1 or Figure 5-8) are analytically provable, the assumption that the outer surface deflections are mostly originated from the subgrade soil under this stress zone is still used as the principle of the new method. Moreover, Chen [17] has used the multidepth deflectometer (MDD) to measure the deflections of bedrock at shallow depths under pavement. He has stated that although the deflection at bedrock surface is not exactly zero because there was some anchor movement under FWD load, but the movement can be considered as insignificant. By these reasons, the presumption that the radial distance to the point where the deflection is zero is closely related to the DTB should still be considered as reasonable. This means that if the deflection basin is able to be extended with respect to subgrade behavior till the point that surface deflection equal to zero, the more reliable result of DTAB could be determined from that point. Some mathematical models of FWD deflection basin are summarized [71] and investigated focusing on the aforementioned point (*d(r)* =0).

Hossain: $$D(r) = A^* e^{B^*r} \tag{6-3}$$

Jendia: $$D(r) = \begin{cases} a*r^6 + b*r^4 + c*r^2 + d & ;0 \le r \le x \\ A*e^{B*r} & ;r \ge x \end{cases} \tag{6-4}$$

Schaefer1: $$D(r) = \begin{cases} a*r^2 + b & ;0 \le r \le x \\ e^{A*r^3+B*r+C} & ;r \ge x \end{cases} \tag{6-5}$$

Graetz: $$D(r) = \frac{w_1 + w_{2*}r^2}{1 + w_3 * r^2} \tag{6-6}$$

Schaefer2: $$D(r) = \frac{w_A + w_B * r^2 + w_c * r^4}{1 + w_D * r^2 + w_E * r^4} \tag{6-7}$$

where *D(r)* is the deflection value at radial distance, *r* is the radial distance from load center, and

$$\left.\begin{matrix} A, B \\ a, b, c, d, A, B \\ a, b, A, B, C \\ w_1, w_2, w_3 \\ w_A, w_B, w_C, w_D, w_E \end{matrix}\right\} = \text{mathematical constant parameters}$$

Although all the equations shown above are able to be used to approximate the related deflection values at some radial distances, each of them has a certain condition for applying to. For example, the eq(6-3) is valid to be used only in the area that the radial distances greater than a certain offset value. Since these mathematical models have different conditions to employ. They yield therefore the different accuracy of matching quality.

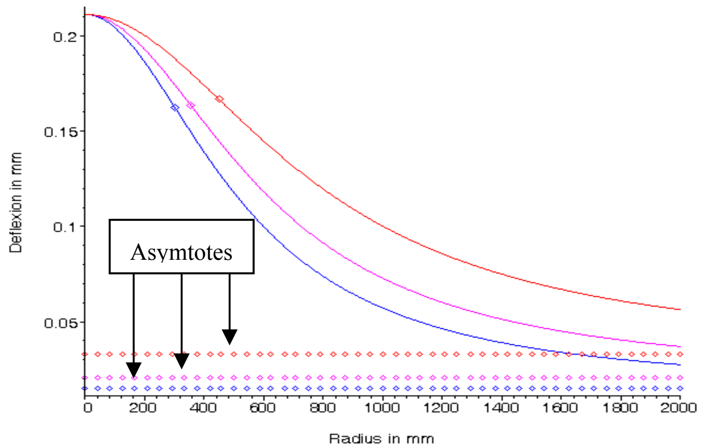


Figure 6-5. Asymptote lines by varying the parameter in Graetz mathematical model [71]

However, all these models have a coincident character that they are originally derived in mathematical process with the purpose for matching the computed deflection basin in the area of the FWD measurement (r < 2.5 m). Each of them yields therefore deflection curve leading to its asymptotical line at the greater radial distances. In other words, the mathematical models like these cannot be used to find out the intercept between curve and radial axis. Figure 6-5 illustrates examples of the asymptotes from deflection basins by varying the parameter w_3 in mathematical model proposed by Graetz or eq(6-6). Consequently, the desired DTAB according to the conventional graphical method cannot be determined using any of these mathematical models.

6.4.1 Description of the proposed method

On the basis of the above discussion, a new simplified method for determining the DTAB from FWD data is proposed. It is based on the followings presumptions:

1) The stiffness of subgrade materials under real pavement structure is governed by many factors (soil type, stress state, water content, bedrock etc.). The effective character of these materials should be somehow related to the change of slope of deflection curve especially at the outer part of this curve.
2) The deflection curve could be extrapolated to the next point which has a radial distance from the former one equal to the average of in-use sensors spacing by keeping slope changing rate consistent. This concept can be implemented by using least square technique with weighing through all default slope values.
3) Since the materials in the conical stress distribution zone under the outer sensors have influence on the outer deflections more than those close to the load center, the weighing values associated with slope value at each sensor pair used in the least square technique could be determined with respect proportionally to the outermost sensor pair.
4) The radial distance to the point where the deflection is zero presents the desired depth to artificial bedrock (DTAB). This depth can be therefore determined by using the intercept of (inverse) radial distance axis.
5) The acquired DTAB should be able to indicate the depth at which the artificial bedrock in pavement model could represent the effective behavior of the materials in the subgrade layer under FWD load. This could be the behavior of subgrade stiffening with depth, real bedrock (if exist) or any high stiffness soil layer which has behavior closely to bedrock.

Since this new proposed method has a main presumption that the slope changing rate of the extrapolated deflection curve should be consistent with those of the measured one, this procedure is called in this work as “Consistent Slope Changing Method (CSCM)”. Table 6-1 illustrates a calculation example of determining DTAB from a typical FWD deflection data using CSCM.

Deflections and sensor positions in the first 8 rows are acquired from FWD test. The slope values of deflection curve at each sensor pair are calculated using linear relation between deflections and inverse distances. All the weight values are computed using ratio of slope values with respect to the slope value of the outermost sensor pair. These set of slope and weight values are applied to least square technique with weight to determine the linear relation of slope increasing equation. The result of this least square technique is shown in Figure 6-6. The acquired equation is used to determine the slope and deflection values at distance of extrapolated curve section (in this example 30 cm).

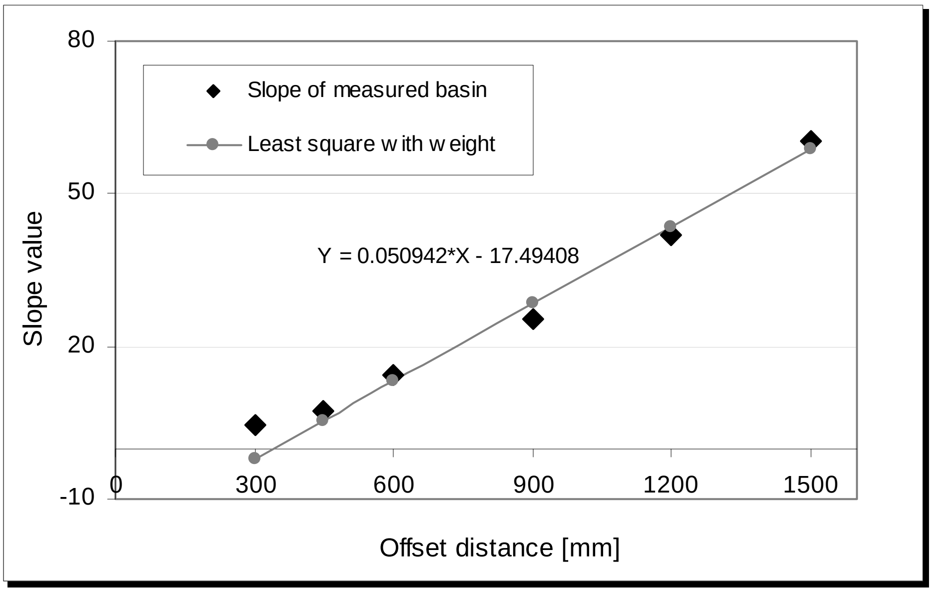


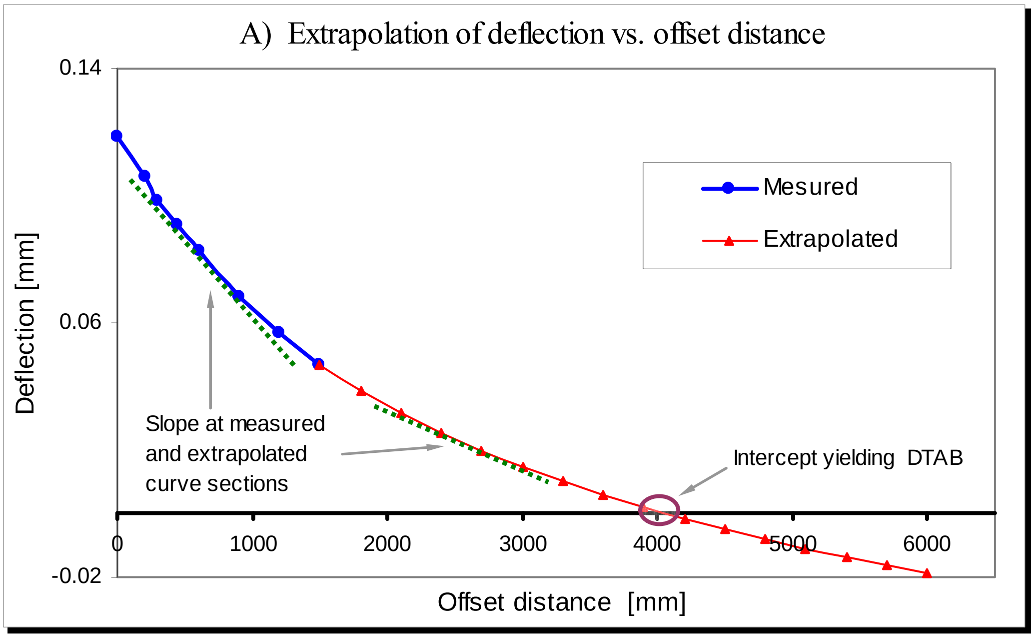
Figure 6-6. Technique of "Least Square with Weight" used in slope developing

The process of extrapolation the deflection curve has to be done until either the horizontal axis intercept is found, which can be observed from the change of deflection value from positive to negative, or the radial surface distance is greater than 14 m where the influence of bedrock on backcalculated moduli is not meaningful anymore. In this example the deflections change from positive value to negative between the radial distance 3900 and 4200 mm. The exact associated intercept value can be computed and is 4.077 m as can be seen in Table 6-1.

The measured and extrapolated deflections are plotted against offset and inverse offset distance in Figure 6-7 (a) and (b), respectively. Both of the graphs show a very smooth curve between the measured and extrapolated deflection. This is a good sign of relation between predicted value and the reality one. It is obvious that both of these graphs can be used to determine the result of CSCM if the DTAB could be found at any shallow depth. On the contrary, if the DTAB value is very large or bedrock is not present it would be not suitable to use the graph of deflection vs. offset distance because of the enormous value of radial axis. Unlike the graph of deflection vs. inverse offset distance, it can be used to illustrate the result of any cases. The latter type is for that reason used to illustrate the CSCM results in this work.

Table 6-1.Calculation of extrapolating deflection values using CSCM

No.	r (mm)	1/r (1/mm)	deflections (mm)	Linear relation slope	const.	X-intercept [1/mm]	DTB [mm]	weight
1	0	-	0.11840					
2	200	0.00500	0.10620					
3	300	0.00333	0.09864	4.54	0.084	-0.01841		0.075
4	450	0.00222	0.09063	7.21	0.075	-0.01035		0.119
5	600	0.00167	0.08256	14.53	0.058	-0.00402		0.240
6	900	0.00111	0.06846	25.38	0.040	-0.00159	-630.40	0.420
7	1200	0.00083	0.05687	41.72	0.022	-0.00053	-1887.96	0.691
8	1500	0.00067	0.04680	60.42	0.007	-0.00011	-9266.87	1.000
9	*1800*	*0.00056*	*0.03856*	*74.20*	*-0.003*	*0.00004*	*27815.32*	
10	*2100*	*0.00048*	*0.03145*	*89.48*	*-0.011*	*0.00012*	*8019.74*	
11	*2400*	*0.00042*	*0.02522*	*104.77*	*-0.018*	*0.00018*	*5682.91*	
12	*2700*	*0.00037*	*0.01966*	*120.05*	*-0.025*	*0.00021*	*4840.08*	
13	*3000*	*0.00033*	*0.01465*	*135.33*	*-0.030*	*0.00023*	*4442.45*	
14	*3300*	*0.00030*	*0.01008*	*150.61*	*-0.036*	*0.00024*	*4235.79*	
15	*3600*	*0.00028*	*0.00589*	*165.90*	*-0.040*	*0.00024*	*4127.96*	
16	*3900*	*0.00026*	**0.00202**	*181.18*	*-0.044*	*0.00025*	***4077.52***	
17	*4200*	*0.00024*	**-0.00158**	*196.46*	*-0.048*	*0.00025*	*4063.13*	
18	*4500*	*0.00022*	*-0.00494*	*211.74*	*-0.052*	*0.00025*	*4072.71*	
19	*4800*	*0.00021*	*-0.00809*	*227.03*	*-0.055*	*0.00024*	*4098.91*	
20	*5100*	*0.00020*	*-0.01106*	*242.31*	*-0.059*	*0.00024*	*4137.02*	
21	*5400*	*0.00019*	*-0.01387*	*257.59*	*-0.062*	*0.00024*	*4183.89*	
22	*5700*	*0.00018*	*-0.01653*	*272.88*	*-0.064*	*0.00024*	*4237.33*	
23	*6000*	*0.00017*	*-0.01905*	*288.16*	*-0.067*	*0.00023*	*4295.80*	



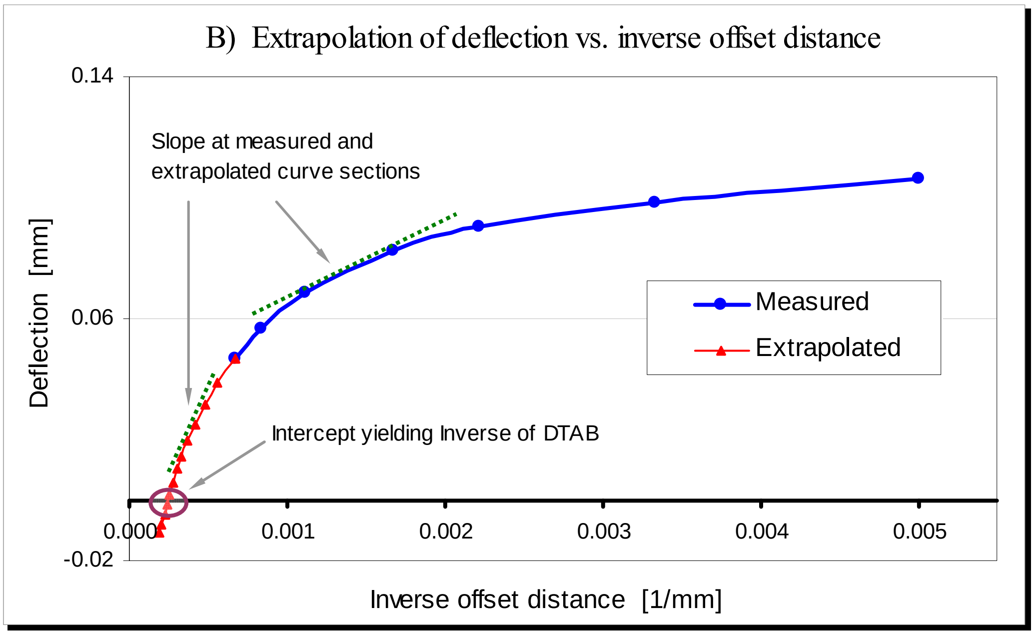


Figure 6-7. Measured and extrapolated deflection curves using CSCM

6.5 Evaluation of The Proposed CSCM

6.5.1 Validation with subgrade stiffening with depth

The CSCM is developed for determining the depth at which artificial bedrock is able to set into the pavement model for representing the behavior of materials in subgrade layer such as real bedrock or the subgrade stiffening with depth. To evaluate this method, the nonlinear behavior of subgrade soil should be investigated.

A numerous studies about nonlinearity of soils have been reported in literature. Some are directly related to backcalculation using FWD data. Nazarian and Boddspati [60] described non-linear behavior occurs in FWD test. An increase in the load magnitude of the FWD results in an increase in deflection that is greater than one to one. In general, there are two types of non-linearity mathematical models considered in pavement engineering area, one is for granular and another one is for cohesive materials [78]. Unfortunately, these models cannot be applied to backcalculation procedure based on MLET. Most of them are used in backcalculation procedure based on Finite Element Method (FEM).

Rohde and Smith [68] illustrated how the stiffness in typical clay and sandy subgrade changes with depth. The analysis approach used in their study is briefly explained here. A finite element program (same source code as ILLI-PAVE) was used repeatedly to generate a database of surface deflection of three-layer pavement systems. Then, the pattern-search technique used in the backcalculation program MODULUS was utilized to obtain stress sensitive parameters defining the nonlinear characteristics of the pavement materials. Two cases of typical three-layer pavement systems were backcalculated. The identical two upper layers, an asphalt surface layer and granular base, were assigned in both systems. The subgrade materials used in these two systems were typical clay and sandy subgrade.

Additionally, the asphalt surface layer was assumed as linear elastic. Three values of asphalt stiffness used in generating the data base were chosen to cover the expected range of possible solutions. For simulating the granular material in the pavement systems, the universal model was used. This model was proposed by Witczak and Uzan [85] to describe the nonlinear behavior of granular soils. In general case, the universal model can be written as below.

$$M_R = (k_1 p_a)\left(\frac{\theta}{p_a}\right)^{k_2}\left(\frac{\tau_{oct}}{p_a}\right)^{k_3} \tag{6-8}$$

where M_R is the resilient modulus of granular soil, k_1 is the constant parameter, p_a is the atmospheric pressure used in the equation to make coefficients independent of the units used, θ is the bulk stress or the first stress variant = (σ_1+ σ_2+ σ_3), where σ_i is the principal stress, and τ_{oct} is the octahedral stress.

$$\tau_{oct}^2 = \frac{1}{9}\left[(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2\right] \tag{6-9}$$

By varying the k_1 parameter in the eq(6-8), three stress-stiffness relationships are defined to cover the expected range of possible solutions.

For simulating the clay subgrade, the bilinear model was used in the pavement model. The bilinear model was proposed by Thomson and Robnett [75] to describe the nonlinear behavior of fine-grained soil. The model can be written as below.

$$M_R = \begin{cases} k_2 + k_3\left[k_1 - \sigma_d\right] & \text{For } k_1 > \sigma_d \\ k_2 + k_4\left[\sigma_d - k_1\right] & \text{For } k_1 > \sigma_d \end{cases} \qquad (6\text{-}10)$$

where σ_d is the deviatoric stress = $\sigma_1 - \sigma_3$, and k_i is constant parameter.

The parameter k_2 was varied to produce three nonlinear subgrade models to cover a wide range of possible solution in generating the database.

To backcalculate one nonlinear property per pavement layer, the technique requires running the FE program 27 times. For each of the 27 runs, the surface deflections expected to occur at the FWD sensor positions were calculated and stored in a database. Using the search routine, the measured deflections were compared with the calculated deflections in the database. The stiffness parameters associated with a deflection basin matching the measured deflection basin were obtained. The results for the explained three-layer system are the surface modulus, a backcalculated k_1 and k_2 for the base and the subgrade, respectively. These results were then used in the finite element program to calculate the stiffness of each element in the FE mesh.

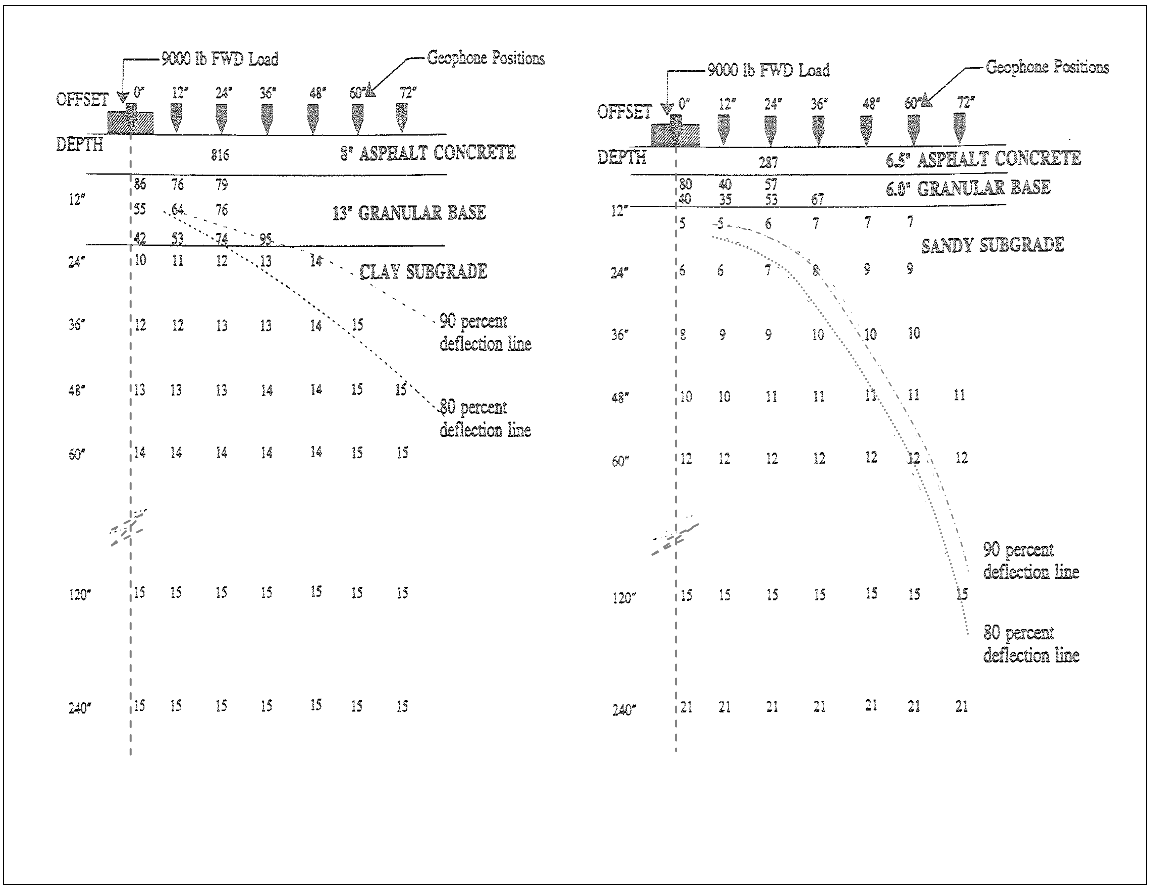


Figure 6-8. Backcalculated moduli [ksi] for pavement models with typical clay and sandy subgrade [68]

The in situ deflection data from two sites which have clay and sandy subgrade were used in the backcalculation. The calculated stiffness values throughout the structure are presented in Figure 6-8. Some differences in nonlinearity between fine grained and sandy subgrade materials can be seen in the Figure 6-8. The stiffness of the clay subgrade is only influenced by the deviatoric stress. At the outer sensors, the subgrade shows no increase in stiffness with depth. Directly beneath the load there is a slight increase in stiffness to a depth of eight feet after which no increase occurs. The sandy subgrade shows a significant increase in stiffness with depth. Below the outer sensors, where little change is expected in terms of load related deviatoric stress, the subgrade increase from 7,000 to 21,000 ksi within 20 feet. It was described that this is a result of the increase in bulk stress due to an increase in overburden pressure [68]. This increase in stiffness is even more significant beneath the load due to the high deviatoric stresses found near the top of the subgrade.

Using the results in the Figure 6-8 as guided values to model a set of pavement system with subgrade stiffening with depth, one could approximate that the stiffening rate beneath the FWD load in those typical subgrade materials has a rough range from 5 to 20 MPa by every 50 cm of depth. The semi-infinite subgrade layer was therefore subdivided at the upper part into 28 layers. The following parameters have been used in the evaluated pavement structures (a whole system = 30 layers):

Asphalt layer moduli: E_1 = 4,000 / 8,000 / and 16,000 MPa

Base layer modulus: E_2 = 400 MPa (constant)

First subgrade layer modulus: E_3 = 45 MPa (refers to RStO'01 [6])

Modulus of the next subgrade layers: $E_4 ... E_{30}$

Case 1 = increasing with 5 MPa/layer

Case 1 = increasing with 10 MPa/layer

Case 3 = increasing with 20 MPa/layer

Thickness of asphalt layer: h_1 = 250 mm

Thickness of base layer: h_2 = 500 mm

Thickness of the upper subgrade layers: $h_3 ... h_{30}$ = 500 mm

The Poisson's ratio is simplified for all layers with the value of 0.35 and no slip between layer interfaces is assumed. The CSCM is used to calculate the DTABs of each model. Using the obtained DTAB, the artificial bedrock with modulus value of 30,000 MPa is assigned to the same set of pavement models which have the identical two upper layers moduli but the subgrade modulus is treated as constant with the value of the uppermost subgrade layer modulus (45 MPa). Figure 6-9 illustrates the whole picture of this evaluation system.

The deflections under FWD load from the pavement models with artificial bedrock are computed at the positions of FWD sensors using BISAR program and compared with those from the pavement model with subgrade stiffening with depth to illustrate how well the DTAB from CSCM represents the behavior of stiffening subgrade soil.

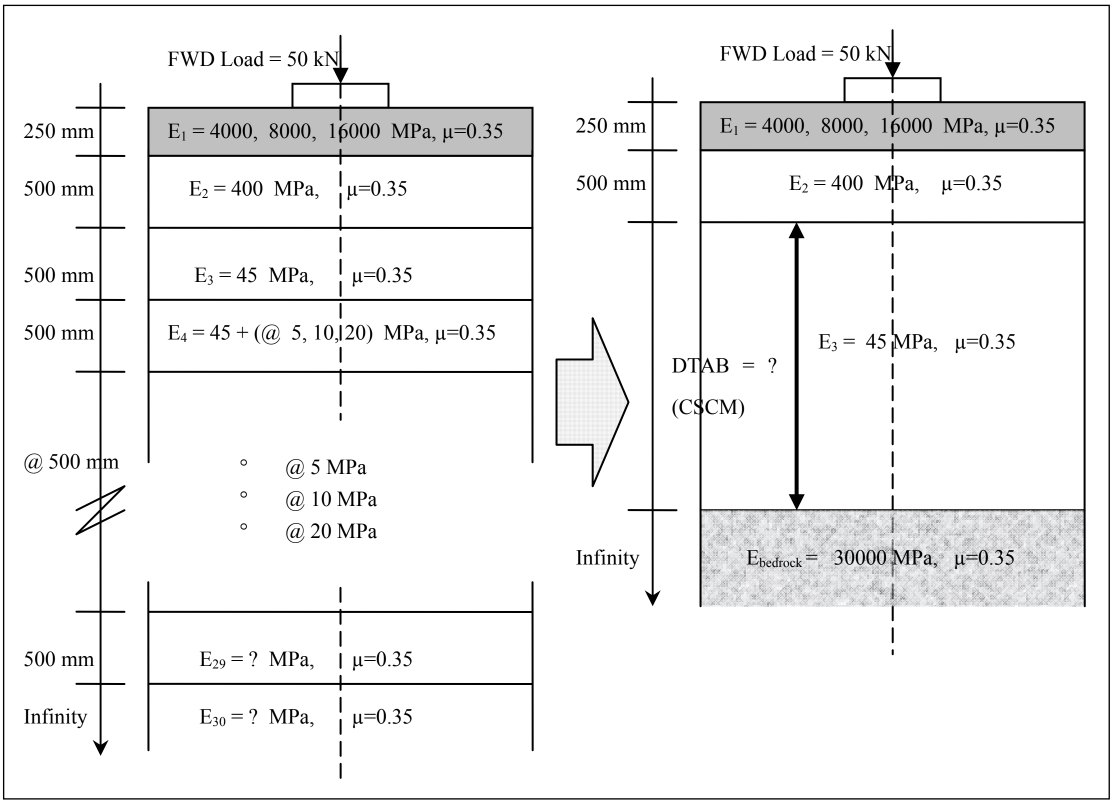


Figure 6-9. Illustration of evaluation of the CSCM with subgrade stiffening with depth

One example of the comparing results obtained from the pavement system with asphalt layer modulus = 16,000 MPa, base modulus = 400 MPa on various stiffening subgrade systems is presented in Figure 6-10. The deflections from pavement system with constant subgrade modulus are also shown in the figure to compare with the others.

The deflection curve of typical pavement model with constant subgrade modulus shows clearly different values with greater deflections at every point of sensor than those from pavement with subgrade stiffening with depth. For the pavement with artificial bedrock at DTAB obtained from the CSCM shows the ability in representing the behavior of pavement with stiffening subgrade. This can be seen in Figure 6-10 that the deflection curves of pavement with artificial bedrock which are plotted with dotted lines are able to get closer to the deflection curves obtained from pavement with stiffening subgrade. The comparing results from pavement systems with upper asphalt layer = 4,000 MPa and 8,000 MPa show the same trend as can be seen in appendix E.

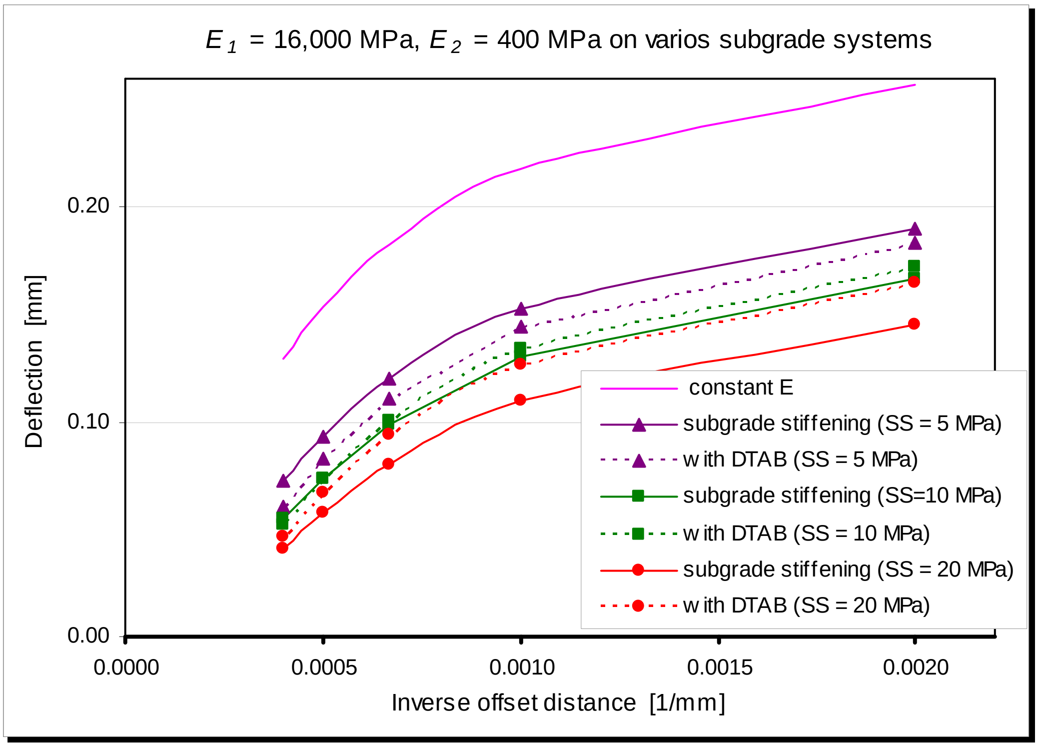


Figure 6-10. Comparing deflection curves obtained from pavements with various subgrade systems

The results from the performance evaluation of CSCM indicate that setting up the pavement model by adding the artificial bedrock at DTAB calculated from CSCM is more appropriate for using in backcalculation programs based on MLET than the traditional pavement model (with constant subgrade modulus). Obviously, this system is able in some degree to represents the behavior of stiffening with depth found in most subgrade materials.

6.5.2 Verification of CSCM with in situ deflection data

To illustrate how well the proposed CSCM responses with in situ pavement surface deflections, the FWD testing data on the test track at the Federal Highway Research Institute or BASt is used again to verify this method. The brief description and essential characters of this test track has already been explained in section 5.2.2.

One exemplification of the graphical results using FWD data from pavement structure construction class III line 1 at testing point no. 2.2.1 is shown in Figure 6-11. The graphical results of other testing points are also illustrated in the same fashion in appendix F. It can be seen in the Figure 6-11 that the deflection curve still shows a very smooth transition between the measured and extrapolated deflection. The depth of the concrete slab at the edge and middle of tank (3.0 and 3.5 m, respectively) are marked with two vertical lines in order to illustrate the expected portion on the horizontal axis. Obviously, the extended curve falls evidently into the expected area. The depth result of 3.27 m demonstrates an uncanny ability to predict the depth of the concrete slab lying under the test track.

All the results of determining DTAB from FWD data testing on the test track at BASt using CSCM are summarized in Table 6-2. The result errors at each testing point obtained from the deflection and inverse offset method and the regression equations are also shown in the table. Recall that these errors are computed with respect to the depth of 3.25 m which is the average depth from the pavement surface of to the concrete slab under the test track.

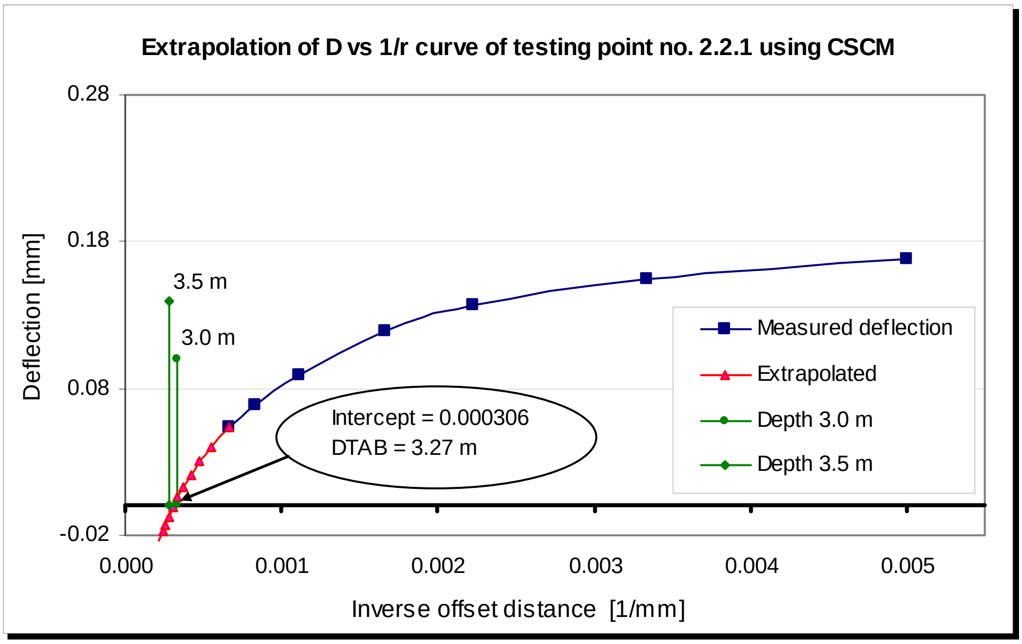


Figure 6-11. Determining the DTAB using CSCM from FWD data testing on test track at BASt

Table 6-2. Summarized results of predicting the depth to concrete slab under the test track at BASt

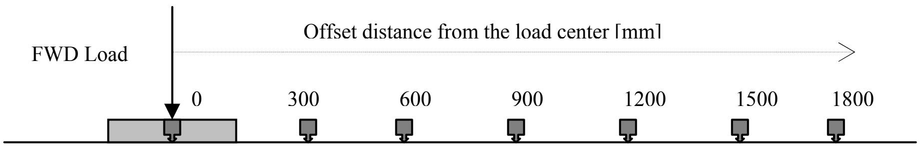
Construction Class	Testing Point	real average DTB	cal. DTAB CSCM	Error [%] CSCM	D vs. 1/r	Regrs. Eqs.
SV Line 1	2.1.1	3.25 m	3.70 m	13.8 %	*	*
SV Line 1	2.1.2	3.25 m	3.74 m	15.1 %	*	*
SV Line 2.1	1.1.1	3.25 m	4.07 m	25.2 %	*	*
SV Line 2.1	1.1.2	3.25 m	4.14 m	27.4 %	*	*
III Line 1	2.2.1	3.25 m	3.27 m	0.6 %	264 %	47.6%
III Line 1	2.2.2	3.25 m	3.24 m	0.3 %	253 %	48.3%
V Line 1	3.2.1	3.25 m	3.51 m	8.0 %	321 %	53.5%
V Line 1	3.2.2	3.25 m	3.25 m	0 %	214 %	36.6%
						* Concrete slab (bedrock) is not found

The pavement structures of construction class SV are designed for heavy traffic. As a result, they usually have the thickest bound layers. Hence, the influence of the upper stiff layers of these pavement structures on the intercept value is also the most significant. As can be seen, the DTAB results from the construction class SV show that the bedrock is present at shallow depth with the average error value of roughly 20%. In case of the construction classes III and V for which the influence of the upper stiff layers on the intercept is less than those of class SV, the obtained results show an excellent prediction which average error values only 0.45% and 4%, respectively. Comparing with the deflection and inverse offset method or the regression equations, the proposed CSCM demonstrates a substantial improvement in predicting the depth to concrete slab from the in situ FWD data from BASt.

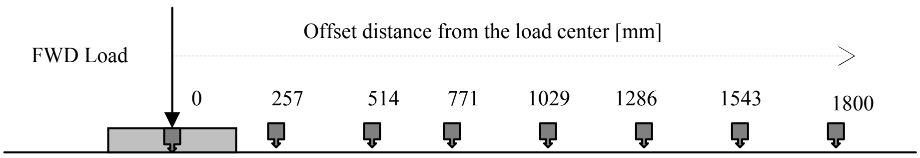
6.6 Robustness of CSCM

As discussed earlier, one of the main drawbacks of the regression equations is that the method requires that four sensors have to be placed at 0, 0.3, 0.6 and 0.9m. This makes this method not flexible for practical use if sensors need to be placed at other positions. Although the deflection and inverse method does not have any obligate sensor positions, it can be clearly seen from the section 5.2.1.1 that this approach is very sensitive. In other words, this approach can yield different results of DTB if the tangent line is originated from different set or positions of FWD sensors.

Case 1: 7 sensors



Case 2: 8 sensors



Case 3: 9 sensors

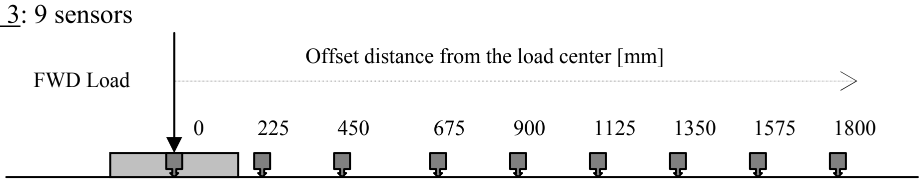


Figure 6-12. Positioning of sensors using in evaluation of robustness of the CSCM

It is clear that a good method for determining DTB should yield the same result or at least only a slightly difference for a pavement system, even if the FWD sensors configuration has to be changed. To evaluate the robustness of the CSCM, three cases of different sensor positions are applied to the set of pavement model with subgrade stiffening with depth as illustrated in the Figure 6-9. The sensors locations in each case are positioned with constant spacing from load center to the usual practical outermost sensor (1.8 m) as presented in Figure 6-12. Using the forward layered elastic program FAT6a (modified version of BISAR), the deflections at each sensor position are computed. The DTABs associated to each deflection basin are determined using CSCM and compared with each other.

The results presented in Figure 6-13 are obtained from the system having an upper layer modulus of 4,000 MPa. The results of other systems which have the upper layer moduli of 8,000 and 16,000 MPa and the same set of subgrade stiffening systems are illustrated in appendix G. Even though the input systems have different in both number and position of sensors, it is obvious that there are only small differences between the results of DTABs computed from CSCM. Additionally, it can be observed that the deepest DTABs are always obtained from the system calculated from 7 sensors which has a greater spacing between sensors. This can be deduced that the spacing distance has also little impacts on the computed DTAB according to CSCM. Nevertheless, the differences of DTABs shown in the Figure 6-13 could be considered as trivial and insignificant for the backcalculation process.

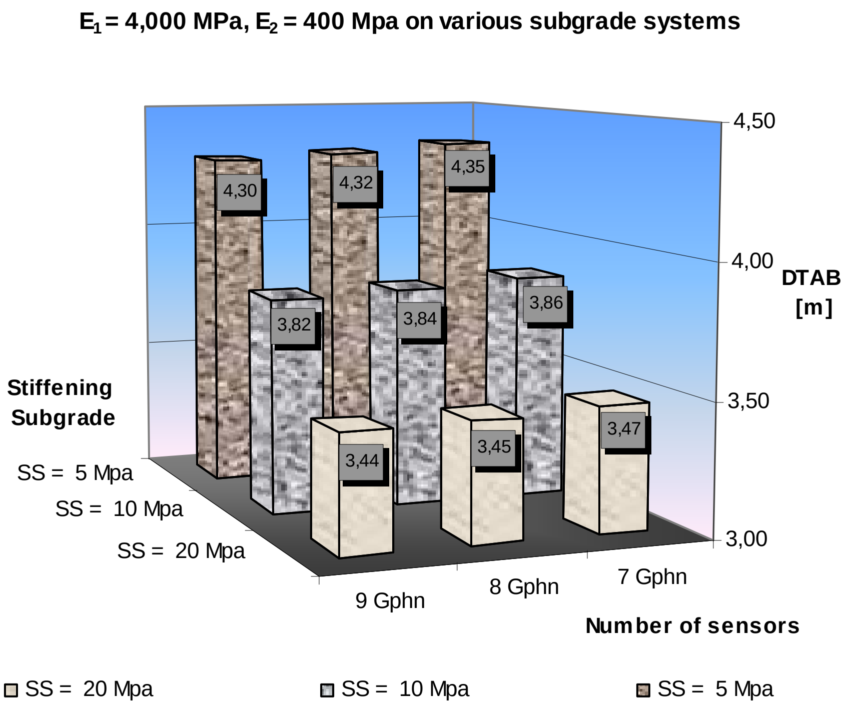


Figure 6-13. Calculated DTABs from different pavement system and set of sensor positions using CSCM

From the evaluation of the robustness of CSCM, it can be concluded that the CSCM has only trivial sensitiveness for changing of number and positions of sensors. This method can be therefore applied to any in situ FWD data in general case. Moreover, it should yield reliable result even if either the number or the positions of sensors have to be altered as long as this alteration are considered as reasonable for FWD test.

Thus far, the CSCM has proved itself suitable for determining DTAB from in situ FWD data. In addition, applying this method to the MLET-based iterative backcalculation program, e.g. VAHREN1, is able to improve performance of such programs. The added artificial bedrock with stiffness value of 30,000 MPa at depth obtained from the CSCM improves the convergence behavior of these programs. At the same time, it is able to increases the reliable of the results by representing in some degree the behavior of stiffening with depth of most subgrade materials. Moreover, it has been found that the CSCM is flexible for applying to general case of FWD test regardless the reasonable changes of number or positions of sensors. The concept of CSCM is added into the GA-based backcalculation program GAMLET as optional function. However, since GAMLET searches by nature for the optimal solution and does not use iteration method, there is no need to fix the stiffness of the artificial bedrock at 30,000 MPa to improve the iteration convergence as in VAHREN1. Thereby, the objective of this function in GAMLET is to help the user in making decision about setting up the subgrade system in pavement model correctly. This means that if the artificial bedrock in pavement model should be added to the pavement model, any suitable range of high modulus values of the artificial bedrock can be independently set in GAMLET by user.

7 Verification of “GAMLET” Program

7.1 Backcalculation Using in situ Data

It is always a tough task, if not impossible, to state that a particular set of layer moduli obtained from backcalculation procedure using in situ deflection data is correct. Unlike performing backcalculation using deflection data generated by forward computer program, one can always state that whether the result is correct or not since the input moduli are exactly known. It is clear that, there are many factors based on differences between the realities and the initial assumptions which can lead to erroneous results. Some examples of these factors can be given here: the random and systematic errors in measured deflection data, the uncertainty of layer thicknesses, variation of moisture content along the pavement section, nonlinear behavior of material, temperature gradient in asphalt layers, and major cracks in the pavement. Such these factors cause the in situ deflection data depart, sometimes drastically, from the initial assumptions.

Comparing the backcalculated moduli with the results from laboratories is also not very helpful to overcome these problems since the real pavement conditions show by nature variations along the road and enormous differences from those in laboratories. Moreover, in most practical cases, some layers have to be combined with their adjacent layers because of two reasons. First, such layers are too thin (insensitive) to be able to backcalculate a modulus for it as a separate layer. Second, some backcalculation programs do not allow for assigning the desired number of layers into the pavement model. Therefore, it is generally the case that the backcalculated moduli are pavement layer parameters, not materials properties.

It is clear that high result accuracy may be obtained when backcalculation is performed using deflection data generated from a pavement model since the assumptions of the model used in backward process matches perfectly with those in forward process. As can be seen in the Table 4-4 through Table 4-6, all backcalculation programs were able to produce the results with the average RMSE value of ca. 0.20%. By the fact of backcalculation with in situ data discussed above, the RMSE value between 1 and 3% are widely used as threshold value for acceptable results. For example, in the Long Term Pavement Performance (LTPP) test section in USA, a RMSE of 3% is used as an acceptable error [2].

However, one should keep in mind that the result with RMSE less than the acceptable value is an encouraging outcome since RMSE is an effective tool providing a statistic for the overall match between the measured and the backcalculated deflection basins but it does not assure that the backcalculated moduli are correct. In practical, the best way to asses the validity of the backcalculated moduli using in situ data is to have a thorough knowledge of the materials in the pavement as much as possible. Construction records and new technology equipments such as ground penetrating radar (GPR) are very helpful for collecting information. Also, experienced engineering judgments are always valuable in this field to anticipate the range of acceptable values.

Based on the available complete information, the in situ deflection basins obtained from FWD test on the test track at BASt is used once again to verify the backcalculation program GAMLET. The details about this test track and the FWD test procedure on this test track have been already described in section 5.2.2. The RMSE value of 0.30 % which can be considered as high accuracy in case of backcalculation using in situ data is used as the threshold of desired value to ensure the high performance of GAMLET.

7.2 Setting up Pavement Model

As described, a number of FWD tests were carried out on the selected 4 different fields based on their pavement structures (construction classes). Two points on each of these fields were tested, altogether 8 testing points. The backcalculations of the deflection basins obtained from all 8 points are performed using the backcalculation program GAMLET. The details of these backcalculations are described thoroughly in this chapter using the data from the first testing point, no.1.1.1 as an example.

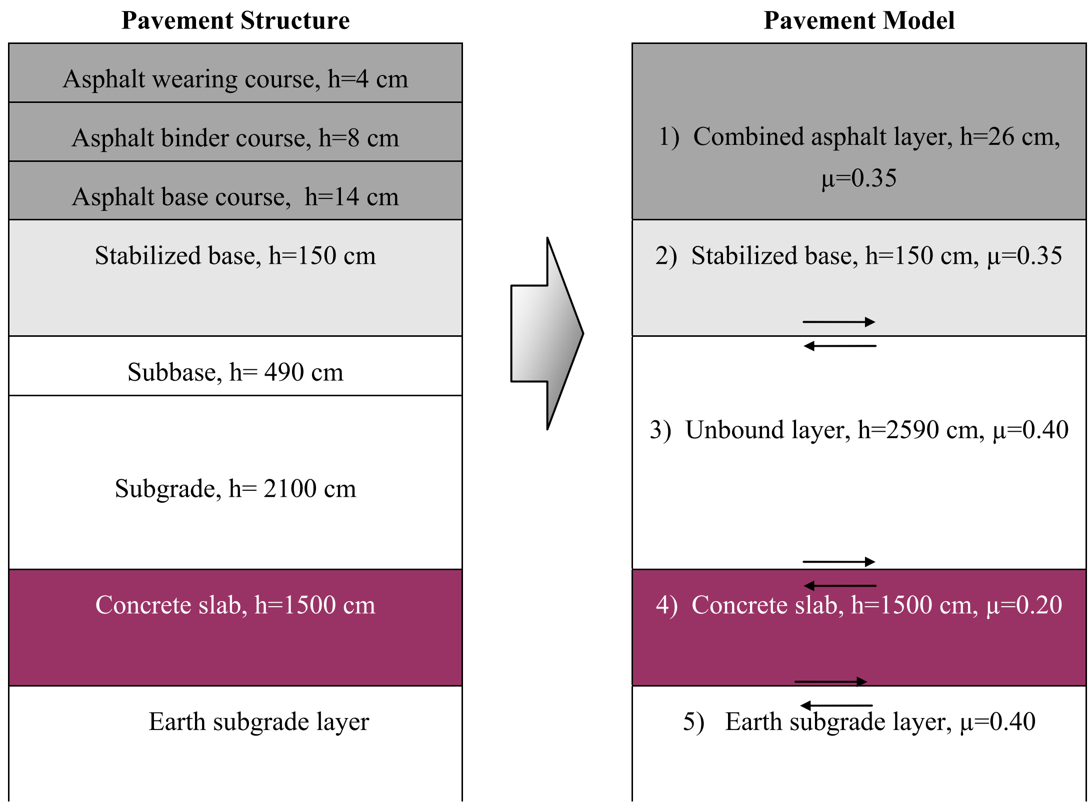


Figure 7-1. Pavement structure and pavement model of testing point no.1.1.1 (class SV, line 2.1)

The testing point no.1.1.1 locates on the pavement structure which is constructed according to the pavement construction class SV line 2.1 in the German guidelines for the standardization of superstructure of road surfaces 2001 (RStO´01) [6]. The pavement construction class SV is designed for use as prototype of pavements which are expected to be suffered from an equivalent 10 ton-standard-axle load more than 32 million times during the design period of 30 years.

Figure 7-1 gives the detail of the real pavement structure and the pavement model used in the backcalculation process. To avoid the insensitive problem, some adjacent layers are combined to each other based on their similarities in expected modulus values. As shown in section 6.5.2 that the DTAB obtained from the CSCM indicated close to the depth to the concrete slab under the testing point. The concrete thickness of 1.5 meter has been assigned according to the available information. Based on the properties of unbound materials, horizontal slip at layer interfaces of such materials is allowed as shown by the parallel arrow symbols in the Figure 7-1.

In order to verify the results obtaining from the backcalculation program GAMLET, the backcalculated layer moduli will be compared with the predicted layer moduli of this testing point in some available literature [[44] and [88]]. Table 7-1 presents these predicted values. Since moduli of asphalt materials in strongly dependent on temperature, the predicted moduli of the combined asphalt layer in Table 7-1 show the possible range of these materials according to seasonal time. Since the FWD tests have been performed on February which is the late winter time in Germany (measured temperatures ≈ 15 °C), a relative high value of this layer could be expected. However, it should be emphasized that the modulus values shown in Table 7-1 are not the “correct” values, but only the predicted values based on the assumptions used in those studies.

Table 7-1. Predicted layer moduli of testing point no.1.1.1 in available literature

Description	Combined asphalt layer, E_1 (seasonal)	Stabilized base, E_2	Unbound layer, E_3	Concrete slab, E_4	Earth subgrade, E_5
Moduli [MPa]	7500-15000	6000	190	25000	-

7.3 Backcalculation Results

7.3.1 Backcalculation of in situ data using GAMLET-loaded GA module

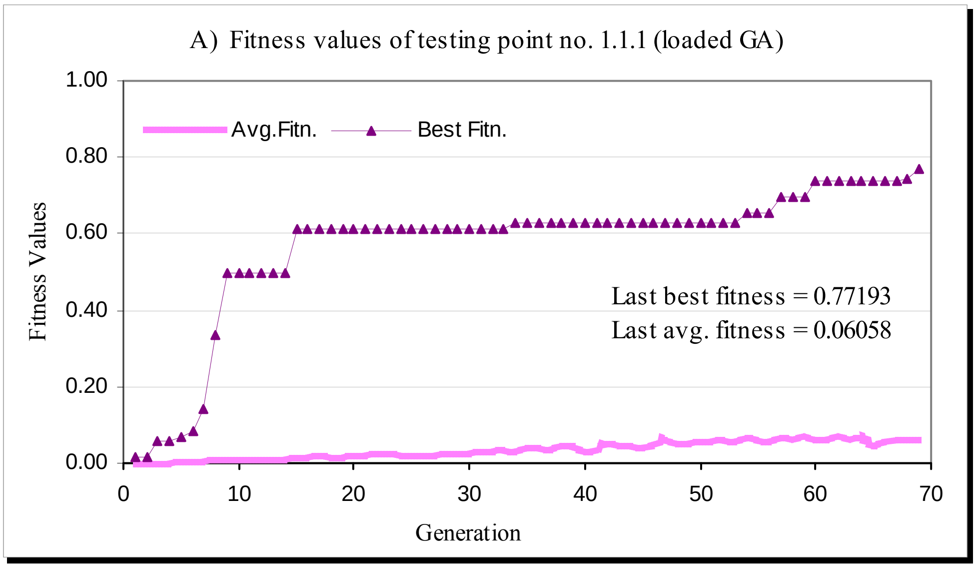
Recall that the loaded-GA module in GAMLET consists of tournament selection, single or uniform crossover operators, jump and/or creep mutation operators, elitism, and niche technique. Based on the in situ deflection data from the test track, this module is used to perform the backcalculation by using parameters shown in Table 7-2.

Table 7-2. Parameters used in GAMLET -loaded GA module

Parameters	Class SV line 2.1
No. of backcalculated layer	5
Size of population	220
Maximum generations	200
Probability of crossover	0.90
Single-point crossover	off
Uniform crossover	on
Probability of jump mutation	0.02
Probability of creep mutation	0.24
Elitism	on
Niche	off
FWDLines	off
Range of possible moduli E_1 [MPa]	1000 to 20000
Range of possible moduli E_2 [MPa]	500 to 10000
Range of possible moduli E_3 [MPa]	10 to 500
Range of possible moduli E_4 [MPa]	10000 to 30000
Range of possible moduli E_5 [MPa]	5 to 300
Precision level [MPa]	5/ 5/ 1/ 10/ 1

Based on the materials used in the pavement structure and the measured temperatures during the tests, the ranges of possible moduli for each layer are entered. According to these parameters and the precision used in this problem, the GA process is used to search for the optimal solution among 2^{51} sets of possible solution. Figure 7-2 (A) and (B) present the convergence behavior of this backcalculation analysis. The process stopped when the RMSE value = 0.28% which is less than 0.30% (illustrated by dotted line in Figure 7-2 B) was found. This convergence occurred at generation 69 with number of function evaluations, CE = 15,180 times.

The backcalculated layer moduli associated with this convergence are shown in Table 7-3. It can be seen that the loaded GA module in GAMLET shows a good capability in solving this backcalculation problem. The search process yields not only the RMSE value of 0.28% which can be considered as excellent in case of using in situ data but also yields the reliable backcalculated moduli based on the comparison with the predicted moduli in Table 7-1 and on the engineering point of view.



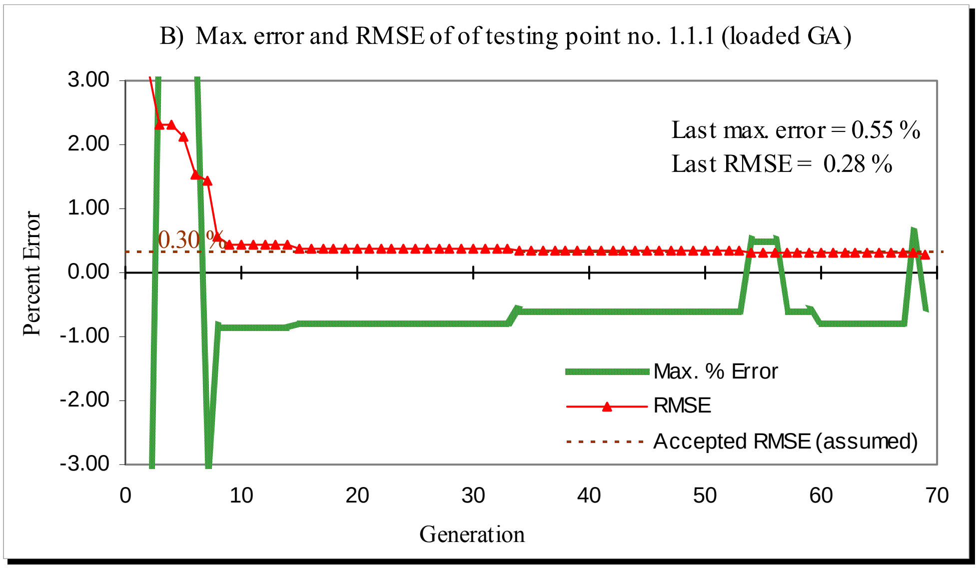


Figure 7-2. Convergence behavior of testing point no.1.1.1 (loaded GA)

Table 7-3. Backcalculated layer moduli of testing point no.1.1.1 (loaded GA)

Layer	Combined asphalt layer, E_1	Stabilized base, E_2	Unbound layer, E_3	Concrete slab, E_4	Earth subgrade, E_5
Moduli [MPa]	12660	2775	194	21640	103

7.3.2 Backcalculation of in situ data using GAMLET-micro GA module

The µGA module in GAMLET has shown thus far an outstanding performance in solving backcalculation problem using deflection data generated from the forward computer program. Besides yielding the accuracy results, the most important improvements of this module are the great reduction of computational effort (computing time) and the eliminating of many GA parameters. As discussed, these improvements make the GA-based backcalculation program more appealing for routine work and more user-friendly. Based on the in situ deflection data from the test track, this module is used to perform the backcalculation by using parameters shown in Table 7-4.

Table 7-4. Parameters used in GAMLET-µGA module

Parameters	**Class SV line 2.1**
No. of backcalculated layer moduli	5
Size of population	5
Maximum generations	200
Probability of crossover	0.50
Single-point crossover	-
Uniform crossover	on
Elitism	on
Range of possible moduli E_1 [MPa]	100 to 20000
Range of possible moduli E_2 [MPa]	100 to 10000
Range of possible moduli E_3 [MPa]	10 to 500
Range of possible moduli E_4 [MPa]	10000 to 40000
Range of possible moduli E_5 [MPa]	5 to 300
Precision level [MPa]	5/ 5/ 1/ 100/ 1

Major differences between the micro and loaded GA can be clearly seen in such table of parameters. For backcalculation problem of testing point no.1.1.1, µGA uses a population size of only 5 in order to reduce the computing time. Moreover, only crossover probability of 0.5 is involves in the process. The convergence behavior of this GA process illustrates in the same fashion as before. In Figure 7-3, the best fitness values of µGA are plotted not along the number of generations but the number of function evaluations. The average and best fitness values from the loaded GA module from Figure 7-2 are also plotted in Figure 7-3 to make the comparison of performance of these two modules.

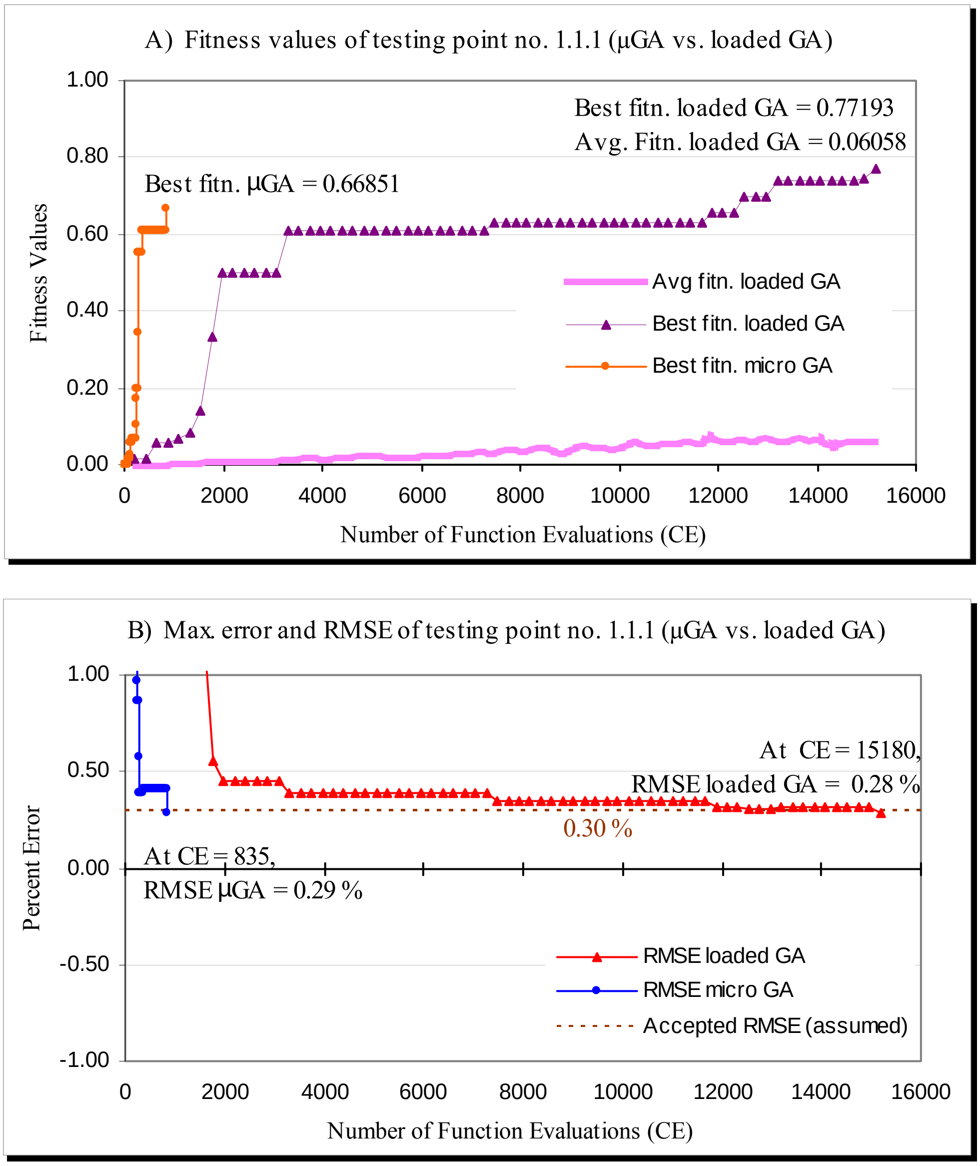


Figure 7-3. Comparison of convergence behavior of testing point no.1.1.1 (µGA vs. loaded GA)

Table 7-5. Backcalculated layer moduli of testing point no.1.1.1 (µGA)

Layer	Combined Asphalt layer, E_1	Stabilized Base, E_2	Unbound layer, E_3	Concrete slab, E_4	Earth Subgrade, E_5
Moduli [MPa]	11739	3282	194	27419	106

Again, the best fitness values of μGA climb up very fast to the near-optimal region comparing with those from the loaded GA. The μGA searching process stopped after the desired RMSE value of 0.29% (<0.30% shown by dotted line) is found. The more interesting is that μGA module uses only 835 times of function evaluations to find out the results. This is roughly 18 times less than those needed in the loaded GA module.

Table 7-5 shows the backcalculated moduli using the μGA module. Comparing these results with those from loaded GA, one could find out that the results obtained from both modules indicate the same trend of bearing capacity of this pavement structure. The moduli of the earth subgrade and unbound layer from both modules are nearly the same. Slightly differences can be seen from other layers. However, such variation on backcalculated moduli has only little effect on the requiring pavement overlay thicknesses [48]. This is due to the fact that the backcalculated deflection basins from both modules yield nearly the same RMSE value which, in turn, corresponds nearly the same bearing capacity of the entire pavement structure. Thus, the needed overlay requirement remains very close to each other.

It can be concluded that, the μGA module has shown the capability in solving the backcalculation problem using in situ deflection data by yielding reliable results and great improvement in required computing time. This module is therefore very suitable for routine backcalculation analysis.

7.4 Behavior of GAMLET Solutions Using Wide Range

It is well known that the traditional backcalculation programs based on the least residual method work fairly well with a good set of seed moduli. On the other hand, it has been reported that the solution obtained from such programs can be inappropriate from an engineering point of view or even clearly incorrect result when poor seed moduli are used. As discussed, one reason for this problem is the algorithms used in those programs do not have the capability to overcome the local minima problem which is the nature of backcalculation solution. By the fact that it is sometimes not easy to find out the appropriate seed moduli for real pavement based on lack of information of materials under the testing pavement.

As described, the GA-based backcalculation program GAMLET does not require any set of seed moduli, but only a range of possible modulus values for each layer. This should make the backcalculation process for user much more flexible. However, it may be interesting to investigate the behavior of the searching process of GA-based program when a wide range of modulus values is assigned to each layer. This issue is investigated using the both modules in GAMLET. The in situ deflection basins obtained from testing point no.1.1.1 is used as database again.

In order to investigate the behavior of the searching process with the wide range of possible solution, a constant of wide range of possible modulus values is assigned to each layer. This range (1 to 30000 MPa) could be considered as covering the possible moduli of all common materials used in flexible pavement structures.

The parameters used in the loaded GA module are presented in Table 7-6. The relative high number of 800 is used for maximum number of generations. With size of population equals to 120, this is equivalent to 960000 times of function evaluations if all generations are performed. Using this range of parameters and the assigned precision level, the GA process is meant to search for the optimal solution among as many as 2^{65} sets of possible solution. After running the µGA module, it is found, as expected, that this module could not perform well in such a large solution surface due to the relatively insufficient information in the searching process. Using the same set of parameters shown in Table 7-4 but wide range of possible modulus values of each layer, a set of backcalculated layer moduli with RMSE value of 3.76% is obtained.

Table 7-6. Parameters used in GAMLET (loaded GA) for solving with wide range moduli

Parameters	**Class SV line 2.1**
No. of backcalculated layer moduli	5
Size of population	120
Maximum generations	800
Probability of crossover	0.90
Single-point crossover	off
Uniform crossover	on
Probability of jump mutation	0.02
Probability of creep mutation	0.24
Elitism	on
Niching	off
FWDLine module	-
Range of possible moduli E_1 [MPa]	1 to 30000
Range of possible moduli E_2 [MPa]	1 to 30000
Range of possible moduli E_3 [MPa]	1 to 30000
Range of possible moduli E_4 [MPa]	1 to 30000
Range of possible moduli E_5 [MPa]	1 to 30000
Precision level [MPa]	5/ 5/ 5/ 5/5

For the loaded GA, the convergence behavior of this backcalculation problem is illustrated in Figure 7-4. Of interest in this investigation is the convergence behavior related to the number of function evaluations. All the convergence parameters are, therefore, plotted against this value.

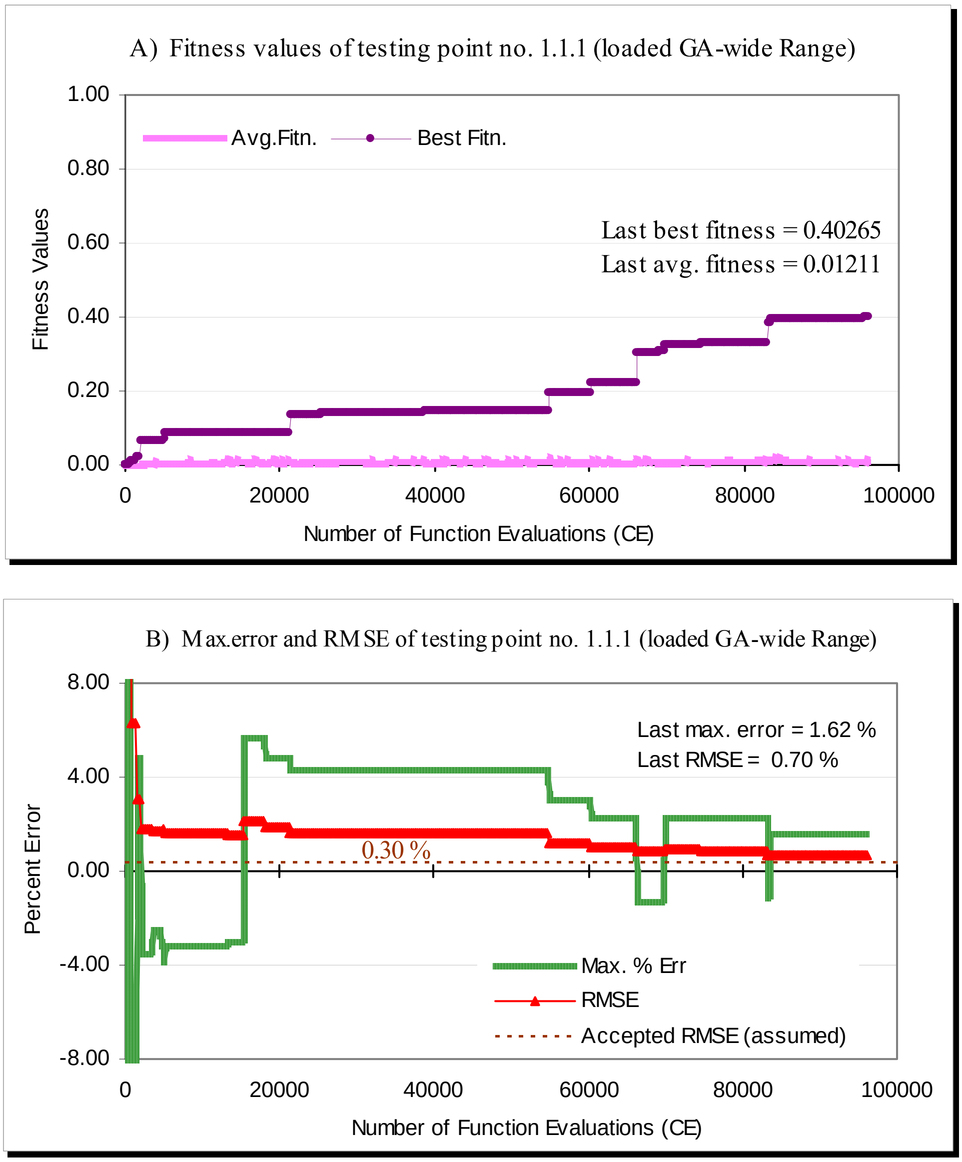


Figure 7-4. Convergence behavior of testing point no.1.1.1 using wide range of moduli

Table 7-7. Backcalculated layer moduli of testing point no.1.1.1 using wide range of moduli

Layer	Combined Asphalt layer, E_1	Stabilized Base, E_2	Unbound layer, E_3	Concrete slab, E_4	Earth Subgrade, E_5
Moduli [MPa]	12793	1781	231	19174	76

Not surprisingly, since the unusual range is used, the average fitness does not show an evident increase along the number of function evaluations. However, the elitism technique still works well by keeping the thus-far-best fitness value within the process. Hence, the best fitness line always shows the rising up in value. It is obvious that elitism technique is beneficial for this application.

The searching process performed until the last generation and stopped. The best acquired RMSE value is 0.70%. Even though the RMSE found in this backcalculation could not achieve the desired value (0.30%), the acquired RMSE value of 0.70% can be considered as fairly high accuracy based on the acceptable value used in most traditional programs (1 to 3%). This can be emphasized by considering the results moduli of this backcalculation shown in Table 7-7. It can be seen that the backcalculated layer moduli presented in this table are able to show the same trend as those found in the previous backcalculation analysis using more reasonable ranges of modulus values (see Table 7-3).

At this stage, it could be concluded that although the wide range of modulus values is used in the backcalculation program GAMLET, the new GA operators and techniques used in the loaded GA module make the backcalculation process robust and powerful enough to search for an accurate and reliable solution. However, assigning such wide ranges of possible moduli in GAMLET is, of course, not recommended since it is highly likely that the search process will need a very long computation time. In this investigation, it requires as many as 960000 times running of computer program BISAR. As discussed, since most of practical FWD projects involve producing a number of deflection basins, backcalculation using GA-based backcalculation program with a good estimated ranges of possible modulus values is always favorable.

8 Conclusions and Recommendations

8.1 Summaries and Conclusions

Nondestructive testing (NDT) has evidently become an important part of the structural evaluation of pavement structures in recent years. Backcalculation of in-service pavement layer moduli based on surface deflection measurements has been widely used for this purpose. Among the NDT equipments used for deflection measurement, falling weight deflectometer (FWD) has evolved as the favorite and most suitable device for pavement structural evaluation. Consequently, a number of procedures and computer programs have been developed for backcalculation of flexible pavement layer moduli using deflection basin obtained from FWD. Since most of pavement evaluation projects using FWD involve testing at multiple points along the pavement section, almost all computer programs using for routine backcalculation analysis are based on multi-layered elastic theory (MLET). A technique of running a number of iterations beginning with a set of initial (seed) moduli to find out the best overall match, or least error, between the measured and the backcalculated deflection basins has been used in common. Unfortunately, it has been found that the solution obtained from this traditional backcalculation technique may not always be appropriate from an engineering point of view, especially in case of analyzing in situ data.

The main reason may be that the solution surface of backcalculation problem contains by nature many local minima (multimodal problem). The algorithms used in backcalculation programs which do not have ability in searching for global minimum could face difficulties when a set of inappropriate seed moduli is used. Another independent problem in backcalculation is that there are many factors based on differences between realities and the initial assumptions of backcalculation process based on MLET which can lead to erroneous results. One main type of these errors is the pavement model errors. By this reason, another key to successful backcalculation is in setting up the pavement model correctly. There are at least two major sources of error in setting up pavement model. First is lacking of knowledge about bedrock which could have a profound effect on measured deflection basin when it lies at relative shallow depth. Second is nonlinear behavior of unbound materials which cannot be taken into account based on the initial assumptions of MLET. Moreover, the limitation of many backcalculation programs that allowing maximum number of layers only three or four layers in the pavement model can be a significant limitation when the pavement structure is complicated.

To overcome the problem of local minima, genetic algorithms (GAs) which have proved themselves capable of solving many large complex multimodal problems have been recently used in backcalculation analyses. Most of the existing GA-based backcalculation programs use simple genetic algorithm (SGA) as searching driver. Even though there have been reports about improvement in yielding high accuracy results compared with other traditional backcalculation programs, the first limitation of applying such programs to routine backcalculation analysis is the relatively long computing time required. Another limitation is the difficulty for user to find out the optimal set of GA parameters. Though GAs are robust enough to handle with some variations in the parameters, a poor choice of parameters set can results in poor performance of the programs. In addition, lack of literature reporting verification of the GA-based backcalculation programs using in situ data makes such programs less appealing for practitioners. Furthermore, some drawbacks of SGA were later found. Even though many new GA techniques have been developed to overcome those drawbacks and limitations, there are rarely reports about modification of the existing GA-based backcalculation programs.

In the view of all the problems explained above a new GA-based backcalculation program GAMLET has been developed in this work for assessing flexible pavement layer moduli. This program embodies the MLET-based computing program BISAR as forward mechanistic model. Unlike the existing GA-based backcalculation programs, GAMLET uses three evaluation functions in parallel to control the goodness of fit between calculated and measured deflection basins. These are maximum error at each sensor, root-mean-square error, and heuristic fitness function. Based on merits of GAs, GAMLET does not require the seed moduli but only lower and upper domain bounds of modulus values for each layer. These eliminate the dependency of solutions on input seed values and should make the backcalculation process easier for users. Moreover GAMLET is capable for backcalculating pavement model with a number of layers. This should give user more flexibility in setting up pavement models.

In GAMLET two different GA modules, loaded GA and μGA, are made available for user. The loaded-GA module contains many new GA operators and techniques: tournament selection scheme, uniform crossover, creep mutation, elitism, niche, and FWDLine. These new techniques are employed in order to improve the performance and robustness of searching process compared with SGA. After a series of computer runs, two recommended sets of basic GA parameters which could be used for analysis wide range of backcalculation problems using GAMLET have been given in this work. On the other hand, the μGA module is utilized to overcome the two main problems encountered in most existing GA-based backcalculation programs, the relatively long computing time and the difficulty in finding optimal set of GA parameters. Since the μGA module uses very small size of population and only crossover operator involves in process, this should make GAMLET more appealing for routine backcalculation analysis.

The performance of both modules has been evaluated by performing backcalculations involving different pavement models and comparing the results with those obtained from other well-known backcalculation programs which are based on different algorithms. The loaded GA module performed very well in term of yielding results with high accuracy compared with the other programs. However, since the strength of the loaded GA is the improvement of robustness in search process but not reducing the computing time and there are more GA parameters involve in this module, the loaded GA is recommended for analyzing the FWD data at project level and should be used by experienced user. Considering backcalculation of the same problems using the μGA module, it has been found that although the results in most cases were slightly inaccurate than those obtained from the loaded GA, the results obtained from the μGA could be considered as equally satisfactory and these results were still better than those obtained from most of the conventional programs used in the comparison. Major difference was that the μGA demonstrated obviously faster (roughly ten times) convergence to the near-optimal region than did the loaded GA.

On the other hand, there have been some methods proposed for determining depth to bedrock (DTB) beneath the pavement from FWD data. The result could be used to improve the quality of setting up pavement model. At the same time, it has been found that adding the artificial bedrock into pavement model at some depth under the subgrade layer is able, in most cases, to improve the convergence behavior of iterative backcalculation programs. Moreover, using this technique is able to eliminate the problem of inverse structure result, since this technique is able in some degree to deal with the behavior of subgrade stiffening with depth which is found in most subgrade materials. Put these concepts together, the DTB obtaining from the existing methods could be used to assign the depth of the artificial bedrock in the pavement model.

In order to indicate the appropriate method for determining DTB, the deflection and inverse offset method and the regression equations were selected based on their feasibilities for applying to MLET-based backcalculation program. These two procedures have been investigated in this work. A number of deflection basins generated from the forward computer program and those obtained from FWD test on the test track which was built above a 1.5 meters thick concrete slab have been used as database in the investigation. Unfortunately, both procedures did not show a good performance in predicting the DTB from the artificial basins. In the case of predicting DTB using FWD data from test track, both procedures yielded not only inaccurate results but they could not indicate the existence of the concrete slab beneath the pavement structure class SV which has thicker upper (bound) layers.

In the view of the limitations of the existing methods for determining DTB from FWD data and an attempt to use the artificial bedrock in the pavement model to deal with subgrade stiffening with depth, some related issues about influences of bedrock on backcalculated moduli have been investigated. A new heuristic method for determining depth to artificial bedrock (DTAB) coined as the “consistent slope changing method” (CSCM) has been proposed in this work. The verification of CSCM has been performed using computed deflection basins obtained from a number of pavement models in which subgrade moduli are stiffening with depth. The results from backcalculation procedure based on finite element method (FEM) available in literature have been used as guidelines for setting the stiffening behavior of typical subgrade soils. The deflection basins obtained from pavement models with artificial bedrock at the depth computed with the CSCM showed the capability of matching those from pavement models with stiffening subgrade. For the case of in situ data, the deflection data from the test track at BASt have been used in the verification. The depth results obtained from the CSCM demonstrated an uncanny ability to estimate the correct value of the concrete slab beneath the test track. Only small difference in results can be observed from the different construction classes of the pavement structures.

Furthermore, some advantages of the CSCM compared with the existing methods have been noted. Since the depth result from the existing methods is computed from only a few deflections of the entire basin. This could make such methods sensitive to FWD measurement errors while CSCM uses almost all FWD deflections (except d_0) to compute the depth result. Thus, the influence of the measurement errors on the CSCM method is automatically reduced which, in turn, improves the reliability of this method. Moreover, the method of regression equations requires that four sensors must be placed at the predefined locations, this makes such method suddenly inapplicable when the positions of sensors have to be changed. The examination of the robustness of CSCM showed that only trivial differences of results can be observed when the number and/or the positions of sensors are changed. This feature makes the CSCM applicable for analyzing the in situ deflection data obtained from general case of FWD test. The concept of CSCM has been added into the GA-based backcalculation program GAMLET as an optional function. The objective of this function in GAMLET is to help the user in making decision about setting up the pavement model correctly.

The complete developed GA-based backcalculation program GAMLET has been verified using the in situ deflection basins obtained from the test track at BASt. The backcalculations of in situ data using both modules in GAMLET have been thoroughly explained. Both the loaded and μGA modules showed capabilities in backcalculating the in situ deflection data by achieving the desired high result accuracy (RMSE = 0.30%) and also yielded the associated reliable backcalculated moduli based on the comparison with the predicted moduli available in literature. The major difference between these two modules was, again, the computational effort used in each module. While the loaded GA required as many as 15,180 times running of BISAR, the μGA used as few as 835 times running of the same forward program or roughly 18 times less than those used in the loaded GA. It can be concluded that μGA module has overcome the two main limitations, the relatively long computing time and the difficulty in finding optimal set of GA parameters, encountered in most existing GA-based backcalculation programs. This module improves obviously the efficiency of the GA-based backcalculation program and makes GAMLET more user-friendly and practical for routine backcalculation analysis.

A further investigation of convergence behavior of GAMLET using unusual wide range of possible modulus values has been made. As expected, the µGA did not perform well in such a large solution surface due to relatively insufficient information in search process. For the loaded GA module, elitism technique still worked fairly well by keeping the thus-far-best fitness value within the process. In effect, the best fitness line showed always the rising up in value. After 960,000 times of function evaluations in the last generation, the RMSE value of 0.70% has been obtained. This value can still be considered as good accuracy compared with acceptable value used in most traditional programs. Moreover, the backcalculated layer moduli still showed the same order of bearing capacity as those found in backcalculation using reasonable ranges of possible modulus values. These results implies that the new GA operators and techniques used in the loaded GA module make the backcalculation process robust and powerful enough to search for a good solution even though a wide range of possible modulus values is used. However, assigning such wide ranges of possible moduli in GAMLET is, of course, not recommended since it is obvious that the search process needs longer computing time. By this reason, a backcalculation using GA-based backcalculation program with a good estimated ranges of modulus values is always favorable and hence the thorough knowledge of the testing pavement structure and its materials is absolutely very important for successful backcalculation.

8.2 Recommendations for Further Studies

It has been pointed out many times in this work that there are many factors involving in backcalculation analysis that can lead to erroneous results. The overall objective of improvement the backcalculation approach is, of course, eliminating those factors and keeping the approach still practical for routine work. An attempt has been given in this work to enhance the backcalculation techniques for assessing flexible pavement layer moduli by taking some of those factors into account. Recommendations for further studies based on this work are mad here.

- Although the loaded GA module in GAMLET has shown the improvement in robustness of searching process and excellent potential of solving backcalculation problems using deflection basins from both computer pavement models and in situ data, the sets of GA parameters used in this work have been chosen based on experience of a number of running this program. Since there are no general guidelines available at present for selection of such parameters, a study conducting for the selection of optimal set of GA parameters for backcalculation of flexible pavement moduli should be carried out.
- A series of more accurate GA-based backcalculation program could be developed for using in research area by combining the same GA techniques used in the loaded and µGA with other more reliable forward mechanistic models

such as nonlinear elastic approach, FEM, discrete element method, visco-elasticity method, and dynamic analysis.

- Since μGA requires usually much less computing time, the development of backcalculation of the in situ FWD data using the μGA module in GAMLET at measurement time (real time process) might be useful for practitioner to evaluate the general state of the pavement structure tested.
- Since the modulus value of 30,000 MPa has been indicated as the most appropriate value for assigning to the artificial bedrock in pavement model for the iterative backcalculation program VAHREN1 based on the improvement of convergence behavior of this program, an appropriate range of modulus value of the artificial bedrock which will be used in GAMLET to deal with subgrade stiffening behavior should be further examined.
- Study about how the stiffness of each of typical materials in unbound layers change in horizontal and vertical direction under real traffic load and FWD load should be performed using more reliable analytical procedures such as FEM or discrete element method. The results may be useful for improving or modifying the procedure proposed in CSCM.
- In recent years, there have been many attempts to use SGA for solving several optimization problems in the field of pavement engineering. Since the GA techniques used in the loaded GA and the μGA have shown the improvement compared with SGA, investigations of using these GA techniques instead the SGA should be performed in order to improve the performance of those SGA-based procedures.

References

[1] AASTHO: *AASTHO Guide for Design of Pavement Structure*. American Association of State Highway and Transportation Officials, 1993.

[2] Alkasawneh, W.: *Backcalculation of Pavement Moduli Using Genetic Algorithms.* Ph.D. Thesis, The University of Akron, Akron, 2007.

[3] Almeida, J: R.: *Backcalculation of Flexible Pavements with Consideration of Temperature Gradients*. 5th International Conference on the bearing capacity of roads and airfields. Vol. 1, Nordal, R. S.; Refsdal, G. (Editors), pp.487-496, Trondheim, TAPIR, 1998. ISBN – 82-519-1346-2

[4] Aoud M.F., Stokoe II, K. H., Joh, S.H., *Estimation Subgrade Stiffness and Bedrock Depth: Combined Falling Weigh Deflectometer and Simplified Spectral Analysis of Surface Waves Measurements*. Transportation Research Record No. 1716, TRB, Washington, D.C., 2000

[5] Arbeitsgruppe „Fahrzeug und Fahrbahn“: *Merkblatt über Einsenkungsmessungen mit dem Benkelman-Balken.* Forschungsgesellschaft für Straßen- und Verkehrswesen, Köln, 1991.

[6] Arbeitsgruppe Fahrzeug und Fahrbahn, *Richtlinien für die Standardisierung des Oberbaues von Verkehrsflächen RstO’01*. Forschungsgesellschaft für Strassen- und Verkehrswesen, Köln, 2001

[7] Beasley, D.; Bull, D.R.; *Martin, R.R.: An overview of Genetic algorithms: Part 2, Research Topic*. University Computing, 1993.

[8] Briggs, R.C. and Nazarian, S.: *Effect of Unknown Rigid Subgrade Layers on Backcalculation of Pavement Moduli and Projections of Pavement Performance.* Transportation Research Record 1227, TRB, pp.183-193, Washington, D.C., 1989

[9] Bundesanstalt für Straßenwesen (BaSt), *Bau einer instrumentierten Modellstraße in Asphaltbauweise zur messtechnischen Erfassung der Beanspruchungssituation im Straßenbau*. AP-Project 03 342, Bergisch Gladbach, 2004

[10] Bush, A.J.: *Nondestructive Testing for Light Aircraft Pavements; Phase II, Development of The Nondestructive Evaluation Methodology.* Report FFA-RD-80-9-II; U.S. Department of Transportation, Washington, D.C., 1980.

[11] Caroll, D.L.: *Chemical Laser Modelling with Genetic Algorithms.* AIAA Journal, Vol. 34 (No.2), pp. 338-346, 1996

[12] Caroll, D.L.: *Genetic Algorithms and Optimizing Chemical Oxygen-Iodine Lasers.* Development in Theoretical and Applied Mechanics, Vol. XVIII, pp.411-424, Wilson, H.B.; Batra, R.C.; Bert, C.W.; Davis, A.M.J.; Schapery, R.A.; Stewart, D.S.; Swinson, F.F. (Editors), The University of Alabama, 1996

[13] Ceylan, H.; Guclu, A.; Tutumluer, E.; Thompson, M.R.; *Use of Neural Networks for Backcalculation of Pavement Layer Moduli.* 2004 FWD Users Group Meeting, West Lafayette, Indiana, 2004

[14] Chang J. R., Hung C. T., Chen D.H., *Application of an Artificial Neural Network on Depth to Bedrock Prediction*. International Journal of Computational Intelligence Research, pp. 33-39, 2006

[15] Chang, R.S., Stokoe, K.H. II, Roesset, J.M., *Effect of Depth to Bedrock on The Accuracy of Backcalculated Moduli Obtained with Dynaflect and FWD Test*. Research Report 1175-5, Center for Transportation Research, The University of Texas at Austin, Texas, 1993

[16] Chen, D-H, Bilyeu, J., He, R., *Comparison of Resilient Moduli between Field and Laboratory Testing.* 78th Annual Meeting of the Transportation Research Board , Washington D.C., 1999

[17] Chen, D-H: *Determination of Bedrock Depth from Falling Weight Deflectometer.* Transportation Research Record, Paper No. 99-0589, TRB, pp. 127-134, Washington, D.C., 1999

[18] Chou, K.M., *Evaluation of Moduli Backcalculation Program for Low Volume Roads*. Nondestructive Testing of Pavement and Backcalculation of Moduli, ASTM STP 1026, Philadelphia, 1989

[19] Chou, Y., Development of an Expert System for Nondestructive Pavement Structural Evaluation. PhD Dissertation, Texas A&M University, College Station, Texas, 1985

[20] CROW, Deflection profile not a pitfall anymore. CROW Record 17, 1997 – ISBN 90 6628 275 4

[21] De Jong, D. L.; Peutz, M.G.F.; Korswagen, A.R., *Computer Program BISAR Layered Systems under Normal and Tangential Surface Loads*. Koninklijke / Shell-Laboratiorium, Amsterdam, 1979

[22] Deb, K.; Goldberg, D.E.: *An Investigation of Niche and Species Formation in Genetic Function Optimization.* 3rd International Conference on Genetic Algorithms. Schaffer J.D. (Editor), pp. 42-50, Morgan Kaufmann, 1989.

[23] Department of Highway Thailand: *Pavement Technology Project: Implementation of Falling Weight Deflectometer Technology and Development of Analytical Pavement Design in Thailand.* Bangkok, 2001

[24] DOH: Department of Highway Thailand, *FWD Testing Procedure.* Personally Contact (Not Published), Ministry of Transport, Thailand, Bangkok.

[25] Dynatest International A/S: *Elmod 5 Quick Start Manual.*

[26] Ehrola, E.; Belt, J.; Ryynänen, T.: *Influence of temperature on the deflection of asphalt pavements.* 3rd International Conference on Bearing Capacity of Roads and Airfields, Trondheim, 1990.

[27] El-Nakib, M.: *Faulting in Rigid Pavement Systems of Highways.* Ph.D. Dissertation, Pavement Engineering Section, University of Hannover, Hannover, 2006.

[28] Fwa, T.F.; Tan, C.Y.; Chan, W.T.: *Backcalculation Analysis of Pavement-Layer Moduli Using Genetic Algorithm.* Transportation Research Record No. 1570, TRB, pp. 134-142, Washington, D.C., 1997

[29] Fwa, T.W.; Rani, T. S., *Seed Modulus Generation Algorithm for Backcalculation of Flexible Pavement Moduli*. Transportation Research Record 1905, 2005

[30] Goldberg, D.E.: *Genetic Algorithms in Search Optimization, and Machine Learning*. Reading, M.A., Addison-Wesley, 1989 – ISBN 0-201-15767-5

[31] Goldberg, D.E.: *Real-coded Genetic Algorithms, Virtual Alphabets*, and Blockings. Complex System, Vol.5, pp. 139-167, Complex System Pub. 1991.

[32] Goldberg, D.E.: *Sizing Populations for Serial and Parallel Genetic Algorithms*. TCGA Report No.88004. University of Alabama The Clearing House of Genetic Algorithms, 1988.

[33] Goldberg, D.E.; Deb, K.; Clark, J.H.: *Genetic Algorithms, Noise, and the Sizing of Populations*. Complex System Vol.6, pp.333-362, Complex Systems Publications, 1992.

[34] Goldberg, D.E.; Debb, K.: *Comparative Analysis of Selection Schemes used in Genetic Algorithms*. Foundation of Genetic Algorithms, pp.69-93, Rawlins,G.J.E. (Editor), 1991.

[35] Goldberg, D.E.; Richardson, J.: *Genetic algorithms with sharing for multimodal function optimization*. Genetic algorithms and their applications: Proceedings of the Second International Conference on Genetic Algoritms, pp. 41-49, 1987.

[36] Harichandran, R.S.; Mahmood, T.; Raab, A.R.; Baladi, G.Y.: *Modified Newton Algorithm for Backcalculation of Pavement Layer Properties.* Transportation Research Record 1384, TRB, pp. 15-22, Washington, D.C., 1993.

[37] Haupt, R.; Haupt, S.E.: *Practical Genetic Algorithms.* 2nd Edition. New Jersey: Wiley & Sons, Inc., 2004. ISBN – 0-471-45565-2

[38] Hiersche,E.U.; Freunde, H.J.; Kiuntke, M.: *Bewährung von Asphaltbefestigungen mit hohen Anteilen von Ausbauasphalt.* Forschungsvorhaben FE 07. 160 G 91 L.

[39] Holland, J.: *Adaptation in Natural and Artificial Systems.* The University of Michigan, Michigan, The University of Michigan Press, 1975. ISBN – 0-472-08460-7

[40] Horz, H.W.: *Tragfähigkeitsmessungen und –auswertungen.* Bundesanstalt für Straßenwesen (BASt), Bergisch Gladbach, 1994.

[41] Hothan J.: *Source Code of Backcalculation Program for Three Pavement Layers System DREIFFM and VAHREN1.* Personally Contact (Not Published). Pavement Engineering Section, University of Hannover, Hannover

[42] Hothan, J.: *Feasibilities and Limitations of Backcalculation Procedures.* 5th International Conference on the bearing capacity of roads and airfields. Vol. 1, Nordal, R. S.; Refsdal, G. (Editors), pp.403-408, Trondheim, TAPIR, 1998. ISBN – 82-519-1346-2

[43] Hothan, J.: *Manuskript zur Vorlesung TV 26209: Fahrbahnbefestigung.* Pavement Engineering Section, University of Hannover, Hannover. 2004

[44] Hothan, J.; Förster, M-O: *Theoretische Betrachtungen zur Prognose des Schädigungspotenzials realer Verkehrsbelastungen*. Straßen und Autobahn, pp.70-77, 2005

[45] Huang, Y.H.: *Pavement Analysis and Design*. 2nd Edition, Pearson Prentice Hall, 2004. ISBN – 0-13-142473-4

[46] Irwin, L.H.: *Backcalculation: An Overview and Perspective.* 6th International Conference on the Bearing Capacity of Roads, Railways and Airfields, Lisbon, 2002

[47] Irwin, L.H.: *Instruction Guide for Backcalculation and the Use of MODCOMP 3 Version 3.6.* Cornell University, CLRP Publication No. 93-6, 1993

[48] Irwin,L.H. ; Yang, W.S. ; Stubstad, R.N. : *Deflection Reading Accuracy and Layer Thickness Accuracy in Backcalculation of Pavement Layer Moduli.* Baladi, G.Y.; Bush III, A.J. (Editor). Nondestructive Testing of Pavements and Backcalculation of Moduli, ASTM STP 1206, 1988

[49] Jendia, S.: *Bewertung der Tragfähigkeit bon bituminösen Straßenbefestigungen.* Ph.D. Thesis, Instiut für Straßen- und Eisenbahnwesen. Universität Karlsruhe. Karlsruhe (TH). 1995.

[50] Jessberger, H.L.: *Die Tragfähigkeit von frostempfindlichem Boden nach Auftauen als Parameter für die Bemessung von Straßen sowie als Maßstab für die Bewertung von Frostkriterien*. Straßen und Autobahn, Heft 12, pp. 511-519, Köln, 1973.

[51] Kameyama, S.; Kasahara, A.; Himeno, K.; *Maruyama, T.: Backcalculation of Pavement Layer Moduli Using Genetic Algorithms.* 8th International Conference on Asphalt Pavements. pp.1375-1385, Seattle, Washington, 1998

[52] Kameyama, S.; Matsui K.; Kasahara, A.; Himeno K.; Maruyama T.: *Effect of FWD sensor locations on backcalculated moduli.* 5th International Conference on the bearing capacity of roads and airfields. Vol. 1, Nordal, R. S.; Refsdal, G. (Editors), pp.477-485, Trondheim, TAPIR, 1998. ISBN – 82-519-1346-2

[53] Kim, Y.R.; Mun, S: *Determination of Subgrade Strength under Intact Portland Cement Concrete Slabs for Rubblization Projects.* Department of Civil Engineering, North Carolina Department of Transportation. Report No. FHWA/NV (2002-010). 2002.

[54] Kinchen. R.W.; Temple, W.H.: Asphaltic concrete overlays of rigid and flexible pavements. Report No. FHWA/LA-80/147, Louisiana Department of Transportation and Development, 1980.

[55] Koss, S.: *Untersuchung und Bewertung der stratischen Rückrechnung von Materialparametern aus Messungen mit dem Falling Weight Deflectometer (FWD).* Diplomarbeit. Pavement Engineering Section, University of Hannover, Hannover, 2006.

[56] Krishnakumar, K.: *Micro-Genetic Algorithms for Stationary and Non-Stationary Function Optimization.* SPIE: Intelligent Control and Adaptive Systems. Vol. 1196, Philadelphia, PA, 1989.

[57] Lytton, R.L.: *Backcalculation of Pavement Layer Properties.* Baladi, G.Y.; Bush III, A.J. (Editor). Nondestructive Testing of Pavements and Backcalculation of Moduli, ASTM STP 1206, 1988

[58] Marchionna A.: Cesarini M., Fornaci M., Malgarini M., *Pavement Elastic Characteristic Measured by Means of Tests Conducted with the Falling Weight Deflectometer.* Transportation Research Record No. 1007, 1985

[59] Mork, H.: *The Effect of Sensor Configuration on The Backcalculation of Layer Moduli.* 5th International Conference on the bearing capacity of roads and airfields. Vol. 1, Nordal, R. S.; Refsdal, G. (Editors), pp.467-476, Trondheim, TAPIR, 1998. ISBN – 82-519-1346-2

[60] Nazarian, S.; Boddspati, K.M.: *Pavement-Falling Weight Deflection Interaction Using Dynamic Finite Element Analysis*. Transportation Research Record No. 1449, TRB, pp. 123-133, Washington, D.C., 2000

[61] Paine, J.G.: Murphy, M.R., *Pavement Deflection and Seismic Refraction for Determining Bedrock Type, Depth, and Physical Properties beneath Roads*. Bureau of Economic Geology, The University of Texas at Austin, Texas, 2000

[62] Park, H.M.; Park, S.W.; Hwang, J.J.: *Use of genetic algorithm and finite element method for backcalculating layer moduli in asphalt pavements.* Transportation Research Record No. 07-2614, TRB, Washington, D.C., 2006

[63] Rakesh, N.; Jain, A.K.; Reddy, M.A.; Reddy, K.S.: *Artificial neural networks-genetic algorithm based model for backcalculation of pavement layer moduli.* International Journal of Pavement Engineering, Vol. 7 (No.3), pp.221-230, 2006.

[64] Reddy, M.A.; Murthy, M.S.; Reddy, K.S.; Pandey, B.B.: *Backcalculation of pavement layer moduli using genetic algorithms*. Journal of Highway Research Board 66, pp.1-10, Indian Roads Congress, New Delhi, 2002.

[65] Reddy, M.A.; Reddy, K.S.; Pandey, B.B.: *Selection of Genetic Algorithm Parameters for Backcalculation of Pavement Layer Moduli.* The International Journal of Pavement Engineering, Vol. 5 (No.2), pp.81-90, 2004

[66] Reeves,C.R.; Rowe, J.E.: Genetic Algoritms – Principles and Perspectives. Kluwer Academic Publishers, 2003. ISBN – 1-4020-7240-6

[67] Riedl, S.: *Rückrechnung dynamischer Tragfähigkeitswerte aus den Messdaten des Falling Weight Deflectomter (FWD).* PhD Dissertation. Fachgebiet Strassenwesen mit Versuchsanstalt. Technische Universitaet Darmstadt, 2006

[68] Rohde, G.T.; Smith, R.E.: *Determining Depth to Apparent Stiff Layer from FWD Data.* Texas Department of Transportation, Report No. FHAW/TX-91-1159-1, Texas, 1991

[69] Roque, R.; Ruth, B.E.; Sedwick, S.C.: *Limitations of Backcalculation and Improved Methods for Pavements Layer Moduli Predictions.* 5th International Conference on the bearing capacity of roads and airfields. Vol. 1, Nordal, R. S.; Refsdal, G. (Editors), pp.409-417, Trondheim, TAPIR, 1998. ISBN – 82-519-1346-2

[70] Salt, G.; Stevens, D.; *McLeod, R.: Pavement Structural Evaluation: Production Level FWD Back-Analysis Using Full Time History.* 6th International Conference on the bearing capacity of roads and airfields. Vol. 1, Correia, A.G.; Branco, F.E.F. (Editors), pp.715-720, Portugal, Swets&Zeitlinger B.V. 2002. ISBN – 90 5809 396 4

[71] Schäfer, F. : *Analyse der Bewertung von Tragfähigkeitsmessungen*. Diplomarbeit, Fachgebiet Konstruktiver Straßenbau, Leibniz Universität Hannover, Hannover, 2003

[72] Seng, C.R.; Stokoe II, K.H.; Roesset, J.M. : *Effect of Depth to Bedrock on the Accuracy of Backcalculated Moduli Obtained with Dynaflect and FWD Test.* Center for Transportaion Research, The University of Texas at Austin, 1993

[73] Stubstad, R.N.; Irwin, L.H.; Lukanen, E.O.; Clevensen, M.L.: *It's 10 O'clock. Do You known where your sensors are?*. Transportation Research Record No. 1716, TRB, pp. 10-19, Washington, D.C., 2000

[74] Syswerda, G.: *Uniform Crossover in Genetic Algorithms.* 3rd International Conference on Genetic Algorithms. Schaffer J.D. (Editor), pp.2-9, Morgan Kaufmann, 1989.

[75] Thomson, M.R., Robnett, Q.L.: Resilient response of subgrade soils. Studies Series No. 160, Civil Engrg., University of Illinois, Urbana-Champaign, Illinois, 1976.

[76] Thongindam, P.: *Proposed Method for Determining Depth to Bedrock.* Internal Research Report, Pavement of Engineering Section, Leibniz University of Hannover, Hannover, 2007.

[77] Tsai B.W.; Kannekanti, V, N.; Harvey, J.T.: *Application of Genetic Algorithm in Asphalt Pavement Design.* Transportation Research Record 1227, TRB, Washington, D.C., pp. 112-120, 2004

[78] Ullidtz, P.: *Modelling flexible Pavement Response and Performance.* Lungby: Polyteknisk Forl, 1998 – ISBN 87-502-0805-5

[79] Ullidtz, P.: *Pavement Analysis.* Lyngby, Denmark: Elsevier science publisher B.V., 1987. ISBN – 0-444-42817-8

[80] Ullidtz, P.; Stubstad, R.N.: *Analytical-Empirical Pavement Evaluation Using The Falling Weight Deflectometer.* Transportation Research Record 1022, TRB, pp. 36-43, Washington, D.C., 1985

[81] Uzan, J.: *Estimation of Subgrade Thickness from Deflection Basin Analyses.* Rasmus, S.; Refsdal, N.; Geir, N. (Editor). 5th International Conference on the Bearing Capacity of Roads and Airfields, Volume 1, Trondheim, 1998 – ISBN 82-519-1346-2

[82] Uzan, J.; Lytton, R.L.; German F.P. : *General Procedure for Backcalculating Layer Moduli.* Baladi, G.Y.; Bush III, A.J. (Editor). Nondestructive Testing of Pavements and Backcalculation of Moduli, ASTM STP 1206, 1988

[83] Vaswani, N.K.: *Method for separately evaluating structural performance of subgrades and overlying flexible pavements*. Highway Res. Record 3612, Highway Res. Board, pp. 48-62, 1971.

[84] Von Becker, P.: *Zur Annahme wirklichkeitsnäherer E-Moduli als Kennwerte für das elastische Verformungsverhalten flexibler Straßenbefestigungen bei elastizitätstheoretischen Beanspruchsrechnungen*. Forschung Straßenbau und Straßenverkehrstechnik, Heft 204, Bundesministerium für Verkehr, Bonn, 1976.

[85] Witczak, M.W.; Uzan, J.: *The universal airport pavement design system*. Report I of V, Granular material characterization, Dept. of Civil Eng., University of Maryland, College Park, Maryland, 1988.

[86] Yang, G.; Reinstein, L.E.; Pai, S.; Xu, Z.; Caroll, D.L.: *A new genetic algorithm technique in optimization of permanent 125-I prostate implants.* Medical Physics, Vol. 25 (No.12), pp. 2308-2315, 1998

[87] Yoder, E.J.; Witczak, M.W.: Principles of Pavement Design. 2nd Edition, John Wiley & Sons, New York, 1975.

[88] Zörner, C. : *Analyse der Bewertung von Tragfähigkeitsmessungen unter Anwendung eines "bedrock".* Studienarbeit. Pavement Engineering Section, University of Hannover, Hannover, 2006.

Appendices

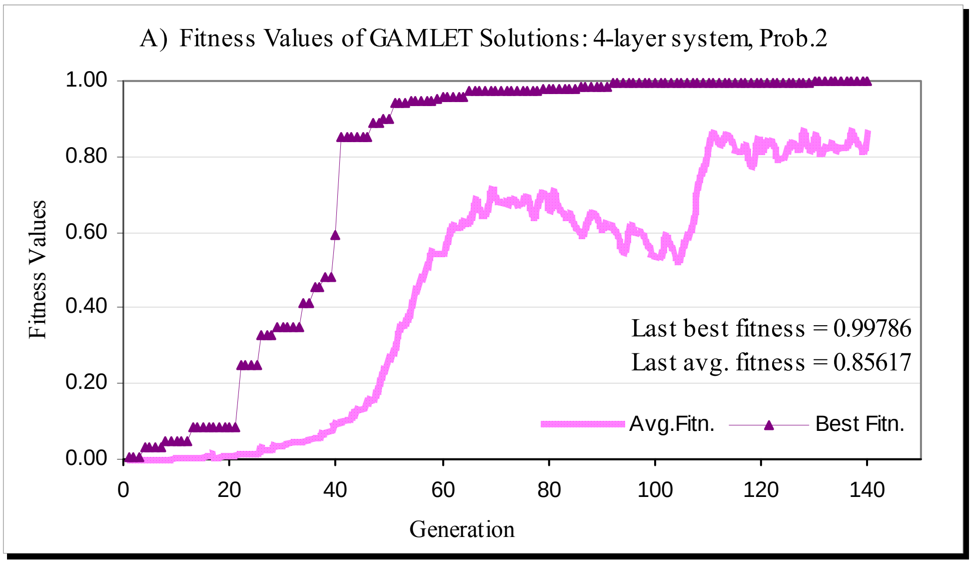
Appendix A: Parameter names and units used in GAMLET

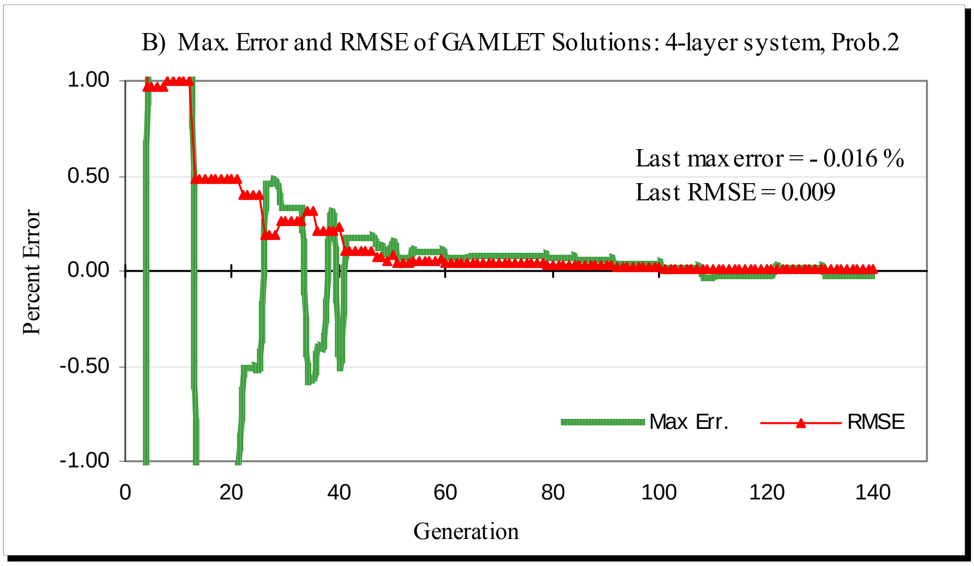
Parameter names	Descriptions	Unit
nlayer	Number of layers to be backcalculated	
pminmodl	Lower bound of possible moduli	[MPa]
pmaxmodl	Upper bound of possible moduli	[MPa]
nmodul	Number of possibilities in each layer	-
maxgen	Maximum number of generations	-
numpop	Number of population in each generation	-
pcross	Probability of crossover	-
pjumpmu	Probability of jump mutation	-
pcreepmu	Probability of creep mutation	-
ibstmodl	Elitism (keeping thus-far-best moduli set)	-
niching	Niche	-
fwdline	FWD line module	-
CSCM	Consistent Slope Changing Method	-
	Input Deflection values	[μm]
-	Input load pressure	[MPa]
-	Plate radius	[mm]
-	Sensor offset distance	[mm]
-	Poisson’s ratio	-
-	Layer thickness	[mm]

Appendix B: Convergence behavior of GAMLET solutions

<u>The Loaded GA</u>

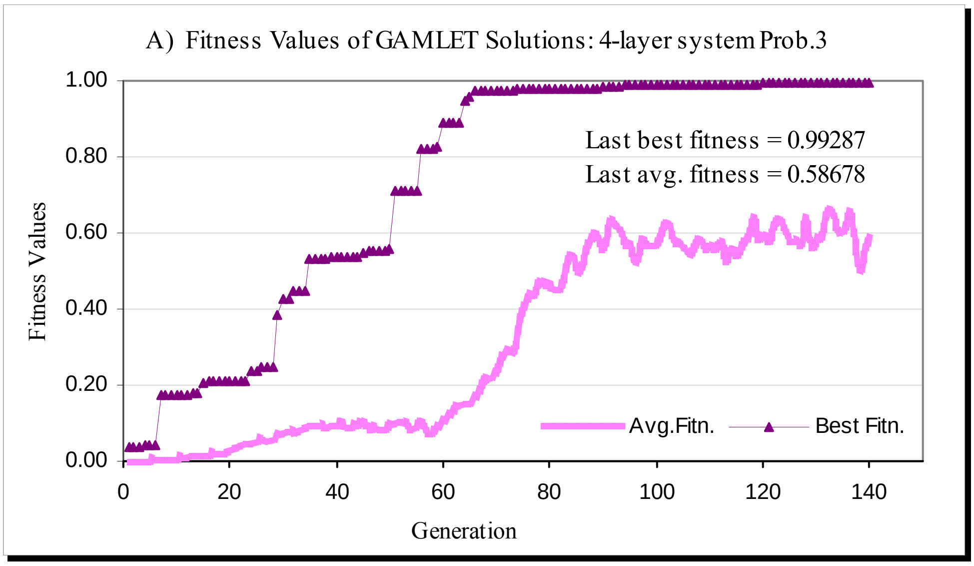
Problem 2: The four-layer flexible composite pavement with a stabilized subbase layer

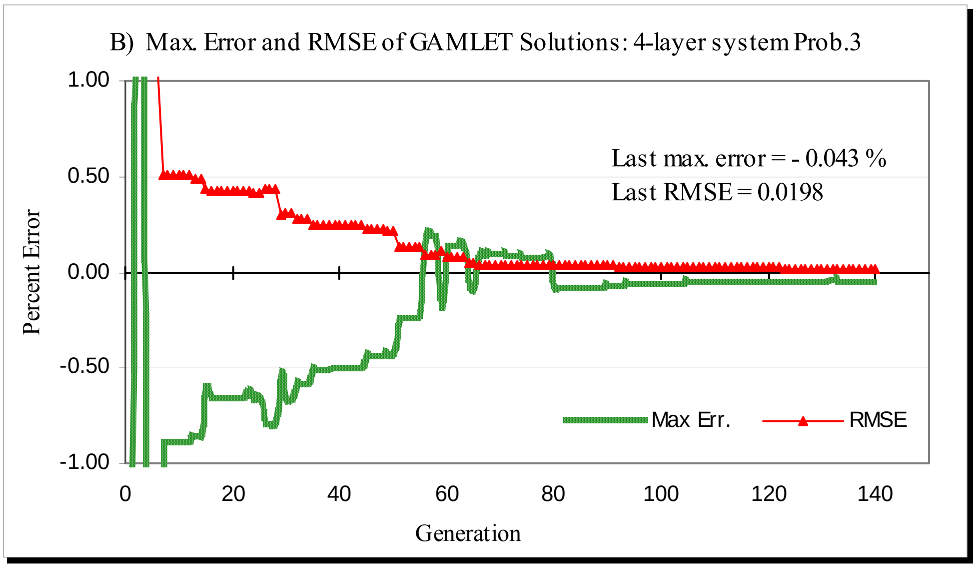




GAMLET module	Backcalculated Modulus [MPa]				max. ΔM* (%)	RMSE (%)
	AC	Base	Stabilized subbase	Sub grade		
<u>Input Modulus values</u>	<u>3447.38</u>	<u>172.37</u>	<u>31026.40</u>	<u>51.71</u>	-	-
Loaded GA	3447.90	172.13	31145.99*	51.72	0.38	0.009

Problem 3: The four-layer flexible composite pavement with stabilized base

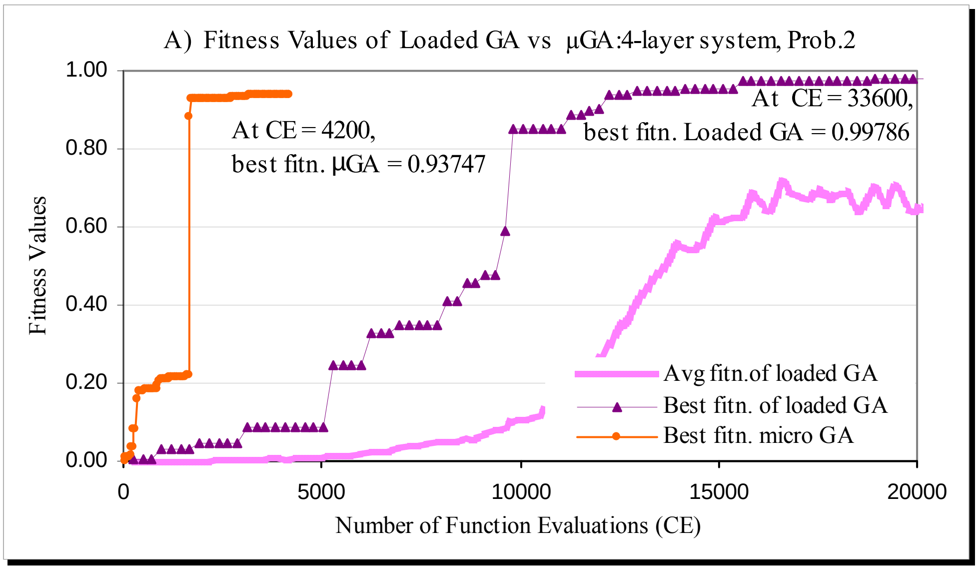


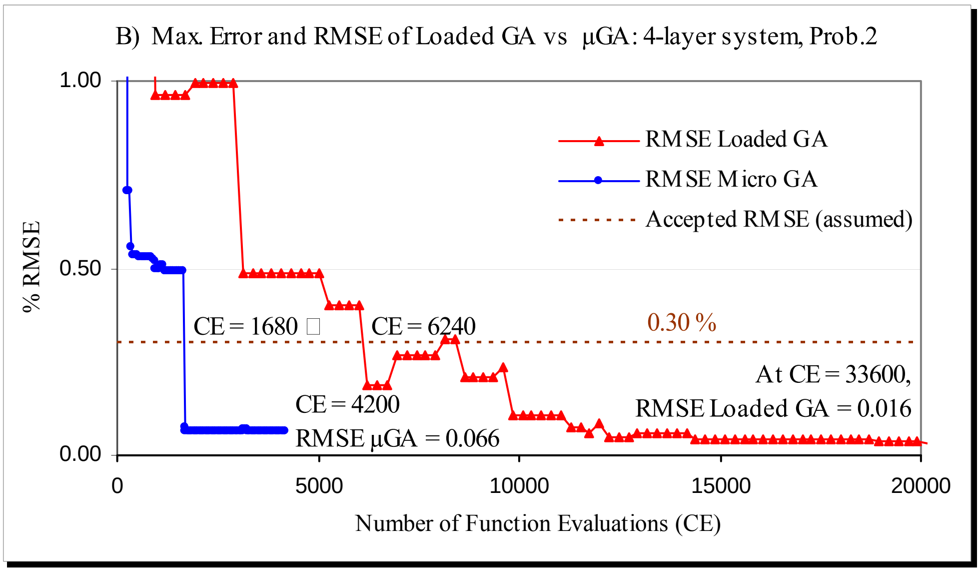


GAMLET module	Backcalculated Modulus [MPa]				max. ΔM* (%)	RMSE (%)
	AC	Stabilized base	Sub base	Sub grade		
Input Modulus values	3447.38	31026.40	172.37	51.71	-	-
Loaded GA	3456.85	30970.36	166.43	51.76	3.44	0.019

The Micro GA

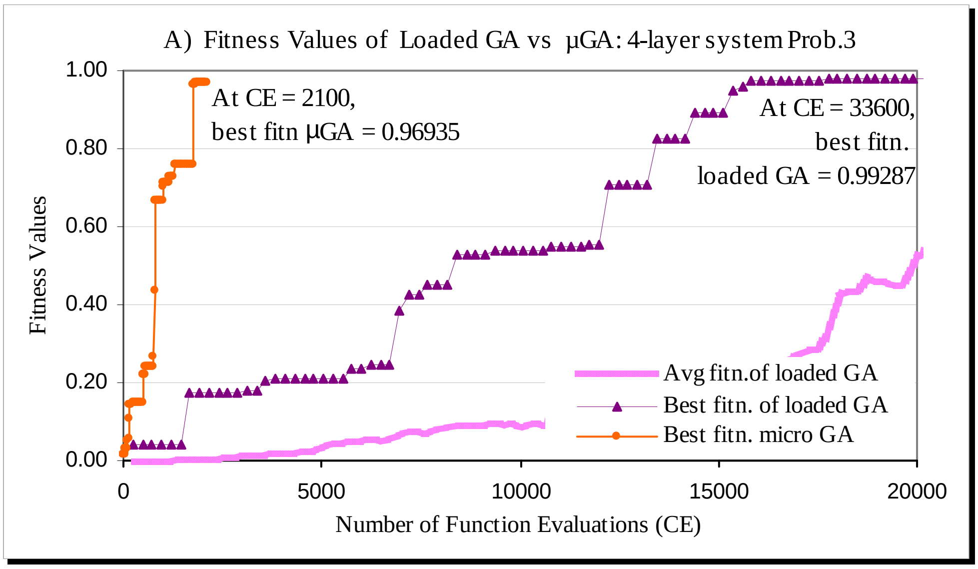
Problem 2: The four-layer flexible composite pavement with a stabilized subbase layer

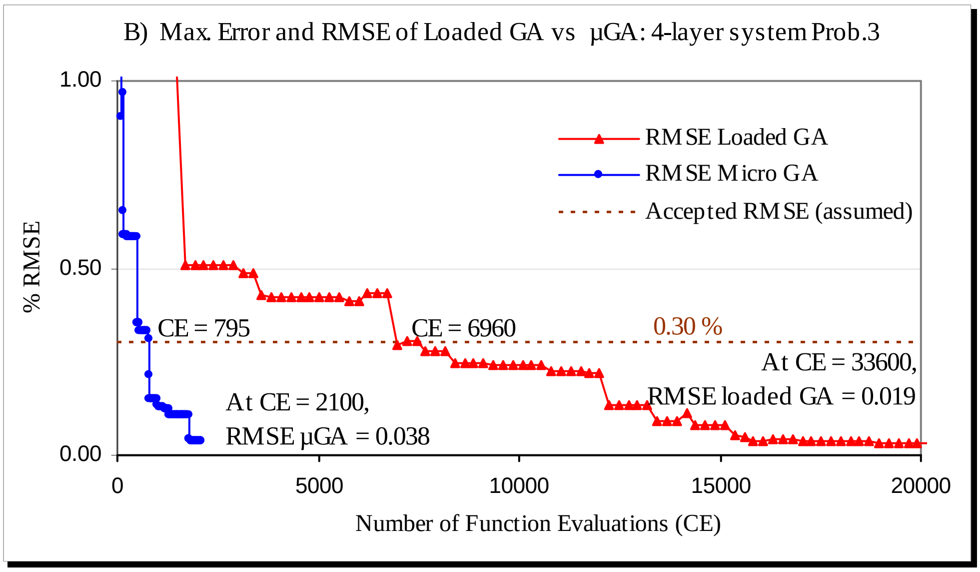




GAMLET module	Backcalculated Modulus [MPa]				max. ΔM* (%)	RMSE (%)	CE (times)
	AC	Base	Stabilized subbase	Sub grade			
Input Modulus values	3447.38	172.37	31026.40	51.71	-	-	-
Loaded GA	3447.90	172.13	31145.99*	51.72	0.38	0.009	33600
Micro GA	3458.02	171.67	30907.06*	51.81	0.38	0.066	4200

Problem 3: The four-layer flexible composite pavement with stabilized base





GAMLET module	**Backcalculated Modulus [MPa]**				**max. ΔM* (%)**	**RMSE (%)**	**CE (times)**
	AC	**Stabl. Base**	**Sub base**	**Sub grade**			
Input Modulus values	3447.38	31026.40	172.37	51.71	-	-	-
Loaded GA	3456.85	30970.36	166.43	51.76	3.44	0.019	33600
Micro GA	3494.29	30330.36	218.34	51.63	26.67	0.038	2100

Appendix C: Test track at BASt in construction process

Construction process of the test track at the Federal Highway Research Institute (in German: Bundesanstalt für Straßenwesen, BASt) in Bergisch Gladbach, Germany.

Construction of the gravel ramp in the field no. 3.1 and 3.2 [9]

Construction of the hydraulically bound layer in the field no. 1.1 [9]

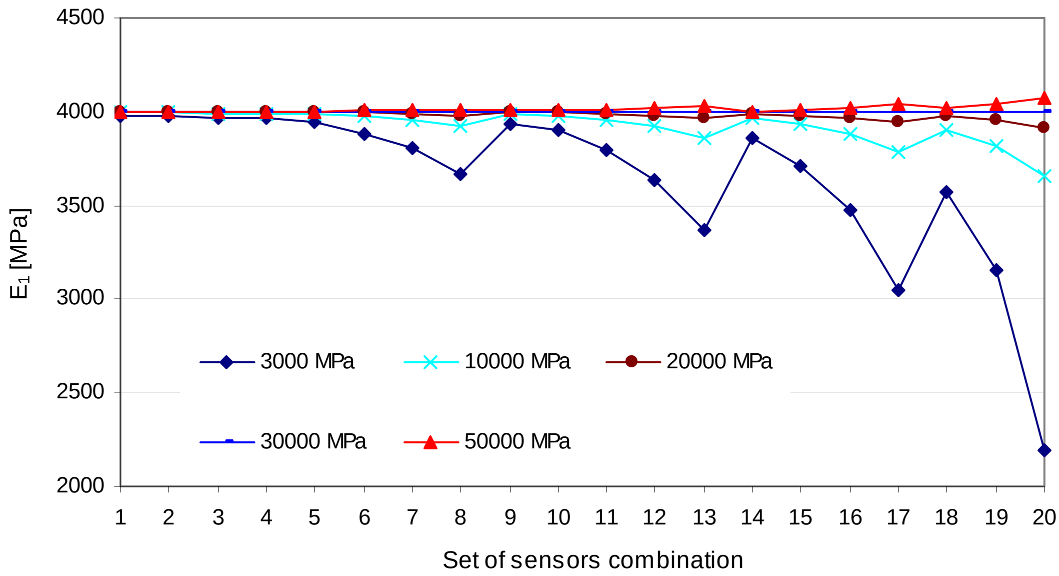
Construction of the asphalt layer in the field no. 2.1 [9]

Construction of the asphalt wearing course [9]

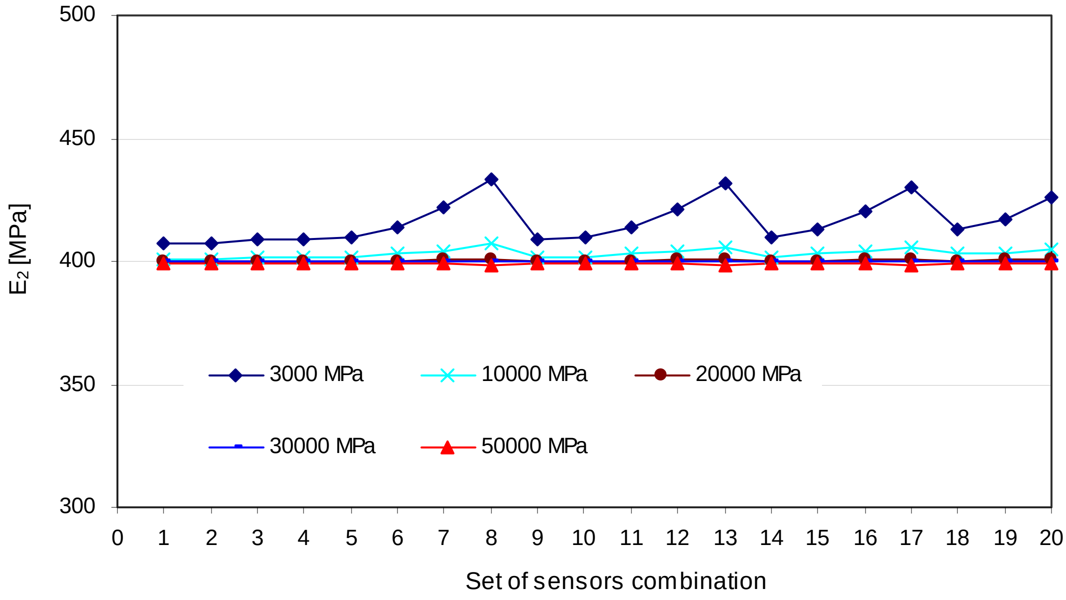
Appendix D: Influences of bedrock stiffness on backcalculated moduli

Influences of different bedrock stiffness on backcalculated layer moduli obtained from the iterative backcalculation program VAHREN1 using system with E_1 = 4000 MPa, E_2 = 400 MPa and bedrock is assigned at depth of 5.25 m with various elastic modulus values.

A) Backcalculated E_1 from systems with different bedrock stiffness

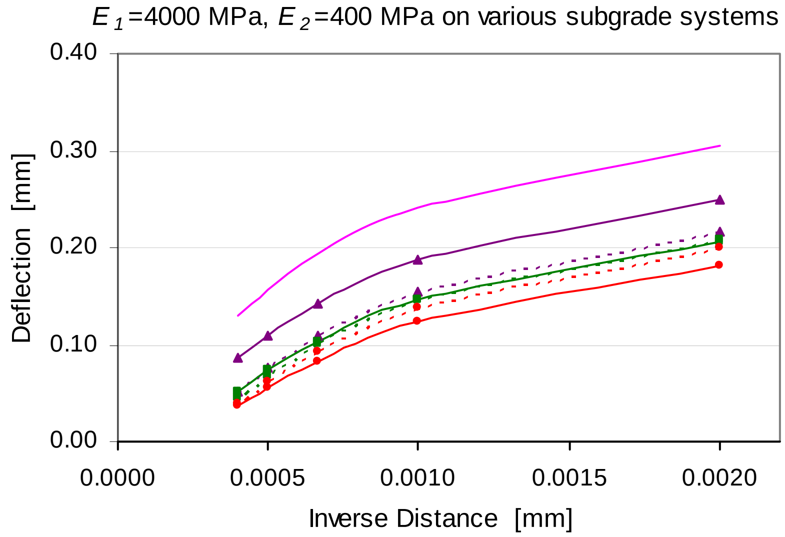


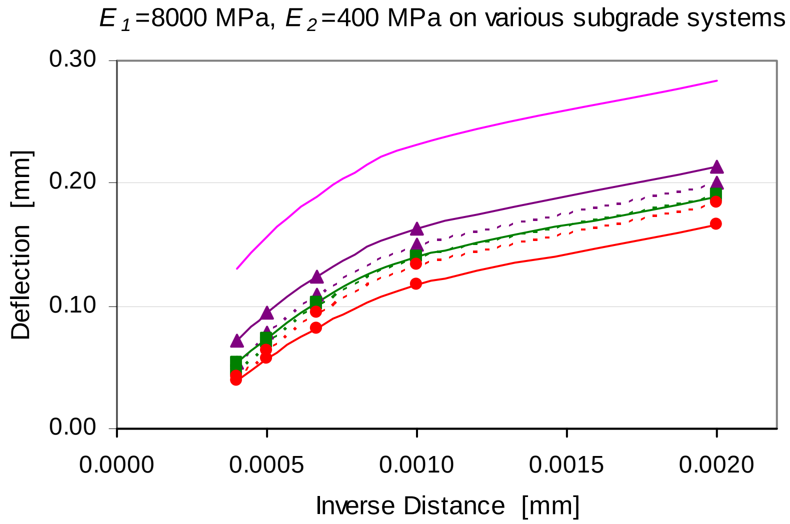
B) Backcalculated E_2 from systems with different bedrock stiffness

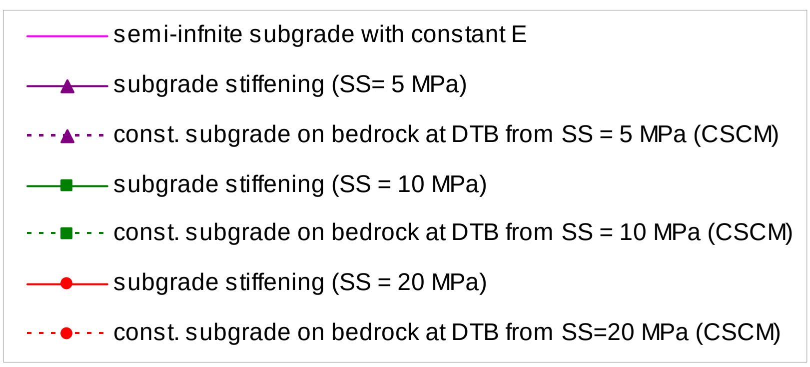


Appendix E: Evaluation of CSCM with pavement models

Evaluation of the consistent slope changing method (CSCM) with typical pavement models on various systems of subgrade stiffening with depth

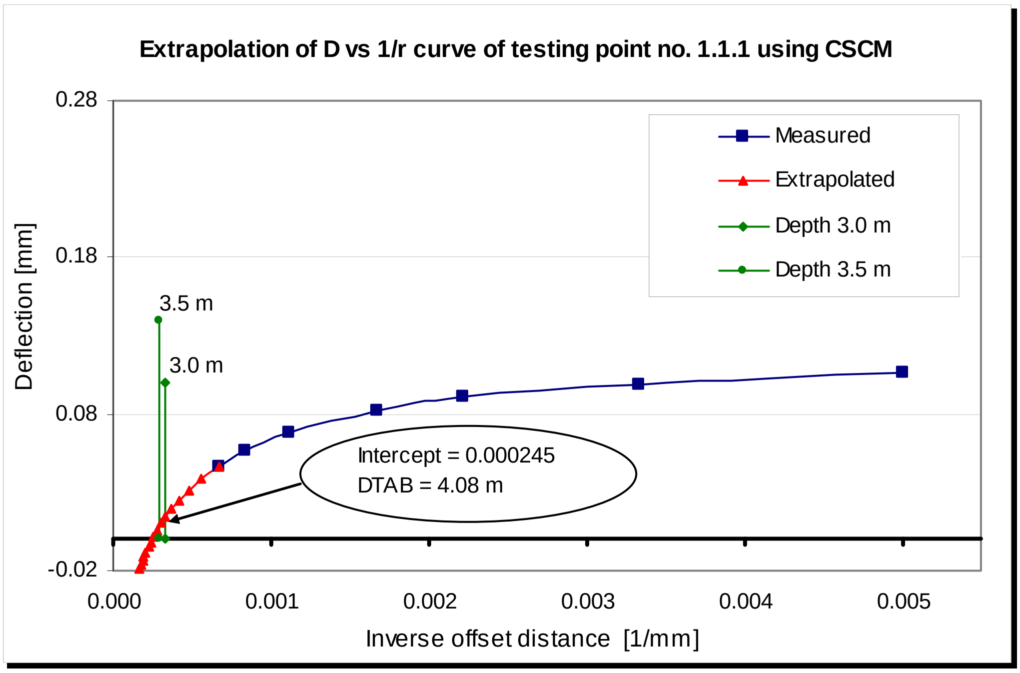




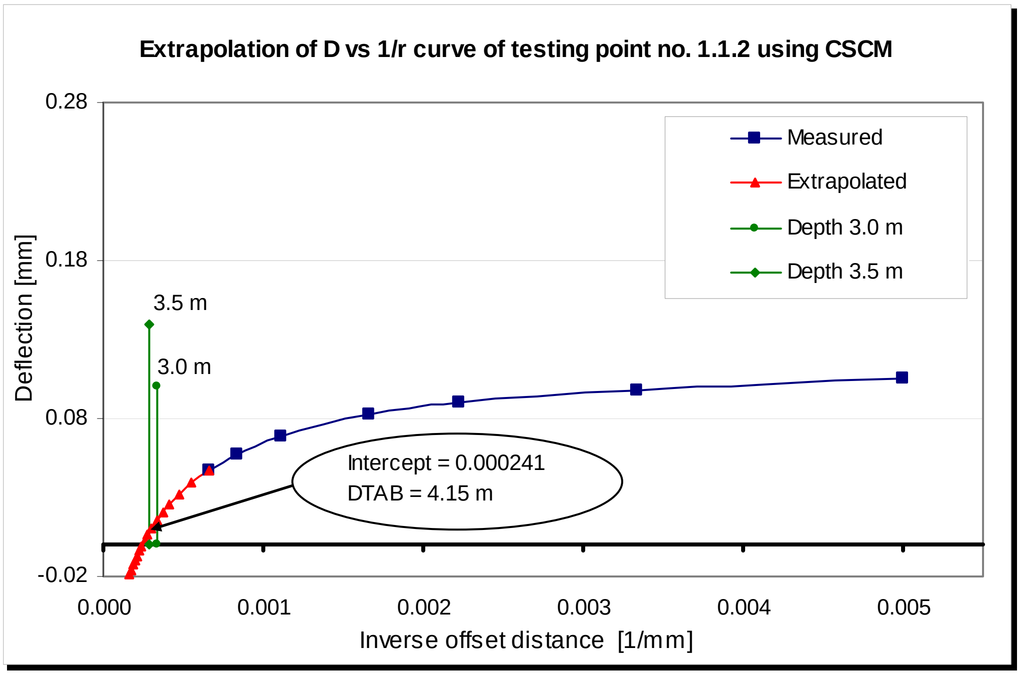


Appendix F: Verification of CSCM with in situ data

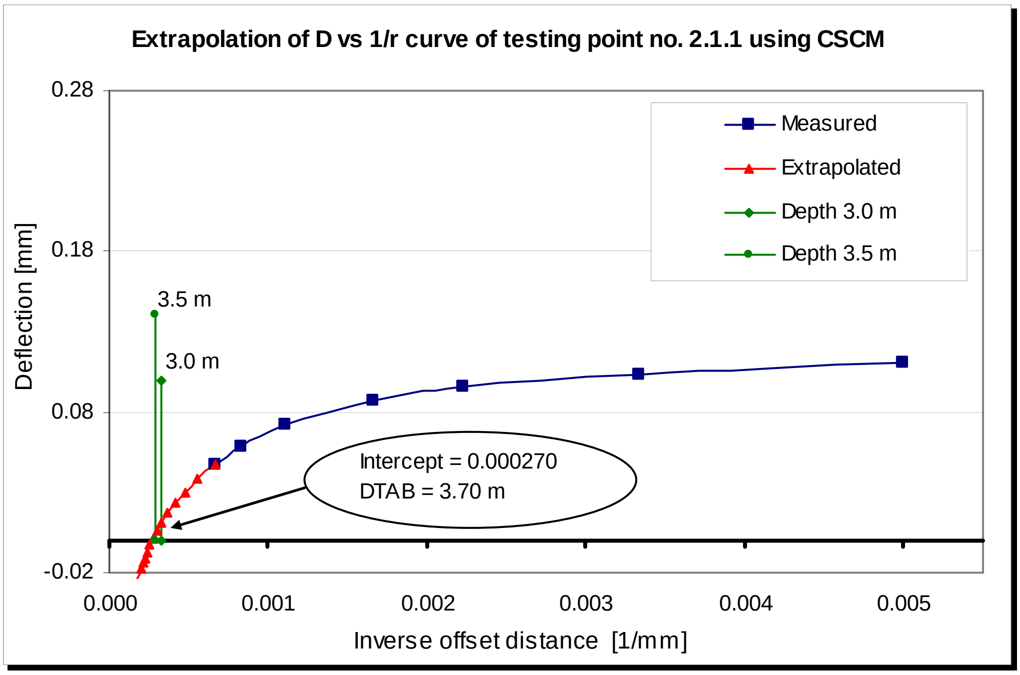
Verification of CSCM with in situ deflection data from FWD test on the test track at Federal Highway Research Institute of Germany (BASt)



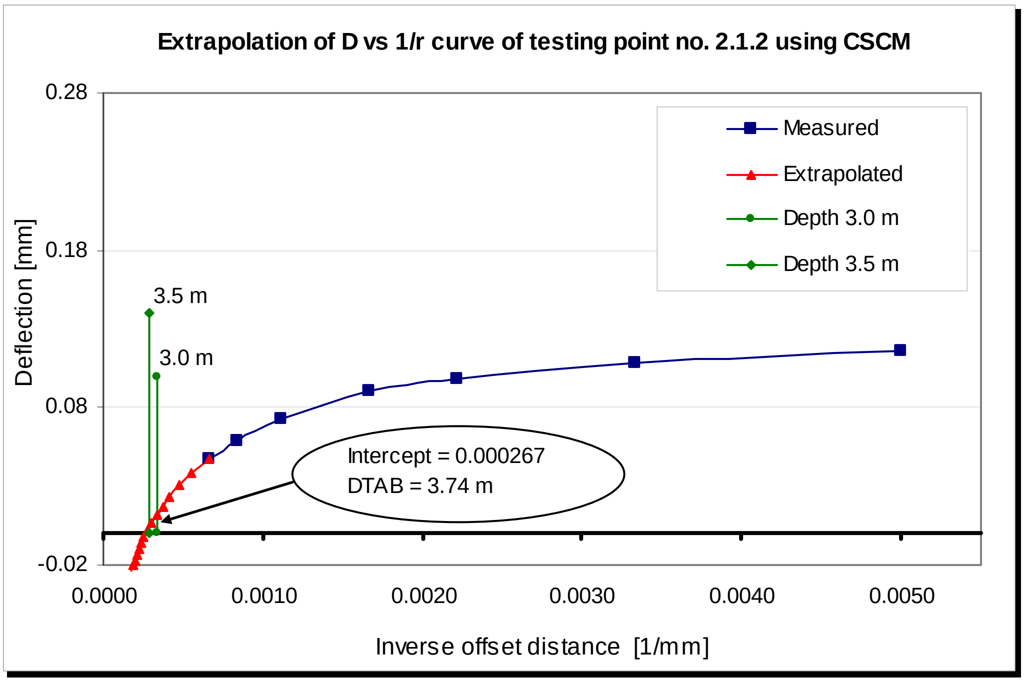
Testing point no.1.1.1 (construction class SV line 2.1 after RStO'01)



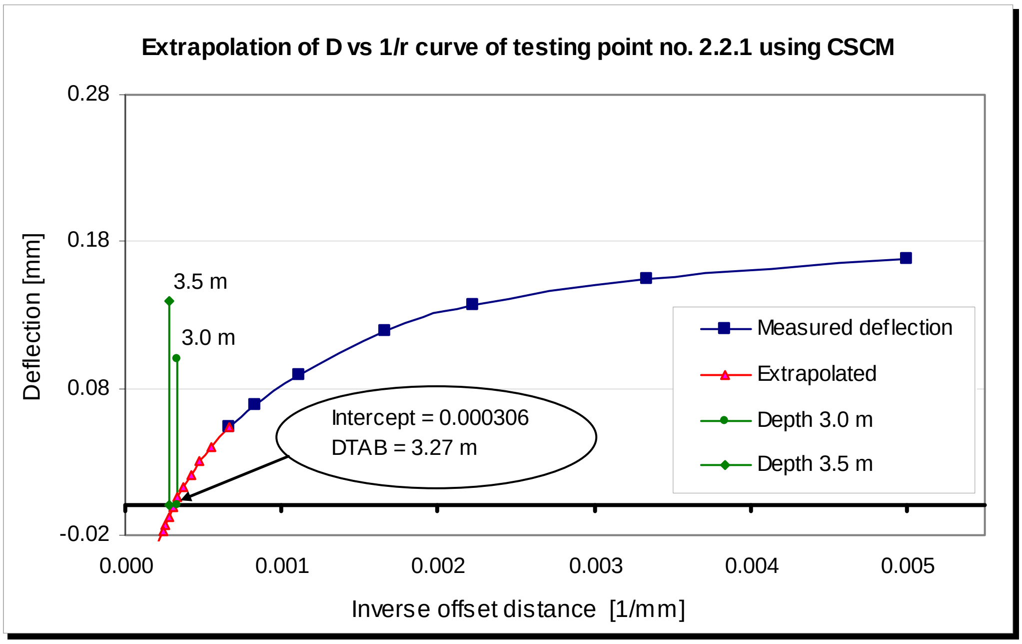
Testing point no.1.1.2 (construction class SV line 2.1 after RStO'01)



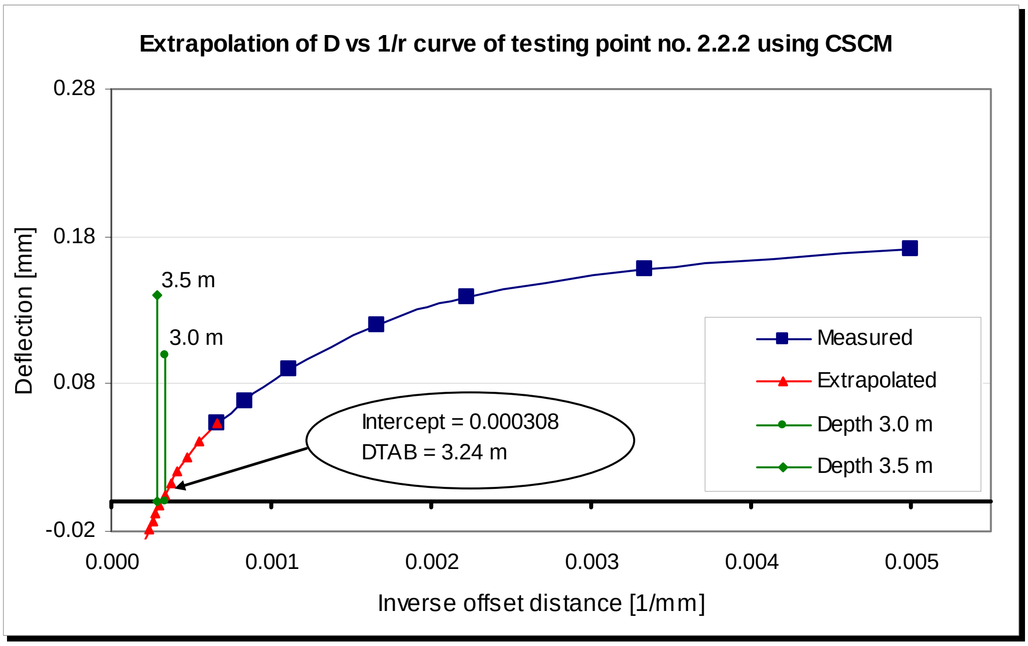
Testing point no.2.1.1 (construction class SV line 1 after RStO'01)



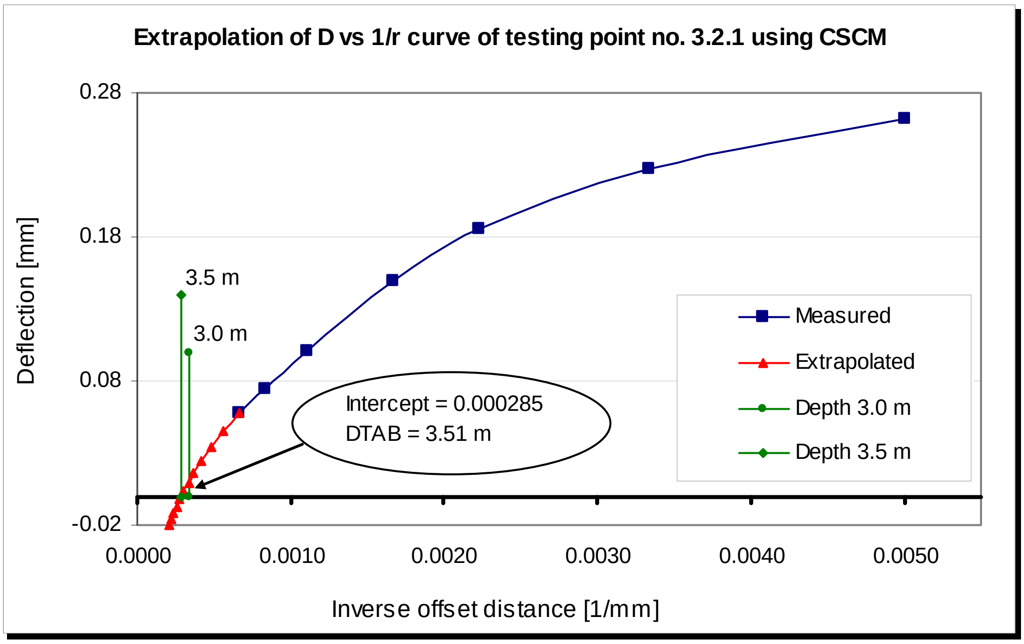
Testing point no.2.1.1 (construction class SV line 1 after RStO'01)



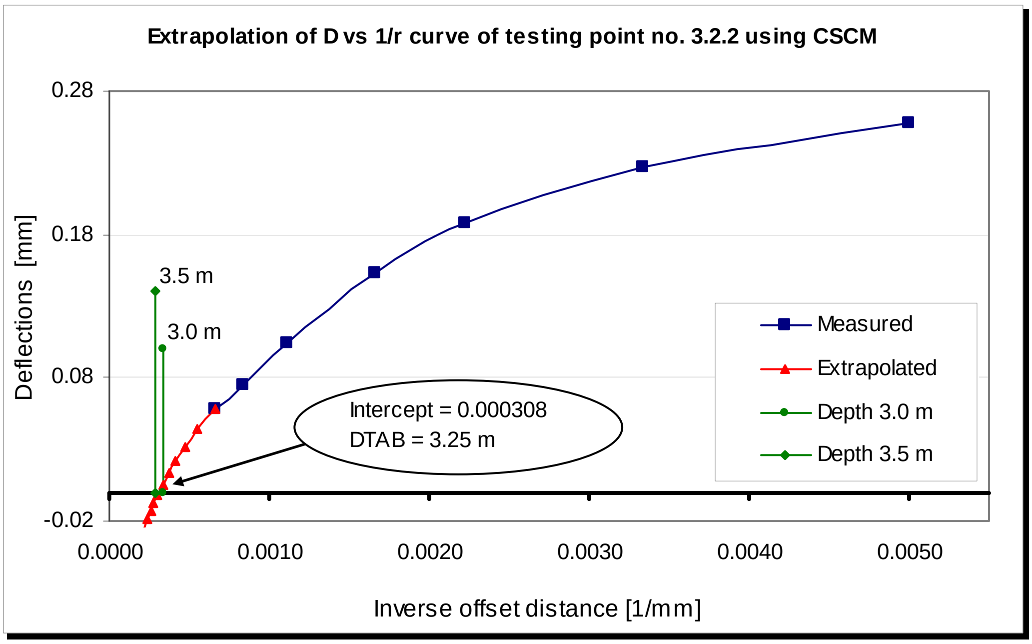
Testing point no.2.2.1 (construction class III line 1 after RStO'01)



Testing point no.2.2.1 (construction class III line 1 after RStO'01)



Testing point no.3.2.1 (construction class V line 1 after RStO'01)



Testing point no.3.2.1 (construction class V line 1 after RStO'01)

Appendix G: Results of investigation of robustness of CSCM

Calculated DTABs from a pavement system with different stiffening subgrade systems and different set of sensor positions using CSCM