SKID RESISTANCE OF ASPHALT CONCRETE BASED ON THAI AGGREGATES

Sitthichai Siriphun

A Thesis Submitted in Partial Fulfillment of the Requirements for the Degree of Doctor of Philosophy in Construction and Infrastructure Management

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ความต้านทานการลื่นไถลของผิวทางลาดยาง
จากส่วนผสมมวลรวมในประเทศไทย

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วิทยานิพนธ์นี้เป็นส่วนหนึ่งของการศึกษาตามหลักสูตรปริญญาวิศวกรรมศาสตรดุษฎีบัณฑิต
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ปีการศึกษา 2559

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Suranaree University of Technology has approved this thesis submitted in partial fulfillment of the requirements for the Degree of Doctor of Philosophy

Thesis Examining Committee

(Assoc. Prof. Dr. Panich Voottipruex)

Chairperson

(Prof. Dr. Suksun Horpibulsuk)

Member (Thesis Advisor)

(Asst. Prof. Dr. Rattaphol Pueboobpaphan)

Member

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Member

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and Innovation

(Assoc. Prof. Flt. Lt. Dr. Kontorn Chamniprasart)

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สิทธิชัย ศิริพันธุ์ : ความต้านทานการลื่นไถลของผิวทางลาดยางจากส่วนผสมมวลรวมในประเทศไทย (SKID RESISTANCE OF ASPHALT CONCRETE BASED ON THAI AGGREGATES) อาจารย์ที่ปรึกษา : ศาสตราจารย์ ดร.สุขสันติ์ หอพิบูลสุข, 104 หน้า

ความต้านทานการลื่นไถลของผิวทางเป็นปัจจัยหนึ่งที่เพิ่มความปลอดภัยบนโครงข่ายถนน ค่าความต้านทานความลื่นไถลสามารถวัดได้บนผิวทางในสนามหลังจากก่อสร้างแล้วเสร็จ ดังนั้น ค่าความต้านทานการลื่นไถลไม่อาจทราบได้ในขั้นตอนการคัดเลือกวัสดุมวลรวม ขั้นตอนการออกแบบส่วนผสม และในระหว่างการใช้งานที่ปริมาณจราจรต่างๆ ซึ่งค่าความต้านทานการลื่นไถลจะลดลงอย่างต่อเนื่องตามปริมาณจราจรที่เพิ่มขึ้น การวัดค่าความต้านทานการลื่นไถลในสนามในแต่ละเส้นทางจำเป็นต้องใช้เวลาและค่าใช้จ่ายสูงมาก การศึกษานี้จึงมุ่งพัฒนาแบบจำลองที่สามารถคาดการณ์ค่าความต้านทานการลื่นไถลของผิวทางลาดยางหลังก่อสร้างแล้วเสร็จใหม่ และพัฒนาแบบจำลองที่สามารถคาดการณ์ค่าความต้านทานการลื่นไถลที่ลดลงตามช่วงอายุการใช้งานโดยใช้คุณสมบัติของมวลรวมและสัดส่วนผสมเป็นตัวแปรหลัก

ในส่วนแรก (การพัฒนาแบบจำลองคาดการณ์ค่าความต้านทานการลื่นไถลของผิวทางลาดยางหลังก่อสร้างแล้วเสร็จใหม่) มวลรวมที่ใช้ในการทดสอบ ได้แก่ หินปูน หินแกรนิต และหินบะซอลต์ ซึ่งรวบรวมจากพื้นที่ก่อสร้างหลักของประเทศไทย ซึ่งครอบคลุม 14 จังหวัด ผิวทางลาดยางทดสอบเป็นแบบเกรดแน่น ที่ใช้หินขนาดใหญ่สุด 9.5 มม. (AC9.5) และ 12.5 มม. (AC12.5) ค่าความต้านทานการลื่นไถลวัดด้วยเครื่อง British Pendulum คุณลักษณะของผิวทางสำหรับผิวทางสองเภท (AC9.5 และ AC12.5) ตรวจสอบด้วยวิธี Sand Patch แบบจำลองที่พัฒนาขึ้นในพจน์ของคุณสมบัติของมวลรวมและส่วนผสมแอสฟัลต์คอนกรีต (ค่าหินขัดเงา, และความหยาบของผิวทาง) สามารถคาดการณ์ค่าความต้านทานการลื่นไถลได้ในเกณฑ์ที่ยอมรับได้ในทางสถิติ ($R^2 > 0.78$, MSE < 0.00089 และ F-significance < 0.05)

ส่วนที่สอง (การพัฒนาแบบจำลองที่สามารถคาดการณ์ค่าความต้านทานการลื่นไถลที่ลดลงหลังผ่านการใช้งาน) ค่าความต้านทานการลื่นไถลคำนวณได้ในพจน์ของคุณสมบัติของมวลรวมและส่วนผสมแอสฟัลต์คอนกรีต และปริมาณจราจรสะสม มวลรวมที่ใช้ในการทำตัวอย่างแอสฟัลต์คอนกรีต ได้แก่ หินปูน หินแกรนิต และหินบะซอลต์ ซึ่งรวบรวมจากโครงการก่อสร้าง 12 โครงการ และครอบคลุม 6 จังหวัด ผิวทางลาดยางทดสอบเป็นแบบเกรดแน่น ที่ใช้หินขนาดใหญ่สุด 9.5 มม. (AC9.5) และ 12.5 มม. (AC12.5) ค่าความต้านทานการลื่นไถล (SRV) ถูกวัดทุกๆ 50,000 pcu ด้วยเครื่อง British Pendulum เป็นเวลา 3 ถึง 4 ปี หลังเสร็จสิ้นการก่อสร้าง แบบจำลองสามารถคาดการณ์ค่าความต้านทานการลื่นไถลที่ลดลงได้ประสบความสำเร็จอย่างมีนัยสำคัญทาง

สถิติ แบบจำลองทั้งหมดที่นำเสนอในงานวิจัยนี้สามารถนำเสนอให้กรมทางหลวงชนบทใช้ประกอบการบริหารจัดการด้านความปลอดภัยทางถนน

สาขาวิชา วิศวกรรมโยธา ลายมือชื่อนักศึกษา ____________________

ปีการศึกษา 2559 ลายมือชื่ออาจารย์ที่ปรึกษา ______________

SITTHICHAI SIRIPHUN : SKID RESISTANCE OF ASPHALT CONCRETE BASED ON THAI AGGREGATES. THESIS ADVISOR : PROF. SUKSUN HORPIBULSUK, Ph.D., 104 PP.

SKID RESISTANCE/ ASPHALT CONCRETE/ PREDICTIVE MODEL/ AGGREGATE CHARACTERISTICS/ SKID REDUCTION/ THAI AGGREGATES

The skid-resistance in road pavements requires considerable improvement in order to increase road network safety. Skid-resistance values can only be measured in situ, that is on the road itself, and prior to post-construction stages. So, skid-resistance values have not been accounted during the aggregate selection and mixing processes and at various cumulative traffic volumes. The road pavements usually deteriorate under cumulative traffic volumes throughout their service life or the skid resistance values (SRV) decreases continuously over the lifetime of road pavements. The field measurement of skid resistance is time-consuming and costly. In this study, a skid-resistance predictive models at the construction stage were therefore developed based on the essential aggregate and mixture characteristics that are the influential factors and to formulate a statistical-based predictive model of skid resistance under various cumulative traffic volumes.

For the first part: developing skid resistance predictive model, three main types of Thai aggregates (limestone, granite and basalt) were mixed in asphalt concrete to make the pavement for the construction sites, which were sourced and collected to make test asphalt concrete. These aggregates were obtained from Thailand’s main regions and covered 14 provinces. Aggregates and their standard densely-graded asphalt concrete mixtures of 9.5 mm and 12.5 mm maximum aggregate sizes were used to perform in the construction site. The SRV was measured by the British pendulum tester. In addition, the textural characteristics of asphalt concrete pavement, based on different aggregate mixtures (AC9.5 and AC12.5), were also analyzed with respect to a sand patch method. The results of the study demonstrated that the developed predictive model in terms of aggregate and mixture characteristics (polished stone value, and mixture surface texture) provided acceptable SRV prediction with high statistic levels ($R^2 > 0.78$, MSE<0.00089 and F-significance < 0.05).

The second part: developing a statistical-based predictive model of skid resistance under various traffic volumes to accurately predict the reduction of SRV, the SRV reduction model has been developed based on the essential aggregate and asphalt concrete mixture characteristics and field traffic volumes. In this study, three main types of aggregates typically used in pavements in Thailand, being limestone, granite and basalt were used to make asphalt concrete. These aggregates were obtained from 12 project sites located in 6 provinces for SRV test. Aggregates and their standard dense-grade asphalt concrete mixtures of 9.5 mm and 12.5 mm maximum aggregate sizes were used at the construction sites. The SRV were measured at every 50,000 passenger car unit (pcu) by the British pendulum tester for 3 to 4 years. The results of the study demonstrated that the model developed can be used successfully to predict the reduction of SRV at field sites. The two proposed models will be recommended for inclusion in the Department of Rural Roads, Thailand preventive scheme for road safety management protocols.

School of Civil Engineering — Student's Signature ________________

Academic Year 2016 — Advisor's Signature ________________

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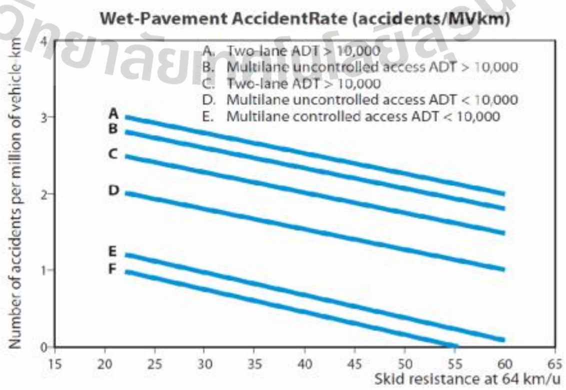
SYMBOLS AND ABBREVIATIONS

PSV	=	Polished Stone Values
PSVdiff	=	The different values between PSVbefore and PSV
AC9.5	=	Dense graded asphalt concrete with 9.5 mm aggregate maximum size
AC12.5	=	Dense graded asphalt concrete with 12.5 mm aggregate maximum size
XRD	=	x-ray diffraction
XRF	=	X-ray fluorescence
AIV	=	Aggregate impact value
ACV	=	Aggregate crushing value
LAA	=	Los angeles abrasion
MTD	=	Mean texture depth (mm)
SRV35	=	Skid resistance value at 35 °C
IFI	=	International friction index or Frictional resistance at test speed equal to 60 km/h
ANOVA	=	Analysis of variance
χ	=	Aggregate size in mm


Wet-Pavement AccidentRate (accidents/MVkm)
A. Two-lane ADT > 10,000
B. Multilane uncontrolled access ADT > 10,000
C. Two-lane ADT > 10,000
D. Multilane uncontrolled access ADT < 10,000
E. Multilane controlled access ADT < 10,000
A
B
C
D
E
F
Number of accidents per million of vehicle-km
0
1
2
3
4
15
20
25
30
35
40
45
50
55
60
65
Skid resistance at 64 km/u

Skid resistance is an important functional property of pavement surface. It is related to the resistance of pavement surface to the sliding or skidding of the vehicles. Inadequate skid resistance can lead to higher chance of skid, which causes the accidents. Skid resistance changes over time as it decreases over the remaining pavement life when the aggregates are polished by traffic and environment. It is well known that the skid resistance is highly influenced by the microtexture and macrotexture of pavement surface. Microtexture is the irregularities in the surface of aggregate particles and its magnitude depends on the aggregate texture characteristics, aggregate mineralogy, and the ability of the aggregate to resist the polishing effect of traffic (Noyce et al., 2005; Kandhal and Parker, 1998; Crouch et al., 1995). Macrotexture is the larger irregularities in the pavement surface involving the voids between coarse aggregate particles. The magnitude of macrotexture mainly depends on the size, shape, and gradation of coarse aggregates. While the microtexture significantly affects the skid resistance at low speed, the macrotexture is more critical at higher speed levels (Noyce et al., 2007; Cenek and Jamieson, 2000; Galambos, 1977).

Microtexture is the road surface characteristic most relevant to skid resistance. The tiny irregularities of microtexture (with texture wavelengths smaller than 0.5 mm) provide the necessary contact points to generate frictional forces with the tyres of vehicles. Macrotexture (with texture wavelengths between 0.5 and 50 mm) is also important, at higher speeds. The skid resistance of a pavement with a high microtexture but a low macrotexture rapidly decreases with increasing vehicle speed, as macrotexture is effective in draining water from the tyre-pavement contact area. The otherwise, road roughness is defined as the larger wavelength irregularities of the road surface: the sizes of irregularity are between 0.5 m to 50 m. These wavelengths are usually less significant for the generation of surface friction, except that the interaction between longer wavelength features and vehicle suspension (or in this case the GripTester skid testing device) influences the contact between the road surface and vehicle tyre (Wilson et al., 2013).

The changes in pavement texture due to temperature, moisture and polishing can significantly reduce the skid resistance over time (Masad et al., 2007; Do et al., 2007; Prowell et al., 2005; Jayawickrama et al., 1996). Therefore, it is necessary to consider the texture of pavement surface and its related aggregate properties over the entire service life of the pavement.

Many studies reported that skid resistance depends on various factors such as traffic volumes, mixture types, aggregate characteristics and contaminants (Do et al., 2007; Liu, 2004; Fwa et al., 2003; Henry and Dahir, 1979; Ryell et al., 1979). Liu (2004) and Fwa et al. (2003) stated that a gap between aggregate particles is one of the important factors to meet the maximum skid resistance. Do et al. (2007) recommended that skid resistance of the asphalt mixture is controlled by the aggregates polishing due to traffic. The proper proportion of coarse and fine aggregate particles helps to achieve the adequacy of skid resistance (Huang, 1993; Henry and Dahir, 1979; Ryell et al., 1979; AASHTO, 1976). Based on the findings from previous studies, it seems that aggregate characteristics and aggregate orientation are the significant factors affecting the surface texture and skid resistance of pavement. Therefore, these factors need to be considered in order to evaluate and predict the skid resistance and surface texture.

1.2 Statement of the problem

Skid resistance is the road safety indicating that the speed of vehicle would be decelerated enough to avoid the severity of a road crash. Especially, in Thailand that have raining season more than 4 months per year. The variation of skid resistance of pavement depends on the road surface condition. This claim can be justified by the fact that there is no skid resistance data available on wet pavement condition in Thailand. Moreover, the pavement surfaces normally loose its skid resistance as the time has passed causing the potential danger to the road users. Therefore, it is essential that the highway and road safety engineers should observe and maintain the skid resistance level on the road surface on a regular basis. Furthermore, the measurement and prediction of pavement skid resistance are necessary when the road maintenance and budget planning are needed which can merit for road safety.

Particularly, to maintain the satisfactory level of skid resistance and to improve pavement surface served the functional road with safety, two friction-related parameters of pavement surface texture - microtexture and macrotexture must be considered. At lower speeds, microtexture is mainly concerned with aggregate particle, whereas macrotexture becomes important at higher speeds associated with the roughness of pavement surface. So that surface texture influencing skid resistance on all levels of speed should be evaluated (Cenek and Jamieson, 2000). Additionally, it is important that skid resistance is incorporated in the criteria for pavement design and pavement maintenance.

Recently, in Thailand the various types of aggregates and binders are also being introduced to design pavements depending on their availability. Limestone, granite, and basalt are normally used as aggregates and asphalt cement are used as binder. Ideally, the skid resistance values of asphalt surfacing would be known prior to the actual construction of the road. However, in practice, skid resistance values can only be measured in-situ, that is on the road itself, and prior to post-construction stages. The expected outcome of this study is that predictive skid-resistance values for Thailand's asphalt concrete mixtures would be estimated at the level of aggregate selection and asphalt concrete mixture design processes prior to real construction. Moreover, skid resistance reduction values versus service time could be estimated when accumulative traffic volume and asphalt concrete mixture types are known. Finally, road safety management would be more controllable when skid-resistance values of asphalt concrete are reliably known before construction or during the planning of a project.

1.3 Objectives of the study

This research has been undertaken with the following objectives:

(i) To develop a predictive skid-resistance model of an asphalt concrete mixture at the construction stage

(ii) To develop a predictive skid resistance model in relation to cumulative traffic volume

1.4 Scope of study

This research focuse on the investigation of skid resistance on asphalt pavement in Thailand. Data collection of field skid-resistance values was conducted, focusing on the 14 provinces from where the aggregates were sourced. Road construction data from the Department of Rural Roads (DRR), Thailand was also collected. For the first part, developing skid resistance predictive model, resources were selected to cover fourteen provinces and a specific range of aggregates (limestone, granite and basalt) and mix types (AC9.5 and AC12.5). The second part, developing a statistical-based predictive model of skid resistance under various traffic volumes, resources were selected to cover six provinces and cover three main aggregate types. The field skid-resistance values of SRV35 and the texture depth values of MTD from the standard sand patch method.

1.5 Organization of the dissertation

This thesis consists of five chapters and outlines of each chapter are presented as follows:

Chapter I presents the introduction part, describing the statement of the problems, the objectives of the study, the organization of the dissertation and the advantages.

Chapter II presents the literature review that consists of various aspects regarding surface texture, factors affecting skid resistance, skid resistance requirement in different types of highways, friction and texture measuring devices, pavement friction models, and aggregate characteristics that are related to the skid resistance. Various factors affecting the skid resistance are briefly explained. The traditional models used to predict the skid resistance of asphalt pavement surface are also presented.

Chapter III presents the characterization of Thai aggregates in relation to the skid-resistance values of asphalt concrete mixture and the development of a predictive skid-resistance model of an asphalt concrete mixture at the construction stage, based on the aggregate and mixture characteristics. The model produced from these procedures will be used in a preventive scheme for road safety.

Chapter IV presents the reduction behavior of the skid resistance of road pavements. In the field, in-situ skid resistances were measured immediately after the constructions were completed and the measurement was continued after the roads were opened for service. Traffic volumes from the studied road sites were also monitored and collected. Therefore, relationship between skid resistance and traffic volume can be correlated accordingly. The studied road sites were rehabilitation projects using types of aggregates (limestone, basalt, granite) from 6 provinces (12 sites) by Department of Rural Roads. The asphalt concrete binder was AC60/70 (AC9.5 and AC12.5). The skid-resistance reduction (service life) predictive equation for Thailand's asphalt concrete mixtures would be developed for various accumulative traffic volumes and asphalt concrete mixture types.

Chapter V presents the conclusion of each chapter and overall conclusion. The suggestion for further study is also presented in this chapter.

1.6 Advantages

This research not only identifies a variety of key factors affecting skid resistance on asphalt pavement in Thailand, but also develops an empirical model capable of investigating the required level of skid resistance to ensure road safety. This model enables highway and pavement engineers to design an appropriate skid resistance on a existing road, planned road, maintenance schedules and managing the allocated budget for maintaining the desired levels of skid resistance on roads effectively. Given the benefits of this study, it could be considered as an asset for highway and pavement engineers in Thailand. More specifically, these study contributions are as follows:

- Factors significantly affecting skid resistance
- Thai aggregate characteristics which are related to skid resistance

Aggregate characteristics can be selected by highway and pavement engineers to improve skid resistance by asphalt pavement surface. These aggregate characteristics correspond to the skid resistance for enhancing safety of pavement during the long-term service.

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CHAPTER II

LITERATURE REVIEW

2.1 Introduction

This literature review consists of skid resistance, factors affecting skid resistance, pavement surface texture, mechanics of tire pavement friction, models for tire pavement interaction, skid resistance requirement, friction measuring devices and skid resistance models.

2.2 Skid Resistance

Skid resistance is an important feature of road pavements. Skid resistance refers to a condition parameter which characterizes the contribution that a road makes to the friction between a road surface and a vehicle tyre during acceleration, deceleration (Weligamage, 2006). Skid resistance has two components, commonly identified as adhesion and hysteresis. The adhesive part is the ‘bonding’ of the tyre as the vehicle brakes and the characteristic tyre is forced against the surface of the stone under significant pressure. The second component ‘hysteresis’ is a result of the deformation of the tyre between the stones and the resistance within the tyre to this deformation. These forces will cause heating which softens the tyre and does have an effect on the friction resistance.

Pavement friction is the resisting force that is created between a road pavement surface and the tyre of a vehicle. Skid-resistance, in the form of a friction coefficient, may be formulated as shown in Eq. (2.1).

$$\mu = F/Fw \tag{2.1}$$

where μ is the friction coefficient, F is the friction force in the opposite direction to the movement, and Fw is the normal force from the dry weight of a vehicle.

Based on the literature to date, the consideration of frictional resistance in road surfaces is believed to be essential in road safety management schemes, as it directly relates to the incidence of road accidents (Henry 2000). According to Rizenbergs et al. (1976), when wet surface conditions reduce frictional resistance, collisions and accidents increase. In their research, the relationship between wet-surface accident rates and pavement skid-resistance values was determined using a case study on a low-to-medium-volume road in Kentucky, USA. Research results revealed that wet-surface accident rates reduce when the skid-resistance value (i.e., in the form of SN40) of a road surface increases. SN40, in Rizenbergs et al. (1976), is the skid (friction) resistance value measured at 40 mph (64 km/h) by a Locked-Wheel Friction Tester. Their results found a definite statistical trend between low skid-resistance and higher accident rates. Various other works, such as those of Agent et al. (1996) support the TxDOT research. Moreover, studies into the effects of wet surface conditions on tangential wheel alignment, using a Sideways-Force Coefficient Routine Investigation Machine (SCRIM) at 13 percent slip (Viner et al. 2004), show that the number of road vehicle accidents is statistically related to skid-resistance.

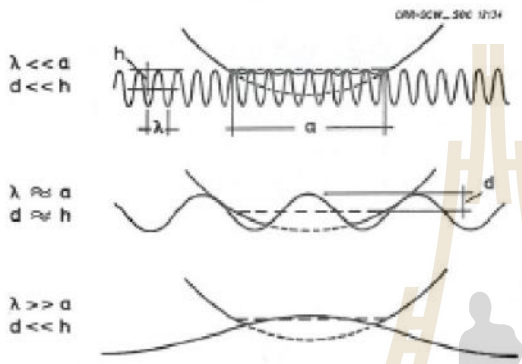
The American Society for Testing and Materials (ASTM) provides specification for the standardization of different friction measuring devices and computation of different indices for comparison of friction values measured by different equipment on the same surface. Specifically, the International Friction Index (IFI) defined in ASTM E 1960 (ASTM E-1960-07, 2009) is used as the standard for comparison of friction values measured by different equipment. IFI has been developed in the Permanent International Association of Road Congresses (PIARC) International experiment (Wambold et. al., 1995) for the purpose of harmonizing friction measurements from different equipment to a common calibrated index.

Furthermore, The National Aeronautics and Space Administration (NASA) has held Annual Runway Friction Workshops at the Wallops Flight Facility since 1993 on the eastern shore of Virginia. The main objective of this workshop is to calculate the IFI index from different devices that participate in it by evaluating the standardization parameters for each equipment. The workshop also serves to create an extensive friction database that would be used for subsequent research purposes.

2.3 Pavement surface texture

Each feature of the pavement surface performance is primarily or partially determined by the surface irregularities on different wavelength scales. Traditionally, surface irregularities can be classified into three main categories: microtexture, macrotexture and longitudinal road roughness. There has been recent research relating performance and pavement characteristics which has created another category for an unchecked range or irregularities. The new category is called megatexture with the wavelengths between 50 mm and 500 mm (Descornet 1998).

Figure 2.1 shows the deformation of a tyre travelling over various road

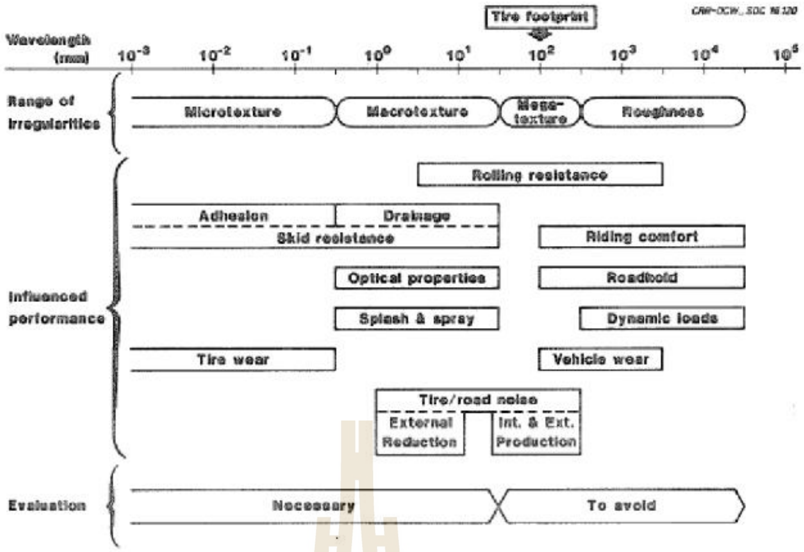


Micro-texture: λ < 0.5mm

Macro-texture: 0.5mm < λ < 50mm

Mega-texture: 50mm < λ < 500mm

Roughness: 0.5m < λ < 50 m


Tire footprint
Wavelength (mm)
10^{-3} 10^{-2} 10^{-1} 10^{0} 10^{1} 10^{2} 10^{3} 10^{4} 10^{5}
Range of irregularities
Microtexture
Macrotexture
Mega-texture
Roughness
Influenced performance
Rolling resistance
Adhesion
Drainage
Skid resistance
Riding comfort
Optical properties
Roadhold
Splash & spray
Dynamic loads
Tire wear
Vehicle wear
Tire/road noise
External Reduction
Int. & Ext. Production
Evaluation
Necessary
To avoid

microscope (Stevenson 1997). Microtexture determines the level of skid resistance by providing sliding contact resistance; this is the dominant factor in skid resistance in dry and low speed conditions (Wilson 2006). It also helps in wet conditions as microtexture breaks the film of water allowing some adhesion to the surface.

Aggregate without any harshness is considered to be undesirable and can easily cause loss-of-vehicle control leading to road crashes. The skid resistance on a chipseal road depends mainly on the microtexture of the aggregate. Microtexture can be worn away by high proportions of heavy commercial vehicles in the traffic mix, by vehicle tyres polishing the aggregate surface (Stevenson 1997).

Recent research in New Zealand has focused on understanding the mechanisms for the polishing of aggregates as this is important for road safety as well as for economics. Research studies have shown that skid resistance restoration and maintenance programmes can significantly reduce crashes each year in New Zealand. Therefore, reducing the rate of microtexture deterioration will result in significant road user social cost savings for roading authorities (Stevenson 1997).

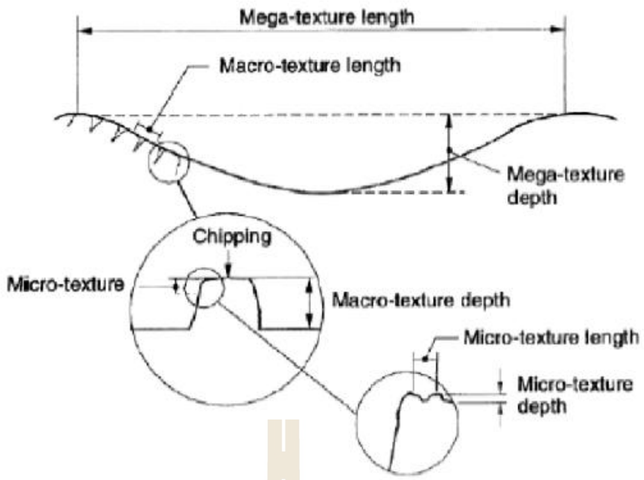
The resistance to aggregate polishing has traditionally been measured in the laboratory in terms of the PSV test although more recently (both in New Zealand and the UK) the PSV test method has been shown to be a poor indicator of in-field skid resistance performance. Other methods have been developed including the Auckland Pavement Polishing Device and Wehner Schulze device which show promise in predicting the aggregate's ability to resist polishing (Wilson 2006; Wilson and Black 2008; Dunford 2008). The microtexture can be measured indirectly or directly by methods such as: 'the indirect friction measurements at low slip speeds' (eg side force routine investigation coefficient (SCRIM), the GripTester, RoAR and the DF Tester), 'subjective assessment using pictures' and 'directly by measurement using optic devices' (Do et al 2000).

Previous research studies on measuring microtexture parameters are defined from the contact mechanisms between wet roads and vehicle tyres. The parameters defined are size, density and shape of the aggregate chips. The ability to resist polishing over time is more complex as it depends not only on the harshness of the aggregate (initial microtexture) but on the geological source properties, the grain sizes and the toughness of the aggregate minerals and the degree of cementation of the clastic matrix between minerals and grains (Wilson and Black 2009).

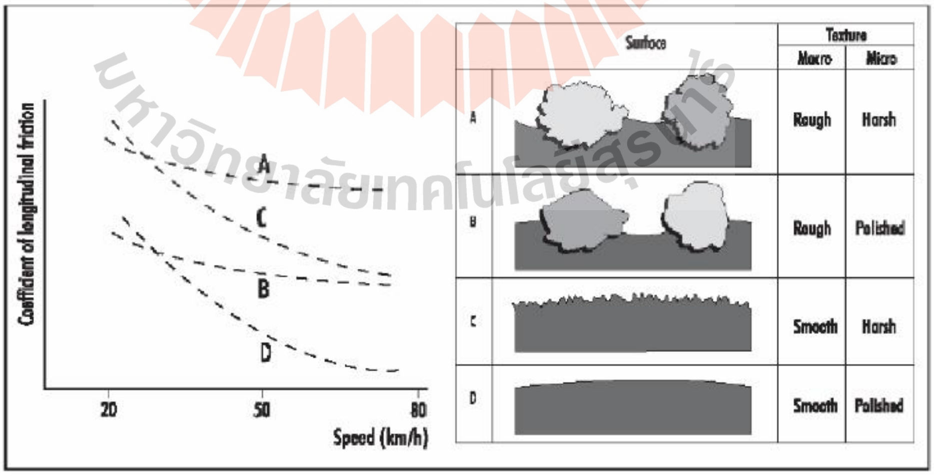
2.3.2 Macrotexture

The range of size for macrotexture is between 0.5 mm and 10 mm and macrotexture provides escape paths for the water and for hysteretic friction to develop, therefore a minimum macrotexture around 0.8 mm is preferable at high speeds. Minimum macrotexture levels are appropriately specified in NZTA T/10 (2010).

Figure 2.3 shows the relationship between megatexture, macrotexture and microtexture. It also shows that macrotexture is determined by size, grade and spacing between the aggregate chips for asphalt layers and chipseal surfaces.



Mega-texture length
Macro-texture length
Mega-texture depth
Chipping
Micro-texture
Macro-texture depth
Micro-texture length
Micro-texture depth



Coefficient of longitudinal friction
Speed (km/h)
20
50
80
A
B
C
D
Surface
Texture
Macro
Micro
Rough
Harsh
Rough
Polished
Smooth
Harsh
Smooth
Polished

skid tester over a range of speeds between 20 and 130km/h that concluded macrotexture still has an impact on skid resistance levels at speeds as low as 50 km/h, which is similar to the situation shown in figure 2.4 (ie the difference between A and C or B and D at a speed of 50 km/h respectively).

2.3.5 Macrotexture measurement

Continuous macrotexture measurements are now predominantly undertaken by laser texture measurements usually described as the mean profile depth (MPD). Alternatively the volumetric sand patch test method can be used to determine macrotexture at a specific location.

2.3.6 Road roughness

After the development of the International Roughness Index (IRI) the term 'roughness' has been typically used to refer to measuring the riding qualities of the surface itself as well as the IRI roughness measurement (Prozzi 2001). However, the riding quality is defined here as pavement performance as perceived by the user, while the roughness will be referred to as the longitudinal unevenness of the road surface as measured in terms of IRI or NAASRA counts/km.

The roughness of the pavement is one of the indicators of pavement structural performance as it directly affects the way in which pavements serve the road user. The pavement roughness may also affect the driving comfort (riding qualities), vehicle operating costs and safety (Hajek et al 1998). However, this should not be confused with the harshness or roughness relating to other texture ranges previously discussed.

2.3.7 International Roughness Index

According to ASTM E867-82A, roughness is defined as 'The deviations of the surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads and drainage.'

The IRI is an indicator and a measuring scale for pavement roughness based on the response of a standard motor vehicle to a single longitudinal profile of the road surface (Hajek et al 1998). IRI was originally developed in 1986 using the results collected from the International Road Roughness Experiment held in Brazil in 1982 (Sayers 1995).

The IRI has now become a well-recognised standard for measurement of pavement roughness globally. However, there is a wide variety of alternative devices used for measuring road profiles; these vary from hand-held dipsticks to high-speed profilers (Perera et al 1996), with each of the devices having their own measuring units. Hajek et al (1998) evaluated the consequences of switching unique device measurements to the universal IRI roughness measurements and this has since become an international standard and a commonly used pavement condition index as it offers many advantages to highway agencies.

The IRI is a geographically transferable and time-stable standard of measurement of road roughness, because it is directly measurable as the vertical accumulating displacement of a vehicle in m/km. The IRI is now used by most worldwide agencies and this has encouraged local transportation agencies to use it as well. The general use of the IRI has led to more equitable allocation of pavement maintenance funding as the pavement serviceability can now be directly compared between different areas within or between countries (Hajek et al 1998).

The use of the IRI requires the measurement of actual pavement profiles and is now most usually measured by laser profilographs. Knowledge of the pavement profile can also be used to help determine the source of pavement roughness as well as designing an effective maintenance and rehabilitation treatment programme (Hajek et al 1998).

2.3.8 Roughness measuring devices

Since the late 1950s, many road profiling devices and roughness indices have been developed and used to measure and quantify pavement performance. Yet, most of the systems fall into the category of response-type road roughness measuring systems (RTRRMS) meaning they measure the response of a specific mechanical device to pavement roughness (Hajek et al 1998). These systems usually required some conversion by developing transformation equations to the IRI. More recently, many transportation agencies have changed from using the response-type measurements to laser-based pavement profile measurements (laser profilographs) which form the basis of the IRI (Hajek et al 1998). In New Zealand, the RTRRMS were typically the National Association of Australian State Roading Authorities (NAASRA) roughness meter system prior to laser-based IRI methods. Most road agencies in New Zealand still report road roughness in terms of both NAASRA counts/km as well as the IRI.

2.4 Mechanics of Tire-Pavement Friction

Although the mechanisms of tire-pavement friction interaction are not fully understood, it is agreed in the literature that the frictional force is composed mainly of adhesion and hysteresis components (NCHRP, 2009). Tire rubber shear is another component that contributes to the frictional force, but its magnitude is negligible when compared to the adhesion and hysteresis force components. So one can express the frictional force as:

$$F = F_{adhesion} + F_{hysteresis} \tag{2.2}$$

If one divides both sides of the Equation (2.3) by the normal load, the following result is obtained:

$$f = f_a + f_h \tag{2.3}$$

Where f is the total coefficient of friction, and fa and fh are the components of the coefficient due to adhesion and hysteresis respectively. It can be seen that both fa and fh depend on the viscoelastic properties of the rubber:

$$f_a = K_1 s \left[\frac{E'}{p^r}\right] tan\delta \qquad (r < 1) \tag{2.4}$$

And

$$f_h = K_1 \left[\frac{p}{E'}\right] tan\delta \qquad (n >= 1) \tag{2.5}$$

Where tanδ is the tangent modulus of the elastometer, defined as the ratio of energy dissipated to energy stored per cycle, p is the normal pressure, E′ is the storage modulus or stress-strain ratio for the component of strain in phase with the applied stress, s is the effective shear strength of the sliding interface, r is an exponent with a value of about 0.2 and n is an index greater that the unity.

$$tan\delta = \frac{E''}{E'} \tag{2.6}$$

And

$$E^* = E' + jE'' \tag{2.7}$$

Where E* is the complex modulus (equal to the stress-strain ratio in a viscoelastic body) and E″ is the loss modulus which is equal to the stress-strain ratio for the component of strain 90^o out of phase with the applied stress.

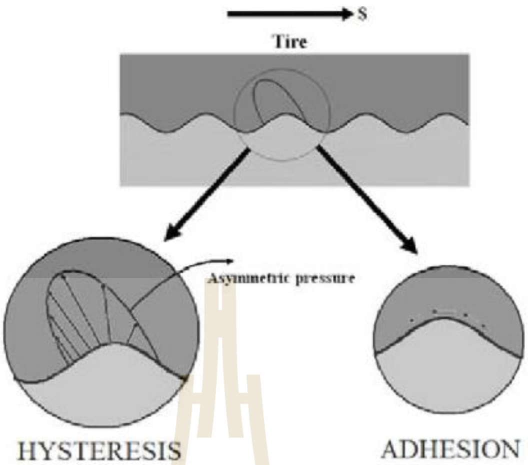
The adhesion component of friction is due to the molecular bonding of exposed surface atoms of both surfaces (tire and pavement), followed by a stretch, break and relaxation cycle. Rubber has an elastomeric structure which is composed of flexible chains which are in constant state of thermal motion. A bond is produced between the separate chains in the surface of the tire and molecules of the pavement during tire-pavement interaction. Essentially, the rubber molecules jump a molecular distance to their new equilibrium position during the above cycle.

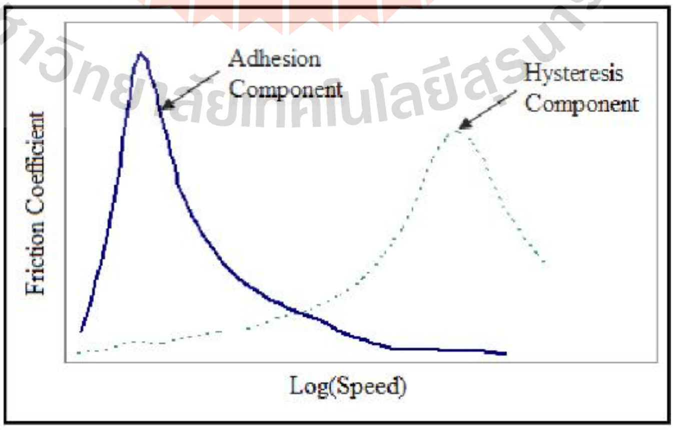
On the other hand, hysteresis forces are due to continuous draping of rubber over pavement aggregate asperities. The pressure distribution about the asperity depends on two distinct conditions:

(1) No sliding (no relative motion)

(2) In the presence of relative sliding

When there is no relative motion, the draping around the contact area and hence the pressure distribution is symmetrical about the asperity giving rise to no net horizontal frictional force. As the sliding begins, rubber accumulates in the leading edge of the asperity creating an asymmetrical pressure distribution producing a net friction force (unbalanced force) opposing the motion. At higher sliding speeds, the extent of the contact area decreases and approaches symmetrical conditions thus reducing hysteresis.


S
Tire
Asymmetric pressure
HYSTERESIS
ADHESION


Adhesion
Component
Hysteresis
Component
Friction Coefficient
Log(Speed)

The knowledge acquired by studying the mechanics of tire-pavement interaction will enable one to better understand how the frictional measurement devices work, what their operational concepts are and hence what they eventually measure.

2.5 Models for Tire Pavement Interaction

The investigators recognize that the tire pavement interaction is viewed from different perspectives by different industries. Customized models developed by these industries provide specific information to those industries in areas of their general interest. Consequently there are two main industries that have proposed different model to predict the frictional properties of a pavement surface, namely: (1) the highway and aviation industries, and (2) the automobile industry.

2.5.1 Models of Highway and Aviation Industries

The highway and the aviation industries have created different empirical models in which they have tried to simulate the coefficient of friction using some measurable texture parameters of the pavement as explanatory variables (NCHRP, 2009). The most popular models are the Penn State University (PSU) model and the Rado model. These models serve as a basis for the PIARC model (Wambold et. al., 1995), which ultimately is used in the computation of IFI

2.5.2 Models of Automobile Industry

The automobile industries have also developed different models that are significant from the vehicle control point of view. Of these, two particular models stand out, namely, (1) The LuGre model and (2) Lumped model

(Seneviratne et. al., 2009). These are dynamic models that focus on the properties of the tire itself rather than those of the pavement. Further details of the above models are found in the cited literature.

2.6 Skid Resistance Requirements

Acceptable level of skid resistance depends on the functional classifications of highways such as rural or urban system, section of highways (e.g. intersection, curve, and steep hills), traffic volumes, speed, and so forth. Kummer and Meyer (1967) summarized the tentative skid resistance in minimum interim skid resistance and made the comparison of minimum friction for main rural highways in State Highways Departments as shown in Table 2.1 and Table 2.2, respectively.

Table 2.1 Recommended Minimum Interim Skid Resistance for Skid Trailer and Portable Tester

Mean Traffic Speed, V (mph)	**Skid Number,[a] SN_{40}**	**British Pendulum Number,[b] (BPN)**
30	31	50
40	33	55
50	37	60
60	41	65
70	46	-
80	51	-

(Kummer and Meyer 1967)

a: Measured at 40 mph in accordance with ASTM E274 Method of Test:

Skid Resistance of Paved Surfaces Using a Full-Scale Tire

b: Measured in accordance with ASTM E303 Method of Test: Measuring Surface Friction Properties Using the British Pendulum Tester

Table 2.2 Comparison of Minimum Frictional Requirements for Main Rural Highways Used by State Highway Departments

Agency	British Pendulum Number, BPN
Arkansas	45
Georgia	50
Louisiana	55
Recommendation	60

(Kummer and Meyer 1967)

Further Road Note 27 (1969) recommended a skid resistance measured by the British Pendulum Tester. The sites are classified into three types depending on the different environment. The result shows that the skid resistance should be set and maintained at the minimum level in the different categories as demonstrated in Table 2.3.

Table 2.3 Criteria of Skid Resistance Measured by British Pendulum Tester

Category	**Type of Site**	**BPN**
A	Difficult Site such as: i) roundabouts ii) bends with radius less than 150 m on unrestricted roads iii) gradients, 1 in 20 or steeper, of lengths > 100 m iv) approached to traffic lights on unrestricted road	65
B	Motorways, trunk and class 1 roads and heavily trafficked roads in urban areas (carrying more than 2,000 vehicles per day)	55
C	All other sites	45

(Road Note 27 1969)

The United Kingdom provided the guidelines for the minimum levels of skid resistance on the main road network detailed in Table 2.4, while Ausroads (2000) suggested skid resistance guidelines by modified site categories and investigatory levels from the United Kingdom, as shown in Table 2.5. Additionally, skid resistance is recommended in New Zealand separated by site category shown in Table 2.6 and in Finland separated as a function of speed as shown in Table 2.7.

Table 2.4 Guidelines for the Minimum Levels of Skid Resistance on Trunk Roads: The United Kingdom

Site Category	**Definition**	**Investigatory Level (SN at 50 kph)**
A	Motorway	0.35
B	Dual carriageway non-event	0.35 – 0.40
C	Single carriageway non-event	0.40 – 0.45
Q	Dual carriageway (all purpose) – minor junctions	0.45 – 0.55
K	Approaches to pedestrian and other high risk situations	0.50 – 0.55
R	Roundabout	0.45 – 0.50
G1	Gradient 5 – 10 % longer than 50 m.	0.45 – 0.50
G2	Gradient ≥ 10 % longer than 50 m.	0.50 – 0.55
S1	Bend radius < 500 m.- dual carriageway	0.45 – 0.50
S2	Bend radius < 500 m.- single carriageway	0.50 – 0.55

(Sinhal 2009)

Remark: Site category R, S1 and S2 were tested at speed 20 kph

Table 2.5 Investigatory Skid Resistance Levels: Australia

Site Category	**Definition**	**Investigatory Level (SN at 50 kph)**	
Speed restriction less than 100 kph		**< 2,500 vpd**	**≥ 2,500 vpd**
1	• Traffic light controlled intersections • Pedestrian/school crossings • Railway level crossings • Roundabout approaches	0.50	0.55
2	• Curve with tight radius ≤ 250 m. • Gradients ≥ 5 % and ≥ 50 m. long • Freeway/highway on/off ramps	0.45	0.50
3	• Intersections	0.40	0.45
4	• Maneuver-free area of undivided roads	0.35	0.40
5	• Maneuver-free area of divided roads	0.30	0.35
6	• Curves with radius ≤ 100 m.	0.50	0.55
7	• Roundabouts	0.50	0.55

(Austroads 2000)

Remark: Site category 6 and 7 were tested at speed 20 kph

Table 2.6 Investigatory Skid Resistance Levels in New Zealand

Site Category	Definition	Investigatory Level (SN at 50 kph)	Threshold Level (SN at 50 kph)
1	Approach to: • railway level crossings • traffic lights • pedestrian crossings • roundabouts • stop and give way controlled intersections (where the State highway traffic is required to stop or give way), • one lane bridges (including bridge deck)	0.55	0.45
2	• Curve < 250 m radius • Down gradients > 10 %	0.50	0.40
3	• Approaches to road junctions (on the State Highway or side roads) • Down gradients 5 - 10 % • Motorway junction area including on/off ramps	0.45	0.35
4	• Undivided carriageways (event-free)	0.40	0.30
5	• Divided carriageways (event-free)	0.35	0.25

(Transit New Zealand 2002)

Note: event-free means where no other geometrical constraint, or situations where vehicles may be required to brake suddenly, may influence the skid resistance requirements.

Remark: Roundabout was tested at speed 20 kph

Table 2.7 Typical Skid Resistance at Various Speeds in Finland

Speed (kph)	**Speed** (mph)	**Acceptable Friction**
≤ 80	≤ 50	0.4
≤ 100	≤ 60	0.5
≤ 120	≤ 75	≥ 0.6

(Noyce et al. 2007)

Jawawickrama et al. (1996) gave comments on the typical thresholds of skid resistance at 40 mph (64 kph) as shown in Table 2.8.

Table 2.8 Typical Thresholds of Skid Resistance

Skid Number: SN_{40}	Comments
<30	Take measures to correct
≥ 30	Acceptable for low volume roads
31 - 34	Monitor pavement frequently
≥ 35	Acceptable for heavily traveled roads

(Jawawickrama et al. 1996)

2.7 Friction Measuring Devices

As mentioned in Section 1, there is a necessity to evaluate accurately the frictional conditions of a pavement surface in order to prevent accidents and ensure safe highway and aviation operations. Reliable pavement surface friction information can be obtained from friction measuring vehicles or by laboratory methods.

2.7.1 Pavement Friction Measuring Vehicles

Different types of vehicles are capable of evaluating the frictional properties of a pavement surface. These devices can be subdivided into four different groups, depending on their operating mechanism. These mechanisms are: (1) the locked wheel, (2) side force, (3) fixed slip, and (4) variable slip. These vehicle subgroups operate under different principles simulating the relevant scenarios. This fact makes direct comparison between devices inappropriate.

The following are the different types of scenarios that these vehicles are designed to simulate:

(1) Locked wheel trailer → Emergency breaking situation without an Anti-Lock Braking Systems (ABS).

(2) Side force → stability in highway curves.

(3) Fixed slip and variable slip → simulated braking action with Anti-Lock Braking Systems (ABS).

One can observe that each device measures a different coefficient of friction on the same surface making difficult the decision making process about the exact or representative frictional conditions of a pavement surface. There are different friction measuring devices that are approved by the Federal Aviation Administration (FAA). Table 2.9 lists them and shows the different coefficient of friction thresholds specified by the FAA for different devices. The different threshold friction levels incorporated in Table 2.9 for different devices clearly shows that the pavement management community has acknowledged the incompatibility among different devices.

The data in Table 2.9 suggest that different devices may be correlated by using a linear model. This correlation is more or less achieved by the International Friction Index (IFI). Of the above, the two most commonly used vehicles used in the industry and the ones that would be the subject of this study are presented in the following sections.

Table 2.9 Friction level classification of runway pavement surfaces

Speed	40 mph			60 mph		
Friction device / Level	0.42	0.52	0.72	0.26	0.38	0.66
Mu Meter	0.5	0.6	0.82	0.41	0.54	0.72
Dynatest Consulting, Inc. Runway Friction Tester	0.5	0.6	0.82	0.34	0.47	0.74
Airport Equipment Co. Skiddometer	0.5	0.6	0.82	0.34	0.47	0.74
Airport Technology USA Safegate Friction Tester	0.5	0.6	0.82	0.34	0.47	0.74
Findlay, Irvine, Ltd. Griptester Friction Meter	0.43	0.53	0.74	0.25	0.36	0.64
Tatra Friction Tester	0.48	0.57	0.76	0.42	0.52	0.67
Norsemeter RUNAR (operated at fixed 16% slip)	0.45	0.52	0.69	0.32	0.42	0.63

(Adapted from FAA 1997)

2.7.1.1 Locked Wheel Trailer (LWT)

The Locked Wheel Trailer (LWT) is an

equipment or device which is the most popular vehicle used by different Departments of Transportation (DOTs) to evaluate pavement condition. It operates under 100% slip conditions, which means that the wheel that is used to measure the coefficient of friction is completely prevented from rolling during testing. It is used to simulate the emergency braking condition without an ABS system. A more detailed operation standard can be found in the ASTM E 274 (ASTM E 274-06, 2009).

2.7.1.2 Runway Friction Tester (RFT)

The Runway Friction Tester (RFT) is a device that is typically used to evaluate the frictional properties of runways. It operates at approximately 15% of slip, in order to simulate the ABS action on the braking operation of aircrafts. The RFT is an approved continuous friction measuring device for which the threshold values for evaluating runway pavement condition can be seen in Table 2.9.

2.7.2 Laboratory Methods

Laboratory methods are available as alternatives for evaluating the frictional properties of a pavement surface. The cost of one of these devices is much lower compared to that of field friction measuring vehicles. There are two commonly devices used in the industry to evaluate surface frictional properties of a pavement in the laboratory. These are: The Dynamic Friction Tester (DFT) and the British Pendulum Tester (BPT).

2.7.2.1 Dynamic Friction Tester (DFT)

The Dynamic Friction Tester (DFT) can be employed to evaluate the surface frictional properties of a pavement. The measuring mechanism of the DFT is based on energy concepts with the loss of kinetic energy of a rotating disk resting on rubber sliders converted to an equivalent frictional force exerted by the pavement. DFT is capable of measuring friction over the sliding speed range of zero to 90 Km/h. A more detailed operation standard can be found in the ASTM E 1911 (ASTM E 1911-09, 2009). ASTM E 1960 advocates the use of DFT for the calibration of friction testing devices due to the high repeatability of DFT in IFI computations (Henry et. al., 2000). The DFT is used in conjunction with the Circular Track Meter (CT Meter) to calculate the IFI of a pavement surface. The CT Meter is a device used to evaluate texture properties of a surface, specifically the Mean Profile Depth (MPD) which is used to explain the friction-velocity dependency in the IFI model. A more detailed operation standard on the CT Meter can be found in the ASTM E 2157 (ASTM E 2157-01, 2009).

2.7.2.2 British Pendulum Tester (BPT)

The British Pendulum Tester (BPT) measures the frictional properties of pavement surfaces. The BPT measures friction at a low-sliding speed contact between a standard rubber slider and the pavement surface. The elevation to which the pendulum swings after contact provides an indicator of the frictional properties of the pavement surface (NCHRP, 2009). The standard practice for measuring surface frictional properties using the British Pendulum Tester can be found in the ASTM E 303 (ASTM E 303-93, 2008).

2.8 Skid Resistance Models

Pavement friction models have been developed for predicting skid resistance. Due to the complex interaction between many factors affecting skid resistance, however, the factors in the model should be evaluated and the prediction model should be developed to explain the effect of each factor on the skid resistance value. There are various pavement friction models, which were developed to the root of new models up to date. This section gives the overview of the various models.

2.8.1 The Penn State Model

The Penn State model shows the relationships between skid number and speed proposed by Leu and Henry (1978). This model was clearly described the significant feature of two texture type: microtexture and macrotexture. Nevertheless, the effects of commercial tires, water film thickness, seasonal effects and other test conditions were excluded from consideration. The model was formulated as follows:

$$SN = SN_0 \, exp\ [-(PSNG/100)\ V(2.1) \tag{2.8}$$

$$PSNG = -\ (100/SN)\ (dSN/dV) \tag{2.9}$$

where;

SN_0 = Intercept of friction at zero speed

PSNG = The percentage skid number-speed gradient (%)

V = Slip speed

Afterwards, Kulakowski (1991) revised percentage of skid number-speed gradient (PSNG) instead of a speed constant (*V_0*). As a result, the Pen State Model was shown below:

$$SN = SN_0 \, exp\ [-(V) / V_0] \tag{2.10}$$

As mentioned above, pavement microtexture and SN_0 was determined from two parameters: root mean square height ($RMSH_{MI}$) derived from profile analysis and British Pendulum Number (BPN) from core samples by using British Pendulum Tester (BPT). The two parameters: root mean square height ($RMSH_{MA}$) determined by the profile analysis and mean texture depth (MTD) derived from the sand patch method are significant to determine pavement macrotexture and PSNG. Thereby, this model can be applied to predict skid number (SN) at any speeds from microtexture and macrotexture measurements. In other words, surface microtexture can provide high skid resistance at low speed, and conversely, surface macrotexture can control friction efficiently at high speed. Based on the Pen State Model, Murat et al. (2005) developed another model for predicting the friction coefficients involving merely micro and macrotextures. It was found that SN_0 and V_0 were affected by micro and macrotexture.

$$SN = [0.37 + \frac{0.11}{MPD_{mac}} + \frac{0.15}{La_{mic}}]\ exp\ [-(V) / [149 + 81 Log(MPD_{mac}) + 80 Log(Rq_{mic})]] \tag{2.11}$$

where;

MPD_{mac} = mean profile depth (macrotexture)

La_{mic} = average wavelength of surface profile (microtexture)

Rq_{mic} = root mean square deviation of surface profile (microtexture)

2.8.2 The Rado Model

Rado (1994) processed a tire from free rolling condition to the locked wheel condition under braking. The friction increased from zero to a peak value and then decreased to locked wheel friction. The Rado model was formulated to measure the skid resistance on speed behavior as follows:

$$\mu(S) = \mu_{peak}\ exp\text{-}[\frac{\ln(s/s_{peak})}{C}]^2 \tag{2.12}$$

where;

μ_{peak} = peak friction level

S_{peak} = slip speed at the peak

C = shape factor (related to the harshness of texture)

2.8.3 The PIARC Model

Later, PIARC (1995) developed the Pen State Model and proposed an international scale of friction value. It is called the International Friction Index (IFI). The IFI consists of two terms: F60 and S_p. F60 is the estimated friction at 60 kph and S_p is the speed constant. The proposed model is shown below

$$F60 = A + B\ FRS\ exp\ [\frac{S-60}{S_p}] + C\ TX \tag{2.13}$$

$$S_p = a + b\ TX \tag{2.14}$$

where;

S_p = IFI speed constant

a,b = constants determined for a specific macrotexture measuring device

TX = macrotexture parameter reported by the specific macrotexture measuring device (e.g., MTD or MPD)

F60 = IFI friction number

S = slip speed

FRS = measurement of friction by a device operating at a slip speed

A, B, C = constants determined for a particular friction measuring device

After the IFI is obtained, the friction value at any other slip speed (S) can then be computed as this equation, which is called as PIARC model:

$$F(S) = F60 \, exp \left[\frac{60 - S}{S_p} \right] \tag{2.15}$$

2.8.4 The Wisconsin Model

Wisconsin developed a frictional equation for asphalt pavement, which is a function between the accumulated vehicle passes, percent dolomite in the coarse aggregate, Los Angeles wear test results, and percent of heavy vehicles (Noyce et al. 2005). The model was formulated as follows:

$$FN = 41.4 - 0.00075D^2 - 1.45 \ln(LAVP) + 0.245 \, LAWEAR \tag{2.16}$$

where;

FN = friction number calculated at 40 mph

D = % dolomite in the mix

LAVP = lane accumulated vehicle passes

LAWEAR = Los Angeles Wear (%)

The further study was recommended by authors to focus on more efficient system of measuring friction values and the quality of pavement mix designs by incorporating friction criteria to be inputted to the pavement management systems in order to receive the optimum schedule of maintenance and resurfacing.

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CHAPTER III

SKID RESISTANCE OF ASPHALT CONCRETE AT THE CONSTRUCTION STAGE BASED ON THAI AGGREGATES

3.1 Background and Introduction

The annual number of road accidents in Thailand has dramatically increased over time. The country has recently become statistically known to have the third highest number of road traffic deaths in the world (WHO, 2013), and this adverse finding has led to a demand for more effective road safety strategies. According to the WHO report, the other main factor in deaths from road accidents, apart from dangerous driving behavior, is the poor physical condition of Thai roads. Consequently, some of the key elements in Thai road design and construction, such as road geometry and on road post construction conditions, require further examination into their effects upon the safety of roads. These elements generally relate to the safety of the road, with skid resistance being both directly related and fundamental to safety.

Generally, the braking distance required by an individual vehicle depends upon the skid resistance between the vehicle tires and the road surface. Ideally, the skid-resistance values of asphalt surfacing (i.e., the majority of road surfaces in

Thailand) would be known prior to the actual construction of the road. This would allow the road to be constructed to a level of safety, which would last throughout its lifetime. However, in practice, skid-resistance values can only be measured in-situ, that is on the road itself, and prior to post-construction stages. Because of these difficulties, skid-resistance values have generally not been accounted for during the asphalt mixture design processes, or during the aggregate selection processes.

Based on the literature, the consideration of skid resistance in road surfaces is believed to be essential in road safety management schemes, as it directly relates to the incidence of road accidents (Henry 2000; Ivey et al. 1992). According to Rizenbergs et al. (1973), wet surface conditions reduce frictional resistance, resulting in the increase in collisions and accidents. Research on measurement of skid resistance was also undertaken by the Texas Department of Transportation (TxDOT) (McCullough snd Hankins 1966) during wet surface conditions, where vehicle speeds were equal to 80 km/h (50 mph). The results showed a definite statistical trend between low skid resistance and higher accident rates. Various other works, such as those of Agent et al. (1996), and Wallman and Astrom (2011) support the TxDOT research. Moreover, studies into the effects of wet surface conditions on tangential wheel alignment, using a Sideways-Force Coefficient Routine Investigation Machine (SCRIM) at 13 percent slip (Viner et al 2004), showed that the number of road vehicle accidents was statistically related to pavement skid resistance.

Factors affecting pavement skid (frictional) resistance can be classified into four groups: pavement surface characteristics, vehicle operating parameters, tire properties, and environmental factors (Wallman and Astrom 2011). Hall et al. (2009) explained that adhesion and hysteresis were the two main types of frictional force produced by the vehicle tire making contact with the pavement surface. Both frictional force components are influenced by the texture of the road surface. In particular, adhesion depends on the micro-texture of the road surface, whereas hysteresis relies greatly on macro texture. Accordingly, the aggregate for pavement mixture is one of the most important influences on the road surface texture and on pavement skid resistance. In the National Cooperative Highway Research Program (NCHRP) guidelines (Hall et al. 2009), aggregates from different locations were tested as per recommended standard procedures. In general, it was found that in the case of standard-compliant aggregates unavailable from one source, pavement aggregates could be selected from at least two main sources and combined to achieve the correct design gradation, along with the other previously mentioned characteristics. The testing process is usually performed prior to the start of a project. This ensures a satisfactory grading of the aggregate characteristics, which in turn should ensure road pavement of adequate, if not high quality. In addition to the standard requirements of other countries, the polishing resistance of a selected aggregate, estimated from Polished Stone Values (PSVs), is one of the recommended design parameters in the NCHRP guidelines for high resistance.

The objectives of this study are (1) characterization of Thai aggregates in relation to the skid-resistance values of asphalt concrete mixture, and (2) development of a predictive skid-resistance model of an asphalt concrete mixture at the construction stage, based on the aggregate and mixture characteristics. Generally, pavement asphalt concrete is classified into dense-grade, gapped-grade, and porous types. In this study, the skid-resistance predictive model was proposed for the dense-grade asphalt concrete because it is the typical pavement in Asian countries including

Thailand. The aggregates used for the model development were granite, basalt, and limestone, which are locally available in Thailand. As such, the proposed model is applicable to dense-grade mixtures with various aggregates available in Thailand and in other similar Asian countries.

The model will be used in a preventive scheme for road safety. The expected outcome of this study is that predictive skid resistance values for Thailand's asphalt concrete mixtures would be estimated at the level of aggregate selection and asphalt concrete mixture design processes prior to real construction. An additional safety criterion (in the form of a skid-resistance value) would be incorporated into the aggregate selection processes and the asphalt concrete mix design protocol, rather than relying solely on engineering criteria (e.g., aggregate hardness, Marshall Flow and Stability values, stiffness) as is usual. Moreover, road safety management would be more controllable when skid-resistance values of asphalt concrete are reliably known before construction or during the planning of a project.

3.2 Methodology and Experimental Work

3.2.1 Sources of aggregate

Comprehensive sources of aggregate used in this study were quantitative and qualitative ranked as representative of the aggregates in Thailand's road network (Department of Rural Roads, 2014). Fourteen provinces in Thailand were selected and three main aggregate types were covered. The regions and provinces were:(1) Northern region: Chiangmai, Sukhothai and Tak, (2) Central region: Nakhonsawan, Sara BuriandRachaburi (3) Eastern region: Chonburi (4) Northeastern region: NakhonRatchasima, UbonRatchathani and Buriram and

(5) Southern region: Trang, Songkhla, SuratThani and Narathiwat. The aggregates were granite, limestone and basalt.

3.2.2 Asphalt concrete mixture

The amount of asphalt binder, air void and aggregates of asphalt concrete samples were based on Marshall mix design procedure in accordance with the specifications of the Department of Rural Roads (2002). Only the Marshall mix design is accepted for asphalt pavement design in Thailand. The asphalt mixtures used were designed to produce wearing courses of dense graded asphalt concrete with 9.5 mm aggregate maximum size (AC 9.5) and dense graded asphalt concrete with 12.5 mm aggregate maximum size (AC12.5). The cylindrical test samples for the skid-resistance tests were prepared and comprised 28 job mix formulas (14 formulas for AC 9.5 and 14 formulas for AC 12.5).

3.2.3 Aggregate Characteristic Testings

The investigation into aggregate characteristics used the following tests:

- Microscopic investigation, in conjunction with X-ray Diffraction (XRD) and X-ray Fluorescence Spectrometry (XRF) for evaluating the mineral composition and contents of aggregates;
- Determination of aggregate impact resistance using Aggregate Impact Value (AIV) in accordance with BS 812-112 (BSI1990a);
- Determination of crushing resistance of aggregate using aggregate crushing value(s) (ACV) in accordance with BS 812-110 (BSI 1990b);

- Determination of abrasive resistance of aggregate using the Los Angeles Abrasion (LAA) value based on a procedure adapted from ASTM C131-06 (ASTM2006) and ASTM C535-12 (ASTM2012);
- Determination of soundness of aggregate using sodium sulphate or magnesium sulphate [equivalent to AASHTO T 104 (AASHTO 1999)or ASTM C88-13 (ASTM, 2013)]; and
- Determination of frictional resistance of material using PSVs in accordance with BS 812-114 (BSI1989). In this test, the aggregate size between 7.94 and 9.52 mm was used. The PSV is the friction determined from the British Pendulum test after polished by the specific machine for 6 hours.

3.2.4 Determination of Texture Depth (T_d) or Mean Texture Depth (MTD) of Asphalt Concrete

The standard sand patch testing procedure (ASTM, 2011; BSI, 1990d) was used for measuring the MTD in the field. Dry and round sand for which a minimum of 90% of total weight passes a No.60 sieve and retains on a No.80 sieve was used for this test.The values of the MTD can be calculated according to Eq. (3.1)

$$\mathrm{MTD} = \mathrm{W}/(\mathrm{A} \times \mathrm{GSz}_{\mathrm{sand}}) \tag{3.1}$$

where W = weight of sand on the asphalt sample surface; A = cross-sectional area of the asphalt sample; and GS_{sand} = specific gravity of the tested sand.

3.2.5 Data Collection of Field Skid-Resistance and MTD Values

Data collection of field skid-resistance values was conducted, focusing on the 14 provinces from where the aggregates were sourced. One hundred and six construction projects, from a total of 14 provinces, were selected to cover a specific range of aggregates (limestone, granite and basalt) and mix types (AC9.5 and AC12.5). The road construction projects selected had complete records of skid-resistance measurements, texture depths, construction history and specific weather conditions during construction. These projects were constructed with the same aggregate types and sources that had already been tested in the laboratory studies. The field skid-resistance values of SRV35 and the texture depth values of MTD from the standard sand patch method were selected and used only on pavement less than three months after being constructed. This ensured that reasonable values of SRV35 and MTD could be obtained to represent construction stage values. Table 3.1 shows the collected data and the data analysis results.

Table 3.1 Data collection of field Skid Resistance and MTD Values

Source	Type of Aggregate	Surface Texture and Skid - resistance							
		MTD (mm)			SRV35			F60	
		No.of samples	9.5	12.5	No.of samples	9.5	12.5	9.5	12.5
Songkhla	Granite	18	0.31	0.38	20	69	71	0.61	0.61
Trang	Limestone	14	0.23	0.35	16	57	71	0.55	0.62
Chonburi	Limestone	22	0.23	0.30	18	52	56	0.50	0.49
Tak	Limestone	16	0.22	0.28	12	49	52	0.48	0.46
Chiangmai	Limestone	36	0.27	0.29	32	50	62	0.45	0.56
Nakhonsawan	Limestone	21	0.25	0.35	19	58	68	0.54	0.59
Sukhothai	Limestone	18	0.30	0.33	20	63	67	0.57	0.58
Ratchaburi	Limestone	16	0.27	0.30	23	60	62	0.55	0.55
Saraburi	Limestone	14	0.25	0.33	25	49	61	0.45	0.53
Nakhon Ratchasima	Basalt	31	0.28	0.37	41	66	72	0.60	0.62
UbonRatchathani	Basalt	28	0.28	0.33	32	60	61	0.54	0.53
Buriram	Basalt	13	0.31	0.39	16	67	71	0.59	0.60
Narathiwat	Granite	15	0.30	0.34	14	64	66	0.58	0.57
Surat Thani	Limestone	16	0.31	0.35	18	59	64	0.52	0.55

Note: 9.5 = asphalt mixture type AC 9.5; 12.5 = asphalt mixture type AC 12.5; F60 = international friction index (IFI) or Frictional resistance at test speed equal to 60 km/h; MTD = mean texture depth calculated from Eq. (3.2); SRV35 = skid-resistance value at 35°C.

3.2.6 Statistical based analysis of testing results

Following the aggregate characteristics reported by Department of Rural Roads (2014) and field data collection, the test results were descriptively and technically analyzed using statistical methods. Statistical correlations among the test results, i.e., aggregate characteristics, asphalt concrete mixture, skid resistance, and the asphalt texture surface characteristics from the field sections, were necessary in order to develop the predictive model. Along with the correlation examination, the statistical analysis was also required prior to model development.

In order to develop the skid-resistance predictive model, the correlation between field skid resistance and the characteristics of the material, i.e., the aggregate and asphalt mixture, which were previously statistically investigated (Department of Rural Roads, 2014) was required. In this study, the Pearson's Correlation Coefficient (r), together with a statistical hypothesis test were used in the correlation analysis. In hypothesis testing, a t-test with a null hypothesis (H_0) of two variables are not correlated, is performed using 0.05 as a significance level. Prior to the correlation analysis, an Analysis of Variance (ANOVA) was required for variance analysis of the test results. Similar to the correlation analysis, ANOVA was used in this study in order to investigate the differences between group means. Therefore, the F-test, which is more suitable to statistically examining more than two populations with different sizes of variables than the *t*-test, was used in this section. In the case of ANOVA, the null hypothesis is that all group means are equal, unless at least one pair is not equal, and then the null hypothesis is rejected. This null hypothesis also means that all data groups are random and sampled from the same population. For example, when examining the effects of asphalt mixture type on road skid resistance, the null hypothesis would be that both types of asphalt mixtures have the same effect on skid-resistance. Rejecting the null hypothesis indicates that different asphalt mixture types result in altered effects.

3.3 Results and Discussion

3.3.1 Aggregate characterization

The aggregate characteristics reported by Department of Rural Roads (2014) has been analyzed and presented in this section. Microscopic investigation of aggregates prepared in the form of a thin section can be used to detect aggregate mineral and texture composition and distribution. In this study, the microscopic investigation was performed in conjunction with XRD and XRF for quantitatively evaluating the mineral composition and contents of aggregates. Fig. 3.1 illustrates examples of microscopic photographs of three main aggregate types, Sukhothai limestone, Songkhla granite, and Ubon Ratchathani basalt. It is clear that the three main aggregate types have visually different textures and compositions. Table 3.2 summarize XRD and XRF examinations for limestone, basalt, and granite, respectively.

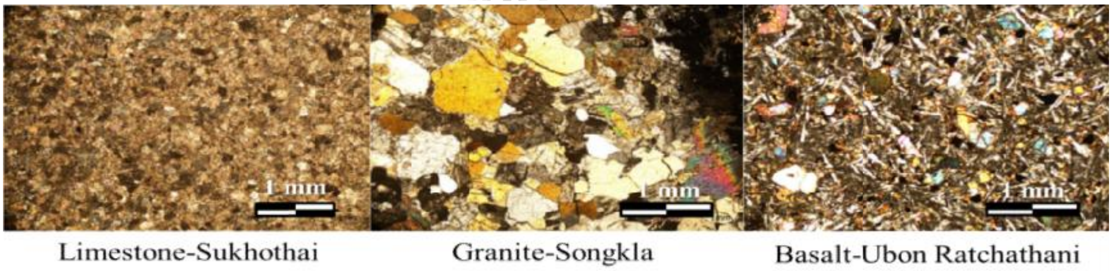


Fig. 3.1 Different textures and compositions for examples of microscopic photographs of the three main types aggregates (reprinted with permission from Department of Rural Roads 2014)

Table 3.2 Results of XRD and XRF Examinations for Limestone (Reprinted from Department of Rural Roads 2014, with Permission)

Source	Mineral	Main mineral compositions		
		CaO (%)	SiO_2 (%)	MgO (%)
Nakhonsawan	Calcite ($CaCO_3$), Dolomite($CaMg(CO_3)_2$)	92.50	5.59	0.78
Saraburi	Calcite ($CaCO_3$)	81.10	8.64	3.14
Tak	Calcite ($CaCO_3$), Amorphous Silicate (SiO_2)	46.30	34.00	2.13
Chonburi	Dolomite ($CaMg(CO_3)_2$), Microcline ($KAISi_3O_8$), Marcasite (FeS_2), Calcite ($CaCO_3$), Cristobalite (SiO_2),Amorphous Silicate (SiO_2), Quartz (SiO_2)	11.00	56.80	2.94
Sukhothai	Calcite ($CaCO_3$)	99.00	0.00	0.22
Trang	Dolomite ($CaMg(CO_3)_2$)	70.30	0.44	28.90
NakhonRatchasima	Dolomite ($CaMg(CO_3)_2$),Calcite ($CaCO_3$)	86.70	0.00	12.80
Chiang Mai	Calcite ($CaCO_3$)	87.60	7.20	0.71
Ratchaburi	Calcite ($CaCO_3$)	97.90	0.81	0.59
SuratThani	Dolomite ($CaMg(CO_3)_2$)	68.80	1.99	27.80

Table 3.3 Results of XRD and XRF Examinations for Granite (Reprinted from Department of Rural Roads 2014, with Permission)

Source	Mineral	Main mineral composition		
		Al_2O_3 (%)	SiO_2 (%)	K_2O (%)
Songkla	Biotite ($K(Mg,Fe)_3(AlSi_3O_{10})(F,OH)_2$),Albite ($NaAlSi_3O_8$), Quartz ($SiO_2$)	14.10	74.60	5.28
Narathiwat	Microcline ($KAISi_3O_8$),Albite ($NaAlSi_3O_8$),Amorphous Silicate (SiO_2), Quartz (SiO_2), Biotite ($K(Mg,Fe)_3(AlSi_3O_{10})(F,OH)_2$), Cristobalite ($SiO_2$)	14.00	72.80	6.41

Table 3.4 Results of XRD and XRF Examinations for Basalt (Reprinted from Department of Rural Roads 2014, with Permission)

Source	**Mineral**	**Main mineral composition**		
		Al_2O_3 (%)	**SiO_2 (%)**	**Fe_2O_3 (%)**
Burirum	Augite ((Ca,Na)(Mg,Fe,Al,Ti)(Si,Al)$_2$O$_6$), Enstatite ($Mg_2Si_2O_6$), Labradorite ((Ca,Na)(Si,Al)$_4$O$_8$), Forsterite (Mg_2SiO_4),Amorphous Silicate (SiO_2)	16.10	49.90	10.00
UbonRatchathani	Augite ((Ca,Na)(Mg,Fe,Al,Ti)(Si,Al)$_2$O$_6$), Enstatite ($Mg_2Si_2O_6$), Labradorite ((Ca,Na)(Si,Al)$_4$O$_8$), Forsterite (Mg_2SiO_4),Amorphous Silicate (SiO_2)	17.00	47.60	11.80

According to the results of the XRD and XRF analyzes, selected limestone in this study can be categorized based on chemical and mineral compositions, as summarized in Table 3.2. Generally, calcium oxide (CaO) constitutes the key and original chemical composition of limestone. The amount of CaO found in limestone usually indicates the state of rock transformation. For example, the major chemical composition of Sukhothai limestone is CaO (99%), which means that the limestone from this source has undergone little metamorphosis. Therefore, out of all the aggregates used in this study, Sukhothai limestone would therefore be the most suitable material for road construction because limestone containing high CaO resists weather deterioration more effectively than limestone with a low CaO content. Limestone from the other sources, besides containing CaO, consisted of silica dioxide (SiO_2) or amorphous silica in its chemical composition. Increased amounts of SiO_2

in the rock lead to enhancement of rock hardness, but show a reduction in rock durability. This hypothesis is supported by the AIV and ACV from the aggregate impact test and aggregate crushing test, respectively (Table 3.5). The values of AIV and ACV from Chonburi limestone, which contains the highest percentage of SiO_2 (Table 3.5), indicate this to be the strongest rock among the limestone gathered from locations around Thailand. Similar to limestone, a high SiO_2 content in granite and basalt also results in greater values of AIV and ACV. However, the SiO_2 in granite is the chemical composition of quartz, not the amorphous silica seen in limestone and basalt. The microscopic photographs of Ubon Ratchathani and Buriram show a very low degree of weathering and erosion. Nevertheless, the amorphous silica found in these rocks may be the cause of rapid degradation in the future.

Table 3.5 Engineering Properties of Aggregate (Reprinted from Department of Rural Roads 2014, with Permission)

Source	**Main Types of Aggregates**	**Aggregate Properties**				
		AIV (%)	**ACV (%)**	**LAA (%)**	**Soundness (% Loss)**	**PSV**
Songkhla	Granite	20	20	21.2	3.07	44
Trang	Limestone	22	24	27.0	1.10	37
Chonburi	Limestone	12	18	13.7	2.11	46
Tak	Limestone	26	29	31.1	1.60	40
Chiangmai	Limestone	22	23	28.8	3.62	38
NakhonSawan	Limestone	26	24	20.7	1.50	38
Sukhothai	Limestone	20	25	23.1	1.40	44
Ratchaburi	Limestone	22	24	26.1	1.10	39
Saraburi	Limestone	22	25	22.5	1.20	37

Source	Main Types of Aggregates	Aggregate Properties				
		AIV (%)	ACV (%)	LAA (%)	Soundness (% Loss)	PSV
NakhonRatchasima	Basalt	19	23	22.2	1.40	33
UbonRatchathani	Basalt	13	17	15.2	1.30	35
Buriram	Basalt	13	17	14.4	1.80	38
Narathiwat	Granite	20	25	20.7	2.96	48
Surat Thani	Limestone	19	20	12.6	1.95	37

Note: ACV = aggregate crushing value; AIV = aggregate impact value; LAA = Los Angeles abrasion; PSV = polished stone value (after polishing).

Table 3.5 summarizes the results of the aggregate tests. The AIV, LAA test, PSVs and soundness number were averaged from two samples from each test. From the impact test results, Tak and Nakhon Sawan limestone are found to have the highest AIV (26%) within the same type of aggregate, whereas Chonburi limestone produces the lowest AIV (12%). Similar results are observed from the aggregate crushing test and the LAA test. Limestone from the Tak province provides the highest ACV (29%) and LAA (31.1% loss), whereas the lowest values (18% and 13.7% loss) are observed from Chonburi limestone. However, Chonburi limestone is found to have the greatest PSV (46) compared to limestone from the other provinces. However, Chonburi limestone may degrade more quickly than other types because it possesses the highest SiO_2 content. The AIV, ACV, LAA and PSV of basalt from three different sources are almost consistent, but the values of these parameters are lower than those from the tests on limestone and granite. The soundness values of limestone and basalt range from 1.10% to 3.62% loss, whereas granite from Songkhla and Narathiwat show a higher percentage loss than the other two types of aggregate, because the main minerals in Songkhla and Narathiwat granite are feldspar, quartz and biotite. The main chemical compositions of these rock minerals, i.e., SiO_2 and Al_2O_3, result in aggregates with high impact value (AIV) and a high crushing value (ACV). However, photos taken by a polarizing microscope indicates that the aggregate from these locations has been subjected to severe weathering and erosion by the environment. However, based on the polished stone test results, the granite has the highest PSV.

The particle size distribution analysis of aggregates from 14 locations around Thailand shows that most of the aggregates are categorized as well-graded material. Only particle size distributions of Trang limestone and Nakhon Ratchasima basalt fall slightly outside the well-graded zone.

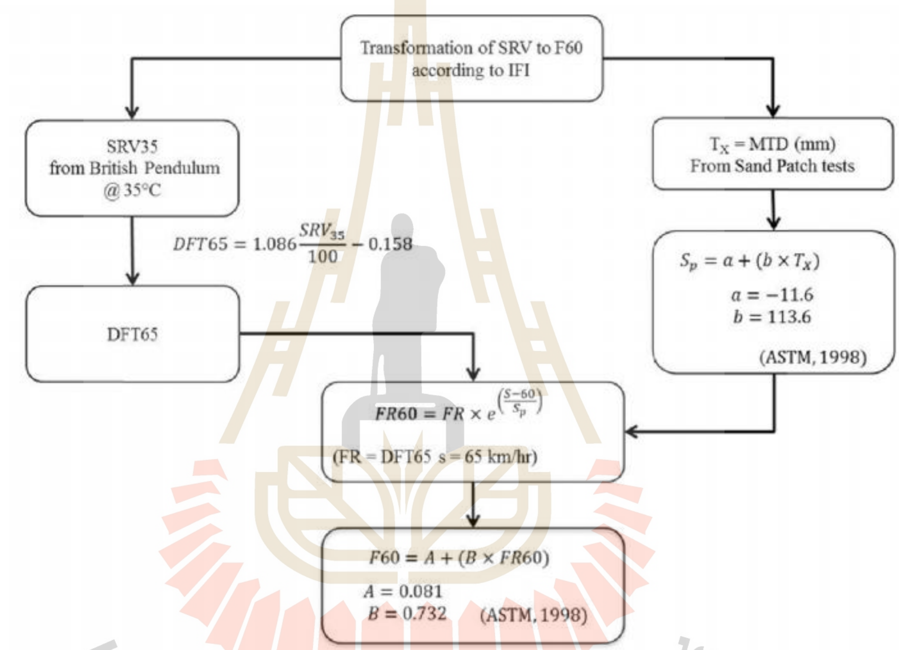
3.3.2 Surface texture and skid-resistance of asphalt concrete

Field Skid-Resistance Values (SRVs) of asphalt concrete were measured by using a British pendulum tester (Munro Instruments, Essex, U.K.) (Table 3.1). The SRV measurements were conducted in a laboratory where the temperature was set at 25°C. According to a study by Beaven and Tubey (1987), the measured SRVs require conversion to the values measured at 35°C (SRV35) for countries located close to the equator. Accordingly, the field SRV35 can be calculated from Eq. (3.2)

$$\mathrm{SRV35} = \frac{(100+t)}{135} \mathrm{xSRV_t} \tag{3.2}$$

where t = temperature (°C) measured during the test; and SRV_t = measured skid-resistance value at the pavement surface temperature t °C. The skid-resistance values at high temperatures are lower than those at low temperatures.

The International Frictional Index (IFI), developed by the Permanent


Transformation of SRV to F60 according to IFI
SRV35 from British Pendulum @ 35°C
T_X = MTD (mm) From Sand Patch tests
$DFT65 = 1.086\frac{SRV_{35}}{100} - 0.158$
DFT65
$S_p = a + (b \times T_X)$
$a = -11.6$
$b = 113.6$
(ASTM, 1998)
$FR60 = FR \times e^{\left(\frac{S-60}{S_p}\right)}$
(FR = DFT65 s = 65 km/hr)
$F60 = A + (B \times FR60)$
$A = 0.081$
$B = 0.732$
(ASTM, 1998)

have the same effect on the MTD and SRV35 (Table 3.6).

Table 3.6 Analysis Results of Field Skid Resistance from *t*-Test and ANOVA

Tested Variable	Independent variable	*P*-Value
MTD	AC9.5 and AC12.5	0.0000
SRV35	AC9.5 and AC12.5	0.0002
MTD of AC9.5	Limestone, Granite and Basalt	0.0898
MTD of AC12.5	Limestone, Granite and Basalt	0.0656
SRV35 of AC9.5	Limestone, Granite and Basalt	0.0719
SRV35 of AC12.5	Limestone, Granite and Basalt	0.2618
MTD of AC9.5	Northern, central, eastern, northeastern and southern regions	0.3713
MTD of AC12.5	Northern, central, eastern, northeastern and southern regions	0.0554
SRV35 of AC9.5	Northern, central, eastern, northeastern and southern regions	0.1650
SRV35 of AC12.5	Northern, central, eastern, northeastern and southern regions	0.2000

3.3.4 ANOVA on the Effect of Aggregate Source and Aggregate Type (Limestone, Granite and Basalt) on the MTD or Skid Resistance (SRV35)

In this section, MTD and SRV35 from the field measurements were used to examine the effects of aggregate type on the skid resistance of asphalt concrete. In Table 3.6, the results from the F-test shows that P-values for every aggregate source and aggregate type are greater than 0.05. The null hypothesis of different aggregate sources and types having the same effect on the MTD and SRV35 cannot be rejected at a 95% confidence level. This indicates that different aggregate sources and types have no influence on the road surface texture and skid-resistance.

3.3.5 Correlation between Field Skid Resistance (F60) and Aggregate Characteristics (AIV, ACV, PSV, LAA, and Soundness Value)

Table 3.7 summarizes the results from the correlation analysis used in this study. The value of R^2 (R-Square) in Table 3.7 indicates the degree of correlation between two parameters. According to statistical theory, R^2 varies from 0 to 1. When the value of R^2 is close to 1, the dependent variable is strongly reliant upon the independent variable, and vice versa. Likewise, the coefficient of correlation (*R*-Value) was used in this research to allow analysis of the degree of correlation between two parameters. The *R*-Value varies from -1 to 1, and a 0 (zero) *R*-value indicates no correlation between the two variables. Besides the R^2 and *R* values, the statistical hypothesis *t*-test was also used in the correlation analysis. The null hypothesis differs from the ANOVA test in that the two parameters are not correlated (R^2 closest to 0). In the correlation analysis, confidence intervals are stated at the 95% confidence level. Therefore, the null hypothesis of no correlation will be rejected, if the *P*-value is less than 0.05.

Table 3.7 Correlation Analysis Results of Field F60 by Regression

Dependent Variable	Independent variable	R^2	R-Value	P-Value
F60 of AC9.5	AIV	0.073	0.270	0.351
	ACV	0.090	0.300	0.298
	LAA	0.079	0.282	0.329
	Soundness	0.003	0.056	0.848
	PSV	0.017	0.129	0.660
	PSV_{diff}	***0.736***	***0.858***	***0.000***
	Percent passing No.200 sieve	0.004	0.063	0.830
	Percent passing No.4 sieve	0.004	0.063	0.831
	Binder content	0.002	0.050	0.866
	Void ratio	0.117	0.342	0.231
	MTD	***0.798***	***0.878***	***0.002***
F60 of AC12.5	AIV	0.000	0.020	0.946
	ACV	0.023	0.152	0.603
	LAA	0.001	0.038	0.898
	Soundness	0.002	0.040	0.893
	PSV	0.042	0.206	0.480
	PSV_{diff}	***0.562***	***0.750***	***0.002***
	Percent passing No.200 sieve	0.030	0.174	0.552
	Percent passing No.4 sieve	0.193	0.440	0.116
	Binder content	0.015	0.121	0.681
	Void ratio	0.004	0.064	0.828
	MTD	***0.621***	***0.788***	***0.001***

The analysis results in Table 3.7 show that the field skid-resistance parameter (F60) does not correlate with nor depend upon the aggregate characteristics, in terms of AIV, ACV, PSV, LAA, and soundness value. The R and R^2 values are relatively small, indicating that the correlation between F60 and the aggregate characteristics is statistically low. This is also proven by the *P*-values from statistical hypothesis testing, of which all *P*-values are greater than 0.05. Therefore, aggregate characteristics considered in this analysis (AIV, ACV, PSV, LAA and soundness value) are not suitable for the development of the skid-resistance predictive model.

Mahmoud and Masad (2007) proposed a model that represented the relationship between aggregate textures and polishing time. In addition to the PSV, the polished stone values of samples before polishing (PSV_{before}), and the different values between PSV_{before} and PSV (PSV_{diff}) were considered in the analysis. Table 3.7 shows that the R^2 and R values of PSV_{diff} and F60 are significant for both AC 9.5 and AC 12.5. Moreover, the null hypothesis of no correlation between PSV_{diff} and F60 is rejected at a 95% confidence level, because *P*-Values are lower than 0.05. It can be concluded that PSV_{diff} is appropriate for use in the parameters for F60 predictive model development.

3.3.6 Correlation between Field skid Resistance (F60) and Asphalt Mixture Design (Percentage of Coarse and Fine Aggregate, Binder Content, and void Ratio), or MTD

The percentages of coarse aggregate (passing a No.4 sieve) and fine aggregate (passing a No.200 sieve) for asphalt mixture, along with the asphalt binder content and void ratio of the asphalt concrete sample do not show a strong correlation with the F60 of AC 9.5 and AC 12.5, as shown in Table 3.7. However, Table 3.7 illustrates a significant correlation between the MTD and F60 of AC 9.5 and AC 12.5. This is also proven by high R and R^2 values and low P-values. It can be concluded that the F60 of asphalt AC 9.5 and AC12.5 correlates to the MTD parameters.

3.4 Development of Skid-Resistance Predictive Model

The field and statistical analysis results from previous sections were used for the development of the predictive model, which focuses on estimating the field F60 at the construction stage.

In the previous section, suitable input parameters derived from the F60 for the predictive model were determined from statistical analysis. Among these parameters, PSV_{diff} and MTD could indirectly represent the skid-resistance-related characteristics of aggregate and asphalt concrete, respectively.

As stated earlier, the input parameters for the predictive model described in the previous section can be subdivided into two groups based on a texture scale. The input parameter representing the microtexture characteristics is PSV_{diff}, whereas MTD represent the characteristics of the macrotexture. Therefore, the input parameters for the F60 predictive model development include (Fig. 3.3):

- PSV_{diff}, which is the difference between PSV values of aggregate before and after polishing;
- MTD, which is tested by using the sand patch method.

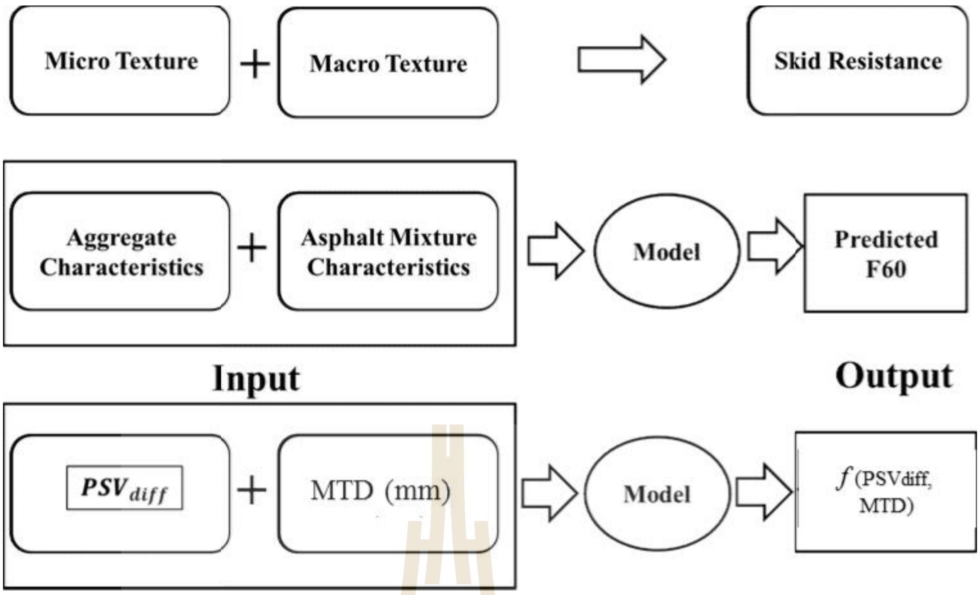


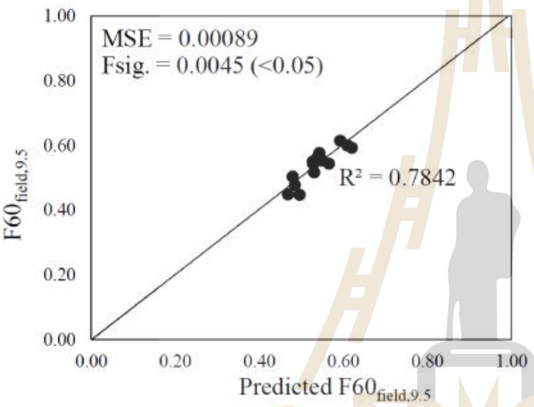
Fig. 3.3 Model approach diagram

Following the selection of the input parameters for the predictive model, the multiple linear regression analysis technique was used to develop the predictive model for field F60. The following equation represents the skid-resistance predictive model:

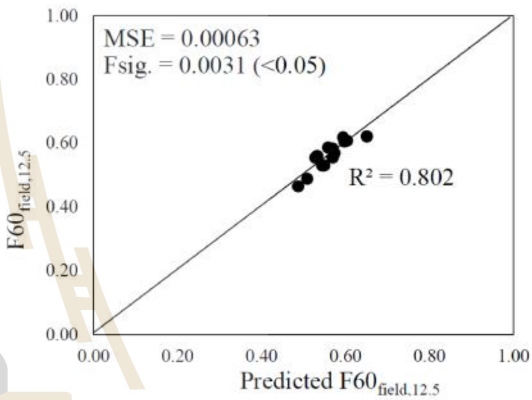
$$F60 = A_x MTD_x + B_x PSV_{diff} + E_x \tag{3.3}$$

where x = type of asphalt, AC 9.5 or AC 12.5. In the prediction of F60 at the mix design, MTD can be determined from the test on asphalt concrete samples. The values parameters for Eq. (3.3) determined using a multiple linear regression analysis are x = 9.5, A_x = 0.2699, B_x = 0.0087 and E_x = 0.3346 for AC 9.5; and x = 12.5, A_x = 0.4556, B_x = 0.0041 and E_x = 0.1702 for AC 12.5.

Fig. 3.4 shows the plots between the F60 predicted from Eq. (3.3) and the F60 collected for AC 9.5 and AC 12.5. High values of the R^2, low MSE (mean square



a) AC9.5



b) AC12.5

- The aggregate petrography examination, i.e., microscopic photography, XRD, and XRF, is only important to understand the in-depth skid-resistance-orientated characteristics of aggregates through their mineral composition. The higher content of silica dioxide or amorphous silica (SiO_2) in Thailand aggregates is relevant to a higher degree of deterioration, which would affect the long-term performance when used in asphalt concrete mixtures;

- The ANOVA, along with a statistical correlation analysis, was used to determine suitable input parameters for the development of the predictive model. From the statistical analyses, MTD and PSV_{diff} (the difference between polished stone values of the sample before and after polishing) were found to be the most suitable input parameters for the predictive model development, which was focused on estimating the International Friction Index (IFI) in the form of F60 (skid resistance measured at 60 km/h);

- A skid-resistance predictive model of asphalt concrete, based on mixture and aggregate characteristics [MTD and PSV_{diff}] as the model variables, was introduced with a statistically acceptable level when comparing predicted values with field data. The intention is to incorporate the model into the preventative scheme under the future strategic road safety management of the Department of Rural Roads, Thailand. In conjunction with the coefficient values suggested in this study, the skid-resistance predictive model can acceptably predict the skid resistance of asphalt concrete in the form of F60; and

- The predictive model of construction stage skid resistance is intended as the first step in the development of skid management, as part of road safety in Thailand. The skid resistance of asphalt concrete can now be predicted during the mix design process. This development contributes to the effective assessment of the safety criteria for asphalt mix design in relation to the properties of aggregates and asphalt mixtures. The model will be put forward for inclusion in the preventative scheme for road safety management by the Department of Rural Roads, Thailand.

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CHAPTER IV

EFFECT OF CUMULATIVE TRAFFIC AND STATISTICAL PREDICTIVE MODELING OF FIELD SKID RESISTANCE

4.1 Background and Introduction

Thailand has been subject to a marked increase in the number of road accidents over the last decade. In 2015, WHO reported that the country was ranked with the second highest number of road traffic deaths in the world (WHO 2015) and this statistical report has led to a demand for more effective road safety strategies. The main contributing factors for deaths from road accidents in Thailand are dangerous driving behavior and the poor physical condition of aged roads. In order to reduce the number of road accidents caused by the poor physical condition of roads, key elements of Thai road design and construction, such as road geometry and post-construction conditions, require detailed examination into their effects upon the safety of roads. Road pavements usually deteriorate under traffic loads throughout their service life, therefore, rehabilitation of road surfaces are regularly required to improve the safety of roads.

Skid resistance has been attributed as being directly related and fundamental to the safety of traffic. Skid resistance is defined as the force developed when a tire that is prevented from rotation slides along the pavement surface (Highway Research

Board 1972) and insufficient skid resistance has been attributed as being related to numerous accidents. The skid resistance value of a road furthermore changes over time and is directly related to traffic volumes. Skid resistance is a significant engineering parameter representing the road safety condition. A skid resistance value that is lower than the acceptable value indicates deleterious road conditions. The road surfaces with low skid resistance are slippery and are attributed to leading many fatal road accidents. Accordingly, skid resistance has been adopted as the prime safety index for the road pavements.

Rezaei and Masan (2013) reported that skid resistance depends upon the macro-texture and the micro-texture of a pavement surface (Kummer and Mayer 1963). The surface micro-texture refers to the relatively large irregularities of a road surface, which relates to pore sizes between aggregate particles, shape of coarse aggregates, destruction of coarse aggregates, and construction method of placing a pavement surface layer (NAPA, 1996 and Noyce et al., 2005). The surface microtexture indicates relatively small irregularities in the pavement surface, depending upon the aggregate surface texture (Jayawickrama et al. 1996, Noyce et al. 2005).

Recently, Siriphun et al. (2016) proposed a skid resistance predictive model prior to the construction stage, whose parameters are determined from conventional laboratory tests. The input parameters included microtextural and macrotextural characteristics and the equation was developed for Thailand limestone, granite and basalt aggregates. The mix design was based on Thailand’s Department of Rural Roads Standards for AC9.5 (9.5 mm maximum aggregates) and AC12.5 (12.5 mm maximum aggregates), which is consistent with the Marshall method. The input parameter representing the micro-texture characteristics was for different polishing stone values before and after polishing (PSV_{diff}), while the Mean Texture Depth (MTD) and gradation (is the shape parameter and   is the scale parameter) represented the characteristics of the macro-texture. The PSV after polishing was determined from the British Pendulum test after polishing with the specified machine for 6 hours in accordance with BS 812-114 (BSI 1989). The following equation represents the skid-resistance predictive model:

$$SRV35 = \frac{A_x MTD + B_x PSV_{diff} + E_x + 0.077}{(7.95\times10^{-3})e^{\frac{5}{S_p}}} \tag{4.1}$$

$$S_p = -11.6 + (113.6MTD) \tag{4.2}$$

where SRV35 is the skid resistance value at 35 °C. A_x = 0.2699, B_x = 0.0087 and E_x = 0.3346 for AC9.5 and A_x = 0.4556, B_x = 0.0041 and E_x = 0.1702 for AC12.5. MTD and PSV_{diff} can be determined from the test on asphalt concrete samples and aggregate samples, respectively. MTD is determined by using the sand patch test on the asphalt sample.

The polishing process of the surface aggregates due to traffic load causes the loss of both micro texture and macro texture (Forster 1989, Harald 1990, Kennedy et al.1990, Roe et al. 1991, 1998, Hosking 1992, Flintsch et al. 2005).

A significant reduction in skid resistance was found to occur during the polishing process and the reduction became gradual until an equilibrium state was reached (Saito et al. 1996, Lay and Judith 1998, Chelliah et al. 2003). The reduction characteristics of skid resistance is very important to manage the road safety strategies after construction. There has been some previous attempt to develop a skid resistance predictive model based on physical properties of aggregates used in asphalt surface mixtures (Rezaei and Masan 2009). NCAT (2006) reported that the reduction in skid resistance was controlled by cumulative traffic volumes. However, even with recent advances in the development of more reliable aggregate tests and skid resistance measurements, an accurate skid resistance predictive equation in terms of traffic volumes is currently still unavailable.

The purpose of this research was to formulate a statistical-based predictive model of skid resistances for Thailand roads under various traffic volumes using passenger car unit (pcu) parameters. The outcome of this study is in the development of skid resistance reduction values versus service time for Thailand's asphalt concrete mixtures, estimated when accumulative traffic volume and asphalt concrete mixture types are known. Road safety management in Thailand's roads would be more controllable when skid resistance values of asphalt concrete are reliably predicted in relation to traffic volumes. Furthermore, preliminary investigations from this study can be employed as a guideline for road safety management protocols.

4.2 Methodology and Experimental Work

4.2.1 Sources of Aggregate

Comprehensive sources of aggregate used in this study were quantitative and qualitative ranked as representative of the aggregates in Thailand's road network. The aggregate were obtained from six provinces in Thailand and covers three main aggregate types, which are granite, limestone and basalt. The provinces were: 1) Songkhla 2) Narathiwat 3) Chonburi 4) Sukhothai 5) UbonRatchathani and 6) Buriram. Research methodology are illustrated in **Figure 4.1**.

Based on the results of the XRD and XRF analyses, the limestone, granite and basalt aggregates can be categorized based on chemical and mineral compositions, as summarized in **Table 4.1**. Generally, calcium oxide (CaO) constitutes the key and original chemical composition of limestone. The amount of CaO found in limestone is representative of the state of rock transformation. The major chemical composition of Sukhothai limestone is CaO (99%), which means that the limestone from this source has undergone little metamorphosis. Limestone containing high CaO can resist weather deteriorations more effectively than limestone with a low CaO content. However, limestone from the Chonburi province, besides containing CaO, consisted of silica dioxide (SiO_2) or amorphous silica in its chemical composition. Increased amounts of SiO_2 in the rock can lead to enhancements in rock hardness, but show a reduction in rock durability. This hypothesis is supported by the aggregate impact value (AIV) (BS 812-112: BSI 1991a) and aggregate crushing value (ACV) (BS 812-110: BSI 1991b) from the aggregate impact test and aggregate crushing test, respectively (**Table 4.2**). The values of AIV and ACV from Chonburi limestone, which contains higher percentage of SiO_2 (**Table 4.2**), indicate this to be the stronger rock than the limestone from Sukhothai. Similar to limestone, a high SiO_2 content in granite and basalt also results in greater values of AIV and ACV. However, the SiO_2 in granite is the chemical composition of quartz, not the amorphous silica seen in limestone and basalt. The microscopic photographs of Ubon Ratchathani and Buriram show a very low degree of weathering and erosion. Nevertheless, the amorphous silica found in these rocks may be the cause of rapid degradation in the future.

Table 4.2 summarizes the results of the aggregate tests. Chonburi limestone is found to have a greater PSV (BS 812-114: BSI 1989) compared to limestone from the Sukhothai provinces. However, Chonburi limestone may degrade more quickly than other types of limestones because it possesses the highest SiO_2 content. The AIV, ACV, Los Angeles Abrasion (LAA) and Polishing Stone Value (PSV) of basalt from two different sources are almost consistent, but the values of these parameters are lower than those from the tests on limestone and granite. Based on the polished stone test results, granite has the highest PSV.

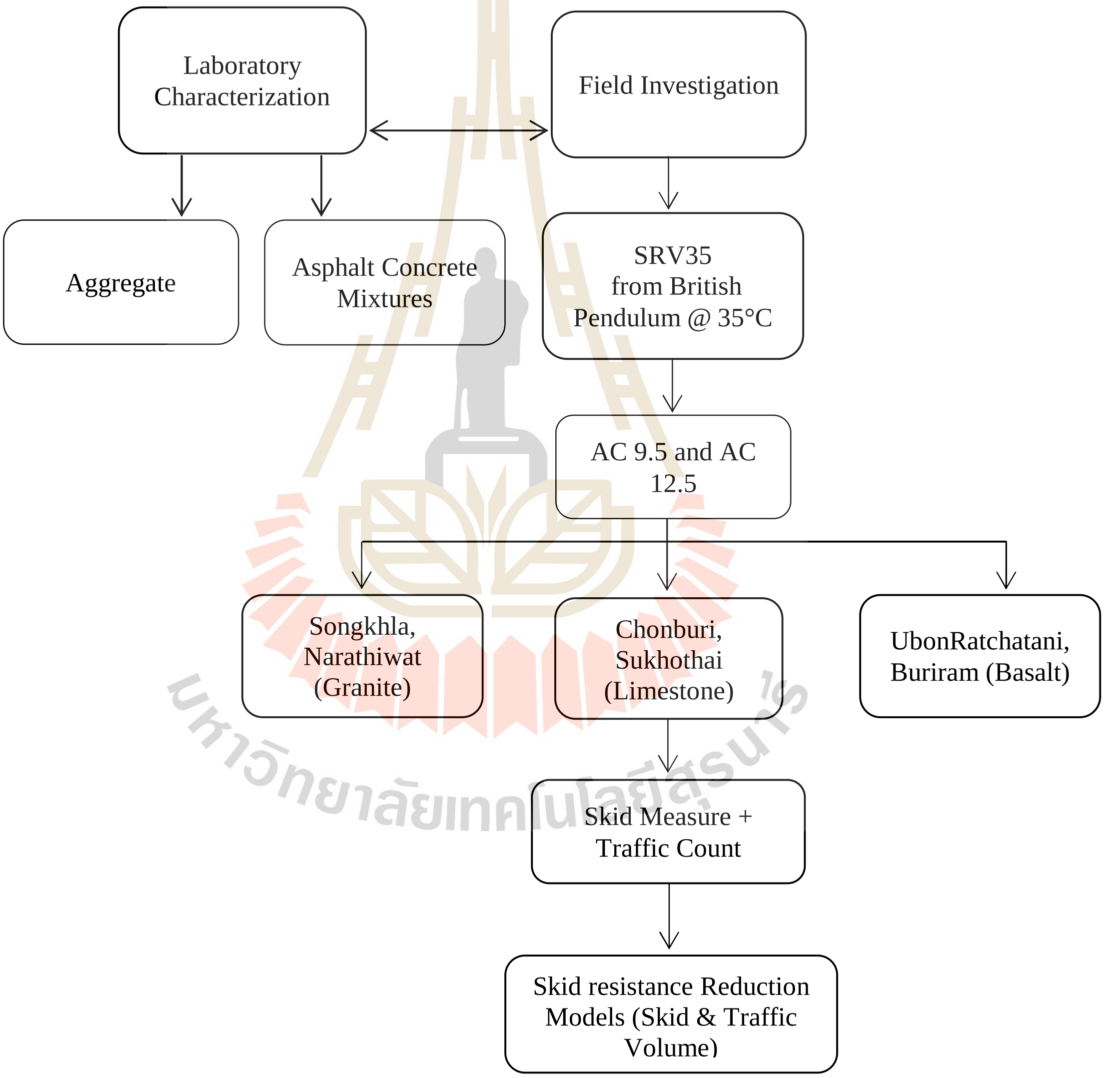


Figure 4.1 Research methodology

Table 4.1 Results of XRD and XRF examinations (Department of Rural Roads, 2014)

	Limestone			
Source	**Mineral**	**Main mineral compositions**		
		CaO (%)	**SiO_2 (%)**	**MgO (%)**
Chonburi	Dolomite ($CaMg(CO_3)_2$), Microcline ($KAISi_3O_8$), Marcasite (FeS_2), Calcite ($CaCO_3$), Cristobalite (SiO_2),Amorphous Silicate (SiO_2), Quartz (SiO_2)	11.00	56.80	2.94
Sukhothai	Calcite ($CaCO_3$)	99.00	0.00	0.22
	Granite			
Source	**Mineral**	**Main mineral composition**		
		Al_2O_3 (%)	**SiO_2 (%)**	**K_2O (%)**
Songkla	Biotite($K(Mg,Fe)_3(AlSi_3O_{10})(F,OH)_2$),Albite ($NaAlSi_3O_8$), Quartz ($SiO_2$)	14.10	74.60	5.28
Narathiwat	Microcline ($KAISi_3O_8$),Albite ($NaAlSi_3O_8$),Amorphous Silicate (SiO_2), Quartz (SiO_2), Biotite ($K(Mg,Fe)_3(AlSi_3O_{10})(F,OH)_2$), Cristobalite ($SiO_2$)	14.00	72.80	6.41
	Basalt			
Source	**Mineral**	**Main mineral composition**		
		Al_2O_3 (%)	**SiO_2 (%)**	**Fe_2O_3 (%)**
Burirum	Augite($(Ca,Na)(Mg,Fe,Al,Ti)(Si,Al)_2O_6$), Enstatite ($Mg_2Si_2O_6$), Labradorite ($(Ca,Na)(Si,Al)_4O_8$), Forsterite ($Mg_2SiO_4$),Amorphous Silicate ($SiO_2$)	16.10	49.90	10.00
UbonRatchathani	Augite($(Ca,Na)(Mg,Fe,Al,Ti)(Si,Al)_2O_6$), Enstatite ($Mg_2Si_2O_6$), Labradorite ($(Ca,Na)(Si,Al)_4O_8$), Forsterite ($Mg_2SiO_4$),Amorphous Silicate ($SiO_2$)	17.00	47.60	11.80

Table 4.2 Engineering properties of aggregate (Department of Rural Roads, 2014)

Source	Main Types of Aggregates	Aggregate Properties				
		AIV (%)	ACV (%)	LAA (%)	Soundness (% Loss)	PSV
Songkhla	Granite	20	20	21.2	3.07	44
Narathiwat	Granite	20	25	20.7	2.96	48
Chonburi	Limestone	12	18	13.7	2.11	46
Sukhothai	Limestone	20	25	23.1	1.40	44
UbonRatchathani	Basalt	13	17	15.2	1.30	35
Buriram	Basalt	13	17	14.4	1.80	38

AIV = Aggregate Impact Value, ACV = Aggregate Crushing Value

LAA = Los Angeles Abrasion, PSV = Polished Stone Value (After polishing)

4.2.2 Asphalt concrete mixture

The amount of asphalt binders, air void and aggregates of asphalt concrete samples were based on a project design mix requirements in accordance with the specifications of the Department of Rural Roads (Department of Rural Roads, 2002), which is based on the Marshall mix design procedure. The asphalt mixtures used were designed to produce wearing courses of AC9.5 (dense graded asphalt concrete with 9.5 mm aggregate maximum size) and AC12.5 (dense graded asphalt concrete with 12.5 mm aggregate maximum size).Typical thickness of AC9.5 is 25-35 mm and AC12.5 is 40-70 mm.

4.2.3 Data Collection of Field Skid Resistance Values

Data collection of field skid resistance values was conducted, focusing on the 6 provinces from where the aggregates were sourced. Road construction data from the Department of Rural Roads (DRR), Thailand were also collected. Twelve construction projects, from a total of 6 provinces, were selected to cover a specific range of aggregates (limestone, granite and basalt) and mix types (AC9.5 and AC12.5). Skid resistance value and traffic volume of each road construction project have been recorded for 3 to 4 years. Data collection of field skid resistance values at the initial and final polishing process by vehicles are given in **Table 4.3**.

Table 4.3 Data collection of field skid-resistance loss values at initial and final levels of polishing by vehicles

Source	**Type of Aggregate**	**Skid - resistance (SRV35)**			
		AC 9.5		**AC 12.5**	
		initial	**final**	**initial**	**final**
Songkhla	Granite	68.88	28.15	72.15	29.49
Narathiwat	Granite	64.48	26.36	65.76	26.88
Chonburi	Limestone	52.32	21.39	55.65	22.75
Sukhothai	Limestone	63.53	30.88	66.77	32.81
Ubon Ratchathani	Basalt	60.02	24.53	61.13	24.99
Buriram	Basalt	66.69	27.26	71.09	29.06

SRV35 = Skid-resistance Value at 35 °C

9.5 = Asphalt mixture type AC 9.5

12.5 = Asphalt mixture type AC 12.5

4.2.4 Statistical based analysis of testing results

Statistical correlations between the test results, i.e., aggregate characteristics, asphalt concrete mixture (for wearing or binder courses), skid resistance and the asphalt texture surface characteristics from the field sections were necessary in order to develop the predictive model. Along with the correlation examination, the Analysis of pair t-test was required prior to model development.

In order to develop the skid resistance reduction predictive model, the correlation between field skid resistance at different traffic volumes and the characteristics of the material, i.e., the aggregate and asphalt mixture, which were statistically investigated. In hypothesis testing, a t-test with a null hypothesis (H_0) of “two variables are *not correlated*”, is performed using 0.05 as a significance level. This null hypothesis means that all data groups are random and sampled from the same population. For example, when examining the effects of asphalt mixture type on road skid resistance, the null hypothesis would be that both types of asphalt mixtures have the same effect on skid resistance. Rejecting the null hypothesis indicates that different asphalt mixture types result in altered effects.

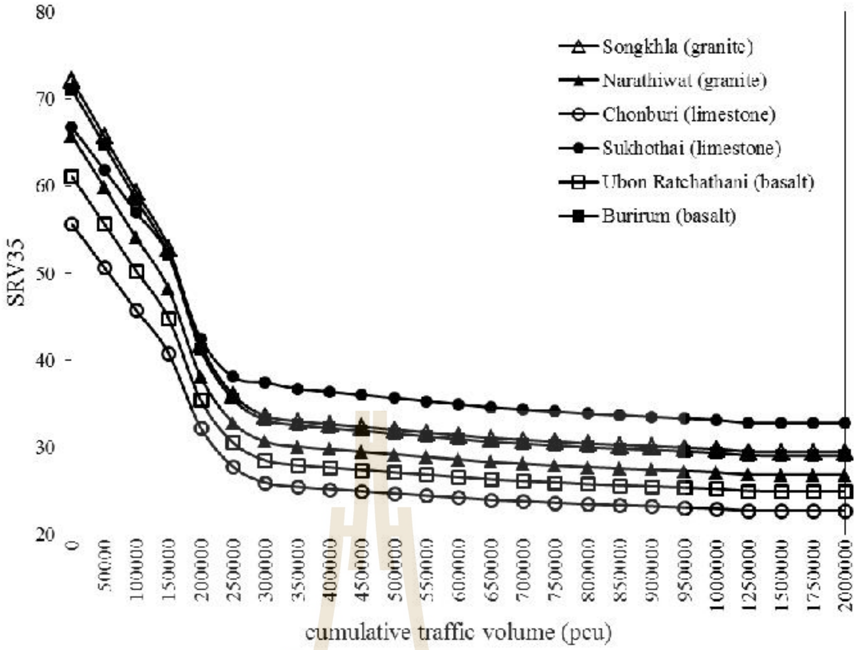
The correlation analysis by regression was also implemented in this study. The value of R^2 (R-square) indicates the degree of correlation between two parameters. According to statistical theory, R^2 varies from 0 to 1. When the value of R^2 is close to 1, the dependent variable is strongly reliant upon the independent variable, and vice versa. Likewise, the coefficient of correlation (R-value) was used in this research to allow analysis of the degree of correlation between two parameters. The R-value varies from -1 to 1, and a 0 (zero) R-value indicates no correlation between the two variables. In the correlation analysis, confidence intervals are stated at the 95% confidence level.

4.3 Result and Discussion

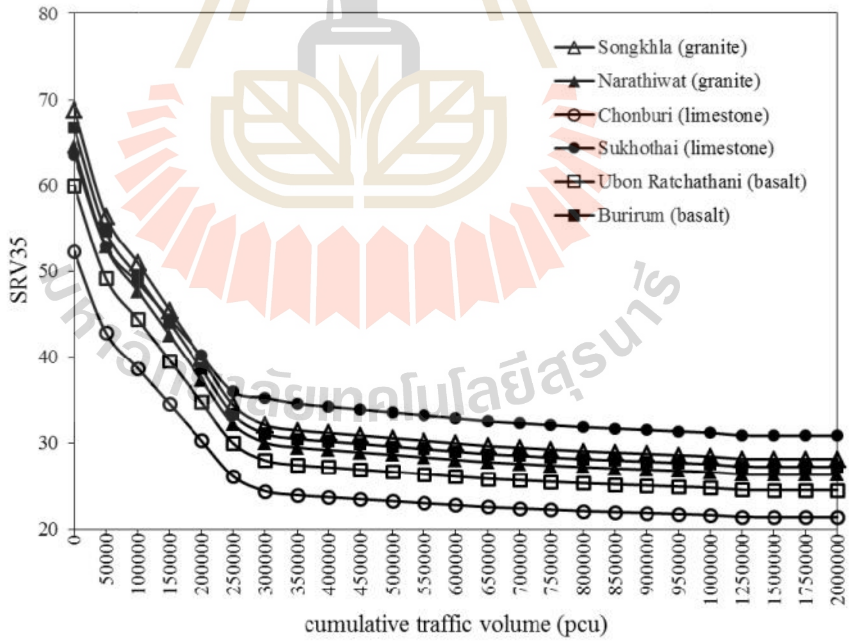
4.3.1 Characteristics of skid resistance reduction

Skid resistance is usually considered as a frictional force, consisting of two components; the adhesion and hysteresis components. The adhesion component is the adhesion force between vehicle tyres and road surface materials. The adhesion friction force directly relates to the micro-texture of asphalt concrete aggregate. The hysteresis component originates from the shape reforming process of tyre treads while running through the road surface. Hysteretic friction force directly relates to the macro-texture of the road surface. The skid resistance of road surface is strongly influenced by the traffic volume and service time.

Figures 4.2a and **4.2b** show the relationship between skid resistance values at 35 °C (SRV35) and cumulative traffic volume for AC12.5 and AC9.5, respectively with different aggregate types and gradations. For a particular asphalt type and aggregate type, the SRV35 decreases significantly as the cumulative traffic volume increases. However, the decrease in SVR35 is minimal when cumulative traffic volume is greater than 250,000 pcu for both asphalt types with the three aggregate types. The SRV35 reduction is caused by the disintegrations in micro-texture due to the polishing action of traffic, which is accelerated by the presence of water and dust particles. For the same aggregate type, the AC12.5 has higher SRV35 for all cumulative traffic volumes, indicating that aggregate size affects the SRV35. The AC12.5 with granite as an aggregate has the highest SRV35.



a) AC12.5



b) AC9.5

Fig. 4.2 Skid resistance versus cumulative traffic volume

for (a) AC12.5 and (b) AC9.5

4.3.2 Statistical analysis of the effect of asphalt mixture types (AC9.5 and AC12.5), aggregate sources and types, CTV and MTD on the SRV35

Pair sample t-test was conducted to examine effect of asphalt concrete type (AC9.5 and AC12.5), aggregate sources and types (limestone from Sukhothai and Chonburi, granite from Songkhla, Narathiwat and basalt from Ubon Ratchatani, Bururum) on SRV35 over cumulative traffic volume (CTV) at 0, 50000, 100000, 150000, 200000, 250000, 500000, 1000000, 1500000 and 2000000. Results from t-test analysis of the effect of asphalt mixture types show the rejection of the null hypothesis at every cumulative traffic volume (P-value ranges from 0.000-0.033 as indicated in **Table 4.4**). Whereas the results from the pair t-test show that P-values for all aggregate sources and types are higher than 0.05. In other words, the asphalt concrete type (AC 9.5 and AC12.5) plays a significant role on the SRV35 while the aggregate source and type play an insignificant role. This result is in agreement with the statistical analysis of initial SRV (SRV_0) at CTV = 0 reported by Siriphun et al. (2016).

Table 4.5 shows the regression analysis of the effect of PSV_{diff} and MTD on the SRV35 at CTV = 0, 50000, 100000, 150000, 200000, 250000, 500000, 1000000, 1500000 and 2000000. It is evident that P-values are lower than 0.05, indicating that PSV_{diff} and MTD affect the SRV35 at every CTV. From **Tables 4.4** and **4.5**,

$$SRV_{35} = f(PSV_{diff}, MTD, AC\ type) \tag{4.3}$$

Siriphun et al. (2016) revealed that PSV_{diff} and MTD can be reflected by initial

$SRV35_0$ (after the end of construction); hence:

$$SRV_{35} = f(SRV35_0, AC\ type) \tag{4.4}$$

Consequently, it is logical to develop a normalized function for a particular AC (either AC9.5 or AC12.5):

$$\frac{SRV_{35}}{SRV_0} \text{at a particular CTV = constant} \tag{4.5}$$

To prove this premise, a normalized SRV35, which is a SRV35 at a particular traffic volume over the corresponding $SRV35_0$ (at CTV = 0) was examined by statistical analysis (**Tables 4.6** and **4.7**). The pair sample t-test was conducted to examine the role of AC type on the normalized SRV and the results is presented in **Table 4.6** for 0 pcu < CTV < 300000 pcu and 300000 pcu < CTV < 2000000 pcu. The P-values in **Table 4.6** are lower than 0.005 and greater than 0.05 for 0 pcu < CTV < 300000 pcu and 300000 pcu < CTV < 2000000 pcu, respectively. In other words, the AC type (AC9.5 and AC12.5) plays a significant role on the SRV when CTV is less than 300000 pcu.

Table 4.7 shows the regression analysis on the effect of PSV_{diff} and MTD on the normalized SRV35 at the studied cumulative traffic volumes (0, 50000, 100000, 150000, 200000, 250000, 500000, 1000000, 1500000 and 2000000). It is found that P-values are higher than 0.05. **Table 4.6** and **4.7** confirm the premise as indicated by Eq. (4.5).

Table 4.4 Analysis results of field skid-resistance reduction from pair t-test

Tested Variable @ any CTV	Independent Variable	P-Value*
SRV35	AC9.5 and AC12.5	0.0331-0.0000
SRV35	Songkhla (granite) and Narathiwat (granite)	0.2000-0.1650
SRV35	Chonburi (limestone) and Sukhothai (limestone)	0.2150-0.1720
SRV35	UbonRatchathani (basalt) and Buriram (basalt)	0.2740-0.1940

* The range of P-Value is reported according to the CTV ranging from 0 to 20000000 (0, 50000, 100000, 150000, 200000, 250000, 500000, 1000000, 1500000 and 2000000)

Table 4.5 Regression Analysis results of field skid-resistance reduction

Dependent Variable @ any CTV	Independent Variable	R^2	R-value	P-Value
SRV35	PSVdiff	0.341-0.595	0.584-0.771	0.0459-0.0032
SRV35	MTD	0.690-0.924	0.831-0.961	0.0008-0.0000

CTV=Cumulative Traffic Volume at 0, 50000, 100000, 150000, 200000, 250000, 500000, 1000000, 1500000 and 2000000

Table 4.6 Analysis results of normalized SRV35 from pair t-test

Tested Variable @ any CTV	Independent Variable	P-Value
SRV35/SRV0	AC9.5 and AC12.5	0.0421-0.0000*
SRV35/SRV0	AC9.5 and AC12.5	0.2482-0.1830**

* The range of P-Values for 0 pcu < CTV < 300000 pcu

** The range of P-Values for 300000 < CTV < 2000000

Table 4.7 Regression Analysis results of Normalized SRV

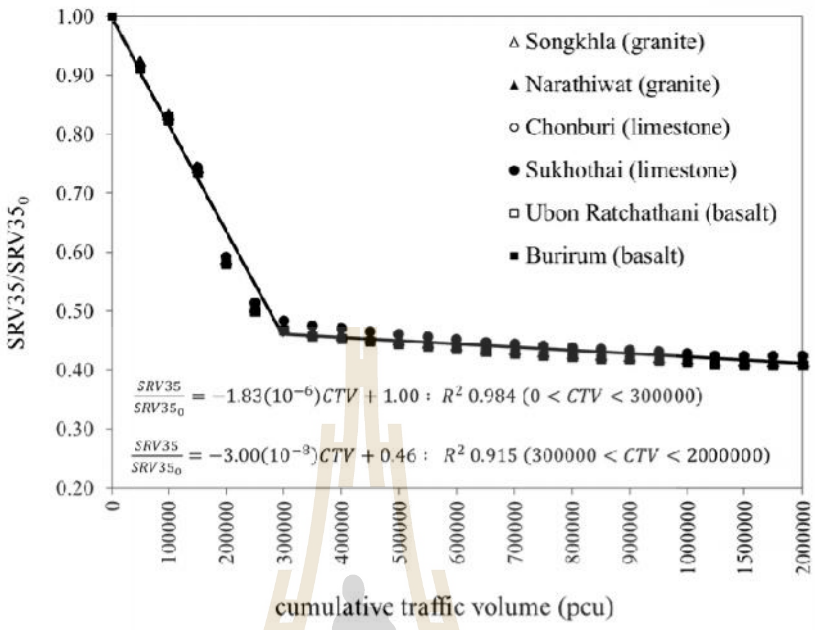
Dependent Variable @ any CTV	Independent Variable	R^2	R-value	P-Value
SRV35/SRV0	PSVdiff	0.000-0.000	0.001-0.003	0.9984-0.9924
SRV35/SRV0	MTD	0.098-0.132	0.313-0.363	0.0745-0.0687

CTV=Cumulative Traffic Volume at 0, 50000, 100000, 150000, 200000, 250000, 500000, 1000000, 1500000 and 2000000

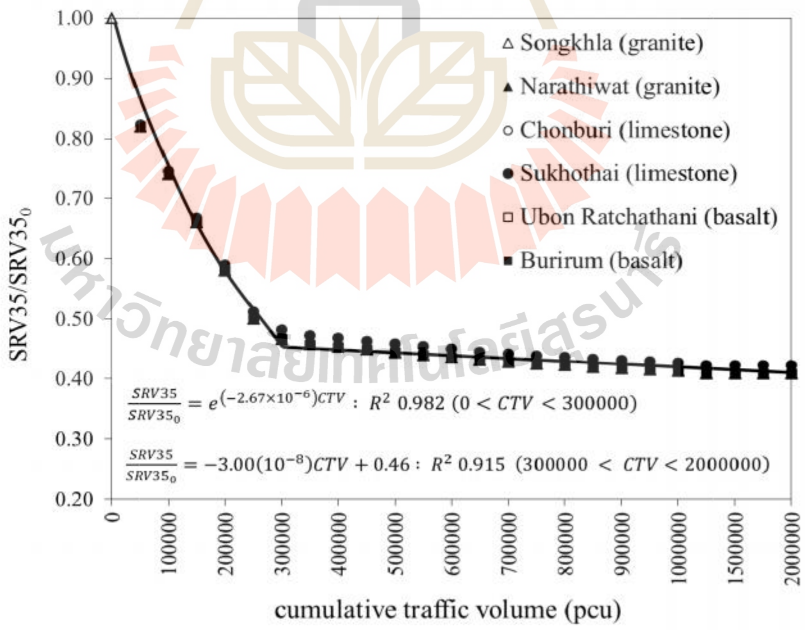
4.4 Development of Skid Resistance Predictive Model

Based on the statistical analysis, it is evident that SRV35 reduction over cumulative traffic volume (CTV) is dependent upon AC type (AC9.5 and AC12.5) and SRV_0. It is thus logical to develop the SRV35 predictive model at various CTV values in terms of initial SRV35 ($SRV35_0$) for a particular AC type as shown in

Figures 4.3(a) and **(b)**.



a) AC12.5



b) AC9.5

Fig. 4.3 Normalized Skid resistance versus cumulative traffic volume

for (a) AC12.5 and (b) AC9.5

The SRV35 predictive equation is expressed by the following equations:

For CTV < 300000 pcu

$$\frac{SRV_{35}}{SRV_0} = 1.00 - 1.83\times10^{-6}CTV \qquad \text{for} \qquad \text{AC12.5} \tag{4.6}$$

$$\frac{SRV_{35}}{SRV_0} = \exp\left(-2.67\times10^{-6}CTV\right) \qquad \text{for} \qquad \text{AC9.5} \tag{4.7}$$

For 300000 pcu < CTV < 2000000 pcu

$$\frac{SRV_{35}}{SRV_0} = 0.46 - 3.00\times10^{-8}CTV \qquad \text{for} \qquad \text{AC12.5 and AC9.5} \tag{4.8}$$

with very high R^2 of greater than 0.915.

Eqs. (4.6) to (4.8) are very useful in practice for determination of SRV35 at various CTV once only the $SRV35_0$ is known. The $SRV35_0$ can be directly measured by British pendulum tester or simply approximated by Eqs. (4.1) to (4.3) using MTD, aggregate gradation and PCV_{diff} as input parameters.

The equation can be extended to estimate the service life of the asphalt concrete pavement in terms of skid resistance criterion. To approximate the service life, Eqs. (4.6) to (4.8) must be used together with the growth rate equation to forecast the future CTV. The annual growth rate (*AGR*) typically used in Thailand is presented as follows:

$$AGR = \left(\frac{T_2}{T_1}\right)^{\frac{1}{(Y_2-Y_1)}} - 1 \tag{4.9}$$

where T_2 is traffic volume (pcu/day) in year Y_2 and T_1 is traffic volume (pcu/day) in year Y_1.

The allowable skid resistance specified by the Department of Rural Roads is 35. Taking SRV35 of 35 in Eqs. (4.6) to (4.8), the CTV at the end of service life (CTV_{end}) can therefore be calculated for a particular asphalt concrete type (either AC12.5 or 9.5). Based on the measured CTV at any time after the construction (CTV_t) and AGR, the end of service life (n) can be approximated from the following statistical equation:

$$CTV_{end} = CTV_t(1 + AGR)^n \tag{4.10}$$

4.5 Conclusions

This study demonstrated the development of the skid resistance reduction predictive model at various cumulative traffic volumes. The outcome of this study indicates that reliable prediction of field skid resistance reduction, could be implemented into Thailand's road safety preventive scheme. The following conclusions can be made:

- The statistical correlation analysis was employed to determine the parameters influencing the skid resistance reduction over cumulative traffic volume for two different types of asphalt mixtures (AC9.5 and AC12.5). It is evident that the influence parameters are MTD, and PSV_{diff}, which can be taken into account by the initial skid resistance after the end of construction ($SRV35_0$).
- Using the skid resistance together with the growth rate of traffic volume, the end of service life can be approximated based on the field measured data, which is very useful in practice. The model will be put forward for inclusion

in the preventive scheme for road safety management by the Department of Rural Roads, Thailand.

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CHAPTER V

CONCLUSIONS

5.1 Summary and conclusions

In Thailand, the skid resistance in road pavements requires considerable improvement in order to increase road network safety. Skid resistance value (SRV) can only be measured in situ, that is on the road itself, and prior to post-construction stages. So, SRVs have not been accounted during the aggregate selection and mixing processes and at various cumulative traffic volumes. It is well known that road pavements usually deteriorate under cumulative traffic volumes throughout their service life or the skid resistance values (SRV) decreases continuously over the lifetime of road pavements. The field measurement of skid resistance is time-consuming and costly. This study aimed to develop skid-resistance predictive models at the construction stage and at various cumulative traffic volumes based on the essential aggregate and mixture characteristics.

For developing skid resistance predictive model at the construction stage, three main types of Thai aggregates (limestone, granite and basalt) were mixed in asphalt concrete to make the pavement for the construction sites, which were sourced and collected to make test asphalt concrete. These aggregates were obtained from Thailand's main regions and covered 14 provinces as (1) Northern region: Chiangmai, Sukhothai, and Tak, (2) Central region: Nakhonsawan, Sara Buri, and Rachaburi, (3) Eastern region: Chonburi, (4) Northeastern region: Nakhon Ratchasima, Ubon Ratchathani, and Buriram, and (5) Southern region: Trang, Songkhla, Surat Thani, and Narathiwat. Aggregates and their standard densely-graded asphalt concrete mixtures of 9.5 mm and 12.5 mm maximum aggregate sizes were used to perform in the construction site. The SRV was measured by the British pendulum tester. In addition, the textural characteristics of asphalt concrete pavement, based on different aggregate mixtures, were also analyzed with respect to a sand patch method. A statistical correlation analysis was used to determine suitable input parameters for the development of the predictive model. From the statistical analyses, MTD and PSVdiff were found to be the most suitable input parameters for the predictive model development. The results of the study demonstrated that the developed predictive model in terms of aggregates and mixture characteristics (polished stone value, gradation, and mixture surface texture) provided acceptable SRV prediction with high statistic levels ($R^2 > 0.78$, MSE<0.00089 and F-significance < 0.05). The model will be put forward for inclusion in the preventive scheme for road safety management.

For developing a statistical-based predictive model of skid resistance under various traffic volumes, the SRV reduction model has been developed based on the essential aggregate (PSVdiff, MTD) and asphalt concrete mixture characteristics (AC9.5, AC12.5) and field traffic volumes (CTV: Cumulative traffic volume). In this study, three main types of aggregates typically used in pavements in Thailand, being limestone, granite and basalt were used to make asphalt concrete. These aggregates were obtained from 12 project sites located in 6 provinces for SRV test as 1) Songkhla 2) Narathiwat 3) Chonburi 4) Sukhothai 5) UbonRatchathani and 6) Buriram. Aggregates and their standard dense-grade asphalt concrete mixtures of 9.5 mm and 12.5 mm maximum aggregate sizes were used at the construction sites. The SRVs were measured at every 50,000 passenger car unit (pcu) by the British pendulum tester for 3 to 4 years. The statistical correlation analysis by t-test and regression were employed to determine the parameters influencing the skid resistance reduction over cumulative traffic volume for two different types of asphalt mixtures (AC9.5 and AC12.5). It is evident that the influence parameters are MTD, and PSV_{diff} and can be taken into account by the initial skid resistance after the end of construction ($SRV35_0$). Using the skid resistance together with the growth rate of traffic volume, the end of service life can be approximated based on the field measured data, which is very useful in practice. The results of the study demonstrated that the model developed can be used successfully to predict the reduction of SRV at field sites. The model will be recommended for inclusion in the Department of Rural Roads, Thailand preventive scheme for road safety management protocols.

5.2 Skid resistance predictive model at the construction stage

The field and statistical analysis results were used for the development of the predictive model, which focuses on estimating the field F60 at the construction stage.

Following the selection of the input parameters for the predictive model, the multiple linear regression analysis technique was used to develop the predictive model for field F60. The following equation represents the skid-resistance predictive model:

$$F60 = A_x MTD_x + B_x PSV_{diff} + E_x \tag{5.1}$$

where x represents the type of asphalt, AC9.5 or AC12.5. The values parameters determined using a multiple linear regression analysis are x = 9.5, A_x = 0.2699, B_x = 0.0087 and E_x = 0.3346 for AC9.5 and x = 12.5, A_x = 0.4556, B_x = 0.0041 and E_x =

0.1702 for AC12.5.

5.3 Effect of cumulative traffic and statistical predictive modeling of field skid resistance

The field and statistical analysis results were used for the development of the predictive model, which focused on estimating the field SRV35 at deteriorated by cumulative traffic volume stages. Based on the statistical analysis, it is evident that SRV35 reduction over cumulative traffic volume (CTV) is dependent upon AC type (AC9.5 and AC12.5) and SRV_0. It is thus logical to develop the SRV35 predictive model at various CTV values in terms of initial SRV35 ($SRV35_0$) for a particular AC type. The SRV35 predictive equation is expressed by the following equations:

For CTV < 300000 pcu

$$\frac{SRV_{35}}{SRV_0} = 1.00 - 1.83 \times 10^{-6} CTV \quad \text{for} \quad \text{AC12.5} \qquad (5.2)$$

$$\frac{SRV_{35}}{SRV_0} = \exp\left(-2.67 \times 10^{-6} CTV\right) \quad \text{for} \quad \text{AC9.5} \qquad (5.3)$$

For 300000 pcu < CTV < 2000000 pcu

$$\frac{SRV_{35}}{SRV_0} = 0.46 - 3.00 \times 10^{-8} CTV \quad \text{for} \quad \text{AC12.5 and AC9.5} \qquad (5.4)$$

with very high R^2 of greater than 0.915.

Eqs. (5.2) to (5.4) are very useful in practice for determination of SRV35 at various CTV once only the $SRV35_0$ is known. The $SRV35_0$ can be directly measured by British pendulum tester or simply approximated by using MTD, aggregate gradation and PSV_{diff} as input parameters.

The equation can be extended to estimate the service life of the asphalt concrete pavement in terms of skid resistance criterion. To approximate the service life, Eqs. (5.2) to (5.4) must be used together with the growth rate equation to forecast the future CTV. The annual growth rate (*AGR*) typically used in Thailand is presented as follows:

$$AGR = \left(\frac{T_2}{T_1}\right)^{\frac{1}{(Y_2 - Y_1)}} - 1 \tag{5.5}$$

where T_2 is traffic volume (pcu/day) in year Y_2 and T_1 is traffic volume (pcu/day) in year Y_1.

The allowable skid resistance specified by the Department of Rural Roads is 0.35. Taking SRV35 of 0.35 in Eqs. (5.2) to (5.4), the CTV at the end of service life (CTV_{end}) can therefore be calculated for a particular asphalt concrete type (either AC12.5 or 9.5). Based on the measured CTV at any time after the construction (CTV_t) and AGR, the end of service life (*n*) can be approximated from the following statistical equation:

$$CTV_{end} = CTV_t(1 + AGR)^n \tag{5.6}$$

5.4 Recommendation for further study

This study was based on three types of paving aggregates: basalt, granite, and limestone and one type of pavement surface (dense-graded asphalt). Statistical skid resistance predictive models are required for PMA (Polymer modified asphalt), para asphalt, porous asphalt, SMA (Stone mastic asphalt) and para slurry seal are recommended for further study.

BIOGRAPHY

I, Mr. Sitthichai Siriphun was born in October 1977 in Songkhla, Thailand. I received my Bachelor's degree in Civil Engineering from School of Civil Engineering, Suranaree University of Technology in 1999 and obtained my Master's degree in Civil Engineering (Transportation) from Department of Civil Engineering, Prince of Songkla University in 2002. After graduation, I have been working as a civil engineer in the Department of Rural Roads, Ministry of Transport. I have been granted a scholarship by Department of Rural Roads in 2014 to pursing my Ph.D. study in Construction and Infrastructure Management, School of Civil Engineering, Suranaree University of Technology. During my Ph.D. study, I have published 1 paper in international journals (ASCE).