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DEVELOPMENT OF A FRICTION PREDICTION MODEL OF ASPHALT CONCRETE AT THE CONSTRUCTION STAGE BASED ON SRI LANKAN AGGREGATES

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Thesis/Dissertation submitted in partial fulfillment of the requirements for the degree Master of Science (Major Component of Research)

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DECLARATION

I declare that this is my own work and this thesis/dissertation does not incorporate without acknowledgement any material previously submitted for a degree or diploma in any other University or Institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text. I retain the right to use this content in whole or part in future works (such as articles or books).

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The above candidate has carried out research for the Master's thesis/dissertation under my supervision. I confirm that the declaration made above by the student is true and correct.

Name of Supervisor: Prof. H.R. Pasindu

Signature of the Supervisor: Date: 19/05/2026

DEDICATION

I dedicate my work to the Transportation Engineering Division of the Department of Civil Engineering of Faculty of Engineering of University of Moratuwa, Sri Lanka

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First and foremost, I would like to express my sincere gratitude to my research supervisor Prof. H.R. Pasindu for the assistance and dedicated involvement in every step throughout the progression of the research. I would like to thank him for his support and guidance throughout the process of the completion of the research.

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ABSTRACT

The skid resistance at the tire-pavement interface ensures adequate braking force for road user safety. Insufficient friction at the wearing course emerges as the primary pavement-related factor of highway crashes. Governing factors of pavement friction include pavement properties, vehicle/tire/driver conditions, and environmental conditions, with pavement characteristics, particularly the mineralogical and textural attributes of surfacing coarse aggregates exerting dominant control. Coarse aggregate polishing resistance critically determines the available skid resistance of an asphalt concrete road, prompting extensive research on aggregate's friction retention capacity under traffic loading. Hence, globally a number of pivotal studies have been conducted to explore polishing behaviour of roadstones and to develop prediction models to forecast skid resistance capacity of flexible pavements. In Sri Lanka, road safety garners increased emphasis amid expressway network integration, yet studies on local aggregate friction properties remain limited, with fewer user-safety guidelines incorporated in the national Standard Specifications for Construction and Maintenance of Roads and Bridges (SSCM-RB). This study thus evaluates friction characteristics of indigenous Sri Lankan road surfacing aggregates via the Polished Stone Value (PSV) test (employing Accelerated Polishing and British Pendulum) alongside petrographic and microscopic analysis of polishing behavior in order to develop a friction prediction model for national asphalt concrete pavements.

Keywords: Coarse aggregates, pavement friction, pavement texture, polishing resistance, skid number, friction prediction model

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LIST OF ABBREVIATIONS

Abbreviation	Description
AAPM	Aachen Polishing Machine
AIMS	Aggregate Imaging System
AIV	Aggregate Impact Value
APM	Accelerated Paving Machine
ASTM	American Society for Testing and Materials
BPN	British Pendulum Number
BPT	British Pendulum Tester
CIDA	Construction Industry Development Authority
CTM	Circular Texture Meter
CVD	Commercial Vehicles per lane per Day
DFT	Dynamic Friction Tester
DoT	Department of Transportation
DTA	Differential Thermal Analysis
EDX	Energy Dispersive X-ray
FHWA	Federal Highway Administration
FI	Flakiness Index
FN	Friction Number
HC	Highland Complex
HMA	Hot Mix Asphalt
ICTAD	Institute of Construction Training and Development
IFI	International Friction Index
IML	Industrial Mining License
KC	Kadugannawa Complex
LAAV	Los Angeles Abrasion Value
MD	Micro-Deval
MPD	Mean Profile Depth
MSSC	Mean Summer SCRIM Coefficient
MTD	Mean Texture Depth

OD	Oven-Dried
OMT	Outflow Meter Test
PFM	Pavement Friction Management
PIARC	Permanent International Association of Road Congresses
PMS	Pavement Management Systems
PSV	Polished Stone Value
RDA	Road Development Authority
RMS	Root Mean Square
RPV	Residual Polish Value
RTM	Road Test Machine
SCRIM	Sideway Force Coefficient Routine Investigation Machine
SEM	Scanning Electron Microscope
SN	Skid Number
SPM	Sand Patch Method
SSCM-RB	Standard Specifications for Construction and Maintenance of Roads and Bridges
SSD	Saturated Surface Dry
SSS	Sodium Sulphate Soundness
TRRL	Transport and Road Research Laboratory
VC	Vijayan Complex
WC	Wanni Complex
WFT	Water Film Thickness
W/S	Wehner/Schulze
XRD	X-ray Diffraction

CHAPTER 1

INTRODUCTION

Pavement friction refers to the resistance force at the tire-pavement contact point that opposes the sliding motion of a rotating vehicle tire. A reliable friction coefficient on the road surface delivers enough braking power to stop vehicles from skidding during turns or stops or any other maneuvers. Low friction levels in the top wearing layer stand out as a leading cause of traffic accidents, alongside issues like vehicle defects, driver errors, other defective road elements, and adverse weather influences. Such friction-related road characteristics account for roughly 28% of all road accidents and 14% of fatal crashes [1]-[3]. Maintaining proper road friction levels thus proves essential to avoid lane departures, skids, run-off-road incidents, and head-on crashes for safer travel.

The friction coefficient at the tire-road contact depends on factors including pavement conditions, tire/vehicle states, and water layer thickness, with wearing course-related friction attributes playing a major role. Coarse aggregates make up around 90% of Hot Mix Asphalt (HMA) surface layers by mass, hence key friction related pavement properties involve the mineral makeup and surface texture of the road aggregates. Over time, traffic wear, harsh weather, and local site factors polish the exposed coarse aggregates of the wearing course, which drops pavement friction raising crash risks, particularly on wet roads. As a result, coarse aggregate polishing resistance strongly shapes the Skid Resistance (SR) capacity of asphalt concrete surfaces.

Pavement friction blends two main forces: adhesion from the micro-texture of single aggregate grains and hysteresis from the pavement's macrotexture [1, 4]. Hysteresis dominates at high speeds and in wet conditions, yet adhesion supports tire grip and SR across all speeds on both dry and wet surfaces. The fine surface bumps and texture of each aggregate particle control its polishing response and wear patterns under traffic loads. Ultimately, road aggregate polishing resistance drives friction loss patterns and sustained SR in asphalt concrete roads. For this reason, during aggregate selection process, apart from standard physical and mechanical properties, polishing behaviour of road surfacing aggregates must be assessed to secure lasting friction on pavements.

Various lab tests evaluate coarse aggregate polishing, but the PSV test ranks as the top choice in numerous countries [5,6]. This PSV test gauges roadstone resistance to tire polishing under lab-simulated road conditions via sped-up wear. Higher PSV means better skid resistance for the aggregate. Nations like the U.S.A., U.K., and China set PSV standards and rules, making it central to selecting aggregates for asphalt pavement anti-skid layers [7,8,9]. To integrate user safety into pavement design and build phases as a pivotal pavement performance criterion apart from mobility and user comfort, countries including Afghanistan, India, Bangladesh, Pakistan, Thailand, Iran, Iraq, and others have tested frictional behaviour of local road aggregates [10,11,12].

However, in Sri Lanka, detailed studies on local aggregate polishing remain rare, and even the newest Standard Specifications for Construction and Maintenance of Roads and Bridges (SSCM-RB) lack matching guidelines [13]. Sri Lanka's 12,380 km national road system covers class “A” trunk roads, class “B” main roads, and class “E” expressways, with top speeds ranging from 70 km/h to 100 km/h. Yearly crashes average 38,000, causing about 3,000 deaths and 8,000 severe injuries, including 600 on expressways [14,15]. Strong pavement friction matters greatly since fast driving combined with poor braking sparks many highway wrecks. Counter steps thus should run from selection of aggregates through construction and routine upkeep, targeting trunk roads, expressways with ramps, sharp curves, steep slopes, crossings, and known skid-prone spots. Nationally, focus grows on road safety as a core measure amid rising speed-related crashes. Calls rise to involve skid resistance into aggregate selection criteria for pavement construction and to plan Pavement Friction Management (PFM) for risky skid zones.

On a global scale, studies on polishing performance of gneissic coarse aggregates are very low compared to rock types like basalt, granites and limestones. Majority of locally available coarse aggregates used in national highway industry, are of gneiss rocks with metamorphic origins, thus with no or fewer access to already established literature and data as a reference or guide [16,17,18]. Therefore, the above research void in the existing research studies emphasises the requirement to assess Sri Lankan road surfacing aggregates furthermore.

On the other hand, Skid Number (SN) is greatly included in the Pavement Management Systems (PMS) under PFM category in many of the developed countries as well as South Asian countries, since SN provides an estimate of the friction capacity of a pavement in terms of aggregate quality and the degree of quality conformity maintained during the construction process [1,19]. Therefore, many road authorities collect and record input parameters of friction and macro-texture of the pavement periodically using different measurement devices, so that existing SN can be calculated. In addition, with the use of collected data over a time period, historical patterns of SN variation over time and traffic application can be used as reference and for forecasting as well, enabling better pavement management. The collection of these network friction data is considered to be comparatively an intensive process demanding higher operational costs and time, leading to development of various prediction models of SN in terms of less demanding input parameters [1,19].

Nonetheless, in the national context despite the inclusion and operation of a PMS for the expressway system, continuous pavement friction related field data collection is questionable and for a country like Sri Lanka, such periodical network level data mining can be challenging. Thus, a prediction model to estimate Skid resistance capacity of the pavement reasonably would be invaluable to forecast investigatory and intervention levels in the PMS.

Hence, the above-mentioned gaps in the national focus on the coarse aggregate properties influential for provision of road user safety and friction management of pavements with higher operational speed, propose and enhance the need for exploration of the polishing characteristics of locally available aggregates and further related research study approaches.

1.1 Research Objectives

Having identified the research gap, which is the scarcity of experimental studies into the polishing behaviour of locally available road surfacing aggregates and gneissic coarse aggregates, leading to lack of introduction such into the national policy framework for highway pavement construction and maintenance, the following research objectives have been formulated.

1. Establish polishing deterioration patterns of coarse aggregates used for pavement construction in Sri Lanka
2. Establish micro-textural and macro-textural deterioration patterns of coarse aggregates used for pavement construction in Sri Lanka, in order to better understand the Objective No.01
3. Establish mineralogical and petrographic characteristics of coarse aggregates used for pavement construction in Sri Lanka, to support findings under the Objective No.01 and 02
4. Establish a friction prediction model for asphalt concrete pavement constructed with mineral coarse aggregates in Sri Lanka, to forecast SR of the pavement in terms of mechanical properties of the coarse aggregates (polishing resistance)

1.2 Significance and Contribution

This research intends to address the absence of research/experimental/monitoring studies on polishing characteristics of coarse aggregates and pavement friction in Sri Lanka, through a friction prediction model to simulate polishing of asphalt concrete pavement under traffic. SR values can be predicted at the stage of aggregate selection and asphalt concrete mixture design processes, therefore the study is initiative to incorporate SR in the national practice of aggregate selection for pavement construction, and to enable advanced planning of maintenance interventions in the PFM of skid hazardous road sections. In addition, road safety management would be more controllable when SR values of asphalt concrete are reliably known before construction and furthermore, the prediction of pavement SR is necessary in the road maintenance and budget planning of specially skid hazardous highway sections such as national expressway system and major trunk roads [1].

The study is initiative to establish the polishing behaviour of locally available road stones, encouraging further experimental studies and incorporation of SR of aggregates and user safety in the national practice of design, construction and maintenance of major trunk road network. The developed database including the

polishing decay curves and the calculated PSVs can be further utilised in the future studies of gneiss based coarse aggregates not only in Sri Lanka, but also in other countries with similar metamorphic grade rock origins as well. The derived prediction model can be universally applied on flexible pavements with gneissic coarse aggregates at the wearing course, in the PFM frameworks under the PMS of any highway agency, to estimate their future SN at critical stages during the pavement lifecycle.

Finally, the recommended measures to be incorporated into the national pavement construction industry, from the selection of coarse aggregates up to the decision making of a road section during operational stages, would drive further future discussions and studies towards the crucial and much needed topic of ensuring road user safety from the very beginning of highway planning and design.

1.3 Research Scope and Limitations of the Study

The study focuses on establishing a friction prediction model of Sri Lankan road stones based on their common physical, mechanical properties in order to predict the skid resistance of pavement at the post construction stage under laboratory conditions. Therefore, establishment of a friction deterioration trend and a prediction model in relation to cumulative traffic volume is a subsequent approach in the study of pavement friction, which is not addressed here. Although polishing resistance of coarse aggregates can be assessed on a controlled laboratory scale, the friction deterioration patterns for a particular mineral aggregate type should be further evaluated subjected to actual traffic application on a pavement surface under different environmental/ambient conditions [1,4].

CHAPTER 2

LITERATURE REVIEW

2.1 Pavement Friction

The performance of a highway pavement during its life cycle means its capability to provide a safe, smooth, strong and durable surface for traffic application over the end of service life, which is evaluated under two main categories, structural performance and functional performance respectively. When it comes to the latter category, serviceability or the ride quality catering to the rider comfort and safety, greatly depends on condition evaluation parameters like road roughness, pavement distresses and pavement friction. Hence, pavement friction is of utmost importance as sufficient pavement friction leads to a safer riding ensuring minimum occurrence of highway crashes and their severities.

In definition, pavement friction is the preventive force against the relative motion of the vehicle between the vehicle tire rubber and the surface of the wearing course of a pavement, as depicted in the Fig. 2.1. Mathematically, pavement friction can be characterised by the dimensionless parameter called pavement friction coefficient (μ), which is the ratio between the tangential force developed at the tire-pavement contact path (F) and the vertically produced resistive force perpendicularly applied on the vehicle tire by the pavement surface (R), as shown in equation (1) [1,4].

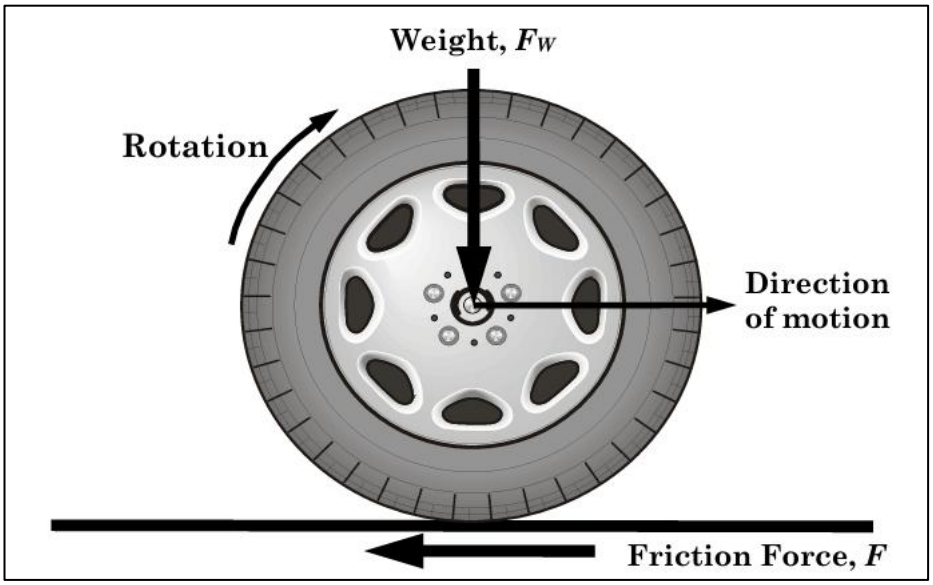


Fig. 2.1: The forces acting on a rolling tire (adapted from [1])

$$\mu = \frac{F}{R} \tag{1}$$

μ = Coefficient of friction

R = FW = Resistive force equal to the weight component of vehicle acting at the tire

F = Tangential Friction force

Coefficient of friction is incorporated as one of the major input parameters in the geometric design of a highway in the determination of the availability of minimum Stopping Sight Distance (SSD), minimum horizontal radius in horizontal curves. In summary, pavement friction plays an indispensable role in providing a motorist the opportunity to safely operate the vehicle both in the longitudinal and lateral directions [1,4].

2.1.1 Longitudinal Friction Force

The frictional force produced by the pavement on the vehicle tire on a free rolling mode with or without constant braking, consists of two major constituents: the rolling resistance force (FR) and the braking slip force (FB) acting in the longitudinal direction, as illustrated in the Fig. 2.2.

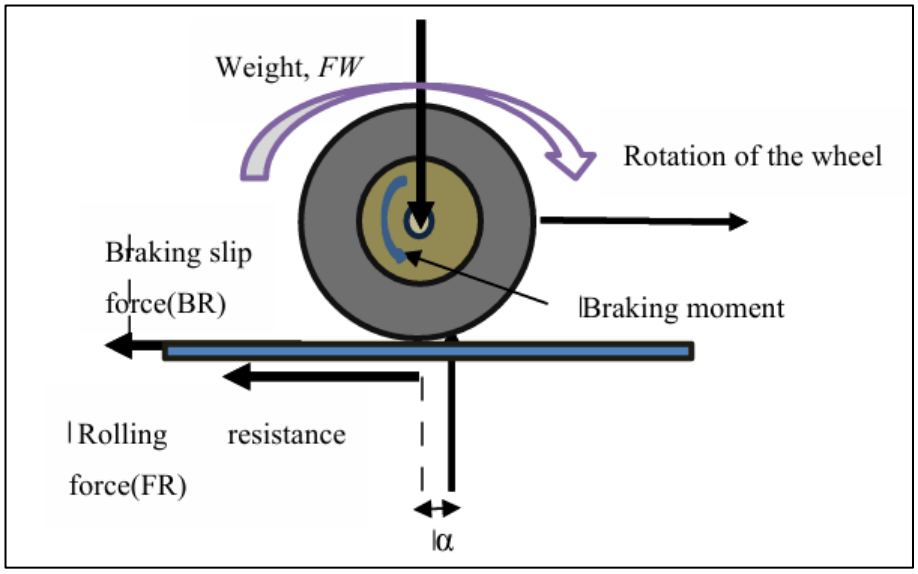


Fig. 2.2: The longitudinal friction forces acting on a free rolling tire (adapted from [14])

The tangential force required to overcome the moment created by the pavement resistive force acting vertically at the centre of pressure of the tire contact area with an offset (α) to the centre of tire, is termed as the rolling resistance force (FR). It causes the tire to roll and directly proportional to the speed of the vehicle. The braking slip force (FB) is the tangential frictional force countering for the moment created by braking at the constant brake mode and is directly proportionate to the degree of braking of the vehicle. In the discussion of the longitudinal friction force, vehicle Slip speed (S) and Slip ratio (SR) as given by equations (2) and (3), become important in the interpretation of the free rolling and braking mode of an operating vehicle [1].

$$S = V - V_P = V - (0.68 * \omega * r) \quad (2)$$

$$\text{Slip Ratio (SR)} = \frac{V - VP}{V} * 100\% \quad (3)$$

S = Slip speed, km/hr

V = Vehicle speed, km/hr

V_P = VP = Average peripheral speed of the tire, km/hr

ω = Angular velocity of the tire, radians/sec.

r = Average radius of the tire, m.

The slip speed is the relative speed between the vehicle tire periphery and the pavement surface, and the slip ratio is defined as the ratio between the slip speed and the vehicle speed. Both S and SR are equal to zero during free rolling state of the tire without braking, whereas in the fully braked mode (locked wheel), the S is equal to the speed of the vehicle with a 100 percent SR [1]. The friction demand of the pavement varies with slip as given in the Fig. 2.3 [1,4].

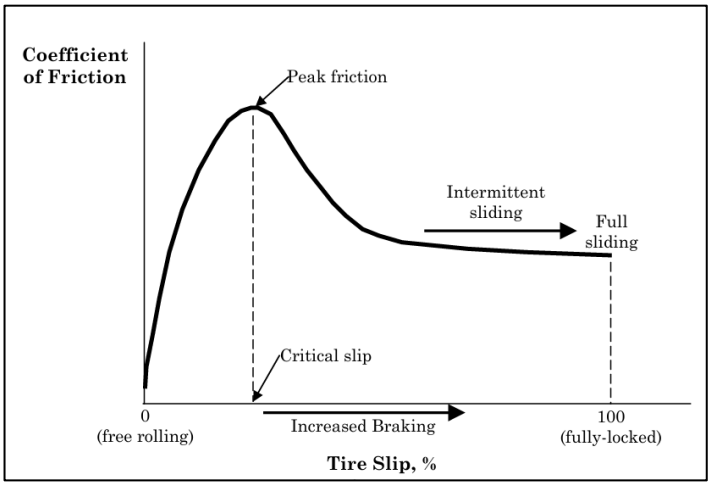


Fig. 2.3: Pavement friction coefficient Vs. Tire Slip (adapted from [1])

Thus, adequate friction should be present at the tire-pavement interface to cater the critical situation.

2.1.2 Lateral Friction Force

The lateral frictional force dominates when a vehicle is engaged in changing the direction of driving. The relation between the forces acting on the vehicle tire and the pavement surface shown in the Fig.2.4, is as follows in equation (4) [1,4].

$$\mathrm{FS} = \frac{\mathrm{V}*\mathrm{V}}{15\mathrm{R}} - e \qquad (4)$$

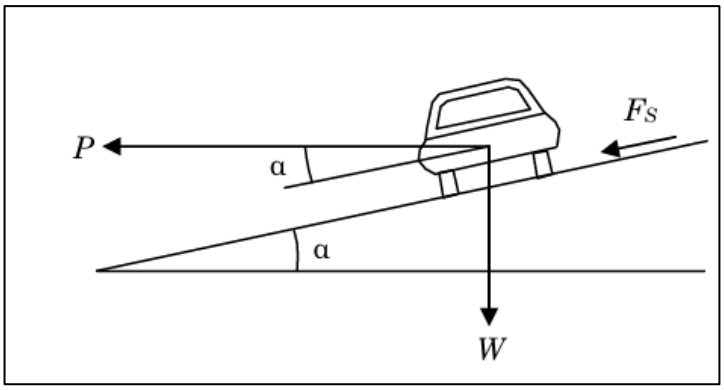


Fig. 2.4: Forces acting on a vehicle during cornering (adapted from [1])

F_S = FS = Side friction

V = Vehicle speed, km/hr.

R = Radius of the path of the vehicle's center of gravity m.

e = Pavement super-elevation, m/m

α = Angle of super-elevation (tan α = e)

P = Centripetal force (horizontal)

W = Weight of vehicle

When braking and cornering happen simultaneously, both lateral and longitudinal friction forces act at the pavement and proportions of the components vary according to pavement characteristics.

The friction demand of a pavement is controlled by the influential geometric design aspects of the highway such as critical horizontal or vertical grades and the vehicle operating factors like vehicle speed and braking, maneuvering actions of turning, cornering or overtaking. The friction capacity of the pavement that should cater the above created friction demand, is greatly governed by vehicle tire condition properties like tread design, rubber composition and hardness and inflation pressure as well as the material and textural properties of the overlay combined with other secondary factors like presence of contaminants on the wearing course and climatic conditions such as rain/snowfall and ambient temperature. Therein, friction related pavement characteristics concern a greater portion in creating adequate friction between the tire-pavement interface [1].

2.2 Pavement Friction Mechanism

As far as pavement friction mechanism is concerned, it is provided by two major vectors, which is adhesion (FA) and hysteresis (FH), based on the surface texture properties as depicted in the Fig. 2.5, given by the equation (5) [1,4].

$$F = F_A + F_H \quad (5)$$

Where,

F = pavement friction

F_A = adhesion

F_H = hysteresis

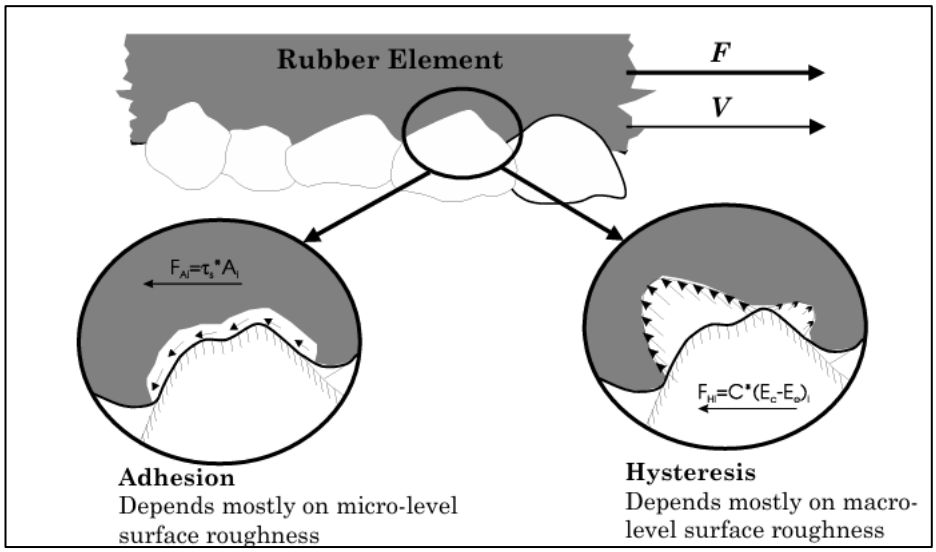


Fig. 2.5: Pavement friction mechanism (adapted from [1])

2.2.1 Adhesion and Hysteresis

Adhesion arises from minor molecular bonding and interlocking between vehicle tire rubber and pavement surface upon contact. This force depends on the interface shear strength and contact area, responding primarily to micro-texture asperities of pavement aggregate particles. Molecular chains in the rubber attempt to bond with pavement molecules, undergoing stretching and rupture cycles that generate retarding adhesion until critical slip occurs. On wet pavements at typical speeds, adhesion supplies about two-thirds of total friction resistance [1].

Hysteresis stems from energy dissipation during tire bulk deformation around pavement texture. As the tire compresses against the surface, rubber stores deformation energy and upon relaxation, some of stored energy dissipates irreversibly as heat, yielding net frictional force to halt vehicle motion. Hysteresis responds mainly to macrotexture from mix design, aggregate gradation or construction. Adhesion dominates on dry, smooth pavements, whereas hysteresis prevails on wet, coarse-textured ones [1].

Road crashes claim nearly 1.3 million lives and injure or disable 50 million people annually, with global economic costs exceeding US$518 billion. Contributing factors include human errors (95% of incidents), road-environment issues (28%), and vehicle defects (8%), wherein slip-related accidents, predominantly tied to low skid resistance rank as the chief pavement defect culprit. The U.S data indicate 15–18% of highway crashes occur on wet roadways, with 13.5% of fatalities linked to wet conditions. Studies confirm wet-weather crash correlations with pavement friction, as shown in

the Fig. 2.6 [3]. Targeted interventions in high-risk zones markedly lower accident rates, underscoring the need for road safety audits and friction-influencing factor analyses. Elevating mean pavement friction from 0.40 to 0.55 slashes wet-pavement crashes by 63%, while research findings reveal 71% and 54% reductions in wet-weather incidents at intersections and freeways, respectively, via enhanced skid resistance [1-3].

Friction interval	Accident Rate (personal injuries per million vehicle kilometers)
< 0.15	0.80
0.15–0.24	0.55
0.25–0.34	0.25
0.35–0.44	0.20

Fig. 2.6: Effect of friction on the road accidents (adapted from [3])

2.2.2 Friction Related Pavement Characteristics

Though pavement friction depends on pavement, vehicle operating and tire, environment related characteristics, pavement related attributes concern a greater proportion. Major pavement related characteristics are as follows.

- Macro texture
- Micro texture
- Material properties – aggregate mineral structure
- Water Film Thickness (WFT)/Water ponding issues
- Distresses
- Contaminants on the surface

The first three parameters are more effective on creating fluctuations of available friction capacity of a pavement. WFT also emerges as one of the prominent factors that determines pavement friction since the water layer acts as a lubricant between the tyre and the wearing course, drastically undermining frictional capacity.

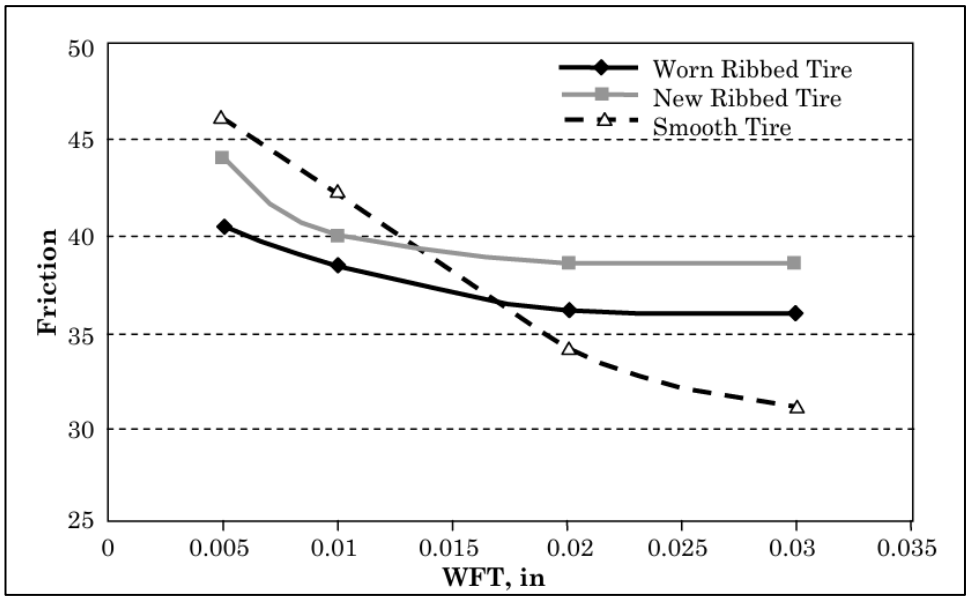


Fig. 2.7: Relationship between pavement friction and water film thickness (adapted from [1])

Skidding occurs during braking or maneuvering when frictional demand surpasses the available tire-road interface friction. Though possible on dry pavements at high speeds, wet surfaces drastically lower friction as shown in the Fig. 2.7, such that a 0.002-inch water film alone cuts tire-pavement friction by 20–30% relative to dry conditions. Countermeasures must thus elevate tire-road friction while minimising surface water [20].

Water film thickness critically dictates friction, particularly above 64 km/h. Elevated speeds (>64 km/h) paired with thicker films (25 mm) of water trigger hydroplaning, severing tire-pavement contact as tires hydroplane atop water [1]. Macrotexture primarily governs surface water drainage, channeling it into subgrade and ground absorption to sustain adequate wet-condition friction coefficients.

2.2.3 Pavement Surface Texture

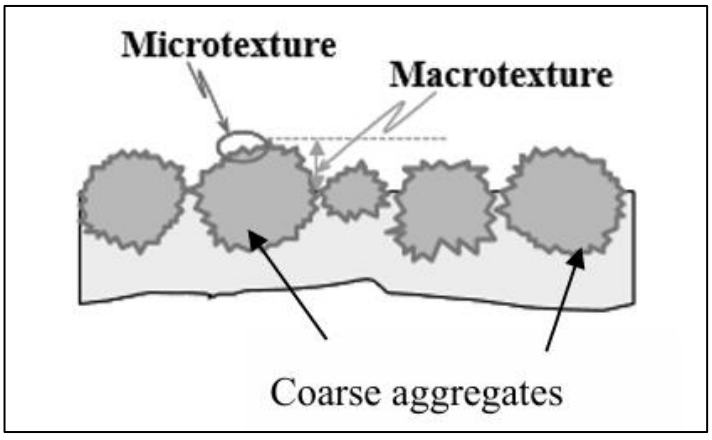


Fig. 2.8: Pavement texture (adapted from [1])

The wavelengths and the amplitudes relevant to micro-texture and macrotexture are as follows, as graphically demonstrated in the Fig. 2.8 [1].

- Micro-texture - microscopic asperities of an individual coarse aggregate ($\lambda <$ 0.5 mm, 1 µm <A< 500 µm) [1]
- Macro-texture - level of roughness influenced by the deviations among aggregate particles [(0.5<λ<50) mm, (0.1<A<20) mm] [1]

Macrotexture encompasses larger surface irregularities, typically voids between pavement aggregate particles that predominantly influence hysteresis friction. Its extent depends on coarse aggregate size, shape, distribution, nominal maximum aggregate size, and surface layer construction methods. Macrotexture aids hysteresis by enabling tire deformation around asperities to boost friction, facilitates rapid water dispersal to avert hydroplaning as well. A minimum depth of 0.5 mm (0.02 inches) ensures sufficient water evacuation from the tire-pavement interface [20].

Micro-texture refers to micron-scale irregularities on aggregate particles, governed predominantly by mineralogy, imparting tactile roughness or smoothness. Initial friction retention against traffic and environment-induced polishing, dictates micro-texture magnitude, critically affecting adhesion friction. In wet conditions, micro-asperities displace thin water films to establish contact and generate friction. Their shape and density (amplitude versus count) control drainage, with superior micro-texture (0.2–0.5 mm) delaying hydroplaning by 20% higher speeds, thus mitigating risks via enhanced materials [20].

In conclusion, the availability of pavement friction is greatly controlled by the degree of textural variations over the aggregate particles and the pavement surface, as depicted in the Fig. 2.9. Optimal macrotexture depth guidance identifies a crossover at 0.6–0.8 mm, beyond which accident risk elevates markedly for roads averaging below 0.7 mm compared to those exceeding it. Crash rates surge with texture depths under 1 mm, stabilizing at minimal levels above 1 mm. [20]

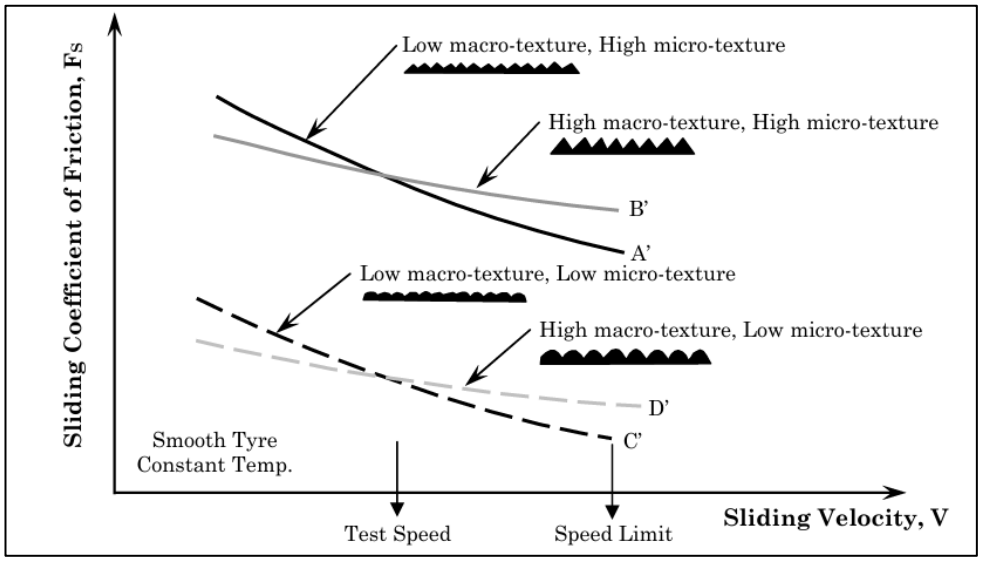


Fig. 2.9: Effect of pavement texture on pavement friction (adapted from [1])

2.2.4 Pavement Friction Measurement

Field SR is quantified by the force generated as a locked tire slides across the pavement surface. Measurements demand precision, repeatability, and reproducibility for reliable field assessment. Typically, a standard tire with controlled wheel slip (0–100%) evaluates pavement skid resistance per American Society for Testing and Materials (ASTM) standards [1,19].

Four primary skid resistance measurement techniques exist:

- Locked-wheel: Quantifies force under 100% slip conditions.
- Sideways-force: Assesses force on a rotating wheel at 20° yaw angle.
- Fixed-slip: Measures friction at constant wheel slip ratios.
- Variable-slip: Gauges friction across user-defined slip levels or peak values.

These methods, particularly locked-wheel and variable-slip correlate tire friction during rolling, braking, or cornering on wet pavements, employing ribbed or smooth standard tires. The British Sideway Force Coefficient Routine Investigation Machine (SCRIM) in the Fig. 2.10 is a commonly used field measurement device in the industry, whereas Locked-wheel, as in the Fig. 2.11, predominates, with skid trailers introduced in the 1960s enhancing testing safety and efficiency. Fixed/variable slip ratios govern friction in slip-ratio modes, while sideways-force simulates cornering criticality [1].



Fig. 2.10: The SCRIM device (adapted from [1])



Fig. 2.11: Locked Wheel Trailer (adapted from [1])

Field approaches complement portable laboratory testers, notably the British Pendulum Tester (BPT) shown in the Fig. 2.12, which is a cost-effective dynamic pendulum device sensitive to micro-texture (dominant at lower speeds) and macrotexture [1,19,21]. Coarse textures may yield misleading BPT results whereas consistent outcomes require controlled coupon curvature, aggregate arrangement, contact path length, and slider load. BPT slider swings polished aggregates variably by type, thus highly sensitive for micro-texture induced by the aggregate mineralogy. [1,22]

The Dynamic Friction Tester (DFT) given in the Fig. 2.13, a disc with three rubber pads, measures skid resistance at 20–80 km/h under constant force, correlating strongly with BPN across speeds.

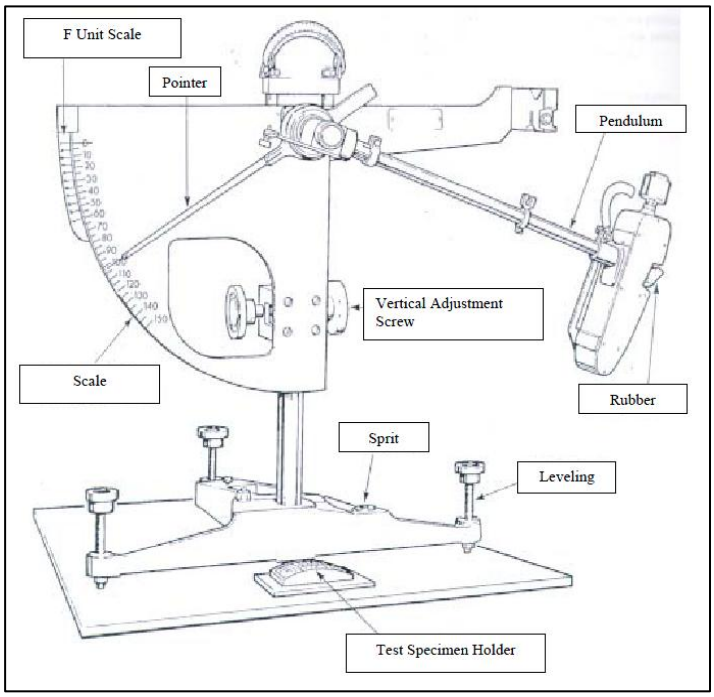


Fig. 2.12: British Pendulum Tester (adapted from [21])

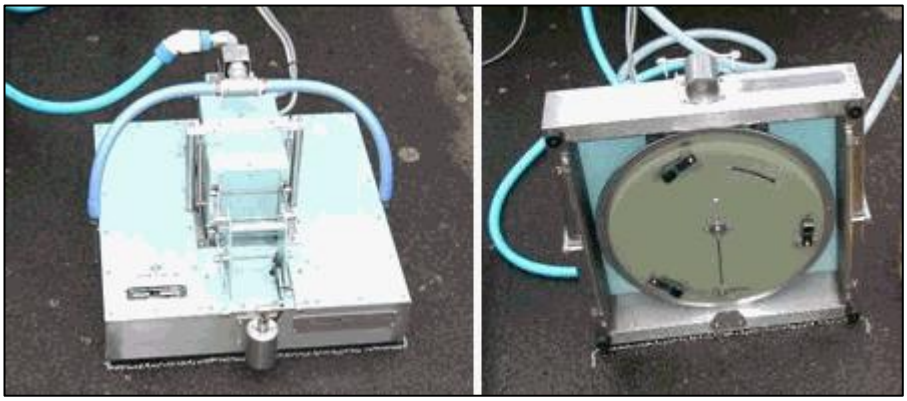


Fig. 2.13: Dynamic Friction Tester (DFT) (adapted from [1])

2.2.5 Pavement Texture Measurement

Linking skid resistance to pavement texture holds paramount importance in research. Volumetric methods like spreading known volumes of sand, glass beads, or grease across the surface and dividing volume by covered area known as routine Sand Patch Method (SPM) as in the Fig. 2.14 or Silly Putty, yield Mean Texture Depth (MTD). The Outflow Meter Test (OMT) shown in the Fig. 2.15, assesses macrotexture via drainage capacity and surface wear [1,19]. A transparent cylinder on a rubber annulus receives water where outflow time between marks quantifies drainage efficacy, correlating to macrotexture. Laser-enabled, high-speed systems such as Circular Texture Meter (CTM) as depicted in the Fig. 2.16, compute profile metrics like Mean Profile Depth (MPD) and Root Mean Square (RMS) height from traffic-speed scans, distilling profiles to singular parameters [1,19,22].



Fig. 2.14: Sand Patch Method (adapted from [1])



Fig. 2.15: Outflow Meter (adapted from [1])

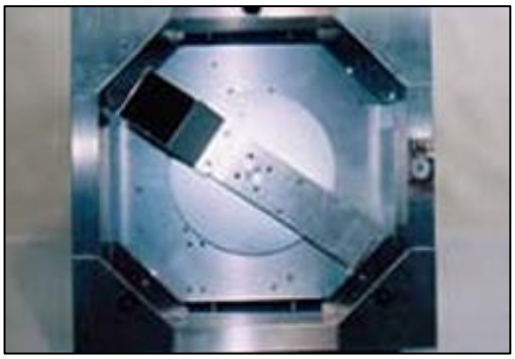


Fig. 2.16: Circular Texture Meter (adapted from [1])

Micro-texture measurement options abound. Pennsylvania State research as in Penn State Model, links zero-speed friction intercepts and BPN to RMS micro-texture height, positioning BPN as a proxy. Scanning Electron Microscope (SEM) imagery quantifies lab-simulated traffic polishing (British accelerated test). Optical/laser devices prevail for simplicity. The Aggregate Imaging System (AIMS) employs microscopy and digital processing for direct texture assessment. 3D color laser microscopy as shown in the Fig. 2.17, profiles surfaces via XYZ scanning, yielding nanometer-resolution, high-depth-of-field color images for precise roughness analysis over an aggregate surface [7,9,23,24,25].

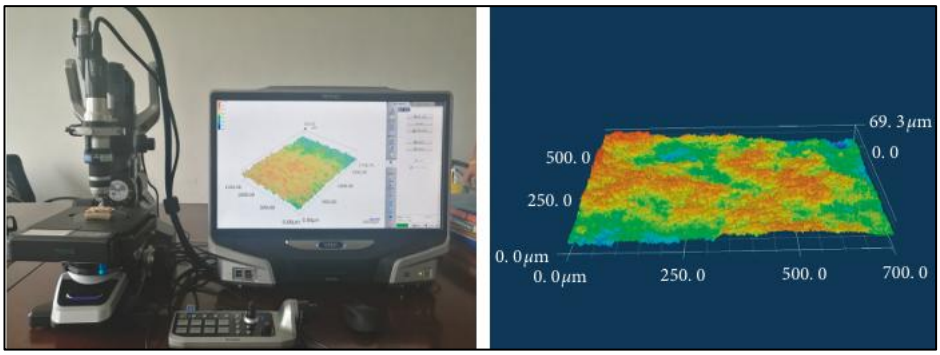


Fig. 2.17: 3D laser microscopy measurement (adapted from [12])

2.3 Skid Number of Highway Pavements

To measure the SR of a pavement surface, as a substitute for the co-efficient of friction, the widespread standardised indices is termed as the Skid Number (SN) or later known as Friction Number (FN). It is a combined measure of both friction capacity and macrotexture availability of a wearing course at a specific speed of the vehicle, usually given as a decimal value between one and zero or as a number between zero to hundred when the co-efficient is multiplied by 100 [1,19]. According to the type of friction and texture (macro) measurement devices, the applicable correlations and equations to express the SN which are called as friction prediction models, would take different forms. Commonly followed indices/models are the Penn State model developed by Leu and Henry (1978), the Rado Model proposed by Rado (1994) and the PIARC Model introduced by Permanent International Association of Road Congresses (PIARC), as given in the TABLE 2.1 [1,19]. Among them, the Penn State model is known to be well representative of micro and macrotexture of a pavement when measured by the BPT and SPM or similar texture measurement device/profilometer [1,19].

TABLE 2.1: STANDARD FRICTION (SN) PREDICTION MODELS

Friction Prediction Model	Model Equation
Penn State	$SN\,(60) = SN_0 \,.\, \exp\left[\frac{-(PNG).V}{100}\right]$ (6) SN = Skid Number SN_0 = Intercept of friction at zero speed (SN_0 = -31 + 1.38BPN) PSNG = The percentage SN-speed gradient (%) (PNG =0.45.$MTD^{-0.47}$) V = Slip speed

Rado	$F(60) = A + B \times FR(60) + C \times TX$ $FR(60) = FR(S) \times e^{(\frac{S-60}{S_P})}$ $S_P = a + b \times TX$ *SP* = International Friction Index (IFI) speed number. *a*, *b* = Calibration constants dependent on the method used to measure macro-texture *TX* = Macro-texture *FR(60)* = Adjusted value of friction measurement *FR(S)* at selected *S* to a slip speed of 60 km/hr *FR(S)* = Friction value at selected *S* *S* = slip speed, km/hr *F(60)* = IFI friction number *A*, *B* = Calibration constants depending on the type of friction measuring device *C* = Calibration constant required for measurements using ribbed tire	(7)
PIARC	$\mu(S) = \mu_{peak}\ exp\text{-}[\frac{\ln(S / S_{peak})}{C}]^2$ μ_{peak} = peak friction level S_{peak} = S at the peak C = shape factor (related to the roughness of texture)	(8)

Various highway authorities have long used the SN in setting up the minimum required friction demand for their road systems based on the functional classification, traffic volume, operational speed, highway alignment and other geometric parameters, as exhibited in the different examples given in the Fig.2.18, 2.19, 2.20 and 2.21[1,19]. Therefore, SN serves as a vital decision-making input in PMS and PFM of specifically skid hazardous road sections.

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

Fig. 2.18: SN requirement in the UK (adapted from [19])

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

Fig. 2.19: SN requirement in the Australia (adapted from [19])

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

Fig. 2.20: SN requirement in Finland (adapted from [19])

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

Fig. 2.21: SN requirement in New Zealand (adapted from [19])

However, measurement of SN along the length of a highway over a selected cyclic time period can be challenging as it depends on the type of available friction or texture measurement device of the particular road agency. Frequency depends on the total network size and resource availability According to the Federal Highway Administration (FHWA) of the United States Department of Transportation (DoT) to evaluate networks comprehensively, annual testing of every pavement section proves ideal due to annual fluctuations in surface friction. An effective strategy involves rotating or cycling tests across network parts every few years, for instance, in a 3-year cycle, one-third gets assessed annually with no interval exceeding 4 years [1]. Hence, annual monitoring is done by many agencies in terms of critical road sections while, a 2-to-5-year turn is appropriate for lower-risk portions of the road network [26]. The initial SN on a pavement just constructed can be recorded only when the construction of the overlay is completed as well. Due to these difficulties and limitations, models to predict SN based on other easily measurable parameters like aggregate quality have been developed as an alternative to the standard models already in use, as given in the TABLE 2.2.

TABLE 2.2: ALTERNATIVE FRICTION (SN) PREDICTION MODELS

Model	Remark
$SN(60) = \alpha.MTD + \beta.PSV\ diff + C$ (9) For Asphalt concrete of 9.5 mm gradation $\alpha = 0.2699$, $\beta = 0.0087$ and $C = 0.3346$ For Asphalt concrete of 12.5 mm gradation , $\alpha = 0.4556$, $\beta = 0.0041$ and $C = 0.1702$	The model is based on a multiple linear regression analysis of the two input parameters of standard PSV and MTD by SPM [19].
$MSSC = 0.98 \times 10^{-2}\ PSV - 0.664 \times 10^{-4}\ CVD + 0.033$ (10)	The Mean Summer SCRIM Coefficient (MSSC) is predicted using PSV of aggregates and the Number of Commercial Vehicles per lane per Day (CVD) [26].
$BPN(t) = (\frac{A}{t})^4 + B.\ PSV$ (11) A = 373 , B = 0.858 for Basalt aggregate A = 372 , B =0.890 for Limestone aggregate	The field BPN values are correlated to polishing time of the PSV test in minutes and the corresponding PSV of the aggregate type [27].

2.4 Coarse Aggregate Mineralogy and Polishing Behaviour

The polishing behaviour of aggregates is closely related to their micro-textural asperities on individual particles and their grain hardness, which are governed by the geological formation of the particular rock type. This can be clearly explained by their mineralogy in terms of the type of minerals present and individual grain hardness and petrography in terms of mineral composition and arrangement of their crystalline structure, a shown in the Fig. 2.22 [26].

Mohs' scale of hardness is considered to be the most common measure of grain hardness with a scale from 1-10, where mineral grains with Mohs' value <5 are regarded as soft while those with Mohs' value > 5 are identified to be hard and tough [1,28]. Aggregates made of coarse and hard single type of grains tend to develop higher resistance against polishing in comparison to those with finer and softer grains, which is called general polishing. However, harder minerals that have undergone chemical alteration called as recrystallisation common in the formation of metamorphic rocks out of igneous rocks, can be susceptible for higher degree of polishing. Differential polishing occurs where the mineralogical build of the aggregates consists of both hard and soft grains with a substantial in-between proportion, where soft minerals give away under polishing leaving wear resistant harder minerals. This is called as plucking of soft minerals as well when the harder grains are embedded in a softer mineral matrix [6]. Hence, coarse aggregates with igneous rock origins perform well under traffic

application due to general polishing of harder grains, whereas some of high-grade metamorphic aggregates show adequate polishing resistance due to differential polishing. Most of the sedimentary rock based coarse aggregates polish quickly due to the lithification of softer mineral and recrystallised harder minerals [6].

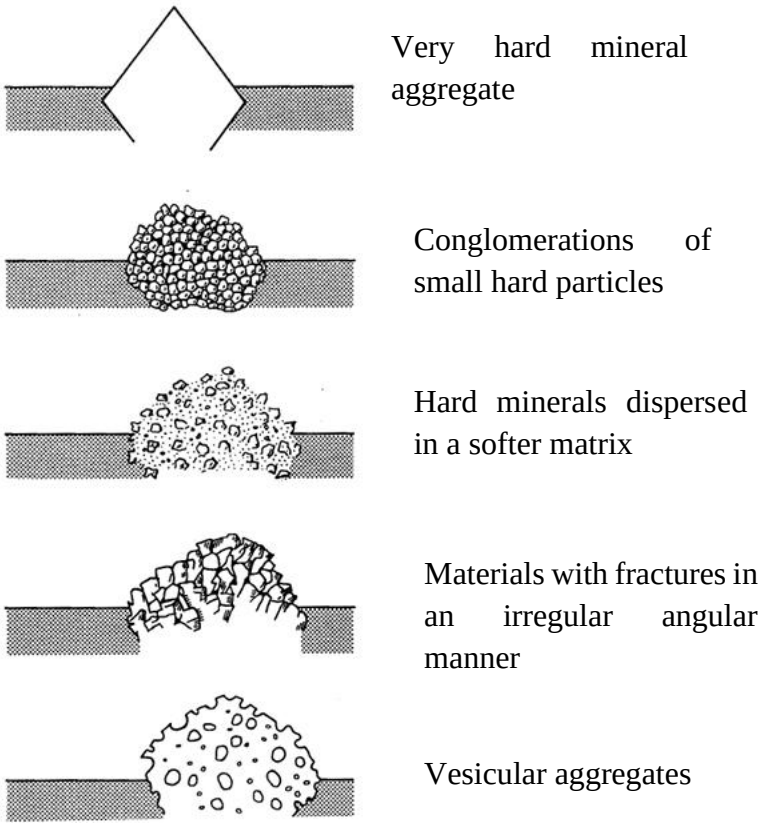


Fig. 2.22: Crystalline structure of different mineral aggregates (adapted from [26])

Limestone, the predominant aggregate in road pavement construction, exhibits the highest susceptibility to polishing, yielding reduced skid resistance and heightened pavement slipperiness. Variability in polishing resistance among limestone deposits is pronounced; for carbonate aggregates like dolomite, susceptibility diminishes with rising clay content, though it declines to an optimal level before escalating with excessive calcite and dolomite. Differential polishing across aggregates stems from wear-resistant mineral content, mainly silica, wherein limestone purity inversely correlates with resistance, requiring ≥25% siliceous particles for adequacy, alongside optimal particle size [12,31,32].

Quartz sandstone, lauded for superior frictional performance and elevated wet friction, benefits from differential particle dislodgement under traffic, fostering optimal macrotexture; its quartz grains, bound by brittle silica cement, resist abrasion effectively as the matrix erodes. Conversely, limestone and flint display minimal resistance due to cryptocrystalline homogeneity and uniform hardness. Basalt, granite, and quartzite of igneous origins offer higher resistance, linked to altered feldspars and quartz dislodgement from resilient matrices. Sand and gravel aggregates, rich in abrasion-resistant particles, reveal favorable frictional properties [12,31,32].

Gneiss-derived aggregates are frequently classified as granite-based in industry and scholarly works owing to their similar physical and mechanical attributes. Characterized by platy foliated texture, gneisses feature coarser minerals embedded within a softer matrix, thus under polishing, the weaker phases disintegrate, exposing durable, unweathered grains that preserve surface texture over extended durations. Minerals like feldspar, plagioclase, and pyroxene further enhance skid resistance through elevated siliceous composition [1].

2.4.1 Polish Stone Value Test

The PSV test is a standardised and well-established method developed by the U.K. to assess the polishing resistance of road surfacing aggregates under laboratory induced traffic application [1]. It involves polishing of aggregate specimens on a wheel by a rubber tyre under a specified applied surcharge load by an Accelerated Paving Machine (APM) feeding water and polishing agents, as shown in the Fig. 2.23, and subsequently measuring friction of the polished aggregate surfaces using a BPT. Higher PSV indicates better polishing resistance for an aggregate with higher skid resistance at the pavement [30, 31]. In numerous studies, repeatability of the PSV test was recorded to be in the range from 1.52 – 1.60, implying the higher reliability of the test to assess the intended aggregate property [28]. At the sixth hour of polishing, a PSV can be deduced using the below equation (12) [21].

$$PSV = S + 56 - C \quad (12)$$

Where,

S = Mean of the BPN of the 04 test samples of the aggregate source tested

C = PSV of the Control stone (a reference roadstone with known PSV value in the range between 50 -60)

Usually, PSV is given as a value between thirty to eighty and aggregates with PSV higher than 60 is considered to be high friction roadstones [1,28].

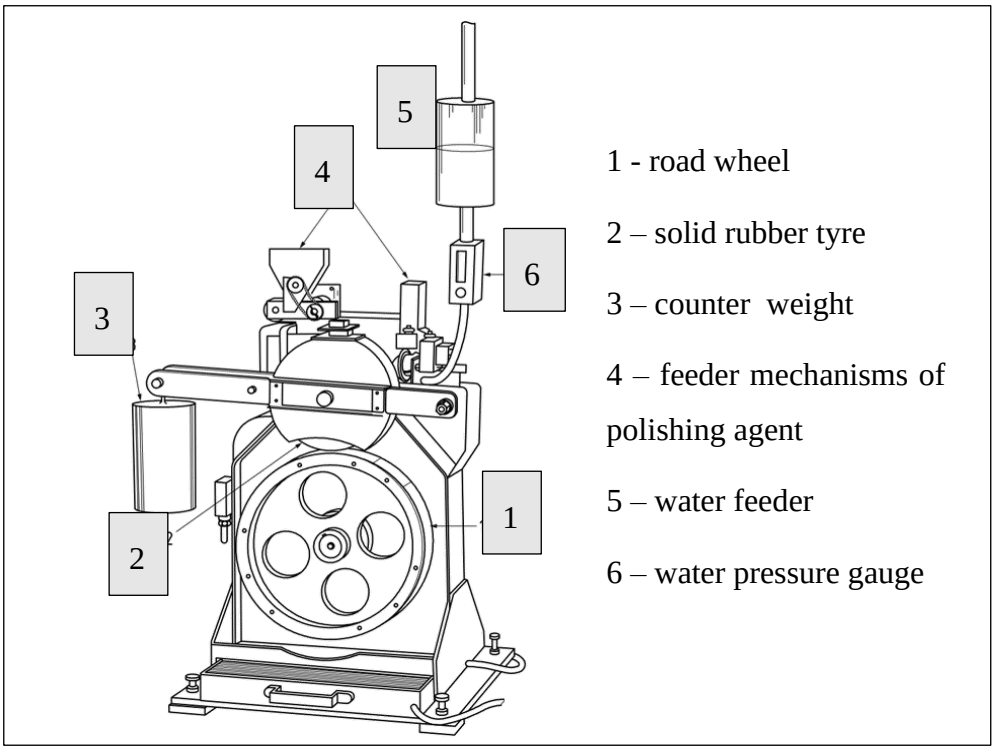


Fig. 2.23: Accelerated Paving Machine (adapted from [21])

PSV is found to be very effective to represent polishing characteristics induced by the micro-texture and mineralogy of coarse aggregates, therefore not only U.K, but also many other regions in the world like European countries, the U.S.A, India have incorporated the above test in their aggregate selection criteria according to the different identified risk categories of highway sections in terms of geometrical features, type of overlay and traffic volume, as shown in the Fig. 2.24, 2.25 and 2.26.

Physical properties	Test method
Grading	Sieve analysis
Fines content	Wet sieving to determine %<0.063mm
Shape	Flakiness index
Resistance to fragmentation	Los Angeles test (reference method) German Schlagversuch impact test
Resistance to wear	French micro-Deval coefficient
Resistance to polishing	British polish stone value
Resistance to surface abrasion	British Aggregate abrasion value
Durability in terms of freeze-thaw resistance	Water absorption as a screening test for freeze/thaw resistance Freeze-thaw test or magnesium sulfate soundness where necessary
Bulk density	Bulk density
Thermal shock	Sonnebrand test – 36 hour boiling in water Soaked, surface dry aggregate heated at 700^0C for 3 minutes

Fig. 2.24: Summary of main European test methods for aggregate in asphalt mixes (adapted from [7])

Site Category	Site description	Relative Risk Level	IL	PSV required for given IL, traffic level and site category									
				Commercial Vehicle Flow (cv/l/d)									
				1-250	251-500	501-750	751-1000	1001-2000	2001-3000	3001-4000	4001-5000	5001-6000	Over 6000
A	Motorway	Standard	0.30	50	50	50	50	50	50	50	53	63	63
		High	0.35	50	50	50	50	50	53	53	53	63	63
B	Non-event carriageway with one-way traffic	Low -	0.30	50	50	50	50	50	50	50	53	63	63
		Standard	0.35	50	50	50	50	50	53	53	53	63	63
		High	0.40	50	50	50	50	53	58	58	58	63	68+
C	Non-event carriageway with two-way traffic	Low -	0.35	50	50	50	50	50	53	53	58	63	63
		Standard	0.40	50	53	53	58	58	63	63	63	68+	68+
		High	0.45	53	53	58	58	63	63	63	63	68+	68+
Q	Approaches to and across minor and major junctions, approaches to roundabouts and traffic signals	Low -	0.45	60	65	65	68+	68+	68+	68+	68+	68+	HFS
		Standard	0.50	65	65	65	68+	68+	68+	HFS	HFS	HFS	HFS
		High	0.55	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS
K	Approaches to pedestrian crossings and other high-risk situations	Standard	0.50	65	65	65	68+	68+	68+	HFS	HFS	HFS	HFS
		High	0.55	68+	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS
R	Roundabout	Low – (<30mph)	0.40	50	55	60	60	65	65	65	65	68+	68+
		Standard	0.45	50	55	60	60	65	65	68+	68+	68+	68+
		High	0.50	68+	68+	68+	68+	68+	68+	68+	68+	68+	68+
G1	Gradients 5-10% longer than 50m	Low -	0.40	55	60	60	65	65	65	65	68+	68+	68+
		Standard	0.45	55	60	60	65	65	68+	68+	68+	68+	68+
		High	0.50	60	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS
G2	Gradient >10% longer than 50m	Low -	0.45	55	60	60	65	65	68+	68+	68+	68+	68+
		Standard	0.50	60	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS
		High	0.55	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS
S1	Bends radius <500m - ≥50mph carriageway with one-way traffic	Standard	0.45	50	55	60	60	65	65	68+	68+	HFS	HFS
		High	0.50	68+	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS
S2a	Bends radius <250m - ≥40mph carriageway with two-way traffic	Low -	0.45	50	55	60	60	65	65	68+	68+	HFS	HFS
		Standard	0.50	68+	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS
		High	0.55	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS
S2b	Bends radius <100m - ≥30mph carriageway with two-way traffic	Low – (<30mph)	0.45	50	55	60	60	65	65	68+	68+	HFS	HFS
		Standard	0.50	68+	68+	68+	HFS	HFS	HFS	HFS	HFS	HFS	HFS
		High	0.55	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS	HFS

Fig. 2.25: Guidelines for the determination of PSV for High Friction Overlay Requirements – UK (adapted from [8])

Required mean summer	PSV of aggregate necessary *Traffic in commercial vehicles per lane per day*					
SFC at 50 km/h	<250	1000	1750	2500	3250	4000
0.30	30	35	40	45	50	55
0.35	35	40	45	50	55	60
0.40	40	45	50	55	60	65
0.45	45	50	55	60	65	70
0.50	50	55	60	65	70	75
0.55	55	60	65	70	75	
0.60	60	65	70	75		
0.65	65	70	75			
0.70	70	75				
.0.75	75					

Fig. 2.26: Transport and Road Research Laboratory (TRRL) Proposals -1973 (adapted from [9])

Further, as a step towards generation of new knowledge on the polishing patterns and incorporation of such approaches in road user safety improvement, countries like

Afghanistan, India, Bangladesh, Pakisthan, Thailand, Iran and Iraq have derived polishing patterns of locally available road surfacing aggregates, as given in the Fig. 2.27 and 2.28.

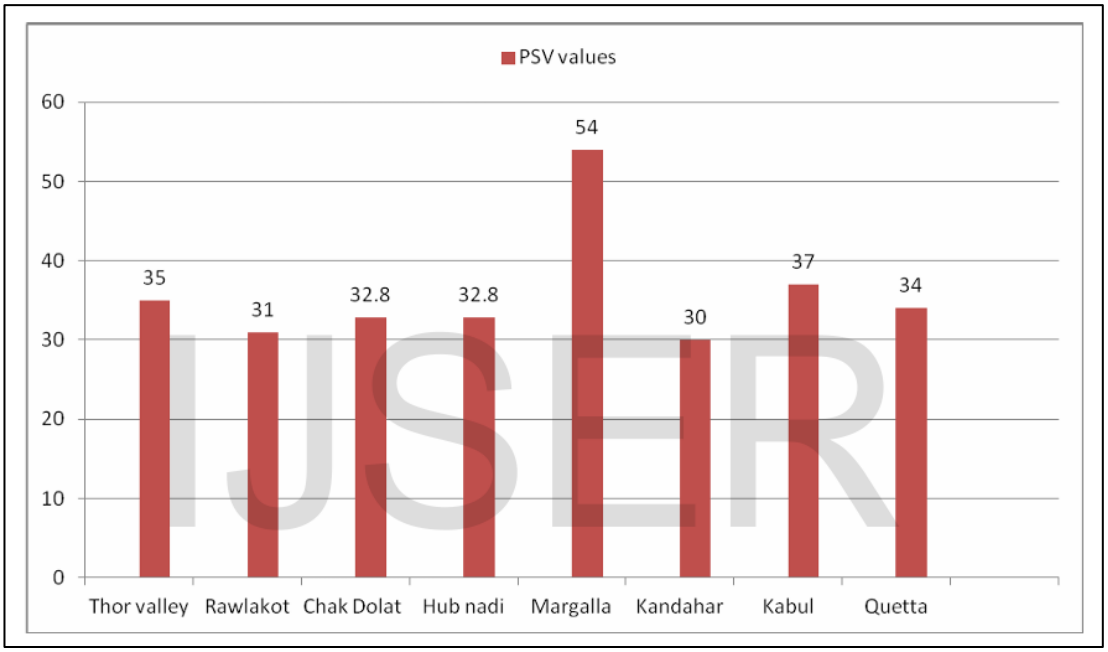


Fig. 2.27: PSV of coarse aggregate quarries in Pakistan and Afghanistan [adapted from 10])

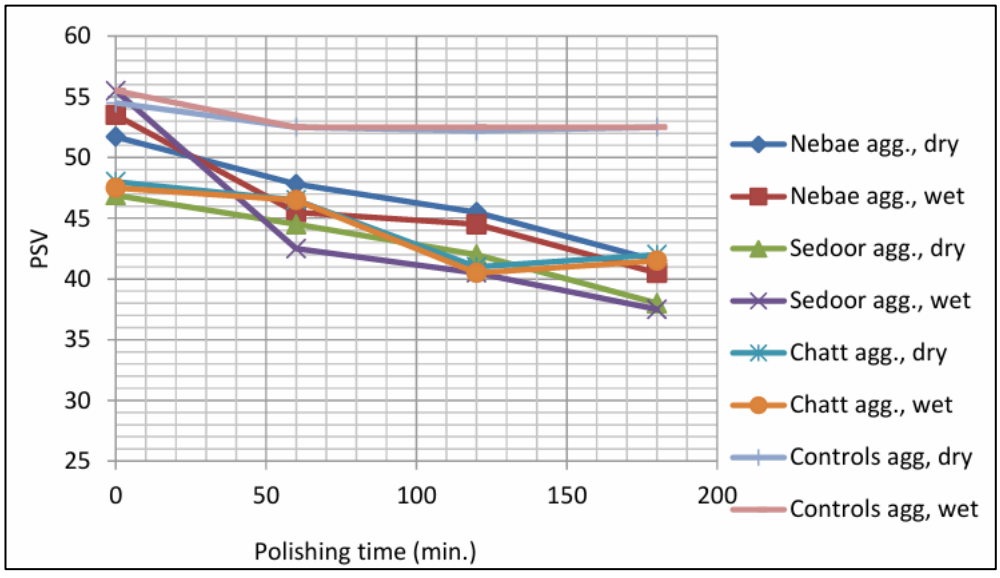


Fig. 2.28: PSV of coarse aggregate quarries in Iraq [adapted from 32])

The most widely used surfacing coarse aggregates for the pavement wearing course in the world, are sandstone and limestone of the sedimentary origins, basalt and granites of the igneous category and hornfels and quartzite of high-grade metamorphic origins, of which PSV values and friction decay patterns have been already established and available in literature for reference, as shown in the Fig. 2.29 and 2.30 [12].

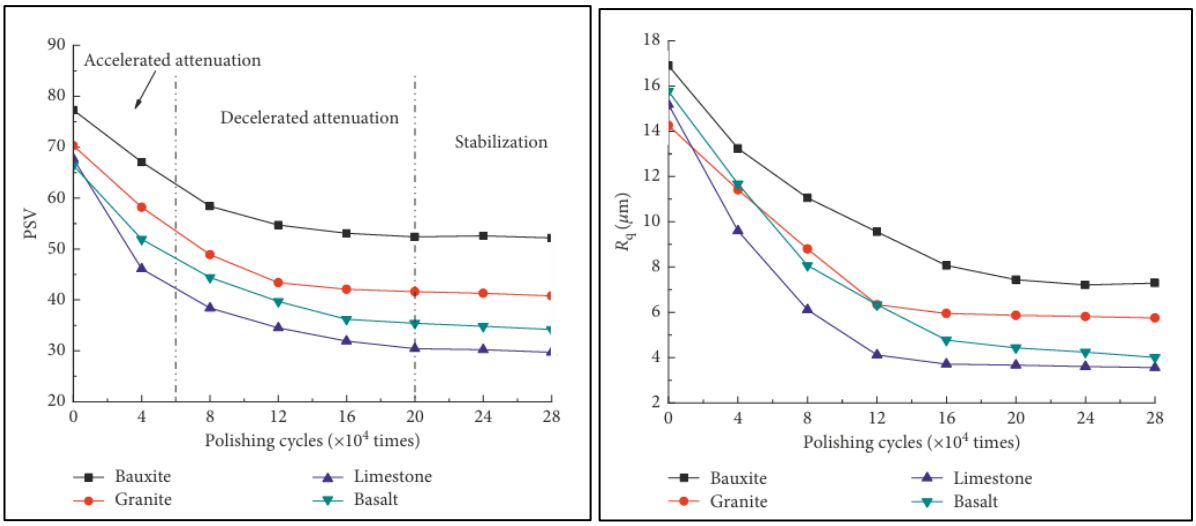


Fig. 2.29: PSV of coarse aggregate quarries in China [adapted from 12])

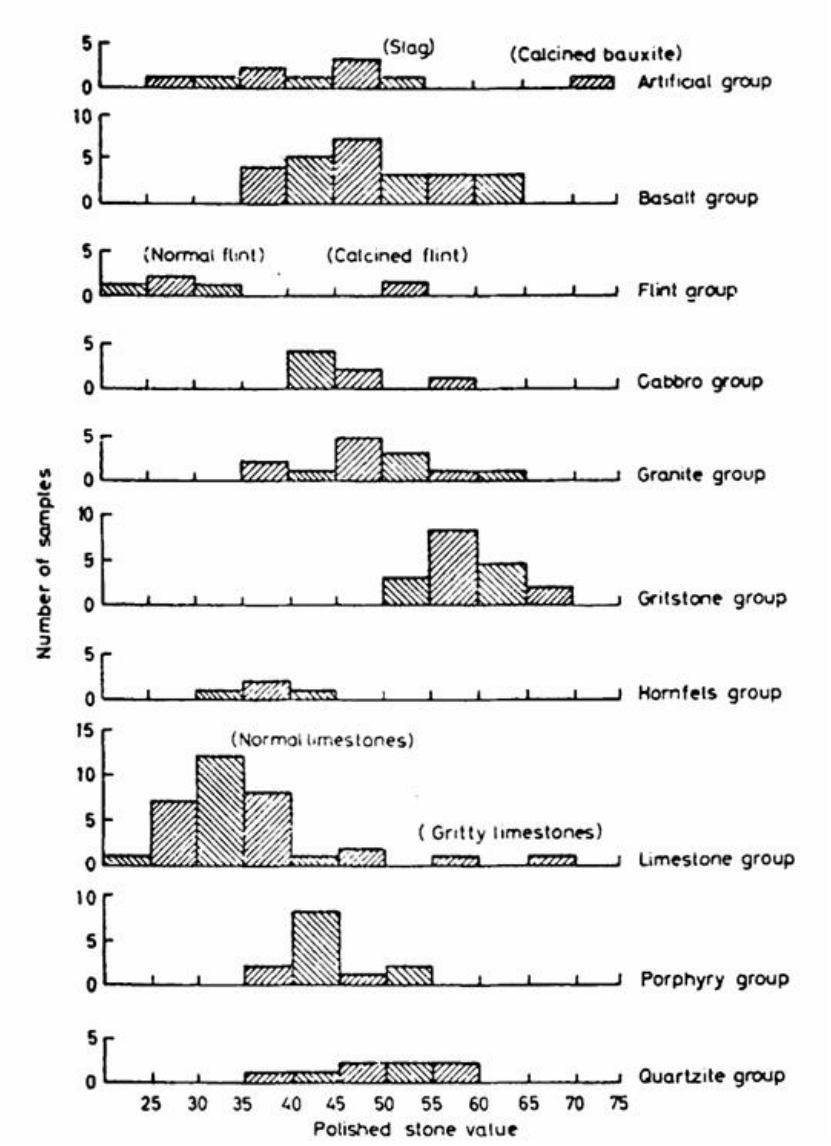


Fig. 2.30: PSV of coarse aggregate quarries in the UK [adapted from 06])

Apart from natural mineral aggregates, various studies have explored the friction characteristics of alternative aggregate types like steel slag, as depicted in the Fig. 2.31 and 2.32 [25,27].

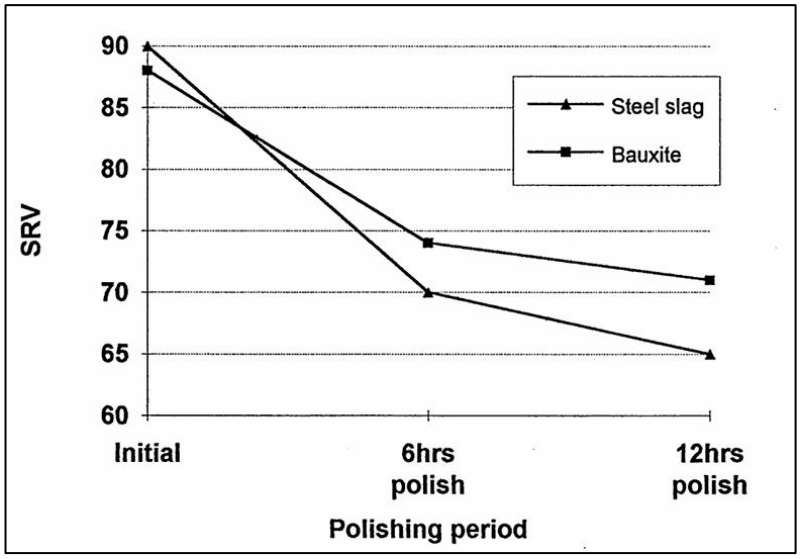


Fig. 2.31: PSV of artificial coarse aggregate products [adapted from 27])

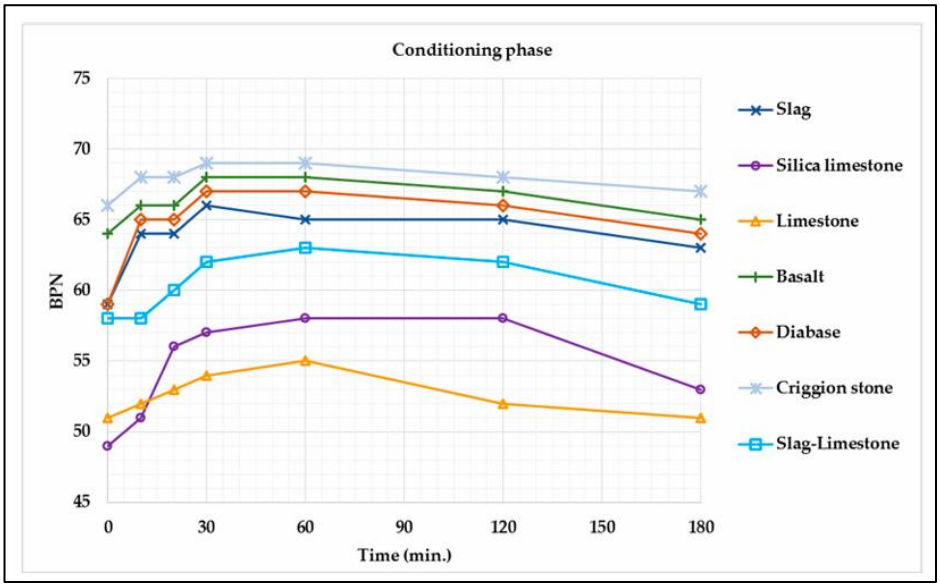


Fig. 2.32: PSV of industrial by products as substitute for coarse aggregate [adapted from 25])

However, despite the wide variety of applications of the PSV test, other similar polishing tests and machines are being developed and assessed as a substitute, since PSV is regarded to be a strenuous process that may have limitations to replicate the actual bituminous mixtures on a flexible pavement [7,31,33]. Examples for afore-mentioned alternatives are Micro-Deval (MD), Small-wheel circular track polishing machine, Aachen Polishing Machine (APM), Wehner/Schulze (W/S) machine, Auckland Pavement Polishing, Los Angles, and Road Test Machine (RTM), as illustrated in the following Fig. 2.33, 2.34 and 2.35 [23,33,34].

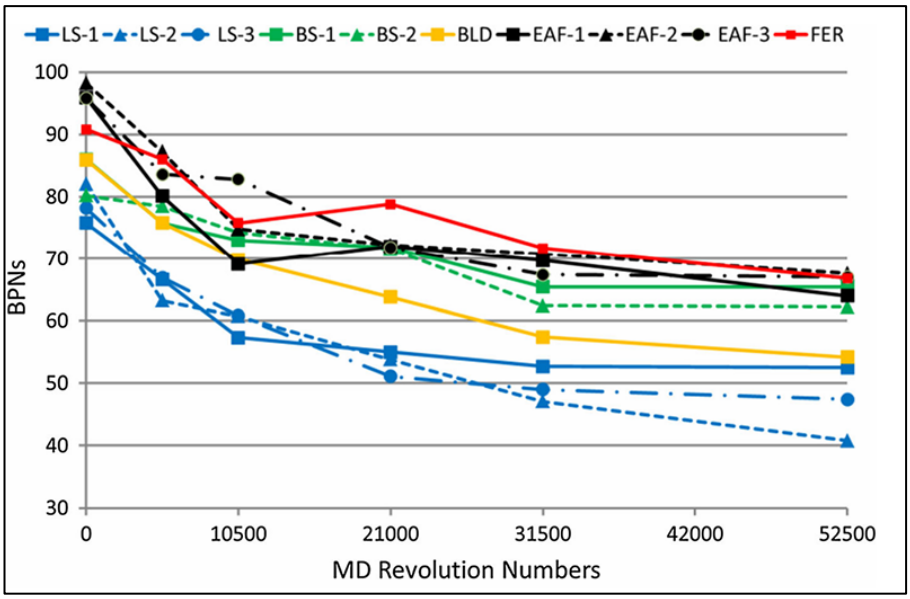


Fig. 2.33: Friction decay graphs derived from MD test [adapted from 23])



Fig. 2.34: Wehner/Schulze (W/S) machine [adapted from 34])



Fig. 2.35: Small-wheel circular track polishing machine [adapted from 34])

CHAPTER 3

RESEARCH METHODOLOGY

In the process of establishing a Skid Resistance/friction prediction model for Sri Lankan Road surfacing aggregates, the following research methodology was adapted, having identified the key requirements through a thorough search of the existing literature on pavement friction. In consideration of the multiple research objectives of the study with a wide spectrum within the sphere of coarse aggregate properties, a variety of feasible and prevailing laboratory tests were introduced, so as to achieve higher clarity and connectivity of the results and findings leading to a well calculated and justified conclusion.

3.1 Research Methodology

The below mentioned is a simplified array of steps followed during the research, which is graphically illustrated in the Fig. 3.1 as well.

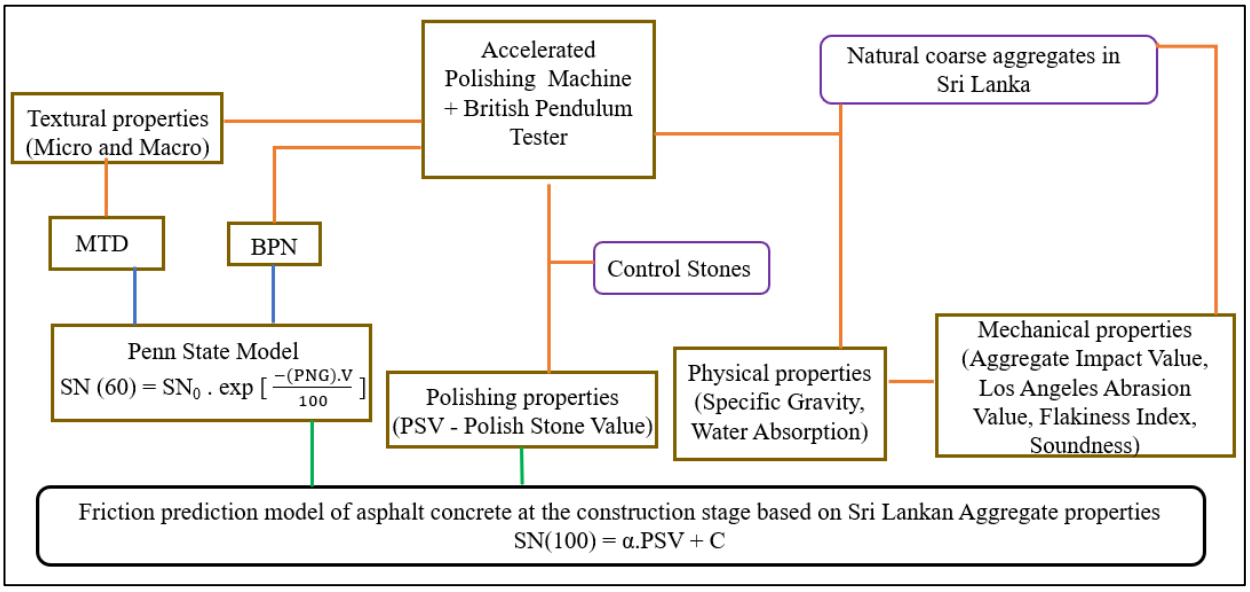


Fig. 3.1: Research Methodology in a simplified flow chart

- Collect substantial amount (10 – 12 kg) of representative mineral coarse aggregate samples with a gradation of nominal maximum aggregate size being 25 mm, from quarries/crusher plants of road construction industry, located in Sr Lanka
- Conduct the laboratory testing stipulated in the CIDA guidelines according to the relevant BS/ASTM standards, to verify the suitability of the collected aggregate samples for the application of wearing course of a highway pavement
    - Specific Gravity (SG)
    - Water Absorption Value

    - Los Angeles Abrasion Value (LAAV)
    - Aggregate Impact Value (AIV)
    - Flakiness Index (FI)
    - Sodium Sulphate Soundness (SSS)
- Verify the quality conformity of the aggregate samples in terms of physical and mechanical properties with the use of the results of the tests, in order to proceed with the rest of the research methodology
- Prepare the required number of test specimens for each quarry location as well as control aggregate
- Calibrate the test machines used for the polishing and texture measurement
- Conduct the PSV test in terms of extended polishing action with the use of the accelerated polishing machine and the BPT, to establish the polishing deterioration trend of the local aggregates
- Conduct texture measurement (micro and macro) for aggregate specimens using the identified texture measurement profilometer at each polishing interval of the PSV test, to establish the texture deterioration trend of the local aggregates
- Calculate the Skid Number for each aggregate location using the Penn State model by substituting the derived friction and texture values
- Develop the Skid-Resistance Predictive Model by simple linear regression analysis with the use of the variables (PSV and SN) deducted in the previous steps
- Validation of the derived model with the use of Field data (BPN data by British Pendulum Test and MTD data (macro-texture) by Sand Patch Method of selected expressway sections in Sri Lanka at post construction stage)
- Extend recommendations to be incorporated in the material selection process of highway pavement construction and Pavement Management System of expressways in Sri Lanka

3.2 Sample Material Acquisition

In the material collection of coarse aggregate samples from crusher plants specifically supplying for highway pavement construction is Sri Lanka, the basis was to identify the most demanding rock sources for the industry as well as to select the quarries representing the majority of the geological formation of the earth crust beneath the country. The annual coarse aggregate production for pavement construction exceeds eight million cubic meters, while Colombo, Kalutara, Kurunegala, Anuradhapura and Hambantota districts are the major providers with the highest supply [16].

Nine tenth of the geological formation beneath Sri Lanka consist of Precambrian age rocks of the earth's crust, called the South Indian Shield, which is older than 570 million years, resulting in the formation of high-grade metamorphic rock sources producing mostly charnockite, hornblende biotite gneiss, biotite gneiss, migmatite and granitic gneiss [17]. Apart from these hard and strong gneissic based aggregates,

smaller amounts granitic gneiss, quartzite, marble, dolomite, granulite and migmatite are available. Depending on the tectonic setting and the age of these metamorphic grade rocks, the country's Precambrian age rock earth's crust is subdivided into four geological complexes; Highland Complex (HC), Wanni Complex (WC), Vijayan Complex (VC) and Kadugannawa Complex (KC), while a coastal belt of comparatively young Miocene aged sedimentary rock formations mostly with limestone can be found in North-Western region of the country as shown in the Fig. 3.2 [16,17].

Therefore, the second basis for aggregate samples collection from 20 Industrial Mining License (IML) qualified quarries of coarse aggregates certified under Road Development Authority (RDA) was to represent the largest two geological complexes; HC and WC of the country, whose location details and source code are presented in the TABLE 3.1. Therein, natural aggregate samples were collected from the largest suppliers of roadstones in the country: Colombo, Anuradhapura, Kalutara, Hambantota and Kurunegala Districts and to represent two major geo complexes in Sri Lanka, in addition, aggregate samples were collected from suppliers from Galle, Puttalam, Vavuniya, Matale and Rathnapura Districts. From each location an approximate ten to twelve kilograms of fresh and unweathered coarse aggregates passing through 25 mm sieve, were collected.

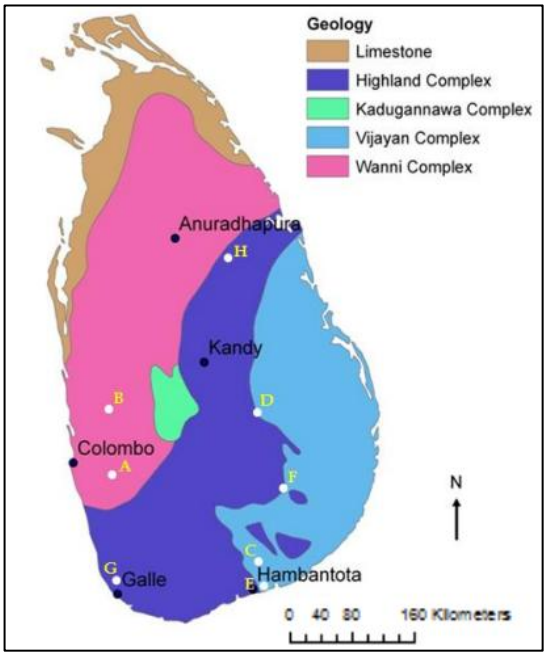
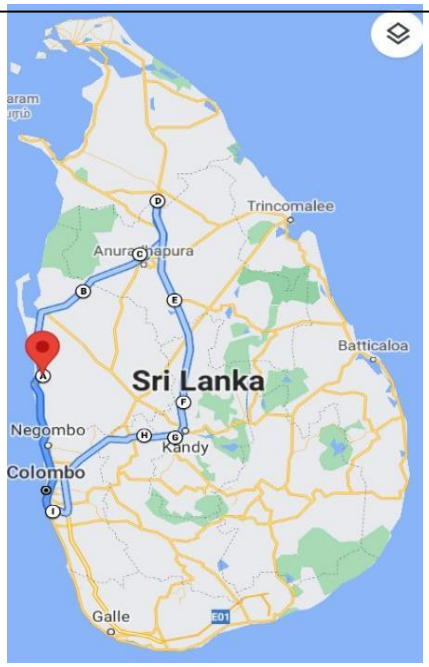


Fig. 3.2: Geological complexes in Sri Lanka (adapted from [35])

TABLE 3.1: LOCATION OF THE SOURCES OF COARSE AGGREGATE SAMPLES OF THE RESEARCH STUDY

Quarry No	Location	Geological Complex	Quarry No	Location	Geological Complex
01	Kalutara	HC	11	Kalutara	HC
02	Kandy	HC	12	Mathara	HC
03	Rathnapura	HC	13	Vavuniya	WC
04	Colombo	HC	14	Galle	HC
05	Colombo	HC	15	Galle	HC
06	Anuradhapura	WC	16	Kalutara	HC
07	Mathale	HC	17	Puttalam	WC
08	Kandy	HC	18	Rathnapura	HC
09	Anuradhapura	WC	19	Anuradhapura	WC
10	Hambantota	HC	20	Kurunegala	WC

The raw material collection was carried out in three separate field visits to the identified crusher plants as depicted in the Fig. 3.3 and the physical appearance of the coarse aggregate specimens are as follows, captured in the Fig. 3.4 respectively.







Day 01





Day 02





Day 03

Fig. 3.3: Material Collection across Sri Lanka

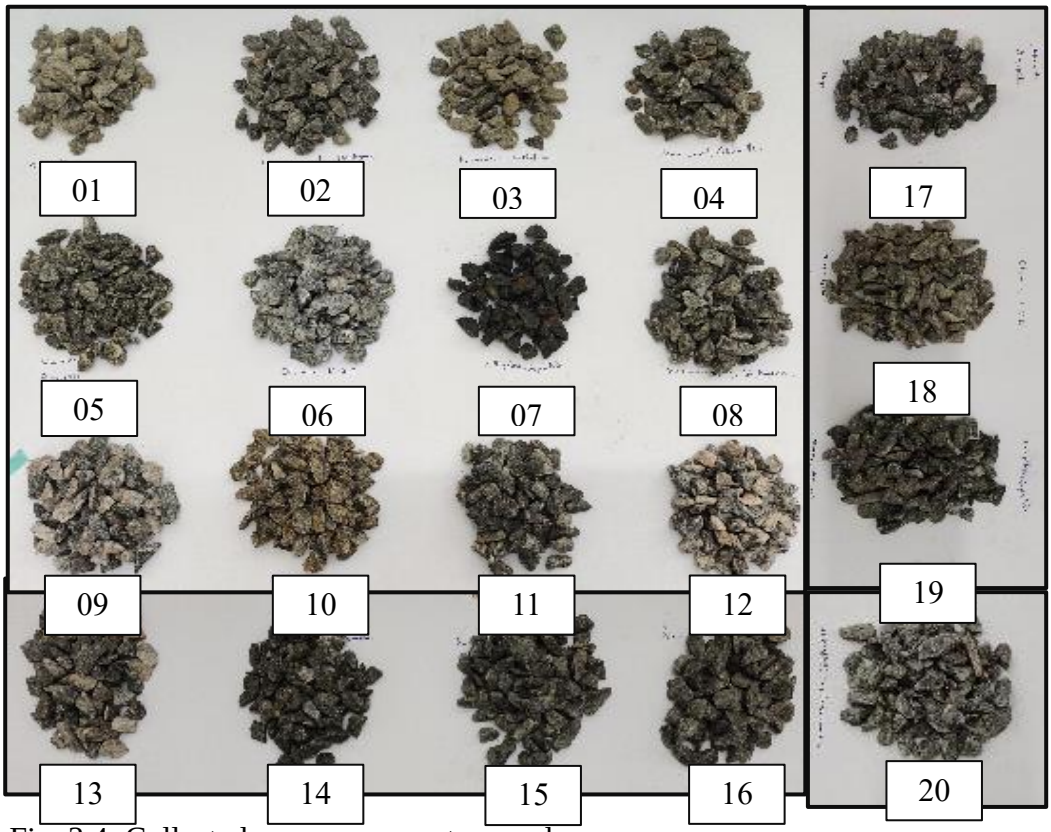


Fig. 3.4: Collected coarse aggregate samples

The laboratory tests are complied in the next chapter in alignment with the preceding corresponding results for better clarity in analysis and reader understanding.

CHAPTER 4

RESULTS AND ANALYSIS

4.1 Evaluation of Physical Properties

In order to proceed with the laboratory testing required to establish the polishing and textural profile of the coarse aggregate samples of each quarry, firstly the suitability of the aggregate samples according to the quality conformity codes stipulated in the SSCM-RB - SCA/5 [13] by Construction Industry Development Authority (CIDA), Sri Lanka previously known as Institute of Construction Training and Development (ICTAD), was evaluated via physical properties and mechanical properties. The two physical property parameters of the coarse aggregate samples under Saturated Surface Dry (SSD) condition tested as per the ASTM C127 were, [36]

- Specific Gravity – the ratio between the density of the test sample to that of water
- Water Absorption Value - the ratio between the mass of water absorbed by the water permeable voids of the test sample to the dry mass of the sample

Two samples of approximately 02 kg of coarse aggregates retained on 9.5 mm ASTM sieve (passing through 12.5 mm ASTM sieve) were oven dried at a temperature of 110 ^{o} C for 03 hours of which the Oven-Dried (OD) mass in air was recorded to a nearest of 0.5 g. The sample was then transferred to a water bath to be soaked at the room temperature for 24 hours and the SSD mass in air was recorded. The coarse aggregate sample was immediately placed in the sample container within the water tank to determine the apparent mass of the saturated aggregate sample by the displacement of water method at the room temperature to a nearest of 0.5 g.

Specific Gravity (Relative Density) – SSD and Water Absorption Value were calculated using the following equations (13,14) for the two samples and the average value was obtained per each quarry location as presented in the TABLE 4.1.

$$\text{Specific Gravity (Relative Density)} - \text{SSD} = B/(B - C) \quad (13)$$

$$\text{Water Absorption Value, \%} = [(B - A)/A] \times 100 \quad (14)$$

Where,

- A = mass of OD aggregate sample in g (in air)
- B = mass of SSD aggregate sample in g (in air)
- C = apparent mass of saturated aggregate sample in g (in water)

TABLE 4.1: SG (RELATIVE DENSITY) – SSD AND WATER ABSORPTION VALUES OF AGGREGATE SAMPLES

Quarry No.	Specific Gravity	Water Absorption Value (%)	Quarry No.	Specific Gravity	Water Absorption Value (%)
1	2.67	0.41	11	2.67	0.47
2	2.70	0.39	12	2.73	0.38
3	2.78	0.45	13	2.66	0.43
4	2.67	0.46	14	2.74	0.46
5	2.71	0.41	15	2.71	0.40
6	2.71	0.49	16	2.63	0.41
7	2.75	0.35	17	2.66	0.37
8	2.65	0.42	18	2.65	0.39
9	2.73	0.42	19	2.77	0.38
10	2.69	0.44	20	2.79	0.45

Specific Gravity relates to the bulk density while Water Absorption Value symbolizes the presence of water permeable surface pores or porosity of coarse aggregates. All the coarse aggregate samples displayed a medium-density normal weight aggregate rock origins, which are substantially acceptable to be used as roadstones, while the water absorption vales are lesser than 2%, therein well complying with the CIDA requirements for coarse aggregates for a bound wearing course [13]. Since the sample size is small (<50) and the parameters being a continuous material property, simple descriptive statistical measures including mean, and standard deviation along with the Shapiro–Wilk normality test were used to analyse the spread and distribution characteristics. Mean is 2.709, highlighting medium-density normal weight aggregate rock origins. Standard Deviation of 0.045 leading to Coefficient of Variance of 0.0167 along with W = 0.969 and p-value = 0.735 of the Shapiro–Wilk normality test implicate a normal distribution with a low variability in specific gravity values across quarry locations in Sri Lanka, suggesting consistent aggregate quality suitable for pavement construction applications. The graphical distribution analysis as in Fig.4.1 showing the spread of specific gravity of aggregate samples in Highland and Wanni geological complexes, reiterate the above observation having a concentration around 2.65–2.75, which is typical for natural crushed rocks of igneous or metamorphic origins [6,23,30,37].

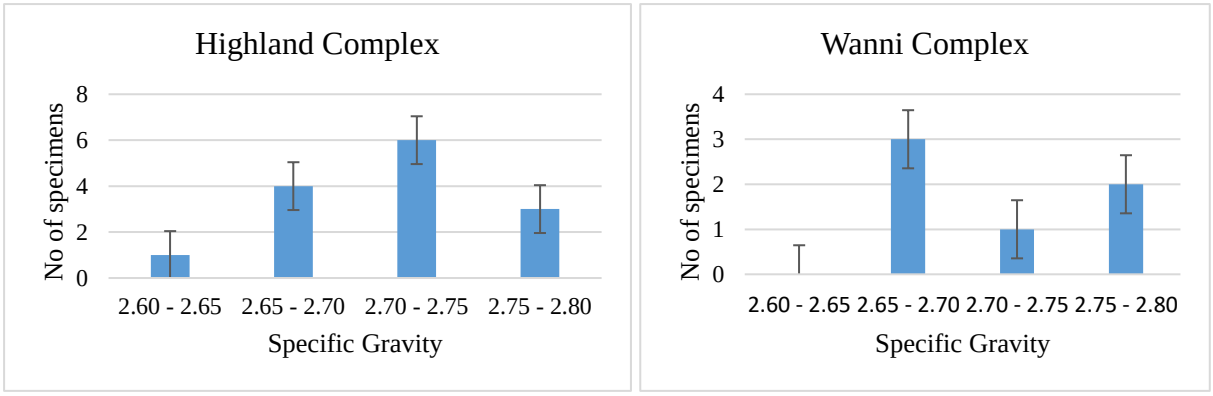


Fig. 4.1: The distribution of specific gravity of aggregate samples in Highland and Wanni geological complexes

4.2 Evaluation of Mechanical Properties

The four mechanical property parameters of the coarse aggregate samples tested as per the applicable ASTM / AASHTO or BS standards were,

- LAAV: AASHTO T – 96 - 02 – the ratio between the loss of aggregate mass to the mass of the original sample due to the effect of mechanical attrition presented as a percentage
- AIV: BS 812-112 - the ratio between the mass of aggregate loss to the mass of the original sample due to the effect of sudden mechanical impact presented as a percentage
- FI: BS 812 – 105.1 - the ratio between the mass of flaky aggregate particles to the mass of the original sample presented as a percentage
- SSS: AASHTO T – 104 - the ratio between the mass of aggregate loss to the mass of the original sample due to the effect of repeated cycles of immersion in the Sodium Sulphate solution and oven drying

4.2.1 Los Angeles Abrasion Value (LAAV) and Aggregate Impact Value (AIV)

Two samples of approximately 2.5 kg of coarse aggregates retained on 9.5 mm ASTM sieve (passing through 12.5 mm ASTM sieve) were oven dried at a temperature of 110 º C for 03 hours of which the OD mass in air was recorded to a nearest of 1.0 g. The sample was then transferred to the LAAV machine drum with the respective 08 number of standard testing steel balls to be rotated for 500 number of rotations at a speed of 30-33 rpm. Thereafter, the test samples were separated as coarse and fine using 1.70 mm sieve and the OD mass of coarser portion was recorded to the nearest of 1.0 g [38].

Three samples with a combined mass of approximately 5.0 kg of coarse aggregates retained on 10 mm BS sieve (passing through 14 mm BS sieve) were oven dried at a temperature of 110 º C for 03 hours of which the OD mass in air was recorded to a nearest of 0.1 g. Each sample was then transferred to the cylindrical measuring cup

upon 25 number of blows from the standard testing metal rod, which was then subjected to 15 number of blows from the free fall of the hammer weight of the AIV machine. The sample was then sieved using the BS 2.36 mm sieve and the passing content was weighed to a nearest of 0.1 g [39].

LAAV and AIV Value were calculated using the following equations (15,16) for the two samples and the average value was obtained per each quarry location as presented in the TABLE 4.2.

$$\text{Los Angeles Abrasion Value (LAAV) \%} =](A - B)/ A] \times 100 \quad (15)$$

$$\text{Aggregate Impact Value (AIV) \%} = [C/A] \times 100 \quad (16)$$

Where,

- A = mass of original coarse aggregate test sample relevant to each test in g
- B = mass of coarse aggregate sample retained on 1.70 mm ASTM sieve in g
- C = mass of coarse aggregate sample passing 2.36 mm BS sieve in g

TABLE 4.2: LAAV AND AIV VALUES OF COARSE AGGREGATE SAMPLES

Quarry No.	LAAV (%)	AIV (%)	Quarry No.	LAAV (%)	AIV (%)
1	31	22	11	36	23
2	32	20	12	37	23
3	36	22	13	31	18
4	38	23	14	38	27
5	37	19	15	39	21
6	33	24	16	39	22
7	32	22	17	40	18
8	30	26	18	40	20
9	35	25	19	32	25
10	36	21	20	32	22

LAAV determines the resistance of coarse aggregates against abrasion or wear where AIV represents the toughness or resistance of coarse aggregates against rapid, sudden impact loads. All the quarry samples of the study demonstrated lesser levels of degradation with LAAV < 40 % and AIV < 30 % complying with the ICTAD/CIDA requirements.

4.2.2 Flakiness Index (FI) and Sodium Sulphate Soundness (SSS)

Two samples of approximately 1.5 kg of coarse aggregates retained on 10 mm BS sieve (passing through 14 mm BS sieve) were oven dried at a temperature of 110 º C for 03 hours of which the OD mass in air was recorded to a nearest of 1.0 g. Then all the aggregate particles of the sample were individually tested using the gauge thickness slot of the Flakiness Index Gauge applicable for their size fraction to be separated as flaky (passing through the 7.2 mm thickness gauge slot) and non-flaky (not passing

through the 7.2 mm thickness gauge slot) particles, whose mass were recorded to a nearest of 1.0 g [40,41].

Two samples of approximately 0.5 kg of coarse aggregates retained on 9.5 mm ASTM sieve (passing through 12.5 mm ASTM sieve) were oven dried at a temperature of 110 ° C for 04 hours of which the OD mass in air was recorded to a nearest of 0.1 g. The sample was then immersed in the prepared Sodium Sulphate Solution for 16 hours, after which the drained sample was oven dried at a temperature of 110 ° C for 04 hours. Alternate immersion and drying were performed for 05 cycles, and thereafter the sample was washed with circulating hot water to substantially remove the chemical solution. The sample was then sieved using a 8.00 mm ASTM sieve to eliminate any smaller fractions, and the retained mass was recorded to a nearest of 0.1 g [42].

FI and SSS Value were calculated using the following equations (17,18) for the two samples and the average value was obtained per each quarry location as presented in the TABLE 4.3.

$$\text{Flakiness Index (FI) \%} = [B / A] \times 100 \quad (17)$$

$$\text{Sodium Sulphate Soundness (SSS) \%} = [(A - C)/A] \times 100 \quad (18)$$

Where,

- A = mass of original coarse aggregate test sample relevant to each test in g
- B = mass of coarse aggregate sample determined as flaky in g
- C = mass of coarse aggregate sample retained on 8.00 mm ASTM sieve in g

TABLE 4.3: FI AND SSS VALUES OF COARSE AGGREGATE SAMPLES

Quarry No.	FI (%)	SSS (%)	Quarry No.	FI (%)	SSS (%)
1	16	1.2	11	23	1.7
2	13	0.8	12	17	3.5
3	17	3.2	13	18	1.8
4	14	1.6	14	14	1.8
5	18	1.8	15	21	1.4
6	20	1.4	16	24	1.7
7	19	1.2	17	22	1.9
8	12	1.9	18	14	1.8
9	16	1.8	19	16	3.1
10	15	1.2	20	15	3.8

Flakiness Index value indicates the fraction of flaky particles (whose smallest dimension or thickness is lesser than 0.6 of their mean sieve size) present in an aggregate sample, whereas Sodium Sulphate Soundness determines their resistance to disintegration by saturated solutions of sodium sulfate implicating soundness or durability of aggregates subject to weathering action or freezing and thawing. All the study samples exhibited favorable FI (< 35 %) and SSS (< 12%) values. Hence, the

quarry samples confirm with the CIDA quality conformities of aggregate suitable for the construction of wearing or surface course of HMA pavement [13]. Therein, the subsequent testings to determine polishing and textural characteristics were to be carried out.

4.3 Evaluation of Polishing Characteristics

In accordance with the major research study objectives to establish the friction and texture deterioration trends leading to the development of a friction prediction model, the polishing patterns of the locally available roadstones were to be determined followed by the determination of micro-texture and macro-texture of coarse aggregates of wearing course application. To further understand and interpret the research findings and the deterioration trends, a mineralogical and petrography analysis was required be conducted.

4.3.1 Preparation of Aggregate Polishing – Wheel Specimens

ASTM D 3319 - 00 - Standard Practice for the Accelerated Polishing of Aggregates Using the British Wheel with the adaptations from BS EN 1097 – 8 - 2020 – Determination of PSV, specifies the method of preparing the test specimens to be tested using the accelerated polishing machine, also known as the British Wheel [21,43]. The following tools and materials were used in the specimen preparation.

- Coarse aggregates passing through ASTM 12.5 mm sieve and retained on ASTM 9.5 mm sieve – ASTM D75 [44]
- 20-30 grade fine sand - ASTM C 778 [45]
- Control stone graded as per ASTM D 3319 - 00
- Friction tester reference stone (Criggion stone) graded as per ASTM D 3319 - 00
- Bonding agent- polyester resin and the catalyst with a thickening agent to change the viscosity
- Mould release agent – grease
- Metal mould with the metal sanding block
- Oven

The standard metal mould of which the inner walls were dabbed with grease, was filled with one layer of washed and oven dried coarse aggregates, in a densely packed arrangement to minimise the gaps between individual coarse aggregate particles. The voids were loosely filled with sand so that up to around one fourth to half of the height of the aggregates were filled. Then the mixture of polyester resin and the catalyst was carefully poured up to the top level of the metal mould so as not to disturb the particle arrangement and the excess resin was removed. Pressure was applied on the prepared specimen using the metal Sanding block during the curing period of about 06 hours. With the completion of the hardening process, the specimen was removed by dismantling the mould and any excess loose sand on the face of the specimen was also removed. Two number of samples per each quarry location and two specimens using

the friction tester reference stone (Criggion) for machine calibration and four specimens using the control stone were prepared in the above-described manner as shown in Fig.4.2 and 4.3. Every specimen was marked on the back of the sample with a given number for identification and the direction of the rotation was also marked to ensure uniformity during polishing.



Fig. 4.2: Accelerated polishing wheel specimens prepared for each quarry location

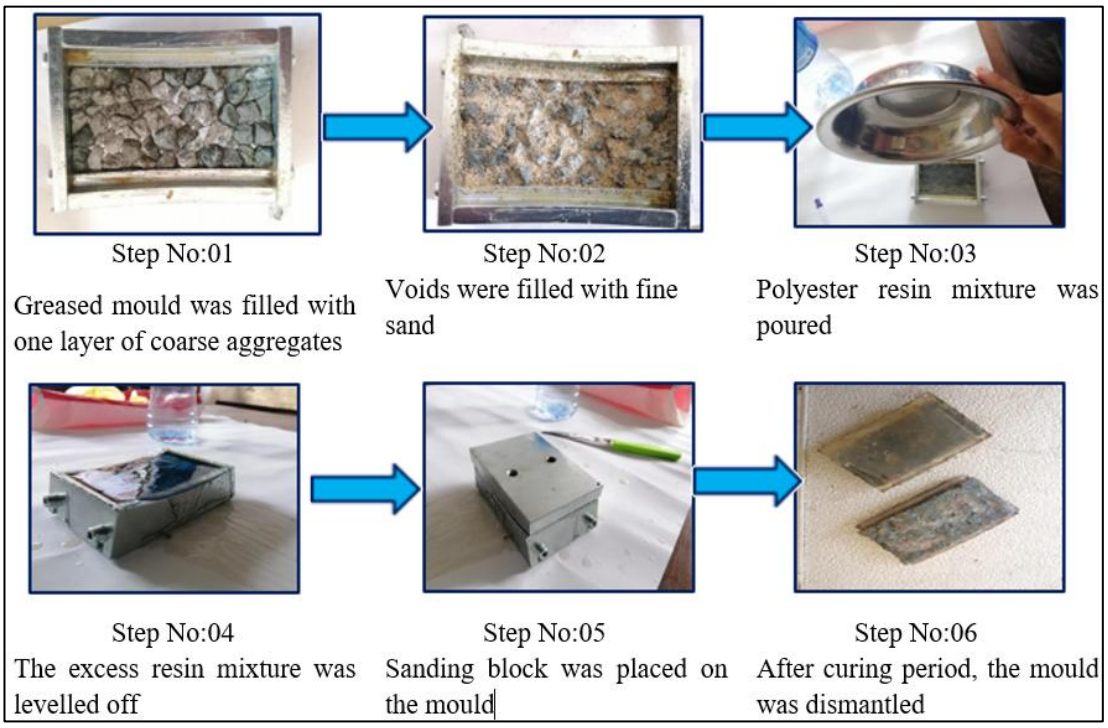


Fig. 4.3: Preparation of polishing British Wheel specimens

4.3.2 Calibration of the Machines and the Reference Graph of Polishing

The BPT was calibrated in accordance with the ASTM – E303 -93 - Standard Test Method for Measuring Surface Frictional Properties Using the BPT, with the following test parameters, as shown in Fig. 4.4 [46].

- Mass of the pendulum with slider and slider mount = 1490.9 g (1470 -1530 g)
- Distance from the center of gravity of the pendulum from the center of oscillation = 412 mm (406 – 416 mm)

- Slider load (pendulum with slider and slider mount) when clamped to the BPT = 2503. g (2400 – 2600 g)



Fig. 4.4: Calibration of the BPT as per ASTM – E303 -93

All the test parameters were well within the specified range, therefore the BPT was in conformity for the successive measurements of the research study.

In case of the accelerated polishing machine, the dimensions of the Cylindrical Wheel, or the road wheel were checked, which was 44.45 mm in width and with a diameter of 406.4 mm. The rubber-tired wheel was measured to be 50.8 mm in width with a diameter of 203.2 mm, which were conforming to the standard values specified both in the ASTM and BS standards. The rubber-tired wheel was conditioned with 14 test specimens (10 number of previously polished test specimens, 02 number of unpolished Criggion stone specimens and 02 number of unpolished control stone specimens) to achieve an equivalent running tire thread condition with a mounted load of 725 N for a total of 06 hours with a rotational speed of the road wheel of 320 rpm. Corn emery was fed in the rate of 30 g/min with water at a rate of 25 g/min consecutively for 03 hours after which emery flour was fed in the rate of 4.0 g/min with water at a rate of 4.0 g/min consecutively for 03 hours during the treating process.
The smaller rubber slider with the following dimensions of breadth = 25.4 mm, width = 31.75 mm and a thickness = 6.35 mm, was used for the BPT readings as specified in the PSV test. The contact path length of the slider on a curved test specimen was adjusted to be 76 mm before the measurement of the friction values. Rubber slider was treated by sliding it off on a previously polished specimen for 25 times and the free slider movement was also checked where the friction pointer reached the 0 mark on the friction scale.

The friction deterioration curve of the control stone as illustrated in the Fig. 4.5, ranges from an average initial BPN of 70 to a concluding 40 at the end of the 10-hour polishing period, with an average BPN of 55 recorded after polishing of 06 consecutive hours.

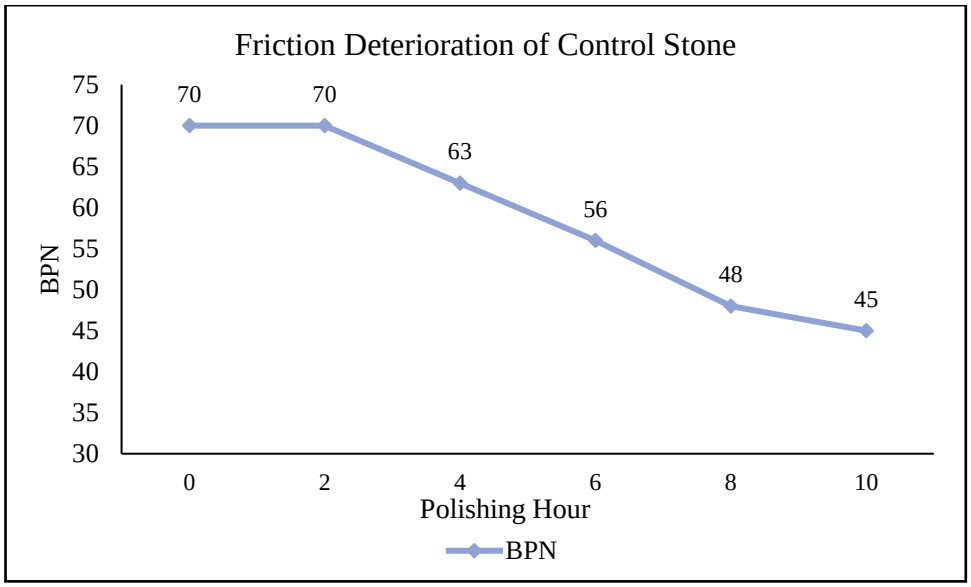


Fig. 4.5: Friction deterioration curve of the control stone

Hence, the calculated PSV value assuming the PSV of the control stone is 55 (the PSV of control stone originated in U.K is from 50 - 60), is 56, which is within the standard specified values of 50 -60 [manual].

$$PSV = S + 56 - C$$

$$PSV = 55 + 56 - 55 = 56$$

Therefore, it can be concluded that the accelerated polishing machine was calibrated producing acceptable and representative polishing action on the coarse aggregates, whereas the calibrated BPT was producing accurate readings. Similarly, the PSV value of the friction tester reference stone was also measured to be 59, which was in the expected range for a Criggion stone, reiterating the reliability of the conducted PSV test [27]. Therefore, the laboratory testing was verified to be reliable for the determination of the friction characteristics of Sri Lankan coarse aggregates.

4.3.3 Polishing of the Aggregate Specimens

The road wheel along with the feeding hoppers of the accelerated polishing machine and the rubber slider and the BPT curved specimen holder were well washed with water at room temperature to ensure absence of free residue particles from the earlier testing. 14 specimens (12 specimens from local quarries with two per each location, 02 specimens with the control stone of calculated PSV 56) whose identification number and direction of rotation had been marked on the back, were clamped onto the periphery of the rood wheel with the support of rubber “O” rings, so that the circumferential top surface of all 14 specimens would create an even running surface for the mounted rubber tire. The feeder units were adjusted with a rate of 5 g/min for the Aluminum Oxide polishing agent (150 µm graded) equivalent to silicon carbide grit, while the water flow was at 50 g/min. The clockwise rotation speed of the road wheel was set to 320 rpm with a mounted surcharge load of 725 N. Initially the specimens were subjected to a polishing of 02 hours for conditioning, after which the

cumulative of 10-hour polishing was carried out with uniform intermittent intervals of 02 hours, as depicted in Fig. 4.6.



Fig. 4.6: Polishing of the specimens using the acccelerated polishing machine

Initially, and at every 02-hour interval, the friction values of the specimens were measured using the BPT. The specimens removed from the road wheel were well washed with water so that loose grit was not present on the testing surface. The BPT was checked for the verticality using spirit level bulbs and the zero F-scale pointer indication during a free slider swing was checked. The curved specimen was tightened firmly on the specimen holder of the BPT, where the slider length was adjusted so that each sliding run would result in a 76 mm long strip of contact between the rubber slider and the test specimen. Then 05 consecutive readings at the F-scale were taken by swinging the slider from its right-side horizontal position as depicted in Fig. 4.7, where the top surface of coarse aggregates were wetted with a splash of water before each run. The last three values were recorded for the calculation of the mean BPN value at the relevant polishing interval for each specimen. Similarly, the polishing and the friction value measurement were performed for all the aggregate specimens from 20 quarry locations in three different rounds of 10-hour polishing.

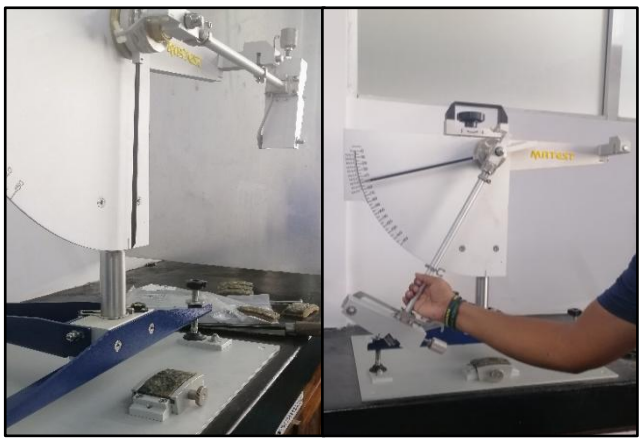


Fig. 4.7: Measurement of Friction value - BPN of the specimens using the BPT

4.3.4 PSV and Friction Deterioration Trends of Aggregate Samples

The measured initial friction number at 0-hour termed as Residual Polish Value (RPV) and the corresponding Polish Value (PVn) or British Pendulum Number (BPN) at every n (02-hour) interval during the 10-hour extended polishing under laboratory condition as per the TABLE 4.4, represent the gradual decrease in the available surface friction capacity due to applied wheel rotations on an actual flexible pavement during its operational stage from post construction phase up to major intervention stages of pavement distresses.

TABLE 4.4: BRITISH PENDULUM NUMBER (BPN) VALUES OF COARSE AGGREGATE SAMPLES

Quarry No.	BPN Value						
	0.0 hour	2.0 hour	4.0 hour	6.0 hour	8.0 hour	10.0 hour	Mean BPN
1	66	59	54	48	45	41	52
2	75	70	63	52	46	45	59
3	68	60	52	48	43	43	52
4	67	59	51	49	45	44	53
5	74	70	65	55	50	45	60
6	71	69	65	53	50	50	60
7	63	62	60	50	47	46	55
8	75	69	65	61	53	44	61
9	79	73	65	62	54	51	64
10	78	75	63	56	53	52	63
11	76	62	51	44	41	41	53
12	72	60	56	51	50	50	57
13	70	61	59	55	46	45	56
14	72	69	53	47	45	45	55
15	75	70	61	50	45	43	57
16	65	57	52	45	40	40	50
17	72	65	59	54	50	48	58
18	62	59	56	51	45	45	53
19	77	66	58	54	51	50	59
20	78	73	66	55	53	53	63

The BPN value at 06-hour was considered in the evaluation of the respective PSV for each specimen with the deduced PSV of the control stone of 56. Apart from PSV, additional influential parameters of polishing:

- Initial friction value before application of accelerated wheel rotations (RPV_{00})
- Terminal friction value at the completion of accelerated wheel rotations (PV_{10})
- Average friction value (Mean BPN)

- Friction number where stabilisation of the polishing behaviour of coarse aggregates was achieved with no/lesser rate of friction deterioration (BPN at start of stabilization)

were identified to further support and interpret data and findings, as shown in the TABLE 4.5.

TABLE 4.5: PSV OF COARSE AGGREGATE SAMPLES

Quarry No.	PSV	RPV_{00}	PV_{10}	Mean BPN	Starting hour of stabilization	BPN at start of stabilization
1	48	66	41	52	10	41
2	52	75	45	59	08	46
3	48	68	43	52	08	43
4	49	67	44	53	08	45
5	55	74	45	60	10	45
6	53	71	50	60	08	50
7	50	63	46	55	08	47
8	61	75	44	61	10	44
9	62	79	51	64	10	51
10	56	78	52	63	08	53
11	44	76	41	53	08	41
12	51	72	50	57	08	50
13	55	70	45	56	08	46
14	47	72	45	55	08	45
15	50	75	43	57	08	45
16	45	65	40	50	08	40
17	54	72	48	58	08	50
18	51	62	45	53	08	45
19	54	77	50	59	08	51
20	55	78	53	63	08	53

Majority of natural rock formations in Sri Lanka, specifically those sourced for pavement construction is originated with high grade metamorphic rocks mostly linked to varieties of gneiss, which is considered to be equivalent and thus often misunderstood in nature to that of granites with igneous rock origins [35]. It has been well documented in literature through laboratory findings as well as firmly established with a wide variety of applications within the construction industry, that gneiss-based coarse aggregates perform well in flexible pavement construction with intermediate levels of resistance to weathering, wear and abrasion [17]. Although research studies to read and establish their resistance to polishing is comparatively scarce, the gneissic road surfacing aggregates exhibit medium quality in friction retention, in comparison to granites which are considered high friction roadstones with PSV higher than 60 [1]. PSVs of the test specimens of this study range from 45 to 60 implying fairly good

quality coarse aggregates in terms of polishing resistance, in par with the above analysis of Sri Lankan status.

Mean BPN provides a lesser representative picture in terms of friction retention of coarse aggregates, since most of the test specimens showcase a comparatively higher average BPN closer to PSV 60 in value, indicating higher friction category. Yet, the majority of PSV of the study samples is scattered around PSV 45 - 55, which is the acceptable range for a gneiss-based coarse aggregates [1]. In the point of view of road user safety and pavement friction management of a road section, the ability of road surfacing aggregates to resist polishing and retain at least a minimum acceptable friction capacity under traffic application up to the end of service period, is more crucial than having pavements with higher friction capacities during the starting years of operational period but with a higher rate of friction deterioration trend resulting in a very low friction capacity at the regular maintenance intervention stages. Therefore, rather than an average friction number, a terminal friction value along with the time/number of traffic application required to attain that terminal value will be more usable in the decision-making process. Thus, PSV becomes more suitable as a test parameter to identify the friction retention capability of coarse aggregates as it measures the polishing resistance at a designated wheel rotation hour to represent latter stages of in-service pavement life. Further, the friction value at the stabilization phase of the polishing properties of coarse aggregates with no significant post friction deterioration trend/rate, is appropriate in the same sense.

Hence, the combination of PSV and BPN at start of stabilization is found to a be more representative parameter to better read the research data set and to establish the polishing characteristics and friction deterioration trends of Sri Lankan coarse aggregates.

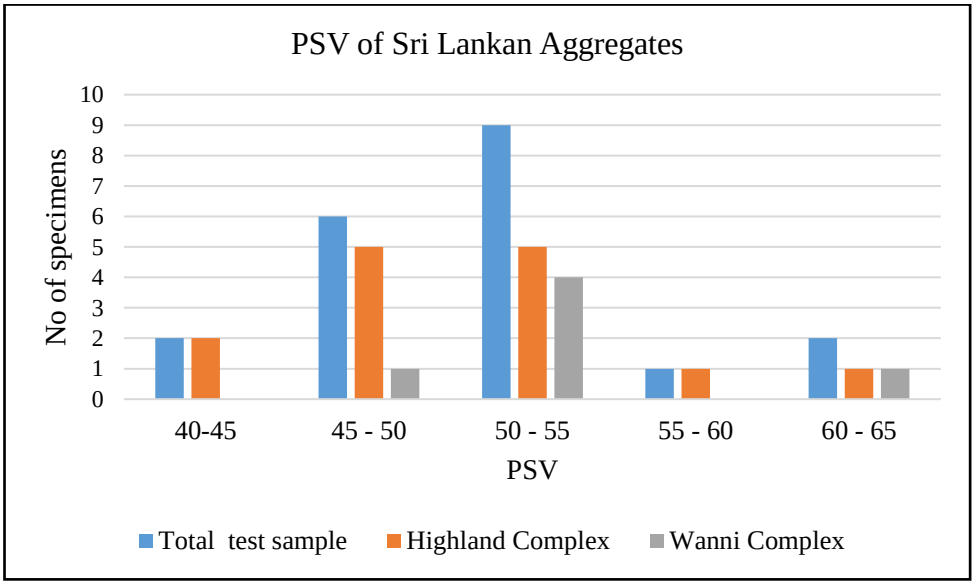


Fig. 4.8: Distribution of Polish Stone Value of the test specimens

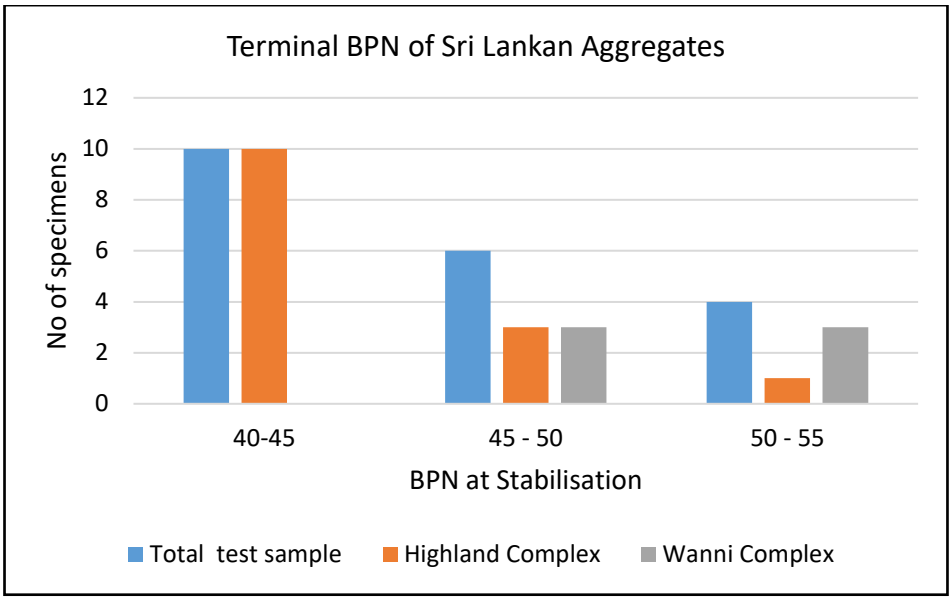


Fig. 4.9: Distribution of BPN at the Stabilisation stage of the test specimens

As far as the distribution histogram in the Fig. 4.8 is considered, where the PSV 50-55 range records the highest concentration in both geo-complexes, it is apparent that the sample test specimen falls into intermediate friction category with moderate/medium levels of polishing resistance. Distribution of BPN at the Stabilisation stage of the test specimens which can be considerd as a terminal PSV value of aggregates as in Fig. 4.9, further reiterates the above statement with most of the aggregate specimens showcasing terminal PSV in the range of 40 – 45. While the above discussed PSV and terminal PSV are idealistic in describring friction retention capacity, the friction deterioration trends of the different quarry samples can be vividly analysed using the following graphical illustrations as in Fig. 4.10 - Fig. 4.14.

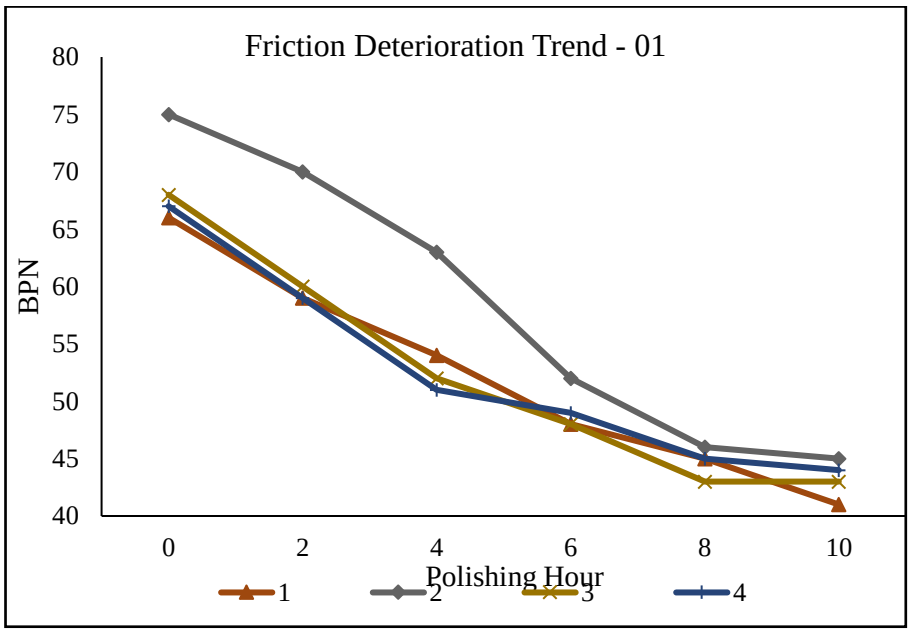


Fig. 4.10: Friction deterioration trends of the quarry samples from 01 - 04

As far as the degradation patterns are concerned in the Fig. 4.10, it is evident that quarry samples from 01, 03 and 04 exhibit very similar trends with PSV values converged around 49, whereas the quarry No. 02 deviates with a higher initial BPN value of 75, a PSV higher than 50 and a terminal BPN of 45, indicating comparatively stronger friction retention capacity for a gneissic based aggregate. Yet, it undergoes a higher rate of friction reduction than all the other samples during the polishing process between 04 to 06 hours.

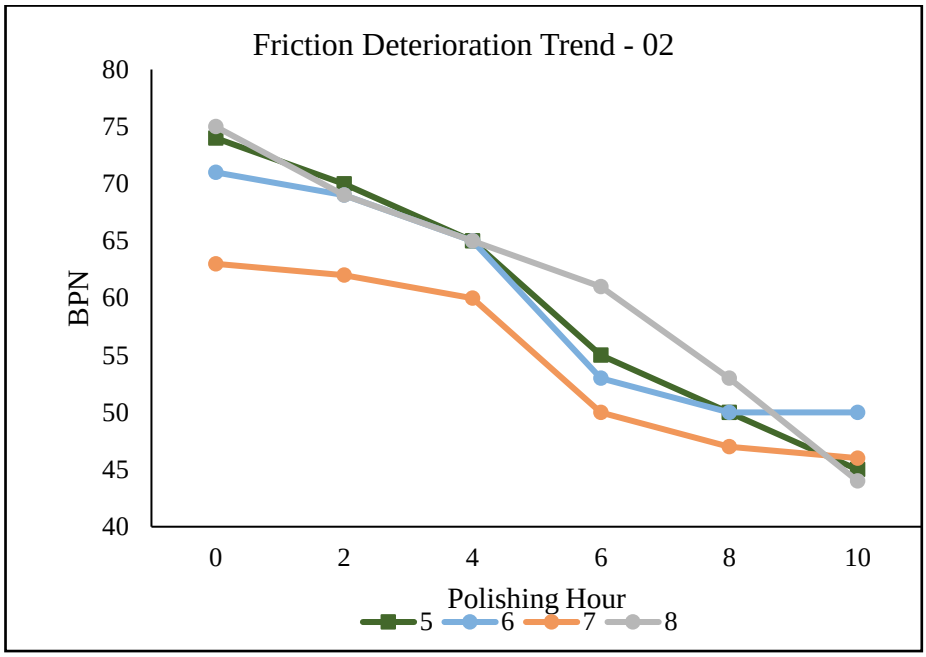


Fig. 4.11: Friction deterioration trends of the quarry samples from 05 - 08

All the samples from quarry No. 05 to 08 showcase a higher PSV greater than 50 in comparison to the previous data set. In the Fig. 4.11, the highest initial BPN, PSV values and BPN at stabilization stage are recorded for quarry No. 05 and 08 compared to others, yet fail to achieve a higher terminal friction capacity. While quarry No. 07 records the lowest friction values for all the categories, quarry No. 06 exhibits stronger resilience having a BPN range of 71 to 50 with PSV of 53, indicating comparatively higher fiction retention capacity under trafficking.

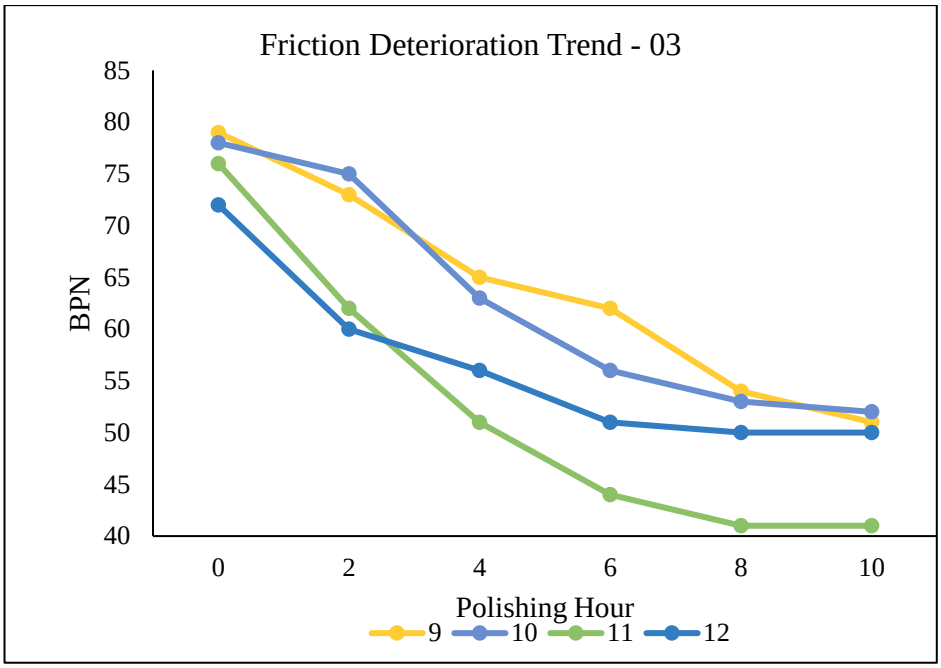


Fig. 4.12: Friction deterioration trends of the quarry samples from 09 - 12

When the friction deterioration trends in the Fig. 4.12 are considered, the highest overall friction capacity is demonstrated by the quarry samples from No. 09 and 10 with the highest recorded range of BPN of 80 to 50 leading to the highest PSV among all the quarry samples from 01 – 20. Although the quarry No. 11 has an initial BPN of 76, due to the high rate of degradation of friction capacity, it eventually ends up with the lowest PSV among the twenty quarries. Quarry No. 12 has satisfactory friction retention characteristics.

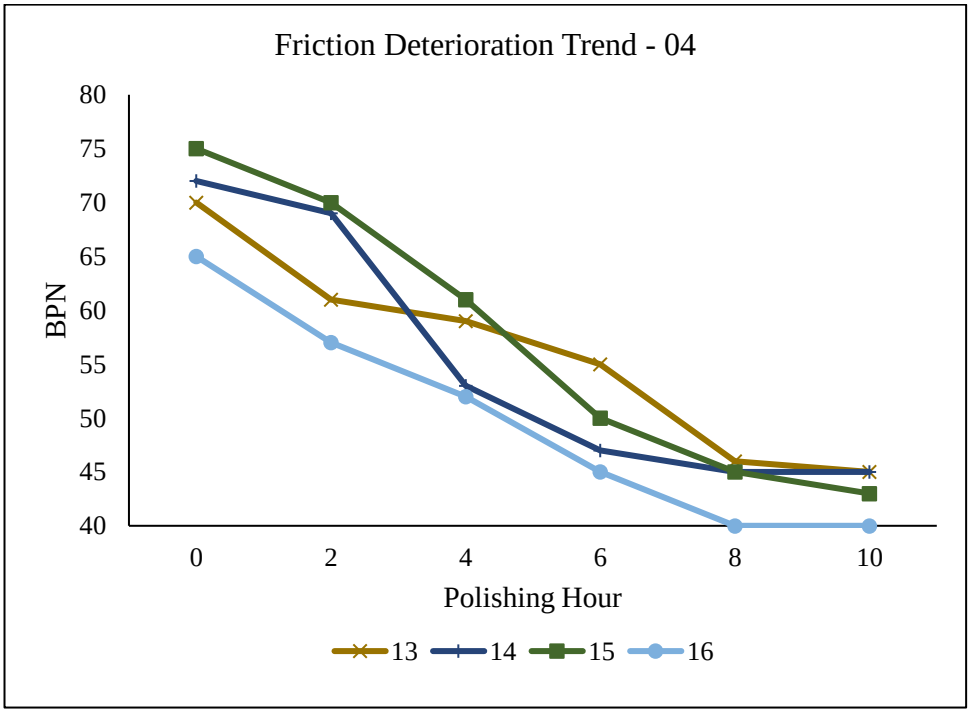


Fig. 4.13: Friction deterioration trends of the quarry samples from 13 - 16

While the samples from quarries No.13 to 15 demonstrate acceptable friction retention capacities as in the Fig. 4.13, the quarry No. 16 produces the second lowest PSV of 45 with the lowest recorded BPN of 40 at the stabilization and terminal stage out of all the test samples.

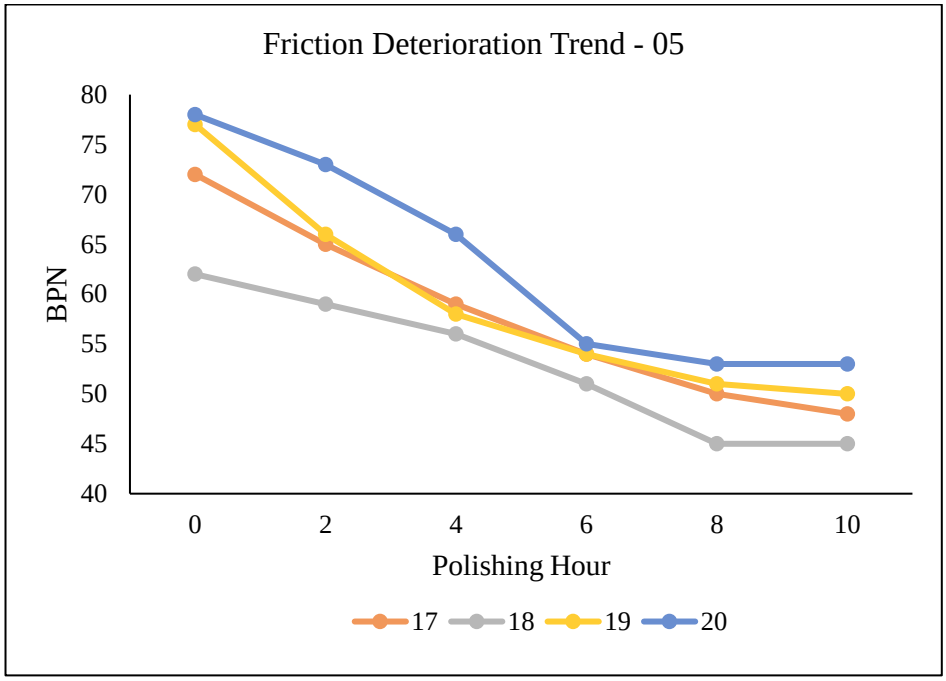


Fig. 4.14: Friction deterioration trends of the quarry samples from 17 - 20

When the patterns in the Fig. 4.14 are concerned in terms of overall friction capacity, the second highest BPN range after the quarry samples from No. 09 and 10, is recorded by the quarry samples from No. 19 and 20. Quarry No. 17 has satisfactory friction retention characteristics. Despite possessing the lowest initial BPN out of all the twenty quarries, the samples from quarry No. 18 maintain a satisfactory friction deterioration pattern with PSV of 51 and a terminal BPN 0f 45 as a result of its lower rates of reduction trends of polishing resistance.

In conclusion, the best quality coarse aggregates in terms of polishing resistance, are those extracted from the quarry locations of No. 09, 10, 19 and 20 with the highest BPN at initial, stabilization and terminal stages, where quarry No. 09 records the highest PSV (62) and the initial BPN (79) out of all the locations. The second highest PSV of 61 belongs to the quarry No. 08 with a comparatively satisfactory friction retention capacity. Out of the tested gneissic based aggregate specimens, the most unsatisfactory quality coarse aggregates with the lowest PSV of 44 and 45 respectively, are those extracted from the quarry locations of No. 11 and 16 with the lowest BPN at stabilization and terminal stages as well, where quarry No.11 demonstrates the highest rate of friction reduction. Similarly, quarry No. 01,03 and 04 exhibit the second lowest degrees of quality in terms of polishing resistance. Although the quarries No 18 and 07 record the lowest initial BPN of 62 and 63 respectively, towards the latter stages of their friction deterioration patterns, these aggregates

maintain comparatively finer retention capacities. Nevertheless, since the above analysis of results are within the sphere of coarse aggregates with metamorphic rock origins, as mentioned earlier all the quarry samples showcase medium levels of friction retention capacities in comparison to those of igneous rock origins.

4.4 Evaluation of Textural Properties

Having established the polishing characteristics of Sri Lankan aggregates, subsequently the textural characteristics were to be examined, since PSV value and initial friction capacity of coarse aggregates are heavily dependent on spread of microtextural asperities and the Skid Resistance number is highly correlated with macro-texture of the pavement surface. Hence, at each polishing intervals of 02 hours, apart from measurement of the BPN, the above two textural properties of the test specimens should be measured.

The depth measuring devices available at the University of Moratuwa suitable for the above purpose such as Digital Automatic Coordinate Measuring Machine with 1µm accuracy and 3D Image Laser Profiler, which would provide the required X, Y and Z coordinates for a focused surface point on test specimens, were out of service due to technical issues owing to non-usage during the Covid -19 wave. Therefore, as an alternative, a manually operated coordinate measuring gauge as depicted in the Fig 4.14, with 1µm accuracy was used to measure micro and macro texture in terms of root- mean square height (Rq) as in equation (19) [25,47,48].

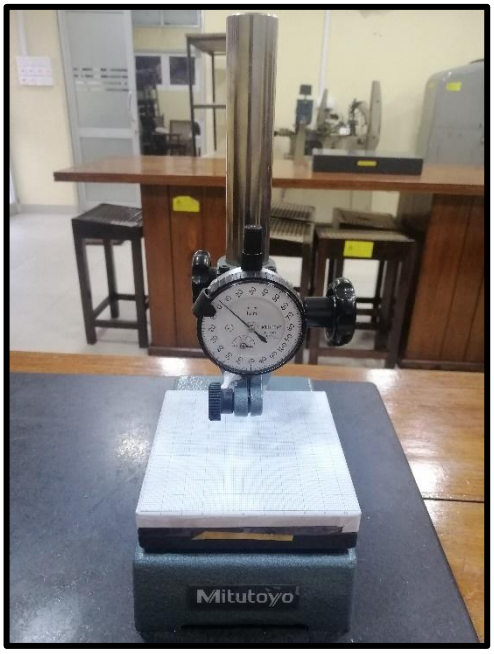


Fig. 4.15: Coordinate Measuring Machine/Guage used for texture measurement

When Z values are in the wave frequency of 0.5 – 1.0 mm, Rq produces a height difference to represent micro texture, whereas macro texture can be modelled by Rq values with Z values in the wave frequency of 10.0 mm on the top surface of the aggregate test specimen.

$$R_q = \sqrt{\frac{1}{N}\sum(Z_i - \bar{Z})^2} \quad (19)$$

Where,

N = Number of points

Z_i = Generic ordinate of height (reference point)

$\bar{Z}$ = Average of $Z_i$values

The miniature pointer sensitive to very small changes in height can be moved manually so that the dial gauge would indicate the height difference with respect to a reference point, which should be considered as zero/origin of the assumed coordinate system. A graph sheet of 1.0 mm (one separate sheet per each test specimen) was attached to the top of the base plate of the instrument to ensure measurement of the same designated point of an aggregate in each polishing interval. The X, Y coordinates of the boundary relevant to the impression on the graph sheet by each individual test specimen were marked in each graph sheet, so that no deviation occurred in consecutive texture measurement in polishing intervals. Two surface aggregates adjacent to each other on a same horizontal line were selected for each test specimen and the readings for height difference at 0.5 mm and 10.0 mm gaps along the length of the selected two aggregate particles for two parallel lines, were taken with respect to the reference point. Same procedure was repeated for each test specimen after each polishing hour.

The above manually operated measurement was highly time consuming, therefore for texture measurement, the aggregate samples from the quarries of the districts with the highest annual aggregate production situated within the largest geological complex in Sri Lanka were selected as given in the TABLE 4.6. [17,35]

TABLE 4.6: DETAILS OF SAMPLES FOR TEXTURE MEASUREMENT

Quarry No.	Source of Aggregate	District	Geological Complex	Code
6	Ganewalpola 01	Anuradhapura	Wanni	A
1	Horana 01	Kalutara	Wanni	B
4	Kaduwela 01	Colombo	Wanni	C
9	Ganewalpola 02	Anuradhapura	Wanni	D
16	Horana 02	Kalutara	Wanni	E
5	Kaduwela 02	Colombo	Wanni	F

Four curved polishing samples were prepared for each location and friction measurement was conducted using the accelerated polishing machine and BPT as previously described. At every 1.5-hour intervals of polishing, friction and texture values were recorded. Friction deterioration trends of the specimens A, B, C, D, E and F result in PSV values of 53, 44, 49, 62, 45 and 55 as well as BPN at the stabilization

stage of 50, 41, 45, 44, 40 and 45 respectively as shown in the Fig. 4.16 and Fig. 4.17. With medium range friction retention capacities, these metamorphic grade rock samples show very similar patterns in the point of view of gneiss based coarse aggregates in the determination of the polishing characteristics.

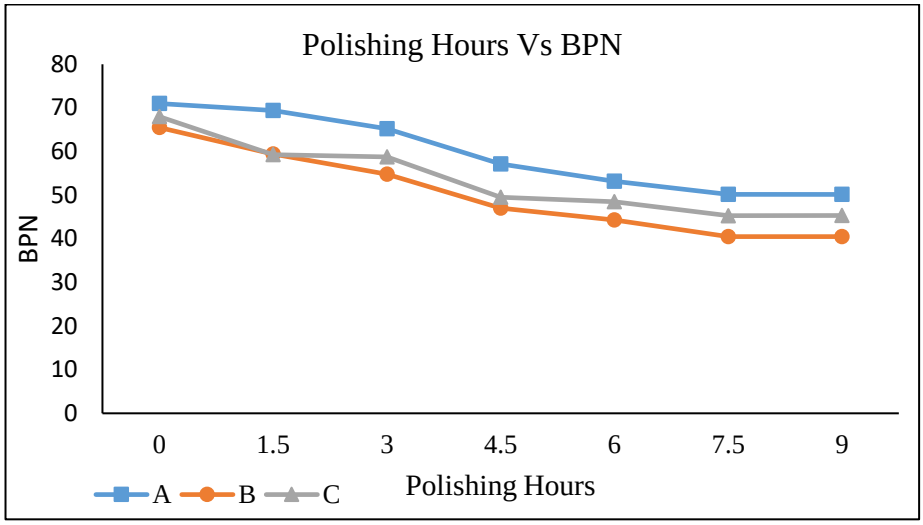


Fig. 4.16: Friction deterioration trends of the quarry samples from A - C

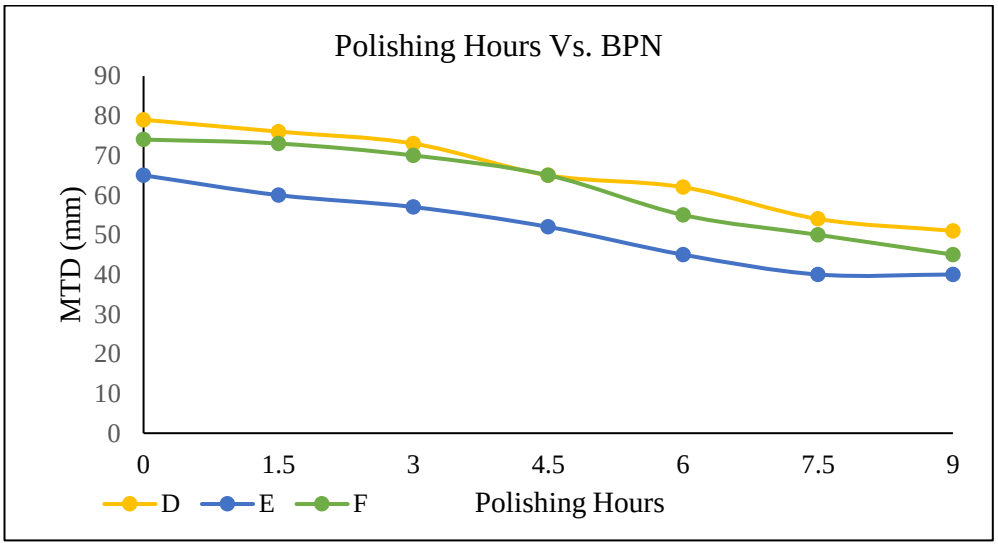


Fig. 4.17: Friction deterioration trends of the quarry samples from D - F

4.4.1. Determination of Micro-texture of the Coarse Aggregates

The observed micro-texture against the number of polishing hours for the selected three quarries, is detailed in the TABLE 4.7, where one texture value represents an average of height differences of the focused two adjacent aggregate particles measured along two parallel lines with a 0.5 mm horizontal gap between each reading point on one line. Each Rq data point refers to the average of the texture values of 08 different specimens per each location. The polishing intervals were limited to 09 hours of extended polishing due to the tedious and time-consuming nature of the texture measurement. The curves of each sample in the Fig. 4.18 display similar deterioration

patterns to that of the polishing trends as in the Fig. 4.16 and Fig. 4.17, with an accelerated attenuation from 0 – 03 hours, decelerated attenuation from 03 – 06 hours and the stabilization stage initialising from 7.5-hour onwards.

TABLE 4.7: MICRO-TEXTURE VALUES OF COARSE AGGREGATE SAMPLES

No. of Wheel Rotations	Polishing Hours	A & D (*A*)	B & E(*B*)	C & F(*C*)
		Rq (µm)	Rq (µm)	Rq (µm)
0	0	150.9	138.9	171.7
28800	1.5	137.0	122.8	150.7
57600	3	123.9	110.9	127.2
86400	4.5	108.4	102.8	107.6
115200	6	97.4	90.1	89.0
144000	7.5	94.6	81.2	79.7
172800	9	92.9	76.6	74.9

Although sample C and F starts with a higher texture value, sample A and D leads to a more consistent controlled retention trend with higher texture vales at the 06-hour interval and stabilization stage than all the other samples. This clearly explains the observed higher PSV and terminal BPN values of A and D, since the other specimens manifest higher rates of texture reduction similar to that of friction reduction, resulting in low friction retention capacity.

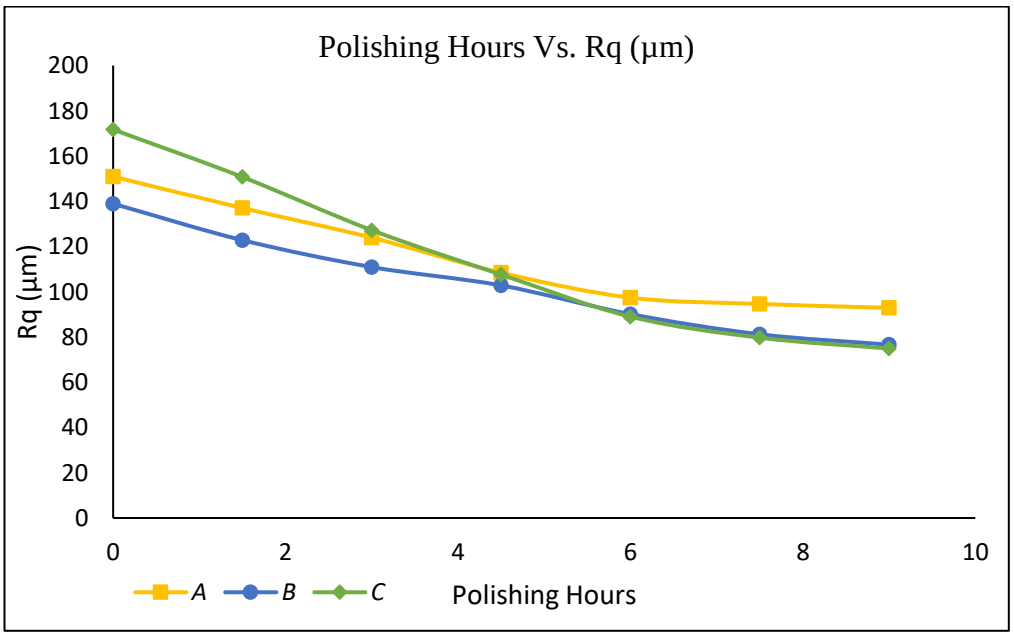


Fig. 4.18: Micro-texture deterioration trends of the quarry samples

In the simple regression analysis between the BPN and Rq values during polishing, as illustrated in the Fig. 4.19, all the three locations lead to a strong linear relationship with impressive square of correlation coefficient (R^2) values very closer to 01, as tabulated in the TABLE 4.8. Hence, it is evident and well establishes the fact that the micro-textural asperities of coarse aggregates heavily govern their polishing resistance, since friction retention capacity greatly relies on the distribution patterns of the mineral matrix [6].

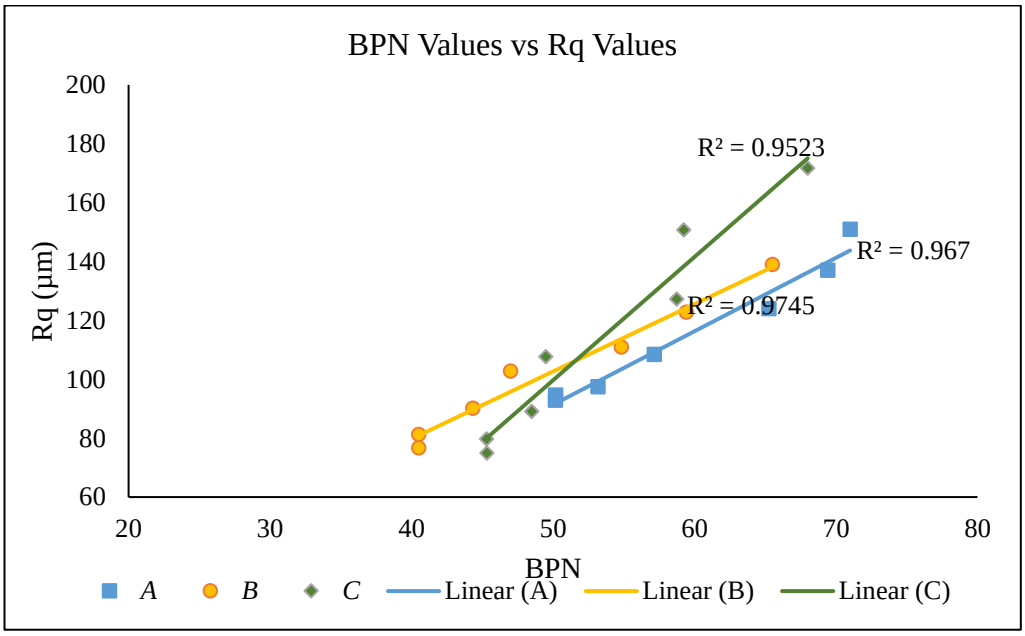


Fig. 4.19: Correlation between Friction values and Micro-texture of the quarry samples

In conclusion, higher micro-texture leads to a higher polishing resistance therein higher BPN and vice-versa, but more importantly low rate of micro-texture reduction with higher resilience against polishing action, is much crucial leading to better friction retention characteristics of natural coarse aggregates.

TABLE 4.8: R SQUARE VALUES OF CORRELATION BETWEEN FRICTION VALUES AND MICRO-TEXTURE

Location No	Correlation between BPN and Micro-texture (R^2)
A	0.967
B	0.975
C	0.952

4.4.2. Determination of Macro-texture of the Coarse Aggregates

The observed macro-texture against the number of polishing hours for the selected three quarries, is detailed in the TABLE 4.9, where each MTD data point refers to an average of height differences of the focused two adjacent aggregate particles measured along two parallel lines with a 10.0 mm horizontal gap between each reading point on

one line. The curves of each sample in the Fig. 4.20 and Fig. 4.21 display similar deterioration patterns to that of the polishing trends as in the Fig. 4.16 and Fig. 4.17, but with no significant highlighted regions of accelerated attenuation and decelerated attenuation or the stabilization stage.

TABLE 4.9: MACRO-TEXTURE VALUES OF COARSE AGGREGATE SAMPLES

No. of Wheel Rotations	Polishing Hours	A	B	C	D	E	F
		MTD (mm)	MTD (mm)	MTD (mm)	MTD (mm)	MTD (mm)	MTD (mm)
0	0	0.878	1.009	1.028	0.661	0.918	0.952
28800	1.5	0.776	1.007	0.932	0.627	0.848	0.868
57600	3	0.622	0.936	0.849	0.582	0.827	0.751
86400	4.5	0.55	0.879	0.724	0.544	0.775	0.629
115200	6	0.522	0.756	0.645	0.506	0.739	0.600
144000	7.5	0.496	0.634	0.544	0.470	0.624	0.526
172800	9	0.476	0.487	0.460	0.434	0.522	0.402

SPM is the most widely used volumetric approach to measure MTD both in the laboratory and field conditions, while profilometers, laser/3-D measurement methods record variations over a profile/surface, producing MPD values, which are usually lower than MTD values [1,19]. In addition, the curved specimens were prepared carefully so as not to have higher irregularities among aggregate particles, to avoid bumping of the polishing rubber tire. Therefore, MTD values of the curved specimens measured in the study are comparatively lower than those of actual pavement wearing course, which usually lies in the range of 0.6 mm – 4.00 mm [20].

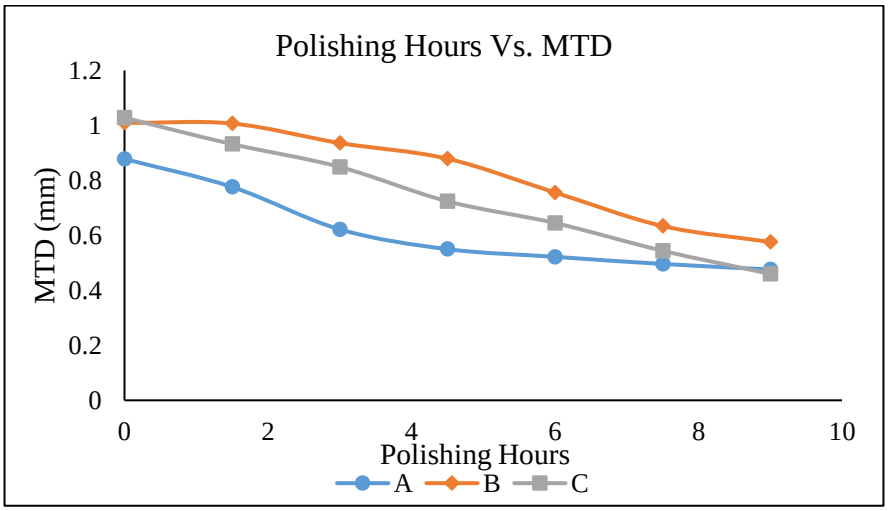


Fig. 4.20: Macro-texture deterioration trends of the quarry samples from A - C

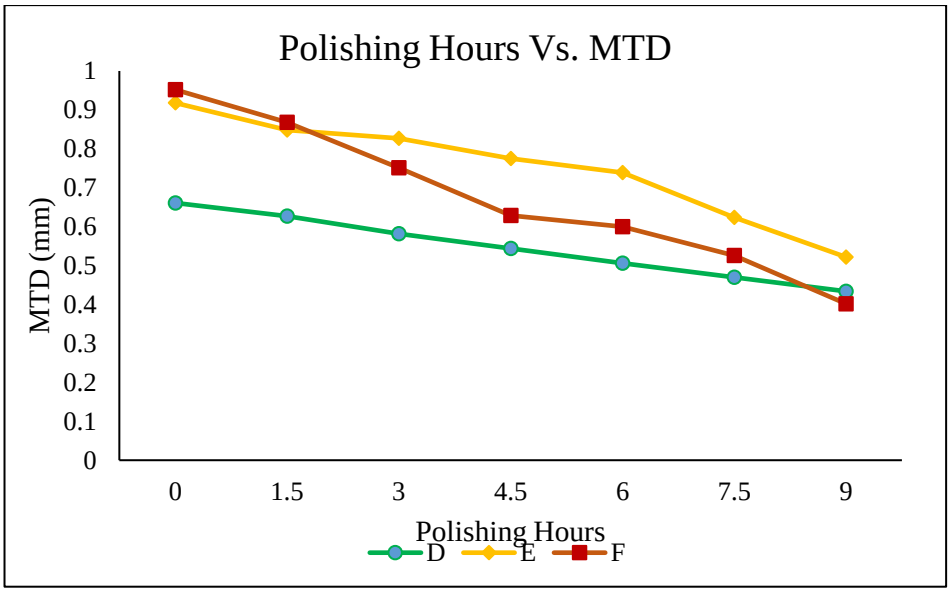


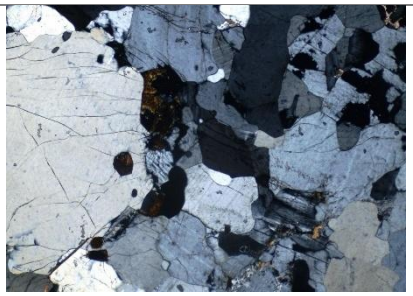
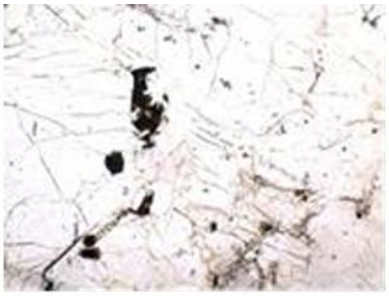
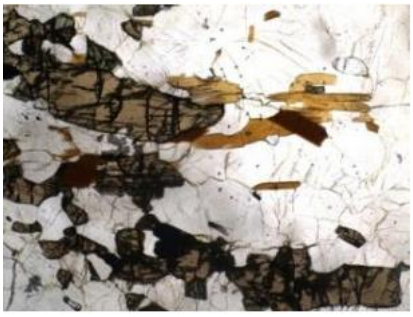
Fig. 4.21: Macro-texture deterioration trends of the quarry samples from D - F

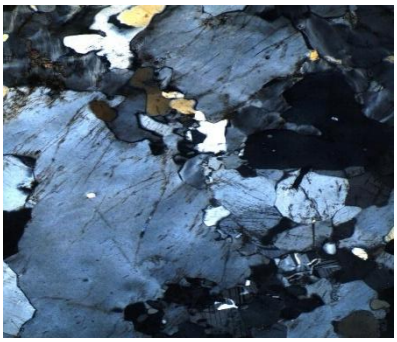
The BPN is more sensitive to micro-textural irregularities of coarse aggregates of a HMA pavement since it measures friction at very lower speeds close to 20 km/h [21], therefore does not heavily depend on macro-textural variations of the pavement. At higher speeds more than 60 km/h, MTD plays the key role in creating frictional grip between the vehicle tire rubber and the pavement, thus it correlates well with friction values measured at higher speeds [1,4]. Micro-texture and mineralogy implicate a property of coarse aggregates, whereas macro-texture represents overall pavement characteristics, resonating rather with factors like aggregate gradation, bitumen/HMA spraying rate and construction practices [1,4]. In the study, to assess polishing resistance of aggregates, micro-texture and petrography evaluation were conducted for they are the most appropriate parameters to identify the desired aggregate property, while MTD values at different polishing intervals are required to calculate the SN of the pavement in the determination of the friction prediction model.

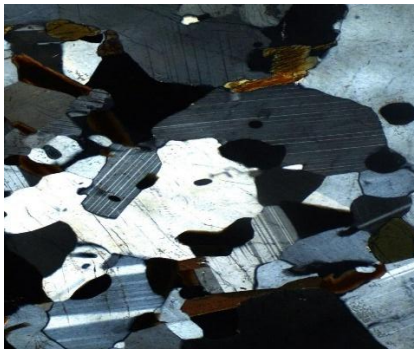
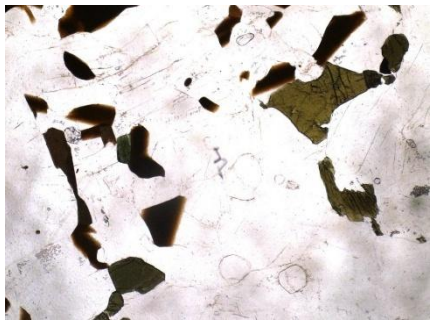
4.5 Evaluation of Mineralogical Properties

In order to further support and fortify the findings of behavioural patterns of Sri Lankan mineral aggregates in terms of polishing resistance, mineralogical and petrographic analysis of the aggregate samples selected for micro-texture measurement, was conducted by the means of a simple thin section analysis. Thin sections with a 30 µm thickness were prepared by cutting, grinding and polishing 10 mm chips of the selected aggregates. These thin sections were observed using the Olympus BH polarizing microscope attached to a microscope video camera, where the types of minerals present and their percentage were deducted and calculated according to Michel-Levy Interference Colour Chart, presented in TABLE 4.10.

TABLE 4.10: THIN SECTION ANALYSIS RESULTS OF COARSE AGGREGATE SAMPLES

S/N	Thin Section Observation	Mineralogy (Rock Type) and Petrography
A	 	Quartz, Feldspar, Biotite (Quatzo feldspathic gneiss) Feldspar ($KAlSi_3O_8$ –$NaAlSi_3O_8$–$CaAl_2Si_2O_8$), Plagioclase ($NaAlSi_3O_8$ –$CaAl_2Si_2O_8$ and Quartz (SiO_2) are the major minerals observed. Both plagioclase and alkali feldspar are present Feldspar with grain size between 8-15 mm, makes up about 70 % of the rock. Little amount (<10%) of quartz with grain size between 2-3 mm can be observed.
B	 	Plagioclase, Quartz, Biotite, Pyroxene (Biotite gneiss with pyroxene) Plagioclase ($NaAlSi_3O_8$ – $CaAl_2Si_2O_8$),Biotite ($K(Mg,Fe)_3AlSi_3O_{10}(F,OH)_2$) , Pyroxene ($XY(Si,Al)_2O_6$) and Quartz($SiO_2$) are the main minerals observed. Biotite grains larger than 3 mm in size, make up about 40 % of the matrix. Plagioclase grains are approximately 2 mm in size and make up 35 %, while Quartz grains between 1mm- 3mm has a percentage of about 20 %.

C	 	Plagioclase, Quartz, Biotite (Biotite gneiss) Plagioclase (NaAlSi3O8 – CaAl2Si2O8), Biotite (K(Mg,Fe)3AlSi3O10(F,OH)2) and Quartz (SiO2) are the major minerals. Biotite with grain size around 2 mm makes up 30 %. Both alkali feldspar (KAlSi3O8–NaAlSi3O8–CaAl2Si2O8) and plagioclase can be seen. Feldspar with grain size ranging between 3-10 mm, makes up 40% of the rock. Plagioclase size ranges between 3-5 mm and Quartz grains (1-2 mm) make up about 15 % of the rock
D	 	Quartz, Feldspar,Biotite, Hornblende (Quatzo feldspathic hornblende gneiss) Feldspar ($KAlSi_3O_8$ –$NaAlSi_3O_8$–$CaAl_2Si_2O_8$), Biotite ($K(Mg,Fe)_3AlSi_3O_{10}(F,OH)_2$) Quartz ($SiO_2$) and Hornblende ($(Ca,Na)_{2-3}(Mg,Fe,Al)_5(Al,Si)_8O_{22}(OH,F)_2$) are the major minerals. Feldspar grain size varies between 3-10 mm and both alkali and plagioclase feldspars are observed. Feldspars make up 60% of the rock. Quartz grains very between 1-3 mm and take 20% in composition. Biotite grains are 2-3 mm in size and make up 15 %

E	 	Plagioclase, Quartz, Biotite, (Biotite gneiss) Plagioclase ($NaAlSi_3O_8$ – $CaAl_2Si_2O_8$), Biotite ($K(Mg,Fe)_3AlSi_3O_{10}(F,OH)_2$) and Quartz (SiO2) are the major minerals found in this sample. More than 40% of the rock is made up of fine-grained biotite with grain size around 1-2 mm. Quartz ranges between 1-3 mm and make up about 30%. Feldspar makes up the rest of the samples and size range is 2-3 mm.
F	 	Plagioclase, Quartz, Biotite (Biotite gneiss) Plagioclase ($NaAlSi_3O_8$ – $CaAl_2Si_2O_8$), Biotite ($K(Mg,Fe)_3AlSi_3O_{10}(F,OH)_2$) and Quartz ($SiO_2$) can be seen as main minerals in this sample. Plagioclase makes up 50% of composition and varies between 1-4mm in size. Biotite grains are fine and make up 30 % of the rock while Quartz and alkali feldspar make up the rest of the rock

The rock type of aggregates from source A and D, is Quatzo feldspathic gneiss with a higher amount of Feldspar, which is a very coarser mineral with blocky and tabular crystals usually leading to a Mohs scale of 6 [1,6,29]. Igneous quartz with a Mohs scale of 7, are coarser than Feldspar with usual grain sizes from 5 - 30 mm, leading to very good polishing resistance [1,6,29]. However, here quartz act as a finer matrix filling the gaps between coarser Feldspar minerals. Therefore, the siliceous quartz present in low amount is finer in grain size comparatively due their recrystallisation in this particular metamorphic rock [1,6,29]. Hence, stronger polishing resistance with

higher PSV value is demonstrated by sample – A and D, due to the larger presence of harder Feldspar minerals. Although the initial micro-texture values are a bit low due to the uniform distribution of abundant Feldspar minerals, still the rock is able to record the highest BPN out of all the three samples since Feldspar is very tough. As further polishing takes place, the softer minerals (quartz) are worn off exposing the harder matrix, resulting in a lower paced micro-texture deterioration leading to better friction retention [1,6,29].

The second-best polishing deterioration pattern against trafficking is manifested by sample – C and F. The Biotite gneiss rock type has considerable amounts of tougher Feldspar minerals with a slightly lesser grain size than the rock types of A and D, as well as Plagioclase-a medium coarse-grained mineral, which is harder and blocky in foliation than recrystallised quartz, with a Mohs scale of 6 [1,6,29]. However, Biotite is a fine to medium grained mineral with a Mohs scale of about 2-3 with sub tabular, corroded and flaky crystalline structure, therefore susceptible for polishing like recrystallized quartz [1,6,29]. In this particular rock type, Biotite can be considered as finer minerals due to their smaller grain size closer to 2 mm. Therein, combination of Biotite and quartz act as the finer matrix, on which the coarser matrix consisting of Feldspar and Plagioclase is embedded. The highest initial micro-texture is shown due to the above formation of the rock with a coarser matrix embedded in a finer matrix leading higher textural variations visible on a plane. But, along with the continuous application of wheel rotations, the finer matrix keeps on weathering exposing available harder minerals. Therefore, the texture reduction rate is higher, and the friction retention capacity is lower than A due to the presence of higher percentage of softer grains accounting for about half of the matrix.

The lowest friction and texture retention capability is exhibited by the roadstones – B and E, due to the absence of a harder coarser mineral like Feldspar compared to the previous examples and owing to the presence of a very high percentage of softer mineral like Biotite and recrystallized quartz of about 60 % out of the rock. The available hard mineral, which is Plagioclase has a finer grain distribution leading to a low spread of hard minerals with a large number of interstitial gaps in between, filled with softer minerals. In conclusion, all the six samples of metamorphic grade rocks possess a crystalline structure with harder minerals embedded in a softer matrix, leading to satisfactory degrees of polishing resistance, acceptable for gneissic based coarse aggregates [1,6,29].

4.6 Friction Prediction Model

Having established the friction and texture characteristics of Sri Lankan road surfacing aggregate as per the research study objectives, finally a model to predict the friction of asphalt concrete pavements constructed with locally available aggregates, needs to be formulated. The most commonly used frction/skid resistance prediction model called as Penn State Model as a substitute to IFI as in equation (20), was identified to be the

most suitable base equation to calculate friction in light of the types of collected laboratory data of the research.

$$SN = (-31 + 1.38BPN)\, e^{-0.45\, MTD^{-0.47}} \quad (20)$$

TABLE 4.11: CRITICAL SKID NUMBER OF COARSE AGGREGATE SAMPLES

Sample Code	Skid Number (SN)			
	0 Polishing Hour	6 Polishing Hour	7.5 Polishing Hour	9 Polishing Hour
A	42	23	20	20
B	38	18	15	14
C	37	20	17	17
D	45	29	23	20
E	37	19	14	13
F	45	25	21	16

The accelerated polishing test simulates the deterioration of coarse aggregates on a flexible pavement subjected to vehicular traffic during its typical span of operational life [21]. In the aggregate selection process during the design and construction of a bituminous pavement specifically in the case of expressways, critical horizontal curves, intersections and accident-prone roadway sections, it is essential to estimate the capability of the mineral aggregates to produce adequate friction or resistance against skidding on the pavement, since such data will be immensely useful in the decision making of planning and maintenance stages. Therefore, polishing hours of 0, 6, 7.5 and 9 were considered to be equivalent to the stages of a constructed wearing course in the order of the initial stage before trafficking, investigatory level, intervention level and the terminal stage. The SR available on the pavement in each of these identified stages were calculated using the Penn State model using the BPN and the MTD relevant to each polishing hour of the six specimens, at an assumed vehicle speed of 100 km/h, as given in TABLE 4.11.

A simple regression analysis was carried out to generate the correlation between PSV and SN of the six specimens for the identified three key stages of pavement as initial stage, investigatory level and the terminal phase. The curves indicate strong correlation between the two variables with coefficients of correlation being closer to 01, as shown in the Fig. 4.22. For each stage, the equation with the highest suggested coefficient of correlation, was selected as the friction prediction model, as detailed in the TABLE 4.12.

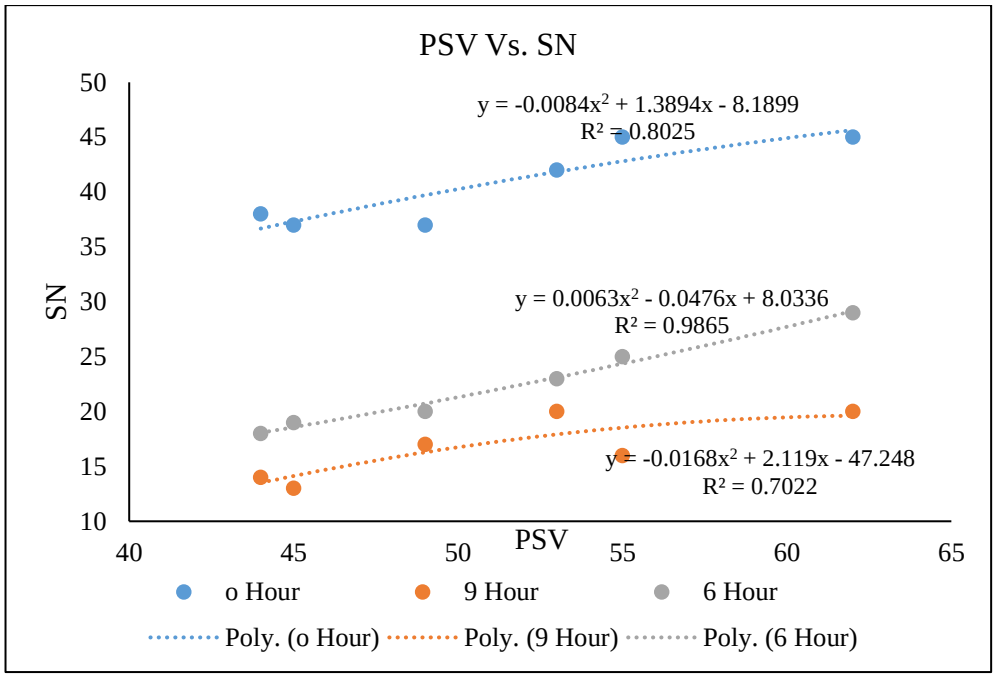


Fig. 4.22: Results of the Regression analysis between Skid Number and the Polish Stone Value

TABLE 4.12: FRICTION PREDICTION MODELS OF SRI LANKAN CORASE AGGREGATES

Stage	Friction Prediction Model	Equation No.	R^2
Initial stage	$SN_0 = -0.0084.PSV^2 + 1.3894.PSV - 8.1899$	(21)	0.8025
Investigatory level	$SN_I = 0.0063.PSV^2 - 0.0476.PSV + 8.0336$	(22)	0.9865
Terminal stage	$SN_T = -0.0168.PSV^2 + 2.119.PSV - 47.248$	(23)	0.7022

Equation (21) can be used to estimate the friction (SN) available on a flexible pavement wearing course constructed with coarse aggregates having a polishing resistance of PSV, just after construction and before trafficking application. To calculate the friction capacity of a bituminous pavement at the investigatory level of pavement friction management system, (22) is suggested, while the terminal friction capacity value of the pavement at the end of its service life will be given by (23).

For the validation of the above models, network friction data of the Southern Expressway – E01 were obtained by the courtesy of the Road Development Authority of Sri Lanka. Collected data of BPN and MTD of the expressway system, were recorded for the year 2017, after 05 years of construction and year wise data was not available. The basis for the field data collection was to obtain annual friction data for

an overlay with no maintenance or renovations done, so that SN at the initial stage, deemed investigatory and terminal stages could be calculated. Due to the absence of year-wise data, the type of stage of each expressway section was assumed based on the cumulative traffic volume over its service years after construction. Since details with regard to the coarse aggregate sources for each section was unavailable, another assumption based on the data extraction point was taken to determine the quarry location of the surfacing aggregates of the selected sections. Following expressway sections with the below detailing as shown in TABLE 4.13 were selected for the validation based on the assumptions, as given in TABLE 4.14.

TABLE 4.13: DETAILS OF THE SELECTED EXPRESSWAY SECTIONS FOR VALIDATION OF THE MODELS

S/N	Expressway Section	Years in Service	Cumulative Traffic Volume	BPN	MTD (mm)	Closet Interchange to Data Extraction Point
01	Gelenigama - Dodangoda	05 (2012 - 2017)	13,481,145	45	0.21	Gelenigama
02	Dodangoda - Welipenna	05 (2012 - 2017)	11,757,307	50	0.25	Dodangoda
03	Welipenna - Kurundugaha	05 (2012 - 2017)	10,672,740	55	0.28	Kurundugaha
04	Pinnaduwa - Imaduwa	03 (2012 - 2017)	5,077,743	60	1.5	Pinnaduwa

TABLE 4.14: ASSUMPTIONS TAKEN FOR EACH EXPRESSWAY SECTION FOR VALIDATION OF THE MODELS

S/N	Expressway Section	Assumption for the Type of Stage of Service	Assumption for the Source of the Wearing Course Aggregate
01	Gelenigama - Dodangoda	Terminal stage	Kalutara (Western Province)
02	Dodangoda - Welipenna	Investigatory stage	Kalutara (Western Province)
03	Welipenna - Kurundugaha	Investigatory stage	Galle (Southern Province)

04	Pinnaduwa - Imaduwa	Initial stage	Galle (Southern Province)

Therein, for each expressway section, the SN respective for their assumed stage was calculated using the Penn State model with the use of collected BPN and MTD data. Considering the average of the PSV of the aggregate source location, SN for each expressway section was again calculated using the applicable derived models according to the assumed stage, as given in the TABLE 4.15.

TABLE 4.15: SN CALCULATED FOR EACH EXPRESSWAY SECTION FOR VALIDATION OF THE MODELS

S/N	Expressway Section	Quarry Location	Average PSV of Quarry Source	Selected SN model	SN - Model	SN – PIARC Model	Error (%)
01	Gelenigama - Dodangoda	1,11,16	45.7	SN_T	16	12	33.33
02	Dodangoda - Welipenna	1,11,16	45.7	SN_I	19	16	18.75
03	Welipenna - Kurundugaha	14, 15	48.5	SN_I	21	20	05.00
04	Pinnaduwa - Imaduwa	14, 15	48.5	SN_0	39	36	08.33

The calculated SN according to the selected models show low variation to that of the Penn State model with an acceptable margin of error, therefore the derived friction prediction models for the three different stages can be validated and verified for use.

CHAPTER 5

CONCLUSION AND RECCOMMENDATIONS

In conclusion, the main research aims of this study were to vehemently explore the polishing characteristics of locally available coarse aggregates in flexible pavement construction as an important material property and to establish a simple approach to determine the friction produced at the tire-pavement interface of a bituminous wearing course using the above identified material property.

Majority of the coarse aggregates used for pavement construction in Sri Lanka are of metamorphic grade rocks origins with gneiss-based mineralogy. Gneissic mineral aggregates are often related and compared with granitic aggregates of igneous rock origins in material properties, however, studies to elaborate more on the polishing behaviour of gneiss-based aggregates in the literature are quite scarce and inadequate. In fact, in the case of Sri Lankan road construction industry and research culture, absence of ample studies to determine polishing characteristics of locally available road surfacing aggregates, is vividly apparent, thus creating an inevitable void in aggregate property evaluation. In addition, at the moment the accelerated polishing test is considered as an expensive laboratory procedure that involves specifically imported testing machinery and materials with specifically trained personnel intervention to conduct a test. Also, the above testing prevails as a rare facility encountered in Sri Lanka to determine aggregate properties and the incorporated procedure is highly time consuming as well.

Therefore, the established polishing deterioration curves of each critical regions in the country within the Wanni and Highland geo complexes can be used as readily available reference graphs of friction retention trends for natural aggregates from those regions. Further the residual British Pendulum Number, calculated Polish Stone Value and the Terminal friction number of each specific sample can be capitalised in the determination of the friction retention capacity of coarse aggregates sourced from similar/nearby regions or quarries. The Fig. 5.1 gives friction deterioration patterns of the five districts with the largest annual roadstones production in Sri Lanka [17,35]. The Fig. 5.2 provides reference trendlines for districts under secondary level subcategory of coarse aggregate production for pavement construction. In aggregate selection for wearing course construction of different roadway categories by construction companies or relevant road agencies or for further experimental studies focused on studying frictional performance of Sri Lankan coarse aggregates and pavement, the following reference graphs can be well utilised as literature facts and data.

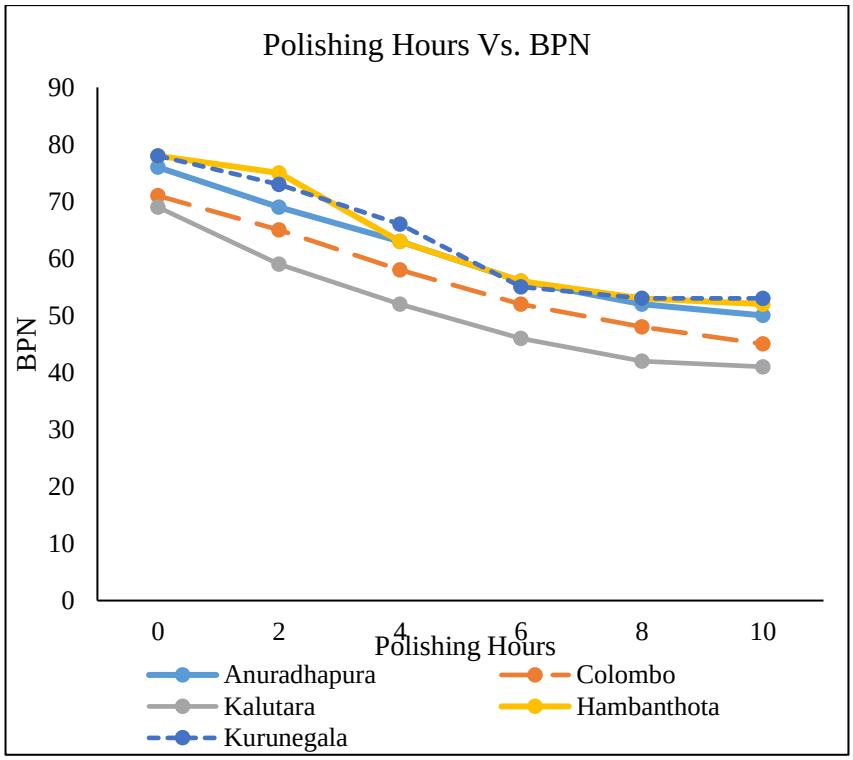


Fig. 5.1: Friction deterioration patterns of the major five districts

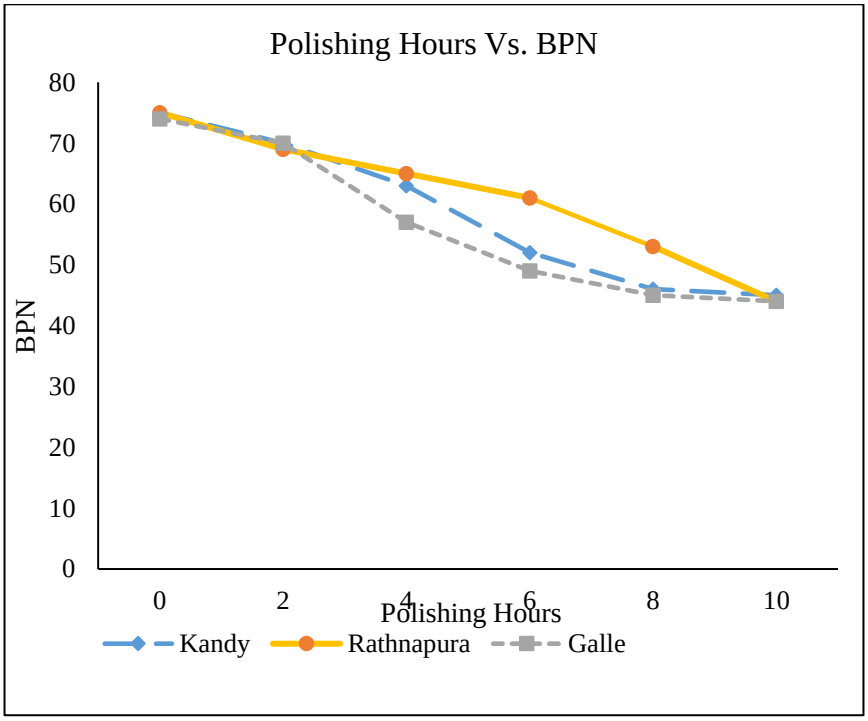


Fig. 5.2: Friction deterioration patterns of the sub-prominent districts

As far as the polishing characteristics of aggregate test samples are concerned, it is evident that the mineral aggregates extracted from Wanni and Highland geo complexes in Sri Lanka fall into the intermediate friction category with moderate/medium levels of polishing resistance with majority of PSV ranging from 45 – 55. It can be further suggested that these aggregate sources are adequately suitable for construction of

flexible pavement wearing course, ensuring road user safety with sufficient friction capacity.

The study on polishing performance of locally produced natural aggregates, were further strengthened and justified with the micro-texture and petrographic analysis conducted on the selected aggregate sources. Therein, the above analysis findings can be referred as guidance for gneiss based coarse aggregates in terms of their friction retention capacities, since no comprehensive and sound research data can be found in the existing literature for polishing behaviour of gneissic aggregates. The Fig. 5.3 presents the polishing deterioration trends of coarse aggregates with gneiss-based rock origins, which are commonly found in South and East Asian countries [17,35].

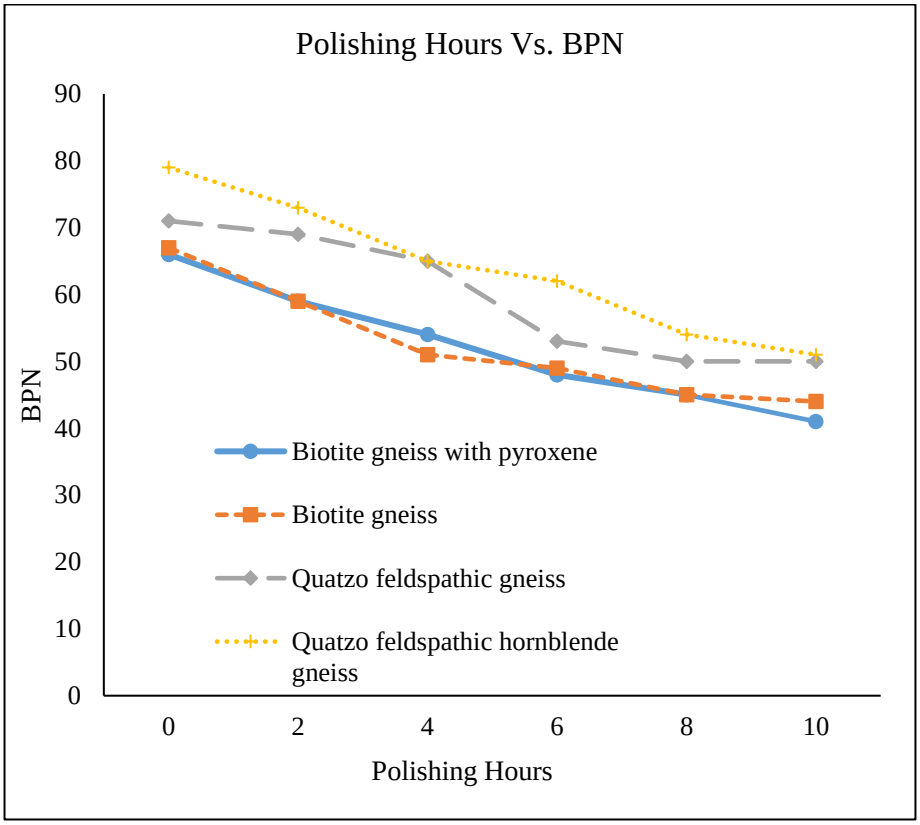


Fig. 5.3: Friction deterioration patterns of the variants of gneiss based coarse aggregates of metamorphic rock origins found in Sri Lanka

The main research objective of the study was to establish a rational yet simple friction prediction model for a HMA pavement with the input parameters of polishing resistance of coarse aggregates used in the construction of the bituminous wearing course. The achievement of such a model will benefit the decision makers in the planning, design, construction and maintenance of highways and pavements, since it provides an estimated SN as the output to forecast the frictional performance of a pavement yet to be constructed. The approach for the derivation of the input parameters for the model, was to capitalise feasible yet accurate testing methods available in Sri Lankan highway industry to evaluate polishing behaviour of coarse aggregates, rather than focusing on highly sophisticated measurement devices. The

PSV – the standard practiced measure to evaluate the polishing resistance of coarse aggregates of a wearing course under simulated laboratory wheel application, is identified in this study to be used to forecast the friction capacity of an actual pavement in service under vehicle trafficking. Therefore, the following three models allow the prediction of pavement friction during the initial stages of pavement construction, specifically in aggregate selection phase.

$$SN_0 = -0.0084.PSV^2 + 1.3894.PSV - 8.1899$$

$$SN_I = 0.0063.PSV^2 - 0.0476.PSV + 8.0336$$

$$SN_T = -0.0168.PSV^2 + 2.119.PSV - 47.248$$

The equation (21) can be applied to estimate the Skid Number available on the pavement just after construction and before opening to traffic, which means that it will provide the initial skid resistance capacity of the HMA pavement. In the determination of the SN at the investigatory level of a pavement during its operational stage, equation (22) can be optimised, whereas equation (23) predicts the terminal friction capacity of a pavement at the end of its service life or at the maximum possible traffic application before introduction of a necessitated resurfacing. All the prediction models are statistically applicable in the case of metamorphic grade gneissic coarse aggregates.

In the pavement management systems of highway categories with higher design speeds such as expressways, to ensure road user safety as well as to mitigate skid related roadway crashes, pavement friction management is introduced to be executed with a greater priority. There, to determine the existing pavement friction performance in terms of SN, macro and micro-texture, a variety of measurement devices and mechanisms are employed worldwide and forecasting of future frictional performance based on the collected data is also carried out using numerous prediction tools and models. In Sri Lanka, to evaluate pavement friction of the expressway system in terms of BPN and MTD, BPT and SPM are employed in an ad hoc approach rather than conducting network level data collection periodically, at regular intervals customising on the degree of critical risk factors or zones, according to a well-established PFM framework. Further, the adequacy of the frequency of collection of the required network level data is questionable. The absence of a complete compilation of periodically collected historical friction data over the starting service span (first five or ten years after handing over to the public) of the expressway system, is also a crucial matter to come up with predicted future conditions for better, calculated decision making in maintenance and intervention. In summary, to predict the future friction or skid resistance deterioration patterns of the wearing course of national expressway system, the existing collected raw data is inadequate.

In such a scenario, the above derived friction prediction models can be used to forecast the future frictional performance of the Sri Lankan Expressway system, at the start of the construction stage/resurfacing, investigatory level and the terminal stage in need of renovation or resurfacing. The only requirement is to conduct a PSV test for the particular coarse aggregate of the respective source/quarry used in the

construction/resurfacing of the overlay. Therein, even without supportive historical or existing network data of the selected expressway section, the friction deterioration trends can be simulated with the estimated SN_0, SN_I and SN_T values that can be used in the decision making process of PFM. Since the models can be used in predicting the frictional performance per each highway section according to their source of coarse aggregate supply, more informed, specific and accurate forecasting can be performed rather than implementing a generic forecasting tool or methodology.

CIDA/ICTAD guidelines on coarse aggregate selection criteria for a bound pavement/course involve testing for resistance against wear, weathering or soundness, toughness, shape and moisture absorption, which will provide comprehensive understanding of the aggregate quality. But, in the case of high risk, accident-prone pavement/highway sections such as expressway, hazardous roadway sections like critical horizontal bends, intersections/interchanges, ramps or any other identified problematic pavement categories in terms of susceptibility to exaggerated polishing/ water ponding issues, the coarse aggregate selection criteria limited to that of CIDA would be inadequate. Thus, for the selection of suitable mineral aggregates for the construction or restoration of afore-mentioned hazardous roadway sections; overlays, incorporation of a methodology or test to include determination of coarse aggregates' resistance against polishing for the existing guidelines is indispensable. Hence, in consideration of the research findings, the Polish Stone Value test is suggested for the above purpose. In addition, in the nomination of appropriate parent rock sources for pavement construction and in the approval of IML grade quarries in Sri Lanka, PSV test is recommended to be incorporated in the selection criteria, so that it will enable compilation of data in terms of polishing resistance of locally available roadstones.

The polishing pattterns of gniess based coarse aggregates in Wanni and Highland geo complexes, were established in this study, therefore further experimental studies are required to explore the mineral aggregate sources loacted in the Vijayan, Kadugannawa complexes and the Northern Limestone region and to precisely determine their friction retention capacities. It is apparent that the derived polishing trends in the study can be effectively used as reference graphs for aggregate sources in Vijayan and Kadugannawa geo complexes, nonetheless further examination is required for accurate determination of their frictional performance. Although in existing literature, friction deterioration trends for limestone have been abundantly established in other countries and typically limestone of sedimentary rock origins depict comparatively poor polishing resistance [], separate study is required to investigate limestone avaiable in Sri Lanka since there are a few variations of limestone present in terms of the degree of the carbonation process.

More definitive petrographic analysis of the aggregates sources are proposed to be performed by means of advanced procedures, such as X-ray diffraction (XRD) analysis, Differential Thermal Analysis (DTA), infrared spectroscopy, or other SEM or Energy Dispersive X-ray analysis (EDX) method, to verify and strengthen the research findings.

The APM along with the BPT were applied in this research in order to simulate polishing due to actual vehicle trafficking and to measure the respective friction, as they are the only available formats in the national context. There are other similar test and methods available of similar calibre to polish and assess the polishing resistance of coarse aggregates used for the pavement wearing course construction like MD, DFT, thus it is recommended to reinvestigate the same parameters in question using an alternative approach so that more accuarate and clarified observations can be made with the benefit of comparison.

Further, the behaviour of the tested aggregate samples should be explored for their polishing patterns under actual field conditions of a highway pavement similar to that in Sri Lanka in terms of climate, environmental factors and different vehicle speeds using test pavement strips of different aggreagte sources, so that more precise versions of friction predition models can be deducted or fine tuning of the suggested model in the study to align more with actual scenario can be achieved.

The adopted methodology in this study to determine the friction prediction model of locally available natural coarse aggregates, can be substitued to establish similar forecasting mathematical model of polishing resistance and frictional performance of pavements constructed with alternative types of aggregates like recycled aggregates, steel slag and other industrial by products.

In conclusion, as an initiative chapter, this research study addresses the absence of research/experimental/monitoring studies on polishing characteristics of coarse aggregates and pavement friction in Sri Lanka, through the development of a friction prediction model to simulate polishing of asphalt concrete pavement under traffic.

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