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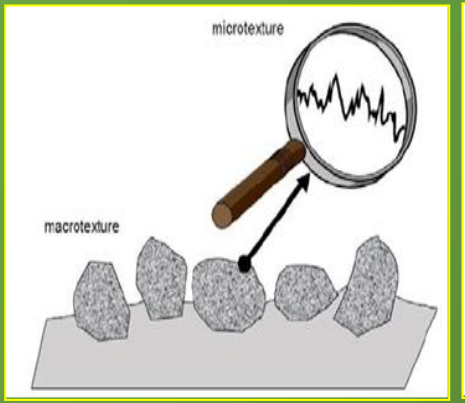
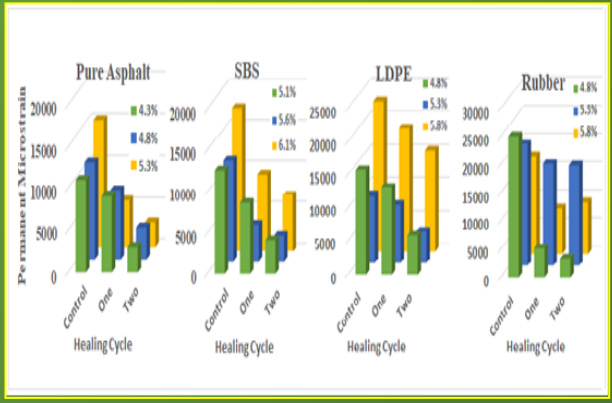
For:
- Roads, City Streets & Airports

- Production and Use of: Concrete & Asphalt Pavements and Bituminous Materials in Civil Engineering and Building Construction

- Design, Manufacturing, Specification, Management, Construction, Evaluation, Rehabilitation

- IT in Pavement Engineering, Material Technology & Research Focus





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EDITORIAL

17th Annual International Conference on Asphalt, Pavement Engineering and Infrastructure

February 21-22, 2018, The Sensor City, Liverpool, UK

TECHNICAL PAPERS

The International Journal of Pavement Engineering & Asphalt Technology, PEAT **ISSN 1464-8164**

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December 2018

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ASPHALT, PAVEMENT ENGINEERING AND INFRASTRUCTURE

February 21-22, 2017, The Sensor City, Liverpool, UK

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The Department of Civil Engineering at Liverpool John Moores University in association with RSTA and Colas Ltd is hosting its 17th Annual International Conference addressing: Asphalt, Pavement Engineering and Infrastructure.

CONFERENCE THEME

Conference is aimed at stakeholders with specific interest in the; development, construction and management of asphalt technology, sustainable infrastructure, environmental protection and energy reduction, aggregate recycling initiatives, airport and highways design and maintenance. The conference will be of interest to; policy advisors, environmental regulators, infrastructure clients, specifiers, planners, designers, local authorities, highway related consultants and designers, materials suppliers, construction companies, contractors and educational institutions.

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The conference will be of interest to; policy advisors, environmental regulators, infrastructure clients, specifiers, planners, designers, local authorities, highway related consultants and designers, materials suppliers, construction companies, contractors and educational institutions.

PUBLICATIONS

The papers will be reviewed by the conference scientific and technical board and published in the conference proceedings. Selected papers will also be refereed and published in a special issue of the International Journal of Pavement Engineering and Asphalt Technology, ISBN 1464-184.

LIVERPOOL JOHN MOORES UNIVERSITY

Faculty of Engineering & Technology
Depaertment of Civil Engineering
Liverpool Centre for Materials Technology
17th Annual International Conference on
Asphalt, Pavement Engineering and Infrastructure
21-22 February, 2018, The Sensor City, Liverpool, UK

CONFERENCE PROGRAMME

DAY 1 – Wednesday 21th February 2018

Venue: SENSOR CITY, 31 RUSSELL STREET, LIVERPOOL, L3 5LJ, UK

08:30 Registration, Refreshment and Exhibition

09:00 Welcome by the Conference Chairman, Professor Howard Robinson, Road Surface Treatments Association, RSTA, **UK**.

09:10 Opening Address, Professor Ahmed Al-Shamma'a, Executive Dean, Faculty of Engineering & Technology, Liverpool John Moores University, **UK**.

09:15 ''Update on the Latest Development in Asphalt Technology''. Tony Sewell the IAT Vice President, **UK**

Morning Programme

Session 1, Chairman: Professor Howard Robinson, Chief Executive, RSTA, UK

09:25 "The New Product Acceptance Scheme based on ISO 9001". Paul Philips, PTS Ltd, **UK**

09:45 "Best Practice in Highway Asset Management". James Wallis, XAIS Ltd, **UK**

09:05 "Influence of Bitumen Emulsion Spray on Pavement Performance". John Richardson, Colas ltd, **UK**

10:25 "Innovations in Asphalt Technology and Trials Performance". Phil Sabin, Aggregate Industries, **UK**

10:35 Questions & Discussion

10:40 Refreshments and Exhibition

Session 2, Chairman: John Richardson, Colas Ltd, UK

11:25 "Latest Development in Polymer Modified Bitumen and Its Industrial Application". Rick Ashton, Total Bitumen, **UK**

11:45 "The new British Standard PD6689 for Surface Dressings and Microsurfacings". Howard Robinson, RSTA, **UK**

12:05 "Alternative techniques in predicting fretting in road surfacing using spectroscopic analysis". Hannah Bowden, **UK**

12:25 "The new Specification Clause 936 for Pavements reinforced with Geosynthetics and Steel Meshes". Paul Sanders, Highways England, **UK**

12:45 Questions & Discussion

12:50 Lunch and Exhibition

Afternoon Programme

Session 3, Chairman: Professor Marco Pasetto, University of Padua, Italy

13:50 "Demystifying Standardisation and Product". Gavin Jones, BSI, **UK**

14:10 "Improving highway whole life cost through materials innovation". Dr Bachar Hakim, ,AECOM Pavement Design and Asset Management, **UK**

14:30 "Assessing the Impact of Polymer Additives on Deformation and Crack Healing of Asphalt Concrete Subjected to Repeated Compressive Stress". Saad Issa Sarsam, University of Baghdad, **Iraq**

14:50 "Rheological evaluation of asphalt binders containing pyrolytic biochar''. Dr Rajan Choudhary, Indian Institute of Technology Guwahati, **India**

15:10 Questions & Discussion

15:15 Refreshments and Exhibition

Session 4, Chairman: Saad Issa Sarsam, University of Baghdad, Iraq

15:45 "Land reclamation and industrial aggregates production: new perspectives for the circular economy". Sig.Cioffi Flavio, **Italy**

16:05 "Green" pavement overlays. Part I: Beams in flexure on elastic foundation- and 'Green' pavement overlays. Part II. Beams in shear on elastic foundation and their numerical representation". John. N. Karadelis, Coventry University, **UK**

16:25 "Steel Slag as Valuable Aggregate in Eco–Friendly Mixtures for Asphalt Pavements". Prof. Marco Pasetto, University of Padua, **Italy**

16:45 "Smart Phones and Pavements". Tony Parry, NTEC, **UK**

17:05 Questions and Closing Remarks by the Conference Director Professor Hassan Al Nageim, UK

CONFERENCE PROGRAMME

DAY 2 – Thursday 22th February 2018

Venue: SENSOR CITY, 31 RUSSELL STREET, LIVERPOOL, L3 5LJ, UK

08:30 Registration, Refreshment and Exhibition

Morning Programme

Session 1, Chairman: Amir Kavussi, Tarbiat Modares University, Iran

09:00 "A novel laboratory test method to measure dynamic water pressure underneath a cracked concrete pavement". Fauzia Saeed, Brunel University London, **UK**

09:20 "An innovative high-speed deflection measurement device for use in network life cycle planning and work programming". Marco Francesconi, Dynatest, **Denmark**

09:40 "A mechanism based reaction-diffusion model for spurt oxidation of bitumen". Uwe Muehlich, University of Antwerp, Road Engineering Research Section, **Belgium**

10:00 "Use of basic oxygen furnace steel slag in open graded friction courses". Abhinay Kumar, Indian Institute of Technology Guwahati, **India**

10:20 Questions & Discussion

10:25 Refreshments and Exhibition

Session 2, Chairman: Mostafa Abo-Hashema, Fayoum University, Egypt

10:55 "Performance of a cold close graded surface course mixture using a new cementations material from GGBS and waste lime". Anmar Dulaimi, Liverpool John Moores University, **UK**

11:15 "Skid resistance of asphalt mixtures prepared using different aggregate sources and gradations". Naveed Ahmad, University of Engineering & Technology, Taxila, **Pakistan**

11:35 "Laboratory assessment of synthetic geopolymer high friction aggregate wear using contact and non-contact measurement techniques". Wilkinson, A., Ulster University, Jordanstown, **Northern Ireland**

11:55 "Investigation of the Effects of Additives on Moisture Susceptibility of

Asphalt Mixes Containing Sulfur-Polymer". Amir Kavussi, Tarbiat Modares University, Tehran, **Iran**

12:15 **Questions & Discussion**

12:20 **Lunch and Exhibition**

Afternoon Programme

Session 3, Chairman: Marco Francesconi, Dynatest, Denmark

13:30 "Determination of affinity between aggregate and bitumen by digital image processing". Johan Blom, Antwerpen, **Belgium**

13:46 "An experimental study to investigate the use of novel structural fibres as shear reinforcement in concrete beams". Ameer Jebur, Liverpool John Moores University, **UK**

14:02 "Development of Distress Prediction Models for Flexible Pavements using LTPP for Main Roads in Egypt". Mostafa M. Radwan, Nahda University, **Egypt**

14:18 "Impact of mineral types and water on bitumen-mineral adhesion and debonding behaviours". Yuqing Zhang, Aston University, **UK**

14:50 **Questions & Discussion**

14:55 **Refreshments and Exhibition**

Session 4, Chairman: Prof. Dr. Mumtaz Ahmed Kamal, University of Engineering & Technology, Taxila, Pakistan

15:25 "Determining Optimum Proportion of Fly Ash as Partial Replacement of Asphalt for Flexible Pavements". M Zafar Ali Shah, Swedish College of Engineering & Technology, Wah Cantt, **Pakistan**

15:40 "Enhancing Stability of Clayey Subgrade Materials with Cement Kiln Dust Stabilization". Magdy A. Abd El-Aziz, Fayoum University, Fayoum, **Egypt**

15:55 "Can Polymer Modified Bitumens Contribute to Cost Efficiencies in Asphalt?''. Gary Schofield / Rick Ashton, Total Bitumen, **UK**

16:10 "Non-destructive assessment of early age mortar containing stainless steel powder". Hayder Al Hawesah, Ali Shubbar, Rafal L Al Mufti, LJMU, **UK**

16:25 "Product assessment to support innovative solution based efficiencies". Paul Phillips, pavement testing services ltd, **UK**

16:40 "Development a High Quality Cold Mix Asphalt". Manar Herez, Liverpool John Moores University, **UK**

16:55 **Questions and Closing Remarks by the Conference Director Professor Hassan Al-Nageim, UK.**

ASSESSING THE IMPACT OF POLYMER ADDITIVES ON DEFORMATION AND CRACK HEALING OF ASPHALT CONCRETE SUBJECTED TO REPEATED COMPRESSIVE STRESS

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ABSTRACT

Microcracks in Asphalt concrete occurs due to the loading and environment impact. However, they can heal by themselves in a slow process under repeated loading at ambient temperature; this can increase the lifetime of the pavement. The aim of this work is to investigate the impact of polymer additives (SBS, LDPE, and rubber) on the crack healing ability of asphalt concrete through its influence on permanent deformation under repeated compressive stress. Asphalt concrete specimens of 101.6 mm diameter and 127 mm height have been prepared with optimum asphalt content requirement and with extra 0.5% asphalt above and below the optimum and tested under repeated compressive stress level of 138 kPa at 25°C environment. The loading cycle was 0.1-second load application followed by 0.9 seconds of a rest period. The test was conducted for 900 repetitions using the Pneumatic repeated load system (RPLS) to allow for the initiation of microcracks. After the specified loading cycles, Specimens were withdrawn from the test chamber and stored in the oven for 120 minutes at 60 ° C to allow for micro crack healing, then were subjected to another loading and healing cycles. Permanent deformation results were detected through LVDT. It was concluded that polymer additives have a positive impact on microcrack healing process. For pure asphalt, SBS, LDPE, and rubber modified mixes, the permanent deformation at optimum asphalt content decreases by a range of (29-67), (63-73), (14-53) and (16-18) % after one and two healing cycles respectively as compared with control mix.

Keywords: Compressive stress, deformation, Modified asphalt concrete, repeated load and resilient strain.

INTRODUCTION

The variation in temperatures during day and night in Iraq have a significant impact on paving asphalt, (Sarsam, 2015). Variable temperatures and loading of traffic during the life of the pavement make the design and selection of suitable materials to resist such impact difficult and practically useless, (Vlachovicova et al, 2007). Polymer modified asphalt is used today to reduce early pavement distress and extend service life by enhancing adhesion, cohesion, and elasticity. The addition of polymers to asphalt cement leads to a change in the rheological properties and can keep the sufficient flexibility at low temperatures in addition to achieving good resistance to deformation under high temperature, (Sarsam and AL-Lamy, 2015). The polymer supports the binder and exhibits higher resistance to temperature changes, fluctuating weather, and high traffic load movement, (King et al, 1986). Elastomers tend to improve the elasticity of asphalt binder at low temperatures, strength at high temperatures and increase the failure strain resistance of asphalt concrete at low temperature, (Crossley and Glen, 1999). Typical elastomeric polymers used to modify asphalt binder include styrene butadiene styrene SBS, crumb rubber CR and reclaimed from scrap tires RR. For SBS, the polystyrene imparts strength to the polymer while the butadiene gives the material its exceptional elasticity. This combination of strength and elasticity gives SBS modified asphalt the ability to resist permanent deformation and to minimize fatigue and low-temperature cracking. The SBS polymer modifier made the HMA mixture softer and more ductile, (Airey, 2004). The effect of LDPE with different percentages on the properties of asphalt concrete mixtures was investigated by (Al-Hadidy, 2001). This study found that the Polyethylene-asphalt binder is characterized by low sensitivity to aging and weathering conditions. The inclusion of LDPE in asphalt concrete mixtures gives a quite satisfactory result in terms of Marshall Stability values and other Marshall properties. It also improves the tensile strength and flexural strength of paving mixtures. (AL-Harbi, 2015) Used five types of polymers: LDPE and HDPE with (2%, 5% and 7%) by weight of asphalt cement; crumb rubber with (12%, 15% and 18%) by weight of asphalt cement; SBR and SBS with (1%, 3% and 6%) by weight of asphalt cement) in order to evaluate the effect of the physical properties of polymers on the performance of asphalt mixtures (stability, indirect tensile strength to fatigue resistance and rutting resistance). It was concluded that the best contents of SBS, SBR, LDPE, HDPE and CR are 3%, 3%, 2%, 2% and 12% respectively, which have higher stability, higher fatigue resistance and lower rutting depth than control asphalt mixtures. (Sarsam and Lafta, 2014) Had prepared the modified Asphalt cement in the laboratory by digesting each of the two-penetration grade Asphalt cement (40-50 and 60-70) with Sulphur, fly ash and silica fumes. Three different percentages of each of the above-mentioned additives have been tried using continuous stirring and heating at 150ºC for 30 minutes. The prepared modified Asphalt specimens were subjected to physical properties determination; the penetration, softening point, ductility before and after laboratory aging. It was concluded that all percentage of additives have reduced the penetration value of asphalt cement. Softening point was increased with the addition of all percentage of additives. (Abd-Allah, and Mohamady, 2014) Evaluated the effect of adding several types of polymers on asphalt cement. The

experimental program involved modifying the asphalt using six types of polymers then evaluating the properties of the modified asphalt. It was found that the optimum percentage of PVC, plastic bags and novolac was 4%, and the optimum percentage of high-density polyethylene HDPE was 5% by weight of asphalt. These percentages caused an increase in kinematic viscosity and reduction in penetration. For more than 25years, researchers have been reporting evidence of micro damage healing, initially on asphalt binders, and later on asphalt mixtures (Garcia, 2012; Kim et al, 2003; Si et al, 2002; Kim and Roque, 2006). The impact of micro crack healing phenomena of asphalt concrete on the resilient characteristics under shear and tensile repeated stresses have been investigated by (Sarsam and Husain, 2016). Specimens of 100mm diameter and 75mm of height have been prepared at optimum asphalt content, and at 0.5% asphalt above and below the optimum. The deformation of the specimens under repeated indirect tensile or shear stresses was captured using a video camera for both conditions. The impact of crack healing was detected through the variation of the resilient characteristics under three levels of stress application before and after healing. The aim of this work is to study the influence of three types of polymer additives on deformation and microcrack healing of asphalt concrete subjected to repeated compressive stress.

MATERIALS AND METHODS

Asphalt Cement

The asphalt cement used in this study had 40-50 penetration grade; it was obtained from Daura Refinery, southwest of Baghdad. The physical properties of the asphalt cement are presented in Table 1.

Table 1. Physical Properties of Asphalt Cement

Property	Result	Unit	SCRB Specification
Penetration (25ºC,100g,5 sec) ASTM D5-97	44	1/10mm	40-50
Softening Point (Ring & Ball) ASTM D5-36	48.9	ºC	50-60
Ductility (25ºC, 5cm/min) ASTM D113-07	120	cm	>100
Kinematic viscosity at 135ºC ASTM D-2170	365	CSt	_____
Flash point (Cleave land open cup).ASTM D-92	323	ºC	Min232
Specific gravity at 25 ºC ASTM D-70	1.04	_____	(1.01-1.05)
After Thin-Film Oven ASTM D1754			
Retained penetration of original,% D946	60	%	>55%
Ductility at 25ºC,5 cm/min.	75	cm	>25
Loss in weight (163ºC,50g,5h) ASTM D1754	0.34	%	< 0.75

Coarse Aggregate

In this work, the crushed coarse aggregate was brought from Al-Nibaee quarry. It consists of hard, strong and durable pieces, free of coherent coatings. The gradation

of coarse aggregate ranges between 19.0 mm and 4.75 mm according to (SCRB R/9, 2003) specification. The mineralogical composition of coarse aggregate is shown in Table 2, while the physical properties of the coarse aggregate are illustrated in Tables 3.

Fine Aggregate

Fine aggregate was brought from Al-Nibaee quarry. The gradation of fine aggregates ranges between passing 4.75mm and retains on 0.075mm. It consists of tough grains, free from clay, loam or other deleterious substance. The physical properties of the fine aggregate are shown in Table 3.

Table 2. Mineralogical Composition of Al-Nibaee coarse Aggregates

Mineralogical Composition	Content %
Quartz	80.3
Calcite	10.92

Table 3. Physical Properties of Al-Nibaee Coarse and Fine Aggregates

Property as per (ASTM, 2003)	Coarse aggregate	Fine aggregate
Bulk Specific Gravity (ASTM C127 and C128)	2.680	2.630
Apparent Specific Gravity (ASTM C127 and C128)	2.632	2.6802
Percent Water Absorption (ASTM C127 and C128)	0.423	0.542
Percent Wear (Los-Angeles Abrasion) (ASTM C131)	21.7	……

Mineral Filler

One type of mineral filler, which is (Ordinary Portland Cement), was implemented. It is thoroughly dry and free from lumps or aggregations of fine particles. The physical properties are shown in Table 4.

Table 4. Physical Properties of Portland cement

Test	Physical properties
% Passing Sieve No.200 (0.075 mm)	98
Apparent Specific Gravity	3.10
Specific Surface Area (m^2/kg)	315

Polymer Additives to Asphalt Cement

Three types of polymer additives were implemented in this work; Low-density polyethylene (LDPE), Styrene-butadiene-styrene (SBS) and Scrap Tire rubber (TR). The modified asphalt cement binders were produced in the laboratory. Details of the production process and the properties were published elsewhere, (Sarsam and Jasim, 2017).

Selection of overall Aggregate Gradation

The gradations that was selected in this study follow (SCRB R/9, 2003) specification for Hot-mix asphalt paving mixtures usually used for wearing course with aggregate nominal size of (12.5 mm). Fig.1. Show the gradation for wearing layer adopted.

Preparation of Modified Asphalt Concrete Specimens

The various fractions of aggregate as supplied from the mixing plant were separated into groups, as retained on each of the following sieve, (19, 12.5, 9.5, 4.75, 2.36, 0.3, 0.075) mm using dry sieve analysis. The material passing 0.075 mm was discarded and replaced by mineral filler (ordinary Portland cement). The aggregates were recombined according to the gradations requirements shown in Fig.1 for wearing course.

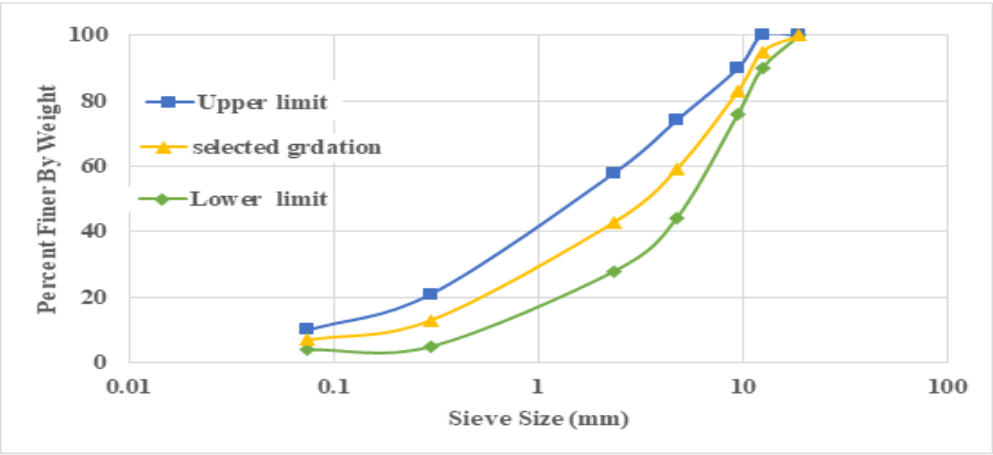

Figure 1. Specification Limits and Mid-Point Gradation of (SCRB, 2003) for Wearing Course Layer.

The overall aggregate mix was heated to 160°C, while the pure or modified asphalt cement was heated to 150°C, then added to the aggregates and mixed thoroughly for three minutes using mechanical mixer until asphalt had sufficiently coated the surface of the aggregates and a homogeneous mixture is achieved. Optimum percentages of asphalt content obtained from previous work by (Sarsam S. and Jasim, 2017) have been implemented with an extra percentage of 0.5 above and below the optimum for each type of modified asphalt. The compaction cylindrical mold (102 mm in diameter and 203 mm in height) used in this work was capable of production of a specimen of 101.6 mm in diameter and 127mm in height as shown in Fig.2. The mold was heated to 150 °C, then the asphalt concrete mixture was transferred to the heated mold, laid and spread uniformly with a heated spatula, then subjected to static compaction of 30 kN load applied through steel cylinder of 101 mm diameter and 8 mm thickness. The applied pressure was maintained for three minutes at 150°C to achieve the target density and thickness. The mold was left for 24 hours and then the cylindrical specimen was extruded from the mold. Fig.3. demonstrates part of the prepared specimens. On the other hand, Table 5 exhibit details of the prepared specimens.

Figure.2 Mold used to prepare specimens **Figure.3.** Part of the prepared specimens

Table 5. Details of the Prepared Cylindrical Specimens

	Asphalt cement type	Pure asphalt			LDPE modified			SBS modified			Rubber modified		
	Asphalt content %	4.3	4.8	5.3	4.8	5.3	5.8	5.1	5.6	6.1	4.8	5.3	5.8
	Bulk Density gm/ cm^3	2.340			2.317			2.292			2.350		
	Volume of voids %	3.9			5.5			5.4			4.0		
	Volume filled with asphalt %	73			68			70.4			75		

Testing under Repeated Compressive stress

The specimens were subjected to axial repeated compressive loading using the pneumatic repeated load system (PRLS) shown in Fig.4. The test was performed on cylindrical specimens, 101.6 mm in diameter and 127 mm in height. In these tests, repetitive compressive loading was applied to the specimen and the axial deformation was measured under the different loading repetitions. Compressive loading was applied in the form of a rectangular wave with a constant loading frequency of 60 cycles per minute and includes 0.1-second load duration and 0.9 second rest period. The stress level of 138kP and temperatures 25˚C were used in the tests.

Figure.4. Cylindrical Specimen in Pneumatic Repeated Load System

The specimen was left in the conditioned chamber for one hour at testing temperature (25˚C) to allow for uniform distribution of temperature within the specimen. LVDT (Linearly Variable Differential Transformer) which convert the mechanical signal (displacement) to electrical signal has been used to monitor the deformation of the specimen under each load cycle, and Positioned onto the specimen and set to zero. Then, the recorded data was analyzed for finding strain at any number of load cycles desired for every test. The repetitive compressive stress was applied to the specimen and the vertical deformation of the specimen was measured under each load repetitions as recommended by (Sarsam and Husain, 2016; Albayati, 2006). The test start to allow for the initiation of micro cracks, and was stopped after 900 repetitions, and then specimens were withdrawn from the testing chamber.

Permanent Deformation Test Results

The experimental design for the permanent deformation test was a factorial with three asphalt contents, and three additives, the test temperature was constant at (25ºC). In addition, the impact of number of healing cycle on permanent deformation was determined. Permanent deformation was assessed using repeated Compressive stresses. Therefore, 25 cylindrical specimens were prepared and tested to simulate the above mention variables. Power model is often fitted to the accumulated permanent deformation curve. It is probably that it is the most commonly used permanent deformation equation. The following classical power model was used in this study as recommended by (Sarsam and Husain, 2016).

$\varepsilon p = \mathbf{a}\, N^{b}$ ………………………………………………… (1)

Where, (a, and b) are the intercept and slope of the curve in log–log Scale, respectively. The intercept (a) represents the permanent strain at N=1, where N is the number of the load cycles. The higher the value of intercept, the larger the strain and hence the larger the potential for permanent deformation as mentioned in the super pave study carried out by (Sarsam and Rahem, 2009). While slope (b) represents the rate of change in the permanent strain as a function of the change in loading cycles

(N) in the log-log scale, high slope values for a mix indicate an increase in the material deformation rate hence less resistance against rutting. A mix with a low slope value is preferable as it prevents the occurrence of the rutting distress mechanism at a slower rate, (Albayati, 2006; Sarsam and Rahem, 2009). To evaluate the permanent deformation, the three parameters selected were, slope, intercept, and the permanent deformation which was measured after 900 loading cycles on the cylindrical samples. The required deformation data analyses include determination of the permanent deformation at the following load repetitions (1, 10, 50, 100, 200, 300, 400, 500, 600, 700, 800, and 900) using the LVDT data. The permanent strain (**εp**) was calculated by applying the following equation.

$$\varepsilon p = \frac{pd*10^6}{h} \qquad \ldots\ldots\ldots\ldots\ldots (2)$$

Where:
ε_p: Permanent micro strain (mm/mm).
pd: Reading of LVDT monitor for permanent strain
h: specimen diameter (mm).

Crack Healing Cycle Technique adopted

The microcrack healing technique implemented in this work was healing by the external heating. After the initiation of microcracks after 900 compressive load repetitions, the test was stopped. Specimens were withdrawn from the testing chamber as mentioned before and stored in an oven for 120 minutes at 60 °C to allow for micro crack healing. Specimens were subjected to another cycle of repeated compressive stress at 25 °C for another 900 repetitions. Specimens were subjected to the second healing process conducted on the specimens and the third cycle of load repetitions. The testing Temperate was (25ºC), while the compressive stress was (138) Kpa. Fig.5 shows specimens stored in the oven for microcrack healing.

Figure.5. Specimens Stored In oven for micro crack healing

RESULTS AND DISCUSSION

Impact of Micro Crack Healing Cycles on Permanent Strain

Impact of Asphalt content and type on permanent deformation was variable. Fig.6. shows the effect of asphalt type and content and healing cycles on permanent strain after 900 load repetitions of compressive stresses; it can be observed that the permanent strain decreases as the healing cycle's increases. At 4.3% of pure asphalt content, the deformation decreases by (17, 72) % after one and two healing cycles respectively as compared with the control mix. For mixture with optimum asphalt content of 4.8%, the deformation decreases by (29, 67) % after one and two healing cycles respectively as compared with control mix. While, at 5.3 % asphalt content, the reduction in permanent deformation by (80, 86) % after one and two healing cycles respectively could be observed as compared with the control mix. On the other hand, when the asphalt content was 0.5% above or below the optimum percentage, the microstrain increased by 30 and 5% respectively. When polymer additives were introduced, similar behavior could be observed, and the permanent microstrain increases as the modified asphalt percentage changes below or above the optimum. On the other hand, the microstrain decreases as healing cycles increases. The impact of optimum additive content on permanent strain is more pronounced as compared to other percentages of polymer additives. Such behavior agrees well with (Sarsam and AL-Lamy, 2015; AL-Harbi, 2015).

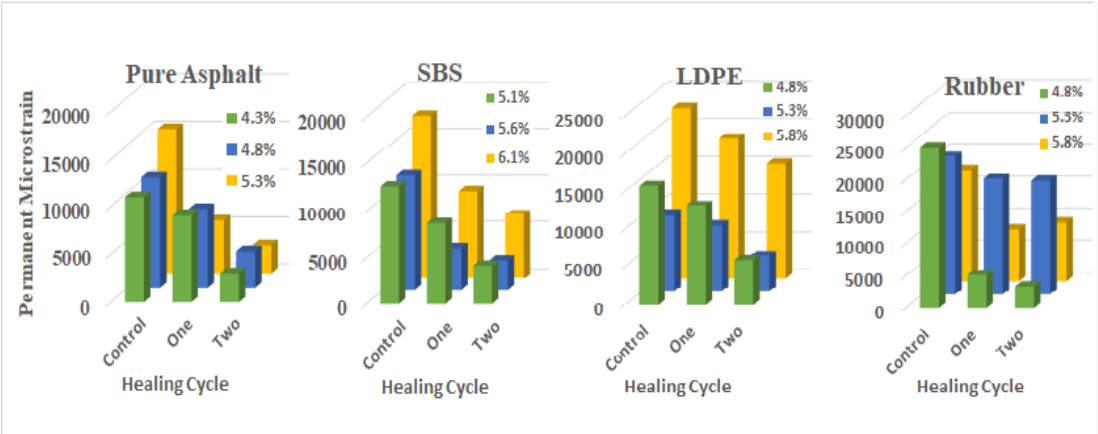

Figure 6. Impact of Asphalt Content and Crack Healing on Permanent Micro Strain

For SBS modified asphalt concrete, Fig.6 exhibit that at 5.1% asphalt content, the deformation decreases by (32, 67) % after one and two healing cycles respectively as compared with control mix. For mixture with optimum asphalt content of 5.6%, the deformation decreases by (63, 73) % after one and two healing cycles respectively as compared with the control mix. While, at 6.1 % asphalt content cause reduction in permanent deformation by (46, 61) % after one and two healing cycles respectively as compared with the control mix.

For LDPE modified asphalt concrete, it can be observed that at 4.8% asphalt content, the deformation decreases by (17, 63) % after one and two healing cycles respectively as compared with the control mix. For mixture with optimum asphalt content of 5.3%, the deformation decreases by (14, 53) % after one and two healing cycles

respectively as compared with control mix. While, at 5.8 % asphalt content cause reduction in permanent deformation by (18, 32) % after one and two healing cycles respectively as compared with the control mix.
For Rubber modified asphalt concrete, Fig.6 demonstrate that at 4.8% asphalt content, the deformation decreases by (80, 87) % after one and two healing cycles respectively as compared with control mix. For mixture with optimum asphalt content of 5.3%, the deformation decreases by (16, 18) % after one and two healing cycles respectively as compared with the control mix. While, at 5.8 % asphalt content cause reduction in permanent deformation by (53, 47) % after one and two healing cycles respectively as compared with the control mix. In fact, the behavior of rubber-modified mix may be referred to the possible chemical reaction between asphalt cement and rubber, which exhibit stiffer mix at lower asphalt content, while it shows behavior that is more elastic at optimum asphalt content. Similar findings have been reported by (Al-Bana, 2010).

Impact of Polymer Additives on Permanent Deformation Parameter

Fig.7 shows the influence of pure asphalt and healing cycles on permanent strain parameters under compressive stresses. When the number of healing cycle's increases, the intercept decreases while the slope value mostly increases. The intercept and slope values changes as well at different asphalt contents. The intercept value increases with the increment of asphalt content.
Table 6 summarizes the influence of pure asphalt and healing cycles on permanent strain parameters. The rate of decreases of intercept value before healing cycles when the asphalt content increases from (4.3 to 4.8)% is 11% while it increases by 203% with further increment in asphalt content to 5.3% as compared to initial asphalt percentage. On the other hand, the slope values decrease by (12 and 55) % for (4.8 and 5.3) % asphalt percentages respectively as compared to mix with 4.3% asphalt content. Such behavior may be attributed to the fact that optimum asphalt content exhibit the more stable mix with minimal deformation and higher resistance to rutting. For mix with 4.8% asphalt content, the intercept value increases 117% after one healing cycle, while it decreases by 40% after the second healing cycle as compared to the control mix.

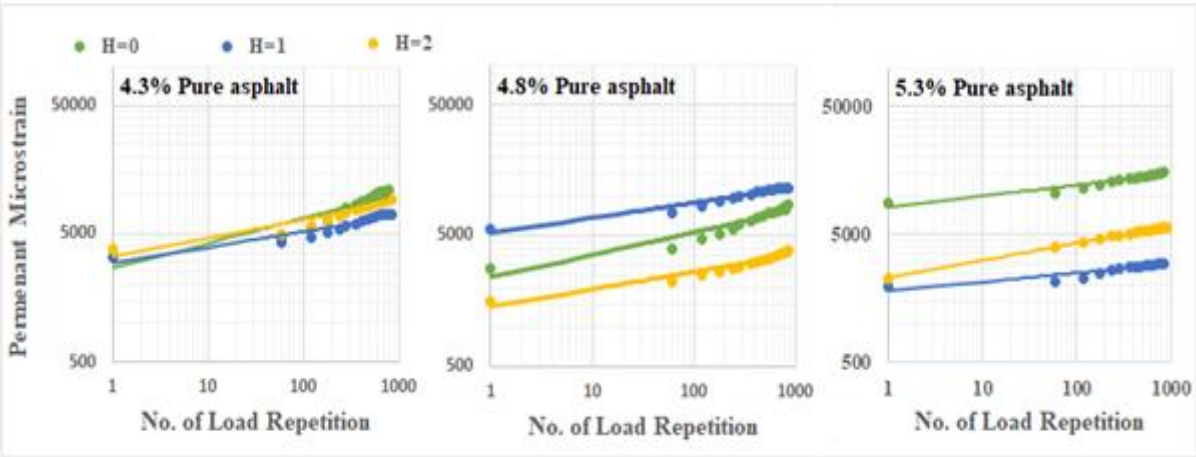

Figure 7. Influence of pure asphalt cement on permanent deformation parameters

For mix with 5.3% asphalt content, the intercept value decreases (78, 72) % after one and two healing cycles as compared to the control mix. The slope value is minimum after one healing cycle regardless of asphalt content.

Table 6. Influence of Pure Asphalt on Permanent Strain Parameters

Permanent Strain Parameters (pure Asphalt)						
Asphalt content%	**4.3**		**4.8**		**5.3**	
No. of Healing cycle	**Intercept**	**Slope**	**Intercept**	**Slope**	**Intercept**	**Slope**
0	2699.8	0.1947	2403.8	0.1713	8201.3	0.0870
1	2956.9	0.1227	5237.7	0.1158	1807.7	0.0707
2	3381.9	0.1402	1428.5	0.1337	2295.5	0.1358

Fig.8 Present the impact of healing cycles of SBS modified asphalt on permanent strain parameter under compressive strength, it can be noted that the intercept value decreases after healing cycles and the slope generally increases. The rate of change in intercept value can be noted from Table 7.
For mix with 5.1% asphalt, the reduction in the intercept value is (72, 70) % after one and two healing cycles respectively as compared to the control mix, while the slope value increases by (18 and 143) % after one and two healing cycles respectively. For mix with 5.6% asphalt, the reduction in the intercept value is (80, 78) % after one and two healing cycles respectively as compared to the control mix. The intercept value increases by (22 and 33) % when asphalt content rises to (5.6 and 6.1) % respectively as compared to 5.1% asphalt mix before healing. For mix with 6.1% asphalt, the reduction in the intercept value is (91, 79) % after one and two healing cycles respectively as compared to control mix. In general, the slope increases after healing.

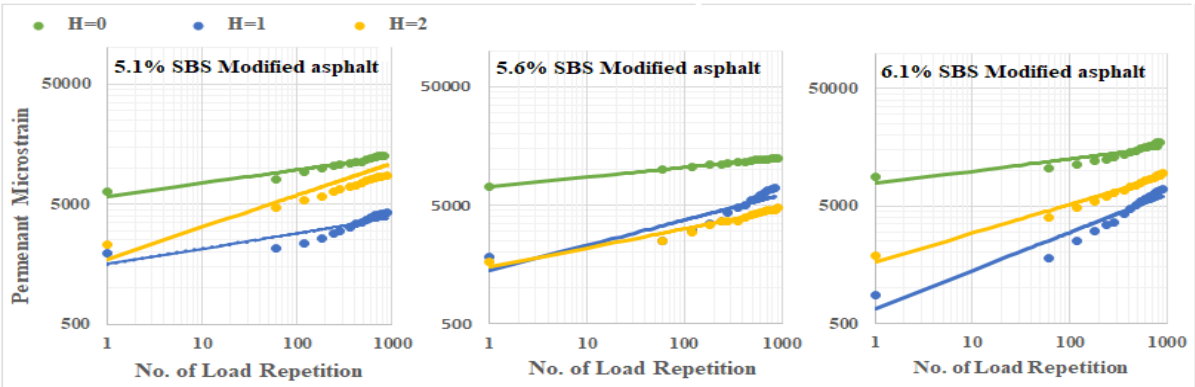

Fig.8. Influence of SBS Modified Asphalt Cement on Permanent Deformation Parameters

Table 7. Influence of SBS Modified Asphalt on Permanent Strain Parameters

Permanent Strain Parameters (SBS)						
Asphalt content%	**5.1**		**5.6**		**6.1**	
0	5831.7	0.1091	7137.3	0.0815	7787.4	0.1038
1	1577.3	0.1292	1395	0.2137	663.78	0.3252
2	1748.5	0.2652	1503.8	0.1605	1673.8	0.2438

Fig.9 shows the impact of healing cycles of LDPE modified asphalt on permanent strain parameter under compressive strength, it can be noted that the intercept value generally decreases after healing cycle, while the slope increases in general after healing cycles. The lowest micro strain levels generally could be noticed at optimum asphalt content before and after two healing cycles. Table 8 summarizes the influence of healing cycles and asphalt content on permanent strain parameters.

For mix with 4.8% LDPE modified asphalt, the reduction in the intercept is (63, 82) % after one and two healing cycles respectively as compared to the control mix. , also the intercept value decreases after the second healing cycle by 51% as compared that after the first healing cycle.

For mix with 5.3%, LDPE modified asphalt, the rate of decreases is (21, 75) % as compared to control mix, while at mix 5.8%**,** the rate of decreases is (63, 30) % as compared to control mix. On the other hand, the slope mostly increases after healing.

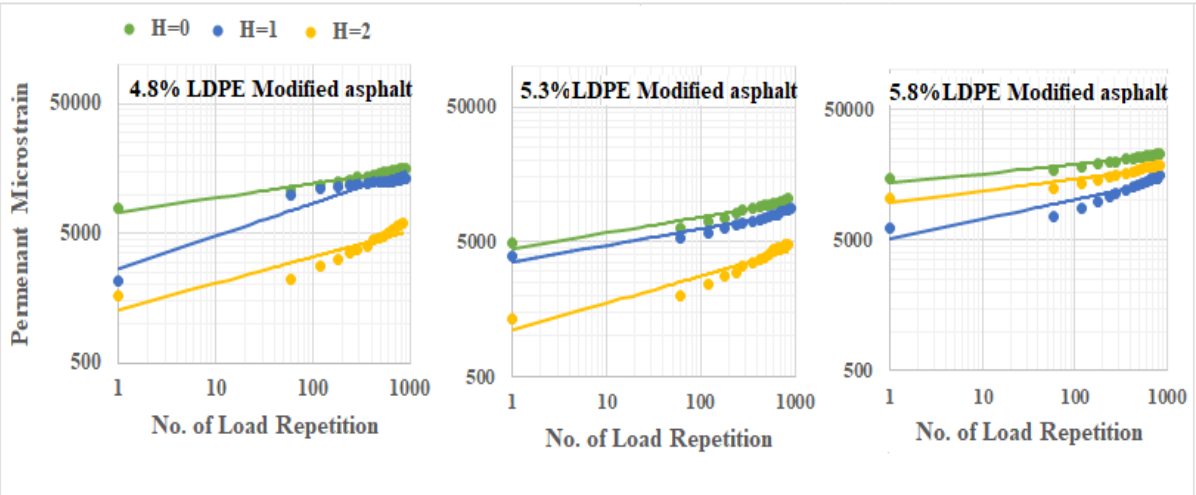

Figure 9. Influence of LDPE Modified Asphalt Cement on Permanent Deformation Parameters

Table 8. Influence of Asphalt Type and Healing Cycles on Permanent Strain Parameters

Permanent Strain Parameters (LDPE)						
Asphalt content%	**4.8**		**5.3**		**5.8**	
0	7271.7	0.1113	4495.9	0.1147	13691.0	0.0703
1	2676.5	0.2525	3549.9	0.123	5131.1	0.1487
2	1302.7	0.2011	1104.0	0.2008	9580.3	0.0919

Fig.10 shows the impact of healing cycles of Rubber modified mixture on permanent strain parameter under compressive strength, it can be noted that the intercept value decreases as healing cycles increases.
For mix with 4.8% rubber modified asphalt, the reduction in the intercept values is (87, 87) % as compared to the control mix.
For mix with 5.3% rubber modified asphalt, the rate of decreases in the intercept value is (54, 86) % as compared to control mixture. While at mix with 5.8% rubber modified asphalt, the rate of decreases in the intercept value is (90, 88) % as compared to control mix. Table 9 summarizes the rate of change in intercept value. On the other hand, the slope increases after healing, while it increases as the rubber modified asphalt cement content increases.

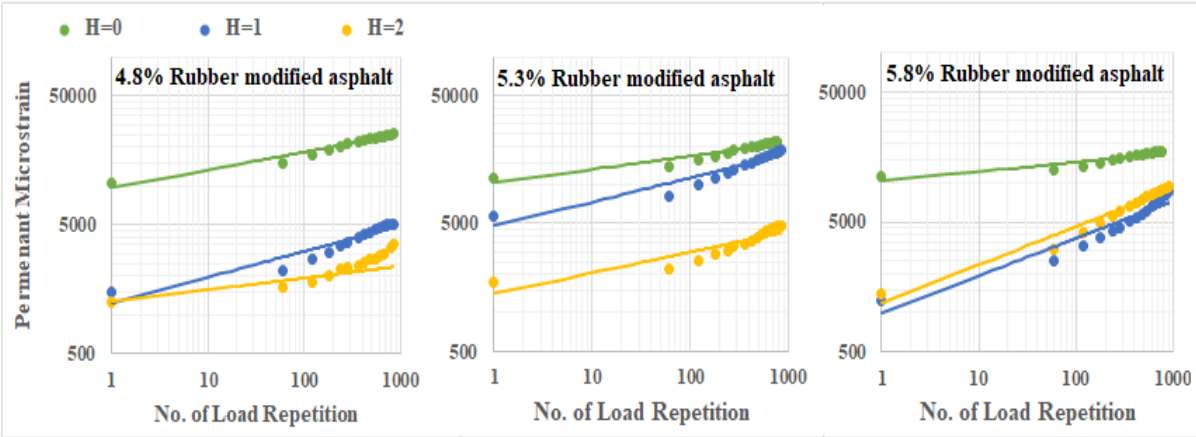

Figure 10. Influence of Rubber Modified Asphalt Cement on Permanent Deformation Parameters

Table 9. Influence of Asphalt Type and Healing Cycles on Permanent Strain Parameters

Permanent Strain Parameters (Rubber)						
Asphalt content%	**4.8**		**5.3**		**5.8**	
0	9781.6	0.1367	10442	0.1034	10293	0.0731
1	1245.6	0.1993	4774.7	0.1878	997.6	0.2871
2	1266.0	0.0923	1419.1	0.161	1190.5	0.2935

CONCLUSIONS

Based on the testing program, the following conclusions may be drawn:

1. For (pure asphalt) at (4.3, 4.8, and 5.3) % asphalt content, the permanent deformation under compressive strength decreases by (17, 72) %, (29, 67) %, and (80, 86) % after one and two healing cycles respectively as compared with control mix.

2. For (SBS) modified mix at (5.1, 5.6 and 6.1) % asphalt content, the deformation decreases by (32, 67) %, (63, 73) % and (46, 61) % respectively,

after one and two healing cycles as compared with control mix. The intercept decreases by (72, 70), (80, 78), and (91, 79) % after one and two healing cycles respectively, while the slope significantly increases after healing.

3. For (LDPE) modified mix at (4.8, 5.3 and 5.8) % asphalt content, the deformation decreases by (17, 63) %, (14, 53) %and (18, 32) % respectively after one and two healing cycles as compared with control mix. The intercept decreases by (63, 82), (21, 75), and (63, 30) % after one and two healing cycles respectively, while the slope mostly increases after healing.

4. For (Rubber)modified mix at (4.8, 5.3 and 5.8) % asphalt content, the deformation decreases by (80, 87) %, (16,18) % and (53,47) % respectively after one and two healing cycles respectively as compared with control mix. The intercept decreases by (87, 87), (54, 86), and (90, 88) % after one and two healing cycles respectively, while the slope increases after healing.

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ENHANCING STABILITY OF CLAYEY SUBGRADE MATERIALS WITH CEMENT KILN DUST STABILIZATION

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ABSTRACT

For clayey soil materials to be effective as pavement subgrades, satisfying the stability conditions is essential especially in wet and dry climatic conditions prevail. This could be achieved using some sort of stabilization methods. Stabilization of pavement subgrade soils has traditionally relied on treatment with lime, cement and sometimes special additives, which are regarded as waste materials. Alternatively, the by-pass Cement Kiln Dust (CKD) is generated during the course of Portland cement manufacture. It represents a mixture of raw feed, partly calcined cement clinker and condensed volatile salts. This study investigated the use of CKD to enhance the durability of clayey subgrade soils. Standard laboratory soil tests were conducted to measure changes in the engineering properties of clayey soils when treated with CKD. Tests were conducted on untreated control soil samples and on varieties of treated samples with CKD ranged from 2% to 10%. The results showed that the CKD clay samples exhibit low plasticity, swelling, maximum dry density and consolidation settlement. On the other hand, results showed significantly increasing in CBR, soil cohesion and soil strength. The internal friction angle concerning shear strength parameters is also enhanced. Overall, the testing programme produced data showing that mixtures with CKD admixture have significant enhancement in engineering properties of clayey soils. This is advantageous for work construction in the civil engineering field.

KEYWORDS: Unusual stabilizer; Clayey Subgrade; Cement Dust Stabilizer, CKD

1. INTRODUCTION

Material engineers are continually confronted by the depletion of quality construction materials for road construction. Even if good quality construction materials are available, the haul costs may preclude their use. Engineers are frequently required to incorporate poor quality soil and aggregate into pavement designs. For example, most of the agricultural roads in the Egyptian roads network were originally constructed as a canal or drain embankments, which are molded with high percent of clay. Clays are notoriously well known for giving rise to swelling problems and difficulties in construction due to excessive settlement and limited strength. These poor quality materials typically have the potential to demonstrate undesirable engineering behavior such as, low bearing capacity, high shrink/swell potential, and poor durability (Chen 1981, Cokça 1999, Abo-Hashema et al. 1994, Kumar and Puri 2013). Hence, such types of soils need to be stabilized before construction for improving their engineering properties. Traditional stabilization methods include the application of various combinations of lime, cement, fly ash, and bituminous materials. These traditional stabilization techniques often require lengthy cure times and relatively large quantities of additives for significant strength improvement. Delays in construction can be costly if adequate planning has not accounted for material cure times (Muntohar 1999).

One may achieve stabilization by mechanically mixing the natural soil and stabilizing material together to achieve a homogenous mixture or by adding unusual stabilization additives. These additives range from waste products to manufactured material, which includes Silica Fume, Rice Husk Ash, Portland cement, Fly ash, chemical stabilizers and Cement Kiln Dust (CKD). These additives can be used with variety of soils to improve their original engineering properties. The effectiveness of these additives depends on the soil treated and the amount of additive used. The high strength obtained from cement and lime may not always be required, however, and there is justification for seeking cheaper additives, which may be used to alter soil properties (Muntohar 1999, Filho et al. 2007, Abd El Aziz 2003, Abd El Aziz and Abo-Hashema 2013).

These unusual stabilizers are marketed as requiring lower material quantities, reduced cure times, higher material strengths, and superior durability compared to traditional stabilization additives (Abd El Aziz 2003, Abd El Aziz and Abo-Hashema 2013). However, most transportation agencies are hesitant to specify such these unusual stabilizers without reliable data to support vendor claims of product effectiveness. Unfortunately, the rapid evolution of existing products and introduction of new stabilizers further complicate the process of defining the performance characteristics of the various unusual soil stabilization additives. While, the nature of soil stabilization dictates that stabilizers may be soil-specific and/or environment-sensitive. In other words, some stabilizers may work well in specific soil types in a given environment, but perform poorly when applied to dissimilar materials in a different environment (Muntohar 1999). For that reason, this research is trying to identify the effect of using unusual stabilizer, such as CKD on the engineering properties of clayey subgrade.

The main objective of this study is to investigate the effect of blending clayey soils with Cement Kiln Dust on their engineering properties. Tests were conducted on untreated control soil samples and on samples treated with CKD. A series of laboratory experiments have then been conducted for varieties of samples: 2%, 2%, 6%, 8%, and 10%. Each treated and untreated soil was characterized in terms of the Atterberg limits, swell potential, compaction characterization, California Bearing Ration (CBR), consolidation test, and un-drained Triaxial shear strength.

2. LITERATURE REVIEW

Soil Stabilization, in its broadest sense, implies the improvement of both durability and strength of soil (Yoder and Witczak 1975). A review of the literature indicates that there has been a large quantity of research completed regarding the application of traditional stabilization additives such as lime, cement, and fly ash. However, little research has been documented pertaining to the use of unusual stabilization additives. Material engineers may find a large quantity of advertisements, pamphlets, and videos in the market testifying to the benefits of a particular stabilization additive. Unfortunately, most of the information disclosed in these media are subjective and traditional engineering properties are poorly documented.

Xeidakis (1996) investigated the stabilization of the swelling clay structure by intercalation of Mg(OH)2 and the development of a brucite interlayer between the clay layers. A final conclusion that could be drawn from this work is that the intercalation of Mg-hydroxide into the clay layers and the stabilization of the swelling clay structures are beyond any doubt; nevertheless, the method, as formulated, is not easily applicable to the field; more research is needed in this direction.

Ajayi-Majebi et al. (1991) conducted an experiment designed to determine the effects of stabilizing clay-silt soils with the combination of an epoxy resin (bisphenol A/epichlorohydrin) and a polyamide hardener. The additive mixture was composed of a 1:1 ratio of epoxy resin to polyamide hardener. Ajayi-Majebi et al. concluded that admixing up to 4 percent stabilizer into a clay-silt material produced large increases in the load-bearing capacity of the material in terms of its un-soaked CBR. They observed that increases in the temperature of the curing environment led to increased strength formation. Cure times for the stabilization agent were reported as low as 3 hours.

Kalkan and Akbulut (2004) measured the positive effects of silica fume on the permeability, swelling pressure and compressive strength of natural clay liners. They concluded that a significant improvement on the permeability, swelling pressure and compressive strength of composite samples was obtained by using silica fume. The investigation showed that the silica fume is a valuable material to modify the properties of clay liners to be used in the landfill sites.

Bell (1996) used lime as stabilizer for clay minerals and soils. The research proved that lime could be used to enhance the engineering properties of clayey soils. All materials experienced an increase in their optimum moisture content and a decrease in their maximum dry density, as well as enhanced CBR, on addition of lime. Some

notable increases in strength and Young's Modulus occurred in these materials when they were treated with lime. Length of time curing and temperature at which curing took place had an important influence on the amount of strength developed.

Miller and Azad (2000) performed a laboratory study to evaluate the effectiveness of Cement Kiln Dust (CKD) as a soil stabilizer. The study revealed that increases in the unconfined compressive strength (UCS) of soil occurred with the addition of CKD. Increases in UCS were inversely proportional to the plasticity index (PI) of the untreated soil. Significant PI reductions occurred with CKD treatment, particularly for high PI soils.

Chen and Lin (2009) conducted experiments to evaluate using of incinerated sewage sludge ash (ISSA) when mixing with cement in a fixed ratio of 4:1 for use as a stabilizer to improve the strength of soft, cohesive, subgrade soil. The study showed that the unconfined compressive strength of specimens with the ISSA/cement addition was improved to approximately 3–7 times better than that of the untreated soil; furthermore, the swelling behavior was also effectively reduced as much as 10–60% for those samples. In some samples, the ISSA/cement additive improved the CBR values by up to 30 times that of untreated soil. This suggested that ISSA/cement has many potential applications in the field of geotechnical engineering.

Al-Rawas et. al. (2005) conducted a research to investigate the effect of lime, cement and Sarooj (artificial pozzolan) on the swelling potential of an expansive soil from Oman. The physical results of the treated samples were determined. The untreated soil values were used as control points for comparison purposes. It was found that with the addition of 6% lime, both the swell percent and swell pressure reduced to zero. Heat treatment reduced swelling potential to zero. The use of lime showed superior results when compared with the other stabilizers.

Hashim et. al. (2005) investigated the effect of using rice husk ask and cement on a stabilization of residual soil. Test results showed that both cement and rice husk ash reduced the plasticity of soils. In term of compactability, addition of rice husk ash and cement decreases the maximum dry density and increases the optimum moisture content.

3. MATERIALS

3.1 Test Soils

The soil used in this study was obtained from a site located in Gaafraa village, Fayoum City, Egypt, next to Fayoum-Etsa Road. A test pit was excavated to obtain disturbed samples. The expansive soil was encountered at a depth of about 0.60m overlaid by a layer of sand and silt. A field density test was carried out in the pit. The disturbed soil was excavated, placed in plastic bags, and transported to the Soil Mechanics Laboratory at Fayoum University for preparation and testing. The physical characteristics of the untreated soil are shown in Table 1.

The untreated soft subgrade soil is categorized as clayey soil (Gs = 2.68 with 90.76% fines) with expansive behavior. Soil categorization tests that follow the standard of

ASTM D1883-87 (1998) and the classifications of AASHTO refer the untreated subgrade soil to the A-7-5 category, which stands for high-plasticity clay. Based on Universal Soil Classification System (USCS), it was classified as CH, which is clay with high plasticity. All geotechnical tests were performed in accordance with ASTM D 4318-95 (1998), ASTM D 1557-91 (1998) and ASTM D 4546-96 (1998). The soil showed a high plasticity index (39.95%) and an activity of 3.059. Generally, the higher the plasticity index and activity of a soil are; the higher the swelling potential is, which is measured as 48%.

Table 1. Physical Characterization of Clay Sample

Physical Properties	Value
Basic Characteristics	
Depth (m)	0.60
Natural moisture content (%)	71.38
Moisture content (disturbed), %	18.32
Specific Gravity, Gs	2.68
Atterberg Limits[1]	
Liquid Limit, LL (%)	74.85
Plastic Limit, PL (%)	34.90
Shrinkage Limit, SL (%)	13.82
Plasticity Index, PI (%)	39.95
Compaction Properties[2]	
Maximum Dry Density, γ_d	1.33gm/cm3
Optimum Moisture Content	34%
Grain size distribution	
- Coarse particles	9.24
- Fine particles	90.76
- Clay	85.00
- Silt	15.00
Activity	3.059
CBR	3%

[1] Liquid Limit, Plastic Limit, Shrinkage Limit, and Plasticity Index, ASTM D 4318 (1998)
[2] Optimum water content for compaction with a modified Proctor effort, ASTM D 1557 (1998)

3.2 Stabilizer Product: Cement Kiln Dust

Cement kiln dust (CKD) is the fine-grained, solid, highly alkaline waste removed from cement kiln exhaust gas by air pollution control devices. Because much of the CKD is actually unreacted raw materials, large amounts of it can and are, recycled back into the production process. Some CKD is reused directly, while some requires treatment prior to reuse. CKD not returned to the production process is typically disposed in land-based disposal units (i.e., landfills, waste piles, or surface

impoundments), although some is also sold for beneficial reuse (Kumar and Puri 2013).
The cement plants generate large quantities of by-pass cement kiln dust during the manufacture of cement clinker constituting a great source of air pollution. The by-pass cement dust contains a mixture of raw feed, partly calcined cement clinker and some condensed volatile salts.

CKD was collected from a cement plant located in Fayoum city, Egypt. It was classified as SM as per specifications of Unified Soil Classification System, ASTM D 2487-06 (1998). Table 2 depicts CKD Chemical and Physical Analysis. Figure 1 shows XRD pattern of Cement Kiln Dust, collected from electrostatic precipitators.

Table 2. CKD Chemical and Physical Analysis

Chemical Analysis	Percentage
Silicon Dioxide, SiO2	17.62
Aluminum Oxide, Al2O3	4.90
Iron Oxide, Fe2O3	2.58
Calcium Oxide, CaO	62.09
Magnesium Oxide, MgO	1.93
Sodium Oxide, Na2O	0.56
Potassium Oxide, K2O	3.76
Sulfur Trioxide, SO3	5.79
Moisture Content	0.07
Loss on Ignition	4.94
Available Lime Index, CaO	33.70
Water-Soluble Chlorides, CL	--
Physical Analysis	
Retained on No. 325 sieve (%)	16.9
Specific Gravity	2.95

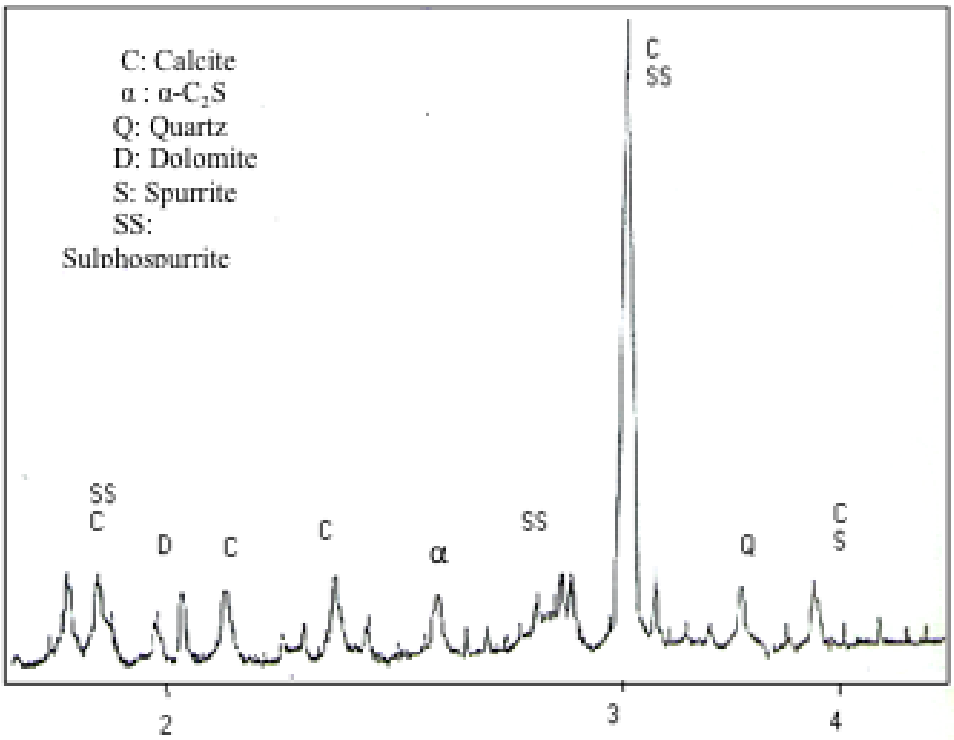

Figure 1. XRD Pattern of Cement Kiln Dust, Collected from Electrostatic Precipitators

4. SPECIMEN PREPARATION

Test samples were mixed from pulverized, air-dry soil and de-ionized water. Treated specimens were prepared following the nine-step protocol outlined here. Untreated control specimens were prepared in the same manner, but without the addition of the stabilizer product (CKD).

1. Using the modified Proctor compaction test, ASTM D 1557-91 (1998), the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) for compaction were determined for the untreated soil. These values are listed in Table 1.

2. Based on the product literature, the recommended percentages of the stabilizer product were as follow: 2%, 4%, 6%, 8% and 10% for CKD.

3. The test soil was pre-moistened to a natural moisture content = 71.38 (Table 1). The soil was mixed dry of optimum at this point to allow for the water that would be added with the stabilizer in Step 6.

4. The pre-moistened soil was allowed to mellow for at least 16 hours in a sealed container.

5. The mass of stabilizer needed to achieve the recommended OMC in the treated sample was measured out.
6. The stabilizer was thoroughly mixed with the soil sample, which was then allowed to stand for 1 hour in a covered container. If there were no evaporation losses, the soil water content would now be equal to the OMC.
7. The treated soil was compacted using a modified Proctor effort, ASTM D 1557-91 (1998), extruded from the compaction mold, and sealed in a plastic bag.
8. The compacted soil was cured in a sealed plastic bag at room temperature for 7 days.
9. The cured sample was trimmed to an appropriate size for testing. If the specimen water content was more than 3% above or below the OMC, new specimens were prepared using adjusted initial water content. Almost all test specimens were within ±2% of the target OMC.

A seven-day curing period was selected as a reasonable delay to allow reactions between the stabilizer and the soil prior to conducting the evaluation tests. Laboratory assessments of soil stabilizers often include a 28-day cure following treatment; the additional three weeks may, depending on the stabilizer, yield additional changes in the soil properties. However, it is expected that significant changes due to an effective soil treatment should be measurable at seven days.

It is noteworthy that all specimens, whether untreated or treated, were compacted at the same optimum water content. In this study, however, the same water content was used to prepare all specimens of a given soil, so that the effect of the stabilizer on the measured soil properties could be distinguished from the effects of varying the water content. For the same reason, the samples were maintained at constant water content during the curing period.

5. LAB EXPERIMENTAL WORKS

A series of laboratory experiments have been conducted for varieties of samples. These experiments are to measure the engineering properties of the soil as follow:

5.1 Measurement of Specific Gravities

Specific gravities of untreated and treated clay subgrade soils have been determined based on the recommended percentages of the stabilizer product: 2%, 4%, 6%, 8% and 10%. Measurement of Consistency Limits

The liquid limit, plastic limit, and plasticity index of the natural and stabilized clayey soil samples were determined by Atterberg tests in accordance with ASTM D 4318 (1998).

5.2 Measurement of Swelling Potential

Many researchers have used the term swelling potential. However, a clear definition of the term has not been established. Generally, swelling potential has been used to describe the ability of a soil to swell, in terms of volume change or the pressure required to prevent swelling. Therefore, it has two components: the swell percent, which is defined as the percentage increase in height in relation to the original height, and the swell pressure, which is designated as the pressure, required to prevent swelling.

The swell percent of each test specimen was measured in accordance with ASTM D 4546 (1998). The apparatus used was the standard one-dimensional oedometer. The specimen in its ring was placed between two porous stones with load plate resting on the upper porous stone. The consolidation cell was assembled in the consolidation frame. The specimen was then loaded to a seating pressure of 2.4 kPa. The pressure was maintained until full settlement was achieved. The specimen was then flooded with water and allowed to swell under the seating load. Deformation readings were taken at 0, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, 30.0, 60.0, 120 and 1440 min, and then every four hours on subsequent days until no further changes in readings were observed and full swell was attained. The increase in vertical height of a sample, expressed as a percentage, due to the increase in moisture content was designated as the ‘‘swell percent’’.

5.3 Measurement of Grain Size Distribution

Grain Sieve analysis has been conducted for the untreated and treated clayey soils to measure the effect of CD stabilizer on the percent of finer and coarse materials.

5.4 Measurement of Compaction Characterization

The compaction parameters such as the maximum dry unit weight (MDD) and the Optimum Moisture Content (OMC) were obtained by Standard Proctor tests using modified effort in accordance with ASTM D 1557 (1998). For this procedure, natural soil and CKD were blended with various amounts of water. During the compaction process, a soil at selected water content was placed in five layers into a mold of standard dimensions, with each layer compacted by 25 blows of hammer dropped from a distance of 457 mm, subjecting the soil to total 56,000 ft-lb/ft3 compaction effort. Each material was evaluated at six different water contents. This established OMC for preparing all of the compacted soil specimens for swell and triaxial testing.

5.5 Measurement of California Bearing Ration and Consolidation Parameter

A portion of 6 kg materials was prepared at the OMC and compacted using a 2.5-kg mechanical hammer. The specimens were compacted in the three layers under 62 blows of hammer for each. After 7 days of moist-curing, the specimen was then soaked for 7 days in water and the other specimen continued to be cured until its old was 14 days. From the test results, an arbitrary coefficient CBR was calculated. This was done by expressing the forces on the plunger for a given penetration, 2.5 and 5 mm, as a percentage of the standard force.

5.6 Measurement of Shear Strength

To measure soil strength, unconsolidated-undrained (UU) Triaxial compression tests were conducted following ASTM D 2850 (1998). The shear strength of highway materials is often characterized using unconfined compression tests, but testing in a triaxial cell yields a more reliable measure of strength. This is especially true for fissured, compacted soils where the confining pressure keeps the specimen intact under load. The compacted and cured soil samples were trimmed into test specimens measuring 38 mm in diameter by 95 mm height at OMC and MDD. During shearing, volumetric strains were measured from changes in the volume of cell fluid. The cross-sectional area of each specimen was corrected using the axial and volume strains and assuming a right-circular cylinder shape.

6. RESULTS AND ANALYSIS

Based on the laboratory experiments, the soil is classified as Clay (Gs = 2.68 with 90.25% fines) with expansive behavior. Soil with PI > 35% is classified to have very high swell potential (Chen 19981), where LL and PI of the sample is respectively 74.85% and 39.95%. The effect of blending CKD on the physical properties of clayey soil can be described in the following subsections:

6.1 Effect of CKD Stabilizer on Specific Gravities

Specific Gravities (Gs) for the untreated and treated soils were determined and the results are plotted in Figure 2. As shown in Figure 2, by increasing CKD the specific gravity of the soil decreases by 6.0 % compared to the untreated sample. This indicates that the treated soil is lighter than that of its natural conditions.

6.2 Effect of CKD Stabilizer on Consistency Limits

The Atterberg limits (liquid limit, plastic limit and plasticity index) of the untreated and treated samples were determined following ASTM D 4318 (1998). The results are plotted in Figure 3 (a), (b), and (c). Results showed, in general, a decrease in liquid limit of all samples. The reduction reached 56.0% when using 10% CKD.

Results showed a fluctuation in plastic limit in some samples when adding CKD. In case of adding CKD 10%, reduction in plastic limit is observed.

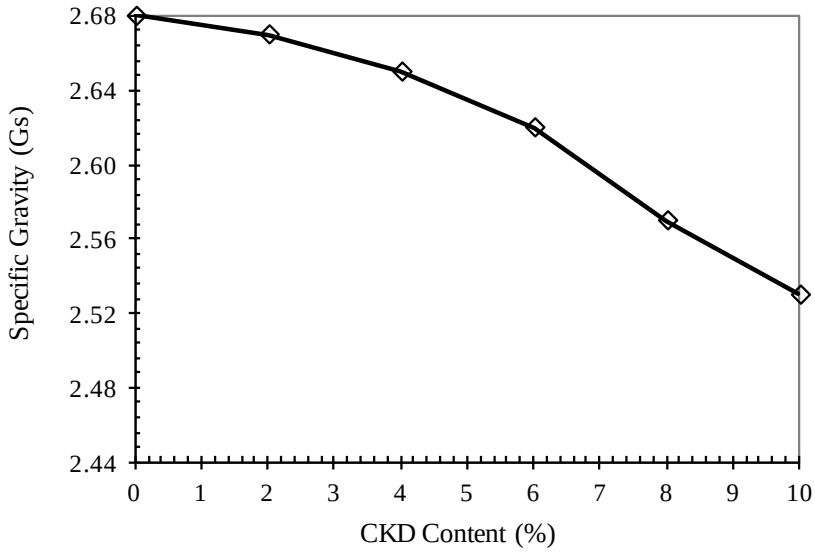

Figure 2. Effect of Blended CKD on the Specific Gravity of Soil

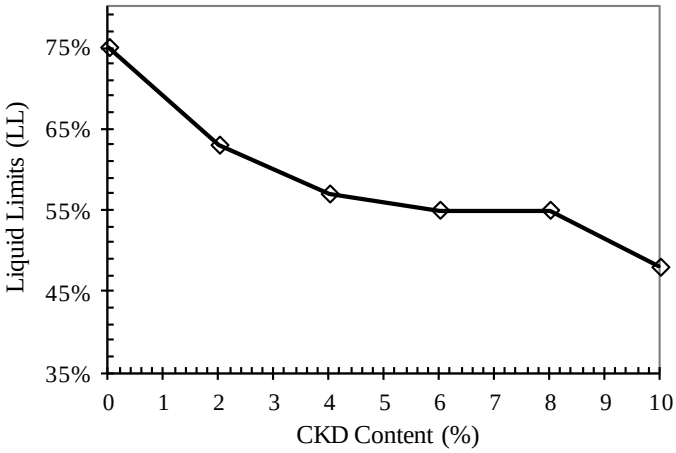

Figure 3-a: CKD Vs Liquid Limit

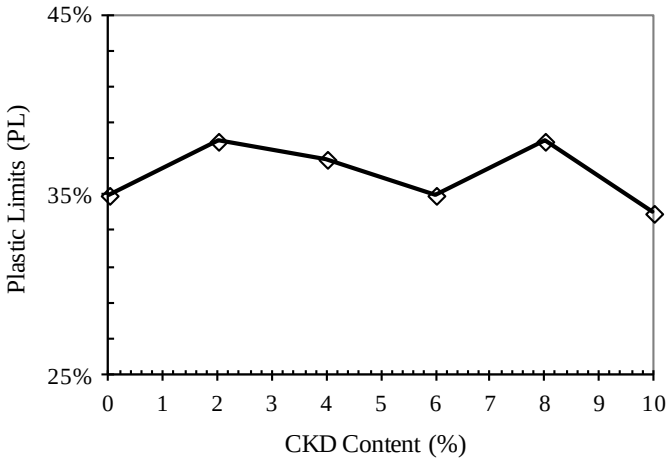

Figure 3-b: CKD Vs Plastic Limit

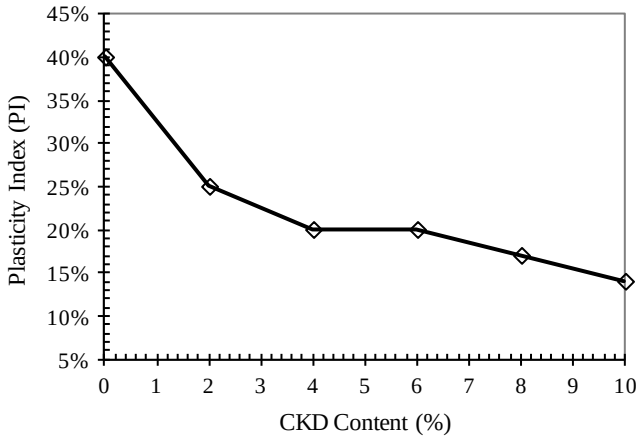

Figure 3-c: CKD Vs Plasticity Index

Figure 3. Influence of CKD on the Consistency Limits

(a) Liquid Limit (b) Plastic Limit (c) Plasticity Index

It can be also observed that CKD reduce the Plasticity Index (PI) up to 65% compared to untreated soils. CKD show reduction in plasticity between 40-65% when using CKD from 2% to 10% compared to the original condition. In conclusion, PI reduction occurred with CKD treatment, which indicates an improvement.

6.3 Effect of CKD Stabilizer on Swelling Potential

Swell percent test was carried out on untreated and treated samples to examine the effect of the various additives on the reduction of the swelling potential of the clayey soil. The swell percent value obtained for the untreated clayey soil was 50%. The swell results are presented in Figure 4, which indicates that swell potential decreases as well.

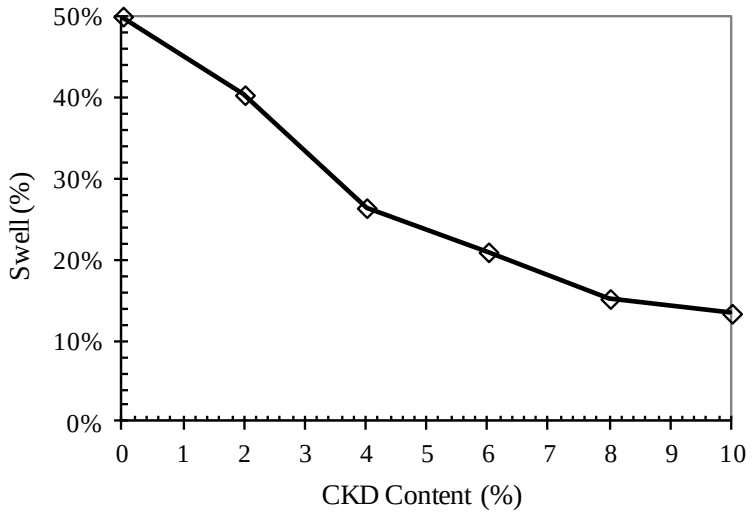

Figure 4. Influence of CKD on the Swelling Potential

It is clear from the test results that CKD significantly decreased the swelling of clayey subgrade soil. The swell percent of the clayey soil is reduced from 50% to 13%, i.e. 75% reduction when using 10% CKD stabilizer caused significant reduction in swell percent.

6.4 Effect of CKD Stabilizer on Grain Size Distribution

The effect of CKD on the Grain Sieve of clayey soil is presented in Figure 5. Figure 5 (a) shows that there is a significant decrease in the percent of finer particles. On the other hand, there is a significant increase in the coarse particles as shown in Figure 5(b). This result indicates that significant improvement in the clayey soil in terms of grain size distribution has been achieved.

6.5 Effect of CKD Stabilizer on Compaction Characterization

Moisture-unit weight curves for all the untreated and treated test soils were determined using a modified proctor compaction effort, ASTM D 1557-91 (1998). Figure 6 shows the variation of the optimum moisture content and maximum dry unit

weight values of stabilized samples with CKD. There is an increase in the optimum moisture content and a decrease in the maximum dry unit weight due to the addition of CKD. The reason for increase in the optimum moisture content is due to the change in surface area of composite samples. The CKD changes the particle size distribution and surface area of the stabilized clayey soil samples. In the same way, the reason for the decrease in the maximum dry unit weight is the addition of higher amounts of CKD, which fills the voids of the composite samples.

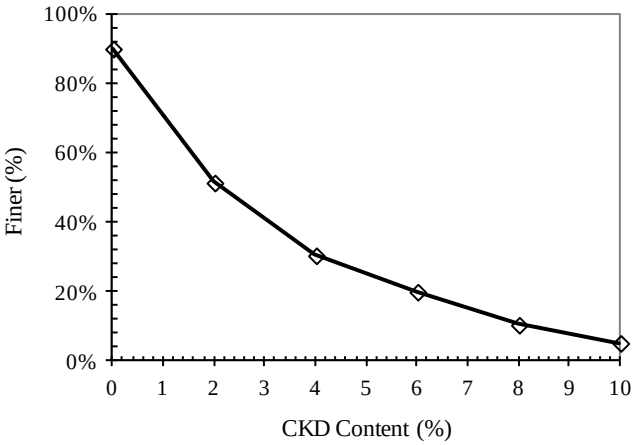

Figure 5-a: CKD Vs Finer Soil

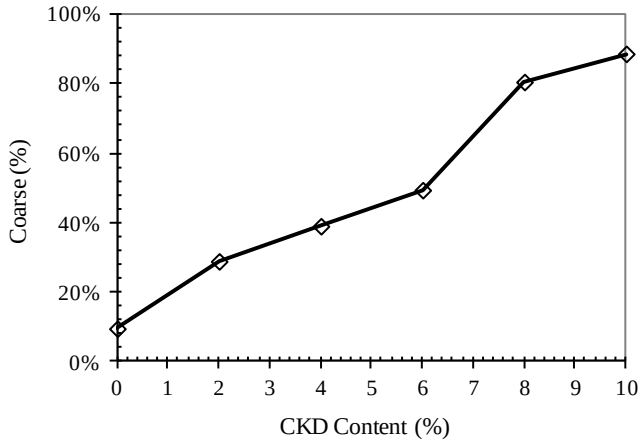

Figure 5-b: CKD Vs Coarse Soil

Figure 5. Effect of CKD on the Grain Sieve of Clayey Soil
(a) Finer Particle of Soil and (b) Coarse Particle of Soil

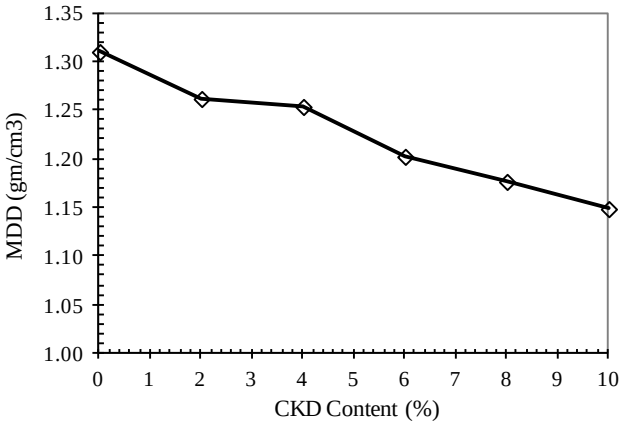

Figure 6-a: CKD Vs Maximum Dry Density

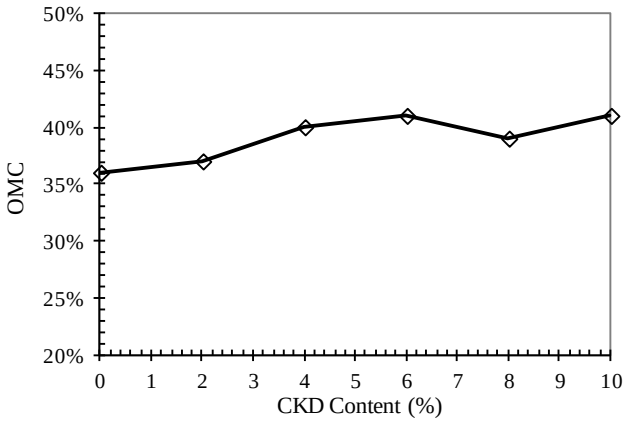

Figure 6-b: CKD Vs OMC

Figure 6. Influence of CKD on MDD and OMC of Clayey Soil
(a) Maximum Dry Density (MDD) and (b) Optimum Moisture Content (OMC)

Statistically speaking, the MDD is decreased by 12% when 10% CKD is added. The maximum reduction in MDD is observed when using 10% CKD. Equal reduction is also noticed when using 2-4% CKD. For OMC, the highest increase is observed with 6% CKD and 10 % CKD, which is 15% increasing compared to the untreated sample.

Compaction characterization of stabilized clayey soil with CKD is plotted in Figure 7. Note that the water contents of many test specimens were a little dry of the target optimum, due to evaporation losses during mixing. However, considering the normal variability obtained in preparing compacted soil samples, there appears to be no significant effect of the stabilizer treatments on the compacted soil unit weight or void ratio. Figure 7 shows that soil, which has been blended with CKD, is best to be compacted in the wet optimum state. Therefore, blended CKD has a place in construction work where a soil's moisture content is very high.

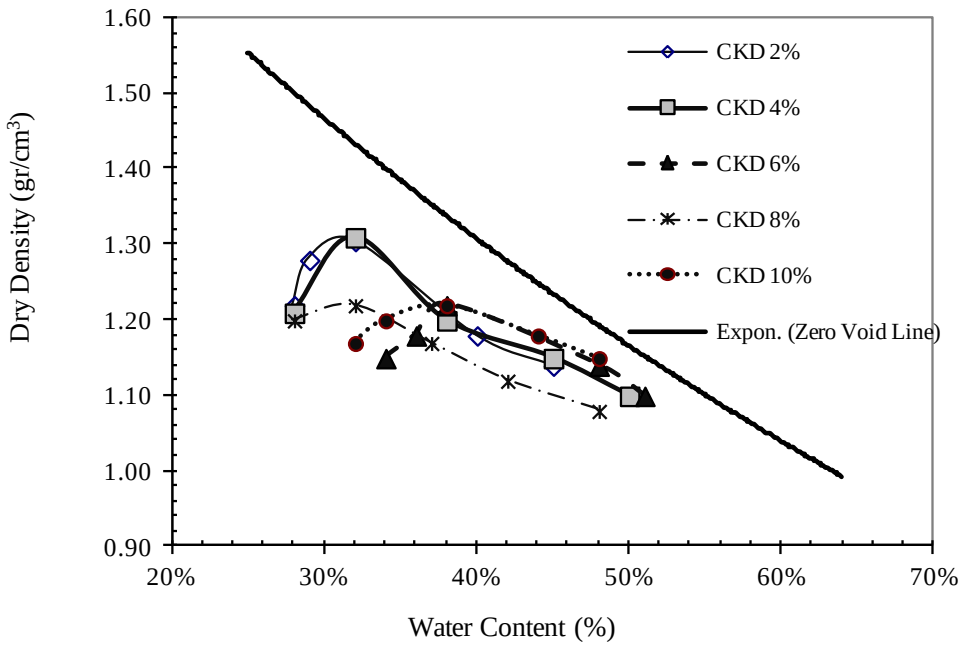

Figure 7. Compaction Characterization of Clayey Soil with Blended by CKD

6.6 Effect of CKD Stabilizer on CBR and Consolidation

Results of California Bearing Ration (CBR Laboratory) and Consolidation Parameter of soil when blending with CKD are presented in Figure 8. Figure 8(a) reveals a trend to enhance and attain the optimum CBR value at 6% CKD and 10% CKD. This means that the CBR of specimens, with the CKD addition 6-10%, was improved to approximately four times better than that of the untreated soil. This enhancement in the CBR is considered excellent improvement to the bearing capacity of the soil. Figure 8(b) shows the compressibility index (Cc) tends to be non-linear. This condition exhibits that when the rate of consolidation is rapid, the settlement of the soil will reduce.

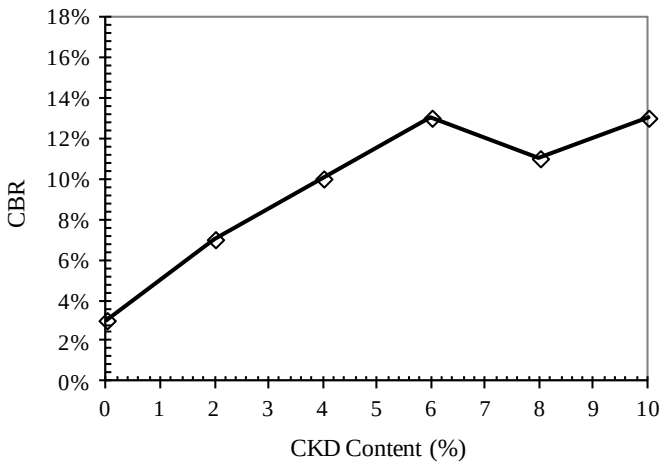

Figure 8-a: CKD Vs CBR

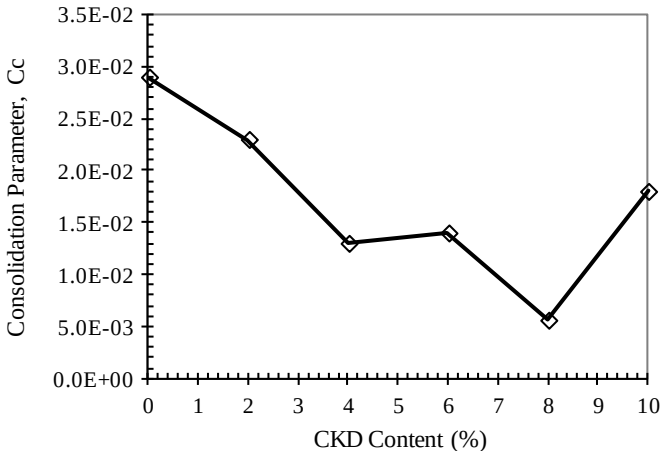

Figure 8-b: CKD Vs Compressibility Index

Figure 8. California Bearing Ration (CBR Laboratory) and Consolidation of stabilized Soil with CKD
(a) CBR Laboratory and (b) Compressibility Index (Cc)

6.7 Effect of CKD Stabilizer on Shear Strength

Typical results, from UU Triaxial tests, are plotted in Figure 9. Stress-Strain behaviour of clayey soil under Triaxial Test is presented in Figure 10. In general, the shear strength parameter, cohesion and internal angle can be enhanced by addition of CKD.

It can be observed from Figure 9 that using up to 6% of CKD has minimal or no effect on shear strength when mixed with clay soils. The effect of CKD starts to appear when CKD is added by above 6%. In conclusion, the soil strength is enhanced and the maximum ultimate strength is attained at 6-10% CKD. Figure 10 shows brittle behavior of the soil when mixed with CKD.

Therefore, it can be concluded that CKD can improve the engineering properties of clayey subgrade soils. Practically, the effective CKD could be blended in the range of 4-8%.

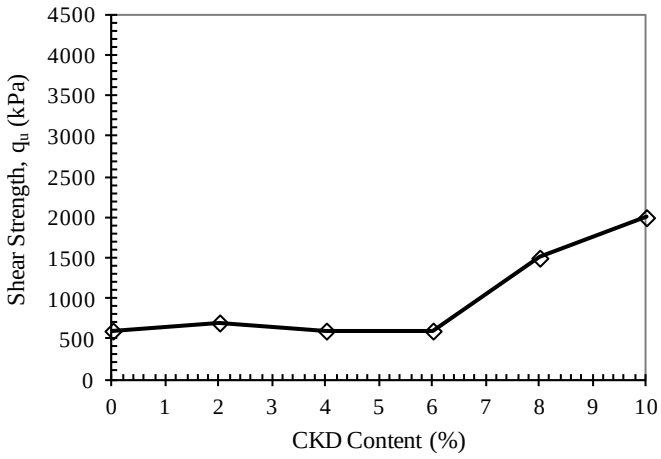

Figure 9-a: CKD Vs Shear Strength

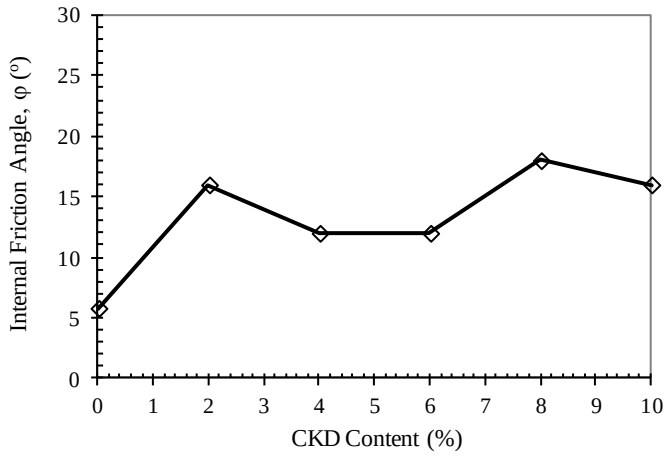

Figure 9-b: CKD Vs Internal Friction

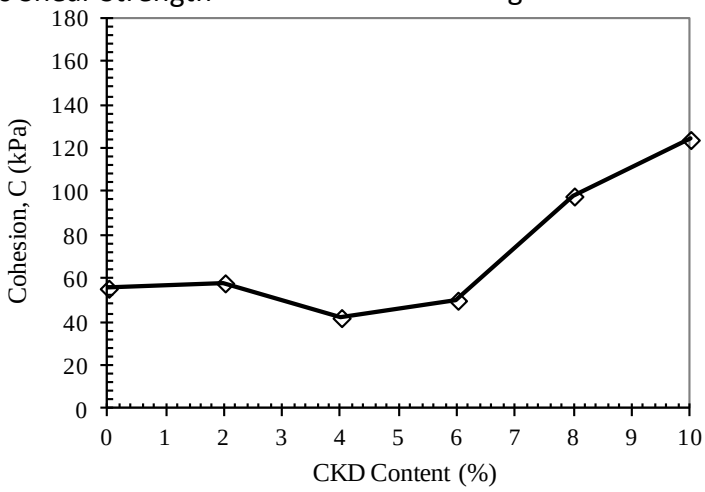

Figure 9-c: CKD Vs Cohesion

Figure 9. Effect of CKD on the Shear Strength of Clayey Soil
(a) Shear Strength of Soil ($q_{ultimate}$), (b) Internal Friction Angle (ø), and (c) Cohesion

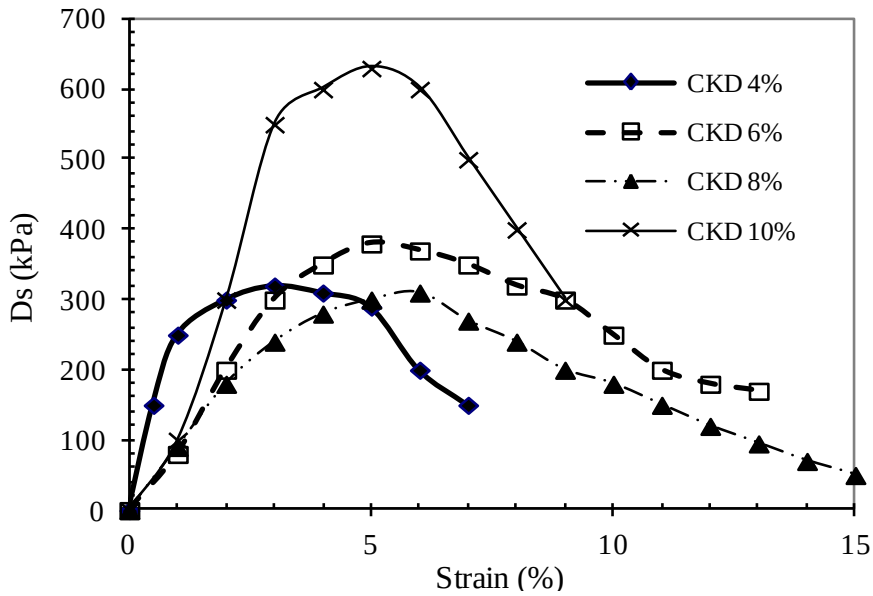

Figure 10. Stress-Strain Behavior of Clayey Soil under Triaxial Test

7. CONCLUSIONS

Cement Kiln Dust (CKD) can be used as unusual soil stabilizer to improve the engineering properties of clayey subgrade. CKD is solid waste materials produced from the manufacture of cement factories. In this study, the effect of adding CKD on the engineering properties of clayey subgrade soils has been investigated. A series of laboratory experiments have been conducted for varieties of samples: 2%, 4%, 6%, 8% and 10 % for CKD. The following conclusions have been drawn:

- The CKD decreases the specific gravities of the clayey soil samples. This indicates that the treated soil is lighter than that of its natural conditions.
- The CKD decreases the liquid limits and plasticity index; and increases the plastic limits in all the stabilized clayey soil samples. For this reason, the soil types of composite samples with high CKD contents change from high plastic to low plastic.
- The CKD has shown a significantly decreasing in swelling potential.
- The CKD has a significant decrease in the percent of finer particles. On the other hand, there is a significant increase in the coarse particles.
- The CKD changes compaction parameters. The addition of CKD increases the optimum moisture content and decreases the maximum dry unit weight.
- The CBR of specimens was improved to approximately 4-times better than that of the untreated soil.
- The shear strength, internal friction angle and cohesion of clayey soil samples increase due to the addition of CKD.
- The modification of clayey subgrade soils using CKD can be a viable and innovative method to enhance the engineering properties.

These findings are considered vital in improving the engineering properties of the clayey soils.

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SKIDDING RESISTANCE: MEASUREMENT AND USE OF DATA

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ABSTRACT

The first Sideways-force Routine Investigation Machine was manufactured in 1967 by W.D.M. limited for the TRRL, following extensive research from the 1930's into measuring the skid resistance of road surfaces. It is proven that the provision of appropriate levels of skid resistance can provide significant benefits in reducing skid related accidents. Fifty years later it has become the principal means by which Highway Authorities in the United Kingdom, New Zealand and many other countries assess skid resistance, and WDM have manufactured over 80 machines. The principal of SCRIM is to measure the force exerted on the test-wheel at a 20^0 angle, which is a measure of the skidding resistance of the road surface.

Modern skid polices are investigatory by nature, and involve assessing the skid resistance against an Investigatory Level (IL). If the skidding resistance is below the IL the Highway Authority puts in place a series of measures to assess and manage the associated safety risks. This does not always involve surface treatment, but where it does, the design should ensure that the new surface would provide an acceptable level of skid resistance for the life of the surface. This can be done by a number of techniques, and typically involves the selection of coarse aggregates that meet a polishing criteria defined by the PSV test. Research from a number of authorities is presented which demonstrates the different performance of aggregates, and how they have been used by a number of authorities.

INTRODUCTION

The Sideway-force Routine Investigation Machine (SCRIM) is the principal method used to assess the skidding resistance of road surfaces in the United Kingdom, and many other countries. The first SCRIM was commissioned in 1967, and since the 1970's has been used on the UK trunk road network. Standards for the operation of SCRIM, interpretation of data and application have been developed, with a number of standards being published, the most recent of which is HD28/15 (DMRB 2015).

Figure 1:1967 SCRIM (TRRL)

SCRIM works on a sideways force principle. The test wheel is orientated at an angle of 20^0 to the direction of travel, and a fixed load of 200 kg applied. The Sideways force generated by the test wheel is measured, and this is used to determine the skid resistance of a road surface. Figure 2 illustrates the concept.

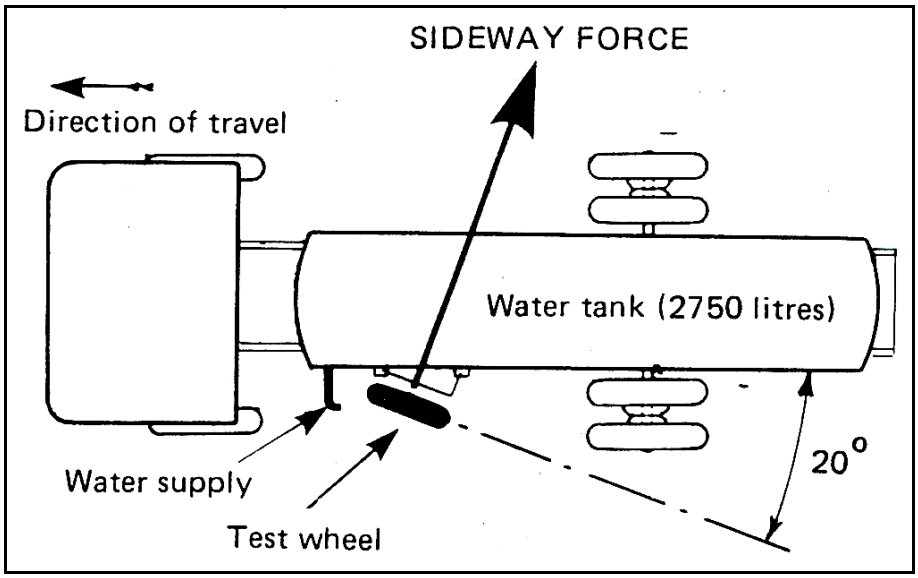

Figure 2: Sideways force concept.

SCRIM allows the testing of skid resistance at speeds from 20 – 85 kph, and measurements are corrected to 50 kph for reporting purposes. The survey season in the UK runs from May to September, and relies upon the accreditation of devices by Highways England, and ongoing quality assurance and re-validation. Reporting is normally summarised for each 10 m length of road, based on the average of readings taken every 100 mm. A process of seasonal adjustment is applied to the data, which attempts to account for both in year, and between year variations in skid resistance.

Skid resistance measured using the SCRIM is expressed as a SCRIM coefficient, which is typically in the range 0.20 to 0.65.

Road condition in England 2015 (Department for Transport 2015) reports that 25% of the principal (A class) and 5% of the trunk network in England required further investigations to check whether the skid resistance was acceptable. The difference is largely due to the different characteristics of the networks, resulting in different requirements for skid resistance.

Skid Policy in the United Kingdom

HD28/15 describes the standard applied for the strategic road network in England, and is also the basis of practice on trunk roads in Scotland. Wales and Northern Ireland. Most Local authorities apply a policy derived from HD28/15 with local variations to suit the characteristics of their network.

The objectives of HD28/15 are to:

- Maintain a consistent approach to the provision of skid resistance,
- Provide a level of skid resistance appropriate to the nature of the road environment at each location.

Figure 3 outlines the overall process as detailed in HD28/15. It involves setting Investigatory Levels (IL), effectively risk rating the network for the demand for skid resistance based on geometry and road layout, and then an investigatory protocol for those sites below the specified IL's. Outcomes from investigations can vary from 'do nothing' to inclusion in the forthcoming programme of works.

Most local authorities base their skid resistance strategy on HD28/15 with local variations. Typically, surveys are undertaken on the busier parts of the local network as defined by class or hierarchy. These variations are typically to Investigatory levels, prioritisation of sites and the investigatory process.

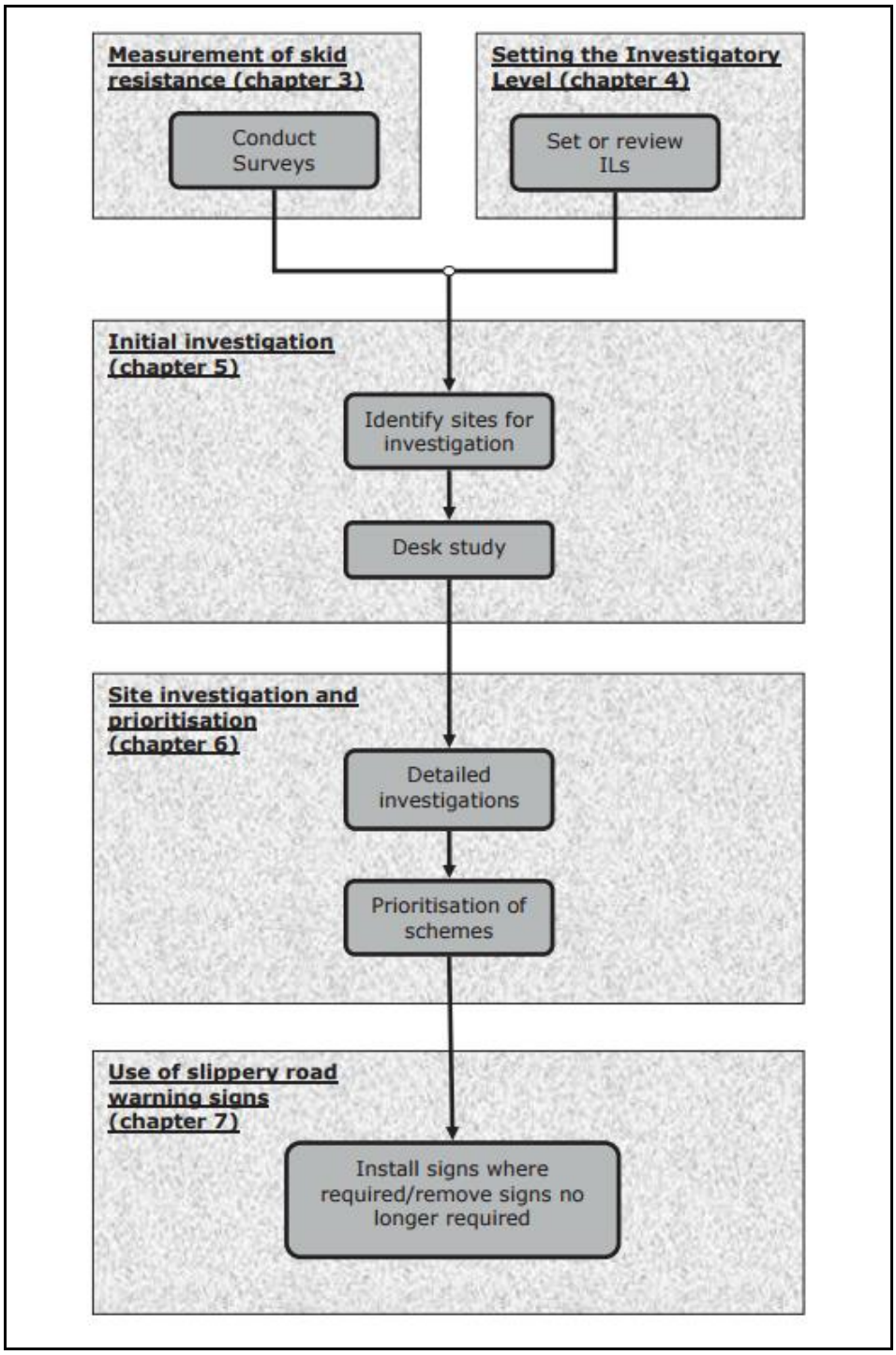

Figure 3: HD28/15 Skid Resistance- operation of standard

Skid Policy in the New Zealand

New Zealand has a skid policy that applies similar principles to that in the United Kingdom, with some modifications. The stated objective of the New Zealand Transport Agency (NZTA) skid resistance policy is to provide a cost effective surface that has appropriate skid resistance for road vehicles in wet and dry conditions.

A key difference between New Zealand and the UK is the extensive use of Chip seal (surface dressing) on the State Highway network. T10 (NZTA 2013) is the current standard. Similar to HD28/15 it defines an investigatory process, but also includes reference to macrotexture requirements, and introduces the concept of 'Intervention levels.' A key process in the management of skid resistance is the exception reporting, a 'fast track' investigation into sites that are below the SCRIM or texture Intervention Level. This approach enables a programme of retexturing to be accelerated to deal with these sites.

Following this initial response there an investigatory process for sites where a combination of crashes, low SCRIM and low Texture depth are prioritised. NZTA also apply an Aggregate Performance methodology to select aggregates for use in surface courses. The current NZTA maintenance contract includes performance criteria with financial penalties for failure to meet threshold/ intervention levels after a specified life.

NZTA research has indicated that an increase in skid resistance of 0.1 reduced crash rates by around 30% over the state highway network (Davies et al 2005), and that since the inception of T10 the rural state wet crash rate had reduced by 20% (Owen et al 2008). The estimated benefit/ cost ratio of the T10 policy has been calculated between 13 and 35 (Cook et al 2011).

Determining Investigatory Levels

Survey data is fitted to digital road networks using GPS fitting. A skid strategy relies upon accurately defined site categories with appropriate Investigatory levels (IL's). These IL's can either be adopted from national standards or adapted to local circumstances. IL's in HD28/15 range from 0.35 for Motorway non-event to 0.55 for the highest risk sites such as <100 m radius bends and approaches to pedestrian crossings.

Figure 4 shows the data for single carriageway bends from three English local authorities. This shows the crash rate v skid resistance for different bends radii. HD28/15 applies one category to bends < 500m radius; however local authority networks are more diverse. This indicates that there is a strong basis to introduce different IL's based on radius of curvature for local authority roads. The determination of the actual IL will depend on safety strategies, funding and the availability of aggregate.

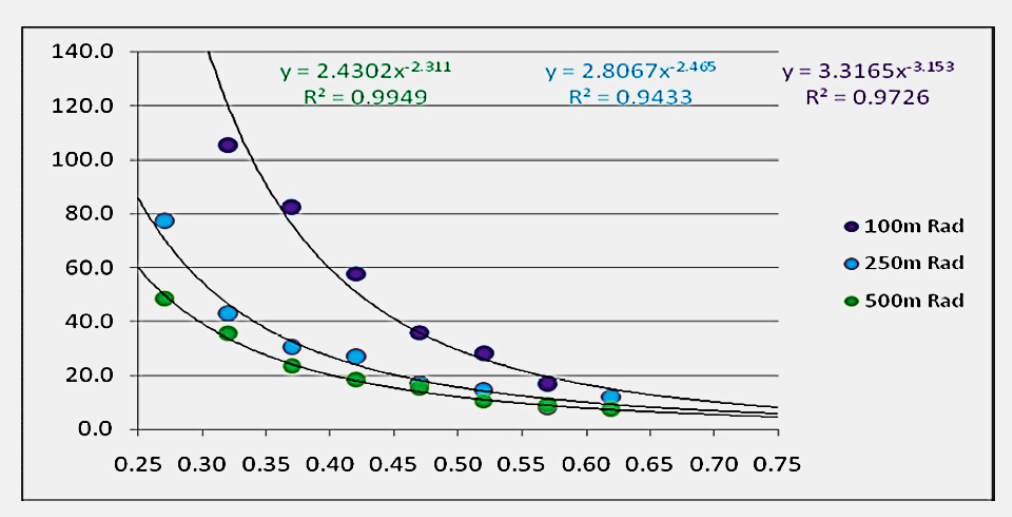

Figure 4: Crash rate v SCRIM for three English local authorities.

In New Zealand, a process of risk rating curves on the state highway network has been undertaken. Police records indicate that a higher number of crashes occur on curves defined as 'moderate' by the police. This risk rating considers a number of factors, including curvature, cross fall and approach speed. It also considers the concept of 'Out of Context' curves, where the approach speed is significantly higher than the curve speed. Figure 5 shows the range of IL's applicable to curves on the state highway network. The 'black' cells indicate default IL's and the hatched cells variations based on risk rating.

Site category	Skid site description	Investigatory level (IL), units ESC					
		0.35	0.40	0.45	0.50	0.55	0.60
2	a) Urban curves <250m radius						
	b) Rural curves <250m radius			L	M	H	
	c) Rural curves 250-400m radius		L	L	M	H	
	a) Down gradients >10%. b) On ramps with ramp metering.						

Figure 5: T10 Investigatory levels for curves.

The adopted Investigatory Levels directly affect the reporting of SCRIM performance at network level.

Achieving Investigatory Levels

The skid resistance of a road surface is generally accepted to be due to a combination of the micro texture of the coarse aggregate used in the surface course, the macrotexture of the road surface, as well as the material and construction characteristics. Figure 6 illustrates Macro and Micro texture.

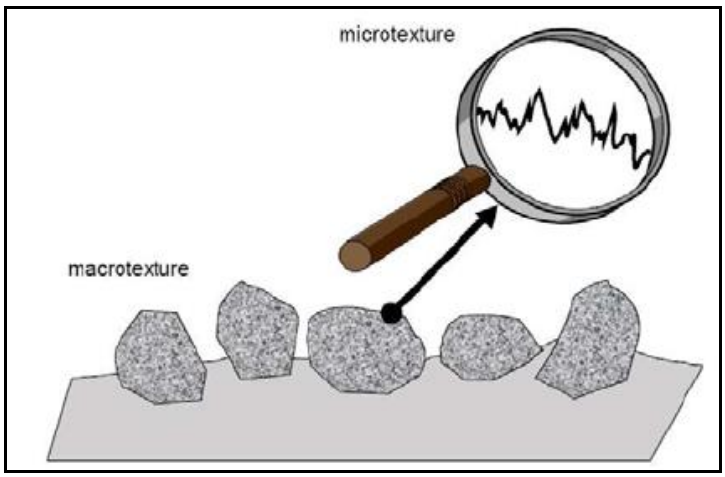

Figure 6: Macro and Micro texture

Micro-texture is defined by the mineralogy of the aggregate, and macro-texture by how the aggregate is orientated in the road surface. It is generally accepted that the absolute level of skid resistance available is determined by micro texture, and the macrotexture influence the level of skid resistance available at a particular location, especially at higher speeds.

The ability of an aggregate to provide skid resistance is described by the Polished Stone Value (PSV), which is derived by laboratory testing. It is accepted that the test has a number of limitations, and other methods such as the Werner Shulze may provide a better means of assessing in service performance.

PSV's used in road surfaces can vary from 50 – 70, with high friction surfacing using Calcined Bauxite providing higher values. In the UK HD36/06 (DMRB 2006) and IAN156/16R1 (DMRB 2016) recommends minimum PSV's for a combination of Investigatory level, site category and commercial traffic. This is used widely for the specification of new surfacing; however, there is often a compromise between the requirement for higher PSV aggregates at 'high risk' sites and a lower PSV for adjoining 'non-event' lengths.

The NZTA approach uses an Aggregate Performance Model, where based on a statistical analysis of over 100 primary sources of aggregate the SCRIM performance of the aggregate is forecast. Where there is insufficient data available the PSV is used applying the following formula

$$PSV = 100*SR + 0.00663*HCV + PSF$$

Where:

SR = investigatory level for the site
HCV = estimated heavy commercial vehicles per lane per day at the end of the surfacing life
PSF = polishing stress factor selected for the site
PSV = polished stone value

Aggregate Performance

The United Kingdom is fortunate to have a number of sources of high PSV (> 65) aggregates; however, stocks are limited. The application of the design guidance HD36/06 tends to lead to greater use of these higher PSV aggregates. It is therefore useful to gain a better understanding of how different aggregates perform in different situations. W.D.M. limited have carried out a number of studies into the performance of different aggregate sources by linking SCRIM data to construction records. Figure 7 shows the average of MSSC (seasonally corrected SCRIM coefficient) by road hierarchy and stated PSV for an English county. This shows an increase in MSSC as the PSV increases, and that there is evidence that the higher hierarchy roads typically have a lower MSSC for the same PSV. The averages are largely in the range 0.45 – 0.50, which aligns well with typical IL range. Figure 8 shows the distribution for the different PSV aggregates.

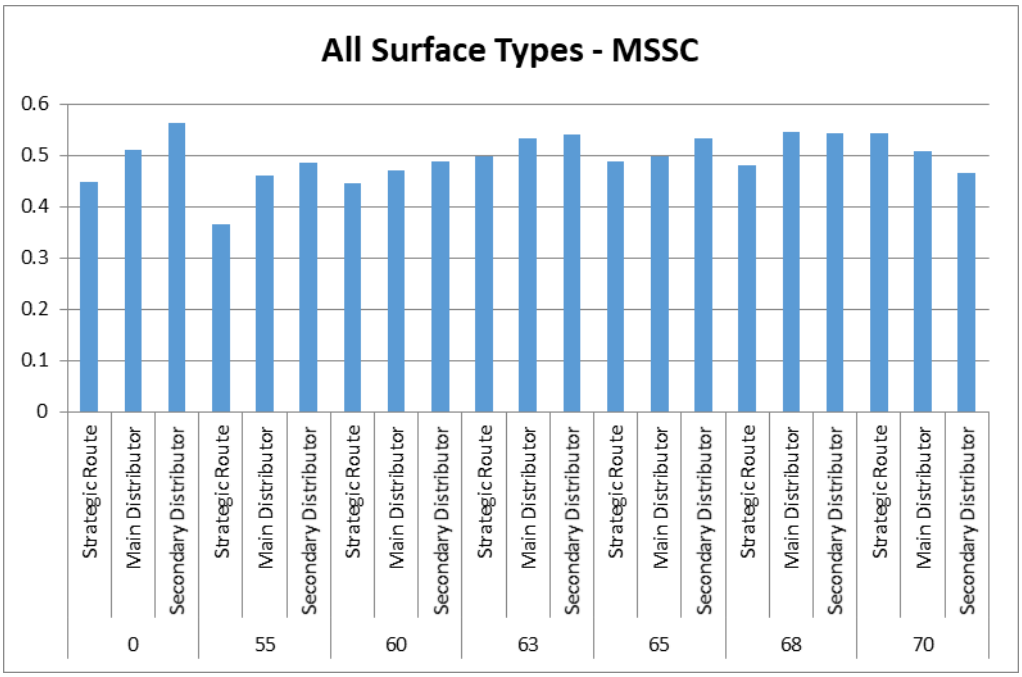

Figure 7: SCRIM v PSV and road hierarchy

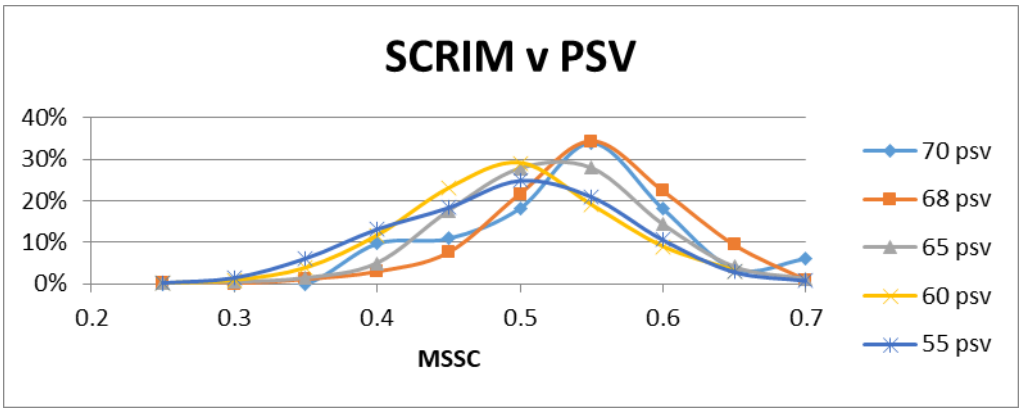

Figure 8: SCRIM v PSV distributions

Figure 8 indicates that all PSV's have a wide distribution and that the standard deviation is typically 0.05 or greater. This suggests that where the IL is 0.40 (non-event) most aggregates used in the county provide a high probability that the IL will be exceeded; however, as the IL increases a greater proportion of the roads surfaced with the aggregate type are likely to be below IL, indicating that there is an increased safety risk that the authority will need to manage. A more recent review of surfaces up to 5 years old in the same authority indicates that deficiency increases by IL, and that different treatment/ material types broadly have a similar deficiency profile.

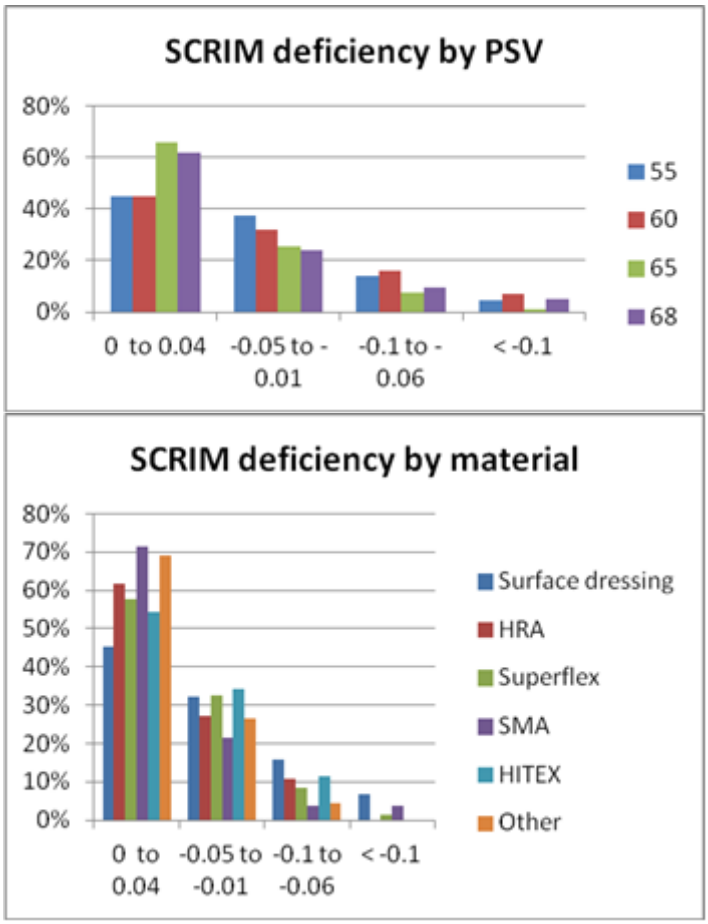

Figure 9: SCRIM performances on new surfaces

The data shown in figure 9 depends on the quality of the PSV specified at the time of design. Figure 10 illustrates how the performance of an aggregate varies when compared to that recommended in HD36/06. Figure 10 shows the relationship between the proportion of the network below IL, against the recommendation in HD36/06 and the actual aggregate used.

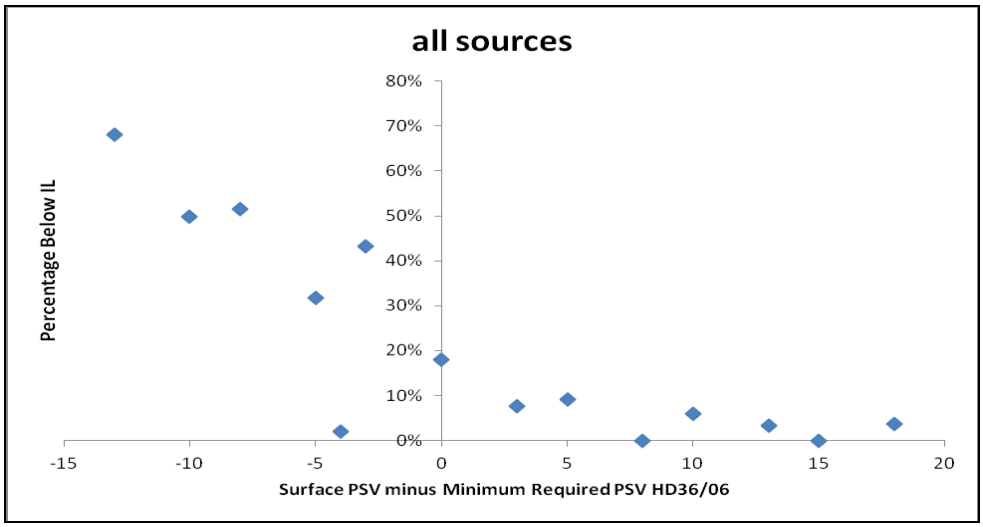

Figure 10: SCRIM against design PSV.

Figure 10 indicates that where the specified PSV is below IL there is a higher probability that the road will be below IL, and that even when the correct IL to HD36/06 has been specified there is still around a 20% probability that the MSSC will be below IL.

Providing skid resistance at high Investigatory level sites

The data presented indicate that authorities have a safety risk to manage where the SCRIM performance is below IL, and this risk is likely to be greater where the IL is at its highest. This is typically at two types of sites; rural bends and approaches to crossings. From accident studies undertaken by W.D.M. limited for a number of authorities these site categories typically have the highest crash rate and the consequences of crashes are likely to be more severe.

A study into the performance of High Friction Surfacing (HFS) in London (Stephenson and Hodgson 2014) considered the performance of HFS in London, and compared it with high PSV 'conventional' surfacing materials.

On the London Principal road network there are 4500 separate sites identified as 'approaches to crossings.' The accident rate at these crossings is higher than for any other site category, with the potential to realise the best rate of return from targeted investment to improve the skid resistance. HFS is routinely used at new crossings, but is not always maintained. Construction records were obtained for a number of sites and the SCRIM performance analysed. Figure 11 shows the performance of a number of different 68+ PSV aggregates compared to two different HFS surfaces, and table 1 shows the proportion of data from different sources above different IL's. This data indicates that some of the 68+ PSV aggregate asphaltic materials provide a high probability of achieving an IL of 0.50; however, a less than 50% probability of achieving 0.55. The HFS provides a higher skid resistance than the asphaltic materials and therefore a greater confidence in meeting the IL's. The adopted IL's for approaches to crossings in London are 0.55 as default, which can be reduced to 0.50 following a site audit.

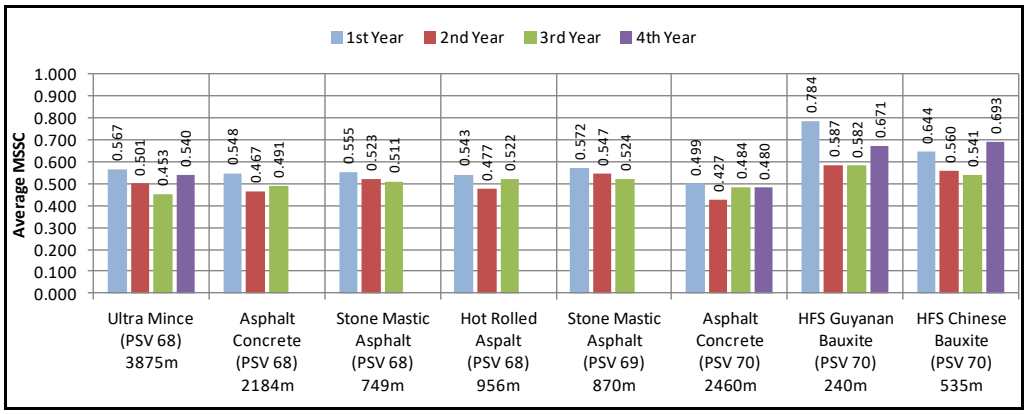

Figure 11: High PSV and HFS performance on approaches to crossings

Table 1: Level of Skid Resistance by Surface Type

Surface Type	**Length Above 0.50 (%)**	**Length Above 0.55 (%)**
ULM Ultra Mince (PSV 68)	86%	69%
Asphalt Concrete (PSV 68)	79%	46%
Stone Mastic Asphalt (PSV 68)	79%	48%
Hot Rolled Asphalt (PSV 68)	84%	52%
Stone Mastic Asphalt (PSV 69)	61%	50%
Asphalt Concrete (PSV 70)	52%	3%
HFS Guyanan Bauxite (PSV 70)	100%	100%
HFS Chinese Bauxite (PSV 70)	81%	72%

Conclusion

SCRIM has been used to measure Skidding Resistance since 1967, and is accepted as the primary measurement technique in the UK, New Zealand and many other countries. The application of a skid policy based on SCRIM has been demonstrated to offer a significant benefit cost ratio in terms of crash reduction when set against the cost of new surfaces.

Skid policies have been developed that take an investigatory approach, with targeted interventions on those sites where there is the greatest potential benefit. An integral part of any skid strategy is the selection and use of appropriate aggregates in new construction and maintenance. There are different approaches that can be adopted; but an understanding of how an aggregate performs over the predicted life of a surface is critical to an effective asset management strategy. The analysis of data for a number of highway authorities indicate that it is difficult to guarantee that all roads will perform above investigatory level; however, through the judicious selection of aggregate the safety risks of lower than desirable skidding resistance can be managed.

This may mean, for the highest risk sites a requirement for more extensive use of High Friction Surfacing.

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INVESTIGATION OF THE EFFECTS OF ADDITIVES ON MOISTURE SUSCEPTIBILITY OF ASPHALT MIXES CONTAINING SULFUR-POLYMER

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ABSTRACT

Large amounts of sulfur is produced in gas production refineries. From these, only small amounts are used in various applications and the rest remain unused. Sulfur deposits not only impose heavy economic costs, but these impart numerous adverse effects on the environment. The application of sulfur in asphalt mixes is rather simple. It requires minor changes to asphalt plants in order to produce a sulfur asphalt mix. However, according to previous researches, it was recognized that bituminous mixes containing sulfur are susceptible to moisture damage and show early stripping problems. Moreover, at mixing and laying stages of sulfur asphalt mixes, excessive H_2S emission could be harmful for the operators.

In this research a sulfur polymer additive, named Googas, was used to produce sulfur polymer mixes that had little H_2S emission and provided some plastic properties to mixes. In order to improve performance of asphalt mixes containing Googas, three additives, namely; Crumb Rubber, Zycotherm and Nano Clay were used in mixes and their properties were determined. All were added at mixing stage of aggregates, bitumen and sulfur polymer. With the purpose of evaluating performance of the above mixes, Indirect Tensile Strength and Marshall Tests were carried out. From these, Tensile Strength Ratio (TSR) and Marshall Residual Stability (MRSR) parameters were determined. The results indicated that the addition of Crumb Rubber, Zycotherm and Nano Clay to sulfur polymer mixes resulted in improved moisture resistance of mixes. The best improvement was achieved in mixes containing 40% binder content of sulfur polymer and 15% Crumb Rubber. This resulted in average 23% increase in moisture resistance of mixes.

***Keywords*:** Crumb rubber; Moisture susceptibility; Nano clay; Sulfur polymer; Zycotherm.

1. INTRODUCTION

A significant amount of sulfur remains deposited near gas production refineries. This not only imposes heavy economic costs, it imparts also numerous environmental problems. Due to very low price of sulfur, its application in asphalt mixes is quite feasible. It requires simple changes in asphalt plants in order to produce sulfur asphalt mixes (Javadi, 2005 and Weidong, 2006). However, ordinary sulfur asphalt mixes produce harmful and unpleasant gases. This is why some producers have modified sulfur using some polymers (Timm et al, 2010). A product of these families, called Sulfur-Polymer (SPAM) or Googas, is produced in Iran and has been used in several projects. The weak point of mixes containing this product is their moisture susceptibility (Timm et al, 2010).

Since early 1970s researchers have concluded that moisture has sever damaging effects on asphalt pavements (Little and Epps, 2001 and Liu et al, 2011). This phenomenon resulted in damages in asphalt mixes in West and South East of USA and millions of dollars were spent to repair the damages (Little and Epps, 2001). Moisture susceptibility can be defined as loss of resistance and reduced durability of asphalt mixes, caused by adverse effects of moisture. Moisture susceptibility is the tendency of asphalt mixes to strip which will then lead to develop several other distresses, including cracking, rutting, raveling, and potholes (Sungun et al, 2014 and Ojum et al, 2017). In a study, it was determined that the damaging effects of moisture resulted in reduced asphalt modulus (up to 25%); increased rutting (up to 60%); and increased fatigue cracking (Mehrara and Khoadii, 2013). In this research, it was revealed that among asphalt mixes containing various additives, the ordinary sulfur asphalt mix was recognized to be the most moisture susceptible mix that requires further treatments before being laid on pavements.

In this research, with the aim of reducing moisture susceptibility of sulfur polymer asphalt mixes (SPAM), three additives, namely crumb rubber, Zycotherm and nano clay were used in SPAM mixes at various amounts. The effects of Zycotherm on moisture susceptibility of asphalt mixes was investigated by Ranka (2012). The results of this latter study showed significant effects of Zycotherm in improving moisture resistance of asphalt mixes. In fact, replacing 0.1 and 0.15% of the mix binder content with Zycotherm resulted in 14 and 7% increase in indirect tensile strength ratio of mixes, respectively.

The application of Crumb Rubber in asphalt mixes has been studied by different researchers. In a research that was performed on mixes containing Crumb Rubber, it was shown that these mixes have better performance against fatigue cracking (Hainian et al, 2013). The effects of Crumb Rubber on rutting and moisture susceptibility of asphalt mixes was also investigated (Sungun et al, 2014). In this research, Crumb Rubber was used at 8, 10 and 12 percent of the binder content of the

mix. The results showed significant improvement in rutting and moisture resistance of asphalt mixes.

In an experimental research, the effects of Nano Clay on moisture susceptibility of mixes was evaluated (Hossain et al, 2014). In this research, Nano Clay was used at 4 and 6 percent of the binder of the mix. Results showed increased tensile strength ratio and reduced moisture susceptibility of asphalt mixes. Similar results were achieved in another research, showing that Nano Clay resulted in improved moisture susceptibility of asphalt mixes (Shu et al, 2011). The effects of Nano Clay on fatigue and rutting performance of asphalt mixes was investigated in Iran too. Results showed significant improvements in various performance parameters of the mix (Ghaffarpour, 2011).

2. MATERIALS CHARACTERISTICS

In this research, in order to investigate the effects of the above three additives on sulfur polymer asphalt mixes; one source of aggregate was selected. This was from a quarry in Yazd province in central part of Iran. A 60/70 penetration grade bitumen, produced in Refinery of Isfahan, was also used in the whole research. All sulfur polymer mixes were continuously graded aggregates. Crumb Rubber modified binder was produced applying wet processing and using a high shear mixer. Major physical testing results of the aggregates and the bitumen binder are reported in Tables 1 and 2 respectively.

Table 1. Major physical testing results of the aggregates

Coarse Aggregates				
Test	**Standard Method**	**Iran Specification limits**		**Result**
		Minimum	Maximum	
Los Angeles Abrasion (%)	ASTM-C131	-	25	20
Sodium Sulfate Soundness (%)	ASTM -C88	-	8	8
Fracture Faces (%)	ASTM -D5821	60	-	100
Flaky and Elongated Particles (%)	ASTM -D4791	-	15	9
Fine Aggregates				
Plasticity Index (%)	ASTM -D4318	NP	NP	NP
Sodium Sulfate Soundness (%)	ASTM -C88	-	12	0.7
Sand Equivalent (%)	ASTM -D2419	50	-	70

Table 2. Testing results of the 60/70 pen bitumen used in mixes

Testing	Standard Method	Specifications limits		Result
		Minimum	Maximum	
Penetration (0.10 mm)	ASTM-D5	60	70	64
Softening Point (°C)	ASTM -D36	49	56	50.5
Ductility at 25°C (cm)	ASTM- D113	100	-	+100
Specific Weight (g/cm^3)	ASTM- D3289	1.013	1.017	1.018
Kinematic Viscosity at 135°C	ASTM- D2170	200	1000	326
Thin Film Oven Test (163°C for 5 h)	ASTM- D1754			
Change in Mass (%)		-	0.8	0.03
Penetration after TFOT (0.10 mm)		31	-	44
Ductility after TFOT (cm)		50	-	+100

3. TESTING RESULTS

Several testing methods can be performed on asphalt mixes in order to determine their moisture susceptibility. In this research, Marshall samples were prepared and advanced Marshall parameters and indirect tensile properties of the samples were determined. The results are reported in the following sections.

3.1. Marshall Residual Stability Ratio

Two different series of Marshall Samples were prepared. The first series were untreated dry samples. The second series consisted of samples that were saturated and were then subjected to one freeze-thaw cycle. In this latter treatment, the compacted samples were freeze treated at -18°C for two hours, as some researchers (Sengul et al., 2012) suggested that. These were then kept at room temperature for further two hours before being immersed in water for 24 hours at 60°C. Then were cooled to room temperature for two hours. Finally, the samples were placed in water at 60°C for 30 minutes before being subjected to Marshall testing at 60°C. From this, stability and flow values of both untreated and treated (i.e. samples subjected to the above mentioned conditions) were determined. The results are reported in Table 3. The results are the average of six samples for each series. From these results, a parameter named Marshall Residual Stability Ratio (MRSR) was determined. This is defined as the ratio of stability of treated samples to that of untreated ones. Fig. 1 reports MRSR results of samples containing various additives at different amounts. With reference to Table 3, it can be observed that the mix in that its binder content was consisted of 15% crumb rubber and 50% sulfur polymer resulted in greater Marshall Stability values (up to 23% increase). While the mix in that its binder content was consisted of

15% crumb rubber and 40% sulfur polymer resulted in 14% increase in Marshall stability.

Table 3. Marshall Stability, flow and MRSR values of the various samples

Mix samples with various additive compositions	Before Conditioning (Dry)		After Conditioning (moisture and freeze-thaw)		
	Stability (Kg)	Flow (mm)	Stability (Kg)	Flow (mm)	MRSR (%)
30% Sulfur-Polymer	1330	1.96	1077	1.56	81
40% Sulfur-Polymer	1440	1.86	1193	1.50	83
50% Sulfur-Polymer	1497	1.80	1203	1.36	81
30% Sulfur-Polymer – 1.5% Nano Clay	1460	2.67	1371	2.46	94
40% Sulfur-Polymer – 1.5% Nano Clay	1591	2.70	1484	2.30	93
50% Sulfur-Polymer – 1.5% Nano Clay	1613	2.76	1520	2.40	94
30% Sulfur-Polymer – 15% Crumb Rubber	1443	2.53	1337	2.10	92
40% Sulfur-Polymer – 15% Crumb Rubber	1561	2.36	1479	1.60	95
50% Sulfur-Polymer – 15% Crumb Rubber	1598	2.35	1478	1.67	93
30% Sulfur-Polymer – 0.15% Zycotherm	1346	3.15	1260	2.80	93
40% Sulfur-Polymer – 0.15% Zycotherm	1457	3.10	1373	2.60	94
50% Sulfur-Polymer – 0.15% Zycotherm	1515	3.00	1417	2.50	93

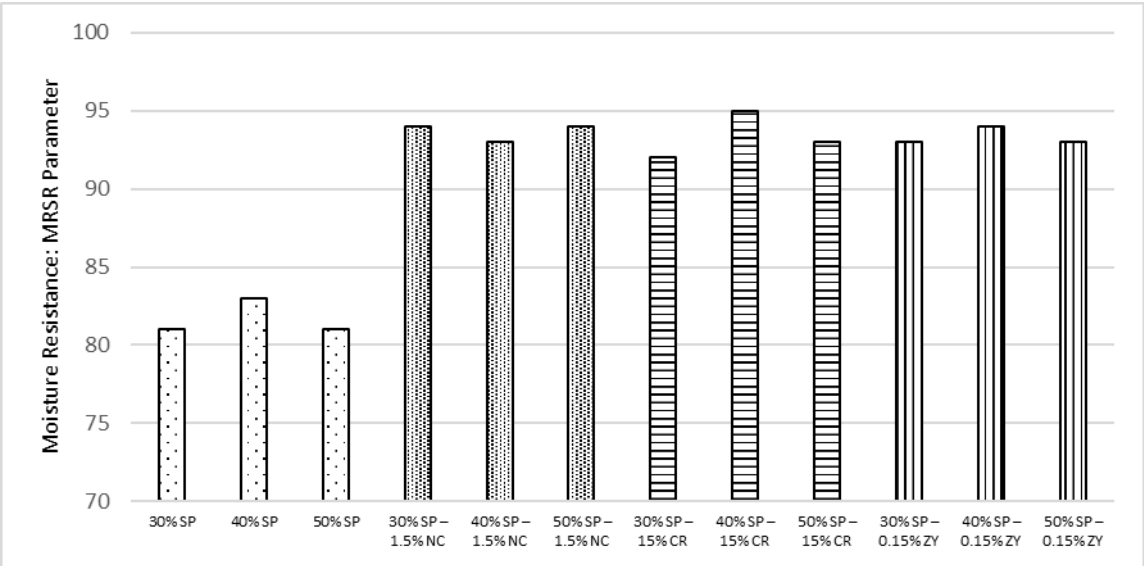

Fig. 1. Marshall Residual Stability Ratio (MRSR) of mixes containing various additives

3.1.1. Moisture Resistance

Samples consisting of the combination of various additives were compacted applying Marshall Hammer. Additives in various samples were consisted of the following combinations:

30% Sulfur-Polymer
40% Sulfur-Polymer
50% Sulfur-Polymer
30% Sulfur-Polymer – 1.5% Nano Clay
40% Sulfur-Polymer – 1.5% Nano Clay
50% Sulfur-Polymer – 1.5% Nano Clay
30% Sulfur-Polymer – 15% Crumb Rubber
40% Sulfur-Polymer – 15% Crumb Rubber
50% Sulfur-Polymer – 15% Crumb Rubber
30% Sulfur-Polymer – 0.15% Zycotherm
40% Sulfur-Polymer – 0.15% Zycotherm
50% Sulfur-Polymer – 0.15% Zycother

With reference to Fig. 1, it can be seen that with adding nano clay to sulfur polymer asphalt mixes (SPAM), MRSR values of mixes were increased greatly. This shows the positive role of Nano Clay in increasing moisture resistance of SPAM mixes.

When crumb rubber (CR) was added to both control samples and those containing SP additive, MRSR values of the samples were increased significantly. For instance, MRSR values of three asphalt mix compositions that contained 30% SP-15% CR, 40% SP-15% CR and 50% SP-15% CR were increased by 13, 17 and 15% respectively.

Similar results were obtained from samples containing Zycotherm additive. The extent of increase for mix samples that contained 3% SP-0.15% Zycotherm, 40% SP-0.15% Zycotherm and 50% SP-0.15% Zycotherm were 16, 15 and 15%.

3.2. Tensile Strength Ratio

With reference to Marshall testing and after achieving the optimal bitumen content of 4.3%, samples containing various additive combinations were prepared for evaluation of their moisture susceptibility. Indirect Tensile Strength (ITS) testing was performed and Tensile Strength Ratio (TSR) of the samples were determined. ITS testing was recognized by other researchers to be a suitable testing method for evaluating the effectiveness of polymer additives (e.g. Shuklamanoj et al, 2014). The summary results

of these parameters are reported in Table 4. Fig. 2 shows comparison of TSR results of SP asphalt samples containing various combinations of additives.

Table 6. ITS testing results of sulfur polymer mixes containing various additives

Additive combinations in mixes	**Indirect Tensile Strength, dry condition (kPa)**	**Moisture susceptibility**	
		Indirect Tensile Strength, saturated condition(kPa)	**TSR Ratio (%)**
30% Sulfur-Polymer (30% SP)	728	532	73
40% Sulfur-Polymer (40% SP)	692	492	71
50% Sulfur-Polymer (50% SP)	558	373	68
30% Sulfur-Polymer – 1.5% Nano Clay (30% SP-1.5% NC)	948	777	82
40% Sulfur-Polymer – 1.5% Nano Clay (40% SP-1.5% NC)	898	727	81
50% Sulfur-Polymer – 1.5% Nano Clay (50% SP-1.5% NC)	890	703	79
30% Sulfur-Polymer – 15% Crumb Rubber (30% SP-1.5% CR)	943	886	94
40% Sulfur-Polymer – 15% Crumb Rubber (40% SP-1.5% CR)	862	784	91
50% Sulfur-Polymer – 15% Crumb Rubber (50% SP-1.5% CR)	798	702	88
30% Sulfur-Polymer – 0.15% Zycotherm (30% SP-1.5% ZY)	780	647	83
40% Sulfur-Polymer – 0.15% Zycotherm (40% SP-1.5% ZY)	820	656	80
50% Sulfur-Polymer – 0.15% Zycotherm (50% SP-1.5% ZY)	850	662	78

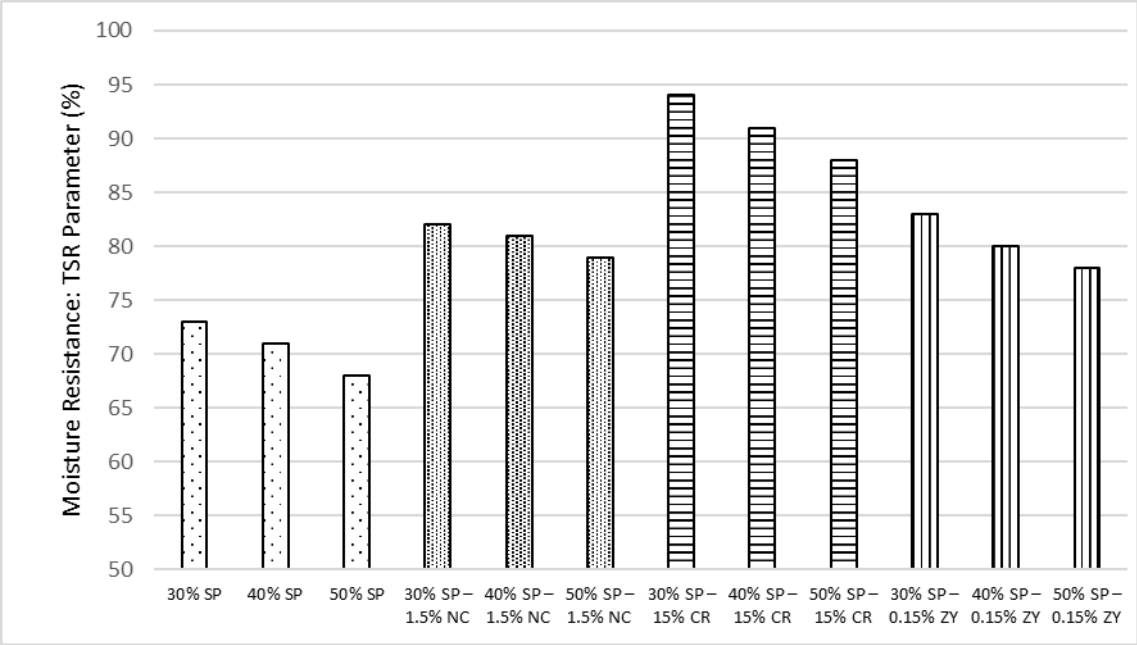

Fig. 2 Comparison of TSR results of sulfur polymer mixes containing various additives

3.2.1. Moisture Damage Resistance

The addition of Nano Clay to SP asphalt mixes resulted in increased indirect strength resistance of both dry and saturated samples. TSR parameter of asphalt mixes containing 30% Sulfur-Polymer-1.5% Nano Clay, 40% Sulfur-Polymer-1.5% Nano Clay and 50% Sulfur-Polymer-1.5% Nano Clay showed TSR increases of 16, 14 and 11%, respectively.

It was also observed that with adding crumb rubber to asphalt mixes containing SP, the indirect tensile strength resistance of both dry and saturated samples were increased significantly (compared with the ordinary SP mixes). TSR parameter of asphalt mixes containing 30% Sulfur-Polymer-15% Crumb Rubber; 40% Sulfur-Polymer-15% Crumb Rubber; and 50% Sulfur-Polymer-15% Crumb Rubber resulted in TSR increased values of 32, 28 and 24% respectively.

Similar results were achieved with adding Zycotherm to SP asphalt mixes, resulting in increased TSR values. The extent of TSR increase in mixes containing 30% Sulfur-Polymer-0.15% Zycotherm; 40% Sulfur-Polymer-0.15% Zycotherm; and 50% Sulfur-Polymer -0.15% Zycotherm were 17, 13 and 10%, respectively.

4. CONCLUSIONS

From the experimental works conducted on asphalt mixes containing Sulfur-Polymer and with using three different additives, the following conclusions can be drawn:

1- Although SP containing bituminous mixes have good mechanical properties, these lack moisture resistance, making them prone to stripping.
2- In order to overcome the stripping susceptibility of SPAM bituminous mixes, three additives, namely Nano Clay, Zycotherm and Crumb Rubber were tested. These, when applied at certain conditions, were considered to be effective in reducing moisture susceptibility of SPAM mixes.
3- All the above additives resulted to increase moisture resistance of SPAM mixes. However, the extent of increase varied according to the additive type and amount used in mixes.
4- Tensile Strength Ratio, determined in Indirect Tensile Test and Marshall Residual Stability Ratio, determined in Marshall Test were considered effective parameters in quantifying moisture susceptibility of SP mixes.
5- ITS testing results indicated that mixes in those the bitumen binder content consisted of 40% sulfur polymer and 15% crumb rubber resulted in 23% increase in TSR value. Marshall Retained Stability Results (MRSR) of mixes containing 40% sulfur polymer and 15% crumb rubber showed 17% increase in MRSR value.

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