Permanent Deformation Behavior of Crushed Rock Base (CRB) as Highway Base Course Material

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ABSTRACT: This paper aims to report the permanent deformation behavior of the Crushed Rock base (CRB) layer under traffic loads using repeated load triaxial (RLT) tests simulating various conditions of basecourse, in order to determine the limit use and dynamic response of CRB. The main deterioration of flexible road surface is rutting because of insufficient material strength. As is well known, current pavement design does not consider permanent deformation of basecourse material, it only requires the index properties and bearing capacity of it. Although an adequate bearing capacity material was placed in the pavement structure, several roads were still damaged. For pavement design criteria, it is not enough to consider only the permanent deformation at the top of the subgrade layer but also the plastic strains taking place in base, subbase and even the asphalt layer which contribute to the overall rutting appearing in the thin asphalt surface. Currently, the plastic deformation characteristics of CRB as a base material is not yet clearly understood. It results from an accumulation of the plastic strain of each cyclic loading cycle up to failure. In this study, the shakedown concept was therefore utilized to describe the permanent deformation characteristic of CRB. The plastic shakedown limit is used to predict the failure criteria at a certain level of the accumulated plastic deformation in the unbound granular material (UGMs) layer. However, the permanent deformation characteristic of UGMs is also dependent on the applied stress levels. The outcome in this study is that the limits of a working stress level and the plastic shakedown limit of CRB were defined.

KEYWORDS: Crushed Rock Base, permanent deformation, unbound granular material (UGM), repeated load triaxial (RLT).

1 INTRODUCTION

An UGM layer with thin bituminous surfacing is extensively used in road networks and a normal pavement structure consists of in-situ sand as subgrade, crushed lime stone and crushed rock as subbase and base layer respectively as shown in Figure 1. Normally, CRB is used as a base course material that can be determined as UGM. The important function of the UGM layer in pavements is to distribute and reduce the amount of vertical stresses and strains because of vehicle wheel loads into sub layers with no unacceptable strain. Therefore, knowledge of all the characteristics of materials relevant to pavement mechanistic design is important to obtain the efficiency of such materials. Nevertheless, the basis of pavement design is still based on a traditional design procedure which is not reliable enough to explain the relationship between design parameter inputs and pavement performances. Roads need to be investigated to improve pavement analysis and designed more precisely than in the past with respect of UGMs plastic strain during its the service life. As a result, the most economical layer thickness and an appropriate material type for the pavement will be determined.

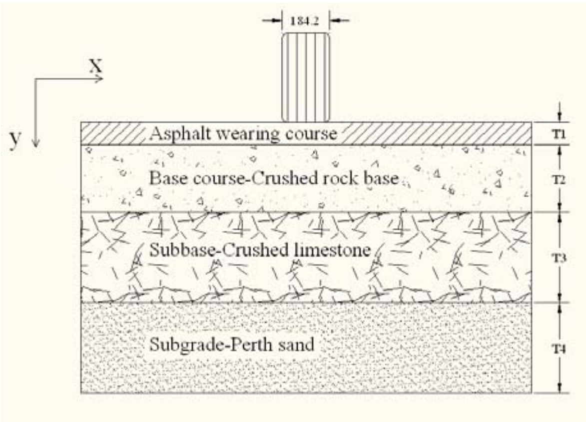

Figure 1: Pavement structure diagram.

This paper focuses on applying the permanent deformation for CRB as a base course material and developing specific models for pavement analysis. The empirical design method is now unacceptable because the test protocols require the design parameter inputs from monotonic loading tests rather than cyclic loading tests which are more representative of real traffic loading conditions. A mechanistic design attempts to explain pavement phenomena under real conditions such as stress level types, material properties and environments of the pavement structure based on design criteria from sophisticated tests which can simulate real conditions into the test protocol (Collins, Wang et al. 1993). The main success of this analytical method is the experimental measurement and appropriate characterization of the mechanical responses from the RLT test, the basic protocol of this study.

2 BACKGROUND

UGMs are very significant pavement construction materials that are used as bases, subbases and subgrades. Understanding the characteristics and behavior of UGM and the response of the materials when subjected to applied loads is therefore necessary for practical work. Basically, UGMs are rather complicated materials to deal with particularly natural soils and gravels which show varied behavior as a consequence of physical history that influences the mineralogical composition, particle form and size distribution. Moreover the actual degree of compaction and moisture content are of enormous importance. Normally, UGM consists of gravels or crushed rock aggregates which have a particular grading that formulates mechanical strength, practicability and are able to be compacted. Their performance is largely governed by their shear strength, stiffness and resistance to material breakdown under construction and traffic loading. The most common modes of deterioration of UGM base layers are rutting due to insufficient resistance to permanent deformation through shear and densification, and collapse during particle breakdown.

The nature of empirical pavement design procedure is based on experience and the results of simple tests such as the California Bearing Ratio (CBR), particle size distribution (PSD), moisture sensitivity, Los Angeles (LA) abrasion, shear strength and deflection. Such testing results are all static parameters and non-mechanical index parameters rather than any consideration of realistic material performance and displacement distribution during cyclic loading, stresses and strain distribution in multilayered pavement design. Consequently, the use of empirical approaches becomes sub-standard. Traditional procedure has been criticized by Wolff who argued that it is too simplistic and does not take into account the non-linear behavior of UGMs (Wolff and Visser 1994). Hence, the pavement analysis and design inevitably involve additional base and subbase permanent deformation. For those reasons, there are some doubts in responses relative to their performance.

The performance of a base course material depends upon its stiffness and deformation resulting from a traffic load. A large deformation causes rutting on the bituminous surface. Basically, conventional pavement construction is designed to provide an adequate thickness to cover the sub layer in such a way that no shear failures and unacceptable permanent deformation take place in each layer. For pavement design purposes, the stress level which is related with a reversible strain response must be determined and consequently not exceeded, once unacceptable permanent strains are prevented. This has improved the possibility of a critical boundary stress between stable and unstable conditions in a pavement.

The shakedown concept has been used to explain the behavior of conventional engineering structures under repeated cyclic loading. Basically, it was originally developed to analyze the behavior of pressure vessels subjected to cyclic thermal loading. Subsequently, it was improved to analyze the behavior of metal surfaces under repeated rolling or sliding loads. The possible employment of the shakedown concept in pavement design was first introduced by Sharp and Booker (1984) and Sharp (1983) who explained its application was based on the tested results of the AASHTO road tests (AASHTO 1986) where in some cases, deterioration was reported due to stiffening or post-compaction after a number of load cycles (Kent 1962). Moreover, studies have been produced to define upper-bound (Collins and Boulbibane 1998) and lower-bound (Yu and Hossain 1998) for the shakedown limit of UGMs in simple pavement structures. At low stress levels, the mechanism of permanent strain has an initial post compaction or re-arrange phase, while the permanent strain rate is relatively high but is reduced with increasing numbers of load cycles. A stable state may be maintained for a period of time unless the states change. Maree reported the behavior of gravel and crushed stone and that under constant confining stress, the specimens stabilized under a certain threshold of repeated deviator stress and developed a design procedure, based on a failure model (Maree, Freeme et al., 1982). Numerous investigations have been conducted regarding the behavior of UGMs used in flexible pavements. Lekarp summarized the main findings regarding the effects of different material parameters on the permanent strain response of UGMs and the maximum applied stress in UGM layers is within the maximum repeated deviator stress limit (Lekarp, Isacsson et al., 2000). The original shakedown concept maintains that there are three ranges of permanent strain response under repeated loading.

- Plastic shakedown range (Range A). The low loading levels apply and the material response indicates plastic in a few initial cycles, although the ultimate response is elastic after post-compaction. The strain is completely reversible and does not lead any permanent strains when it reaches a state of stability.
- Plastic creep range (Range B). The applied loading level is low enough to avoid a quick collapse. The material achieves a long-term stable state response with any accumulation of plastic strain (Post-compaction). However the material will show failure with a large number of load cycles after a stable state.
- Incremental collapse range (Range C). The repeated loading is relatively large so that plastic strain accumulates rapidly with failure occurring in a small number of load cycles after stiffening.

A pavement is likely to show progressive accumulation of permanent strains (rutting) under repeated traffic loading if the magnitude of the applied loads exceeds the limiting value (Range C). If the applied traffic loads are lower than this limit, after any post-compaction stabilization, the permanent strains will level off and the pavement will achieve a stable state of "shakedown" (Ranges A and B) presenting only reversible strain under additional traffic loading (Sharp 1985). This implies an adaptation by the pavement subjected to the working load. This could be due to a change in material response (compaction degree), due to a change in stress state or due to a combination of both effects. With this understanding of material behavior, the shakedown concept typically then determines the load carrying capacity of the structure if it is not to reach excessive permanent strain. For performance prediction, it is of great importance to know whether a given pavement will experience progressive accumulation of permanent strain leading to state of incremental collapse or if the increase in permanent strain will cease, resulting in a stable response (shakedown state).

2.1 Permanent strain under a number of load cycles

In considering the long-term behavior model of pavements, it is essential to take into account the accumulation of permanent strain with the number of load cycles and stress levels that play and important role. Hence the main research purpose focusing on long-term behavior should be to establish a constitutive model which predicts the amount of permanent strain at any number of cycles at a given stress ratio. In the past, the permanent strain of UGMs for pavement applications has been modeled in several ways. Some of these are logarithmic with respect to the number of loading cycles (Barksdale 1972; Sweere 1990) whilst others are hyperbolic, tending towards an asymptotic value of deformation with increasing numbers of load cycles (Wolff and Visser 1994; Paute, Hornych et al., 1996). The first type is that due to this approach, permanent axial strain is supposed to accumulate in linear relation to the logarithms (Barksdale 1972) as follows Equation (1):

$$\varepsilon^{p} = a + b\log(\mathrm{N}) \tag{1}$$

where ε^p is permanent strain; a and b are regression constants; and N is the number of loading cycles. The long-term strain behavior was also investigated by Sweere in a series of RLT tests and suggested that for a large number of load cycles, the following approach should be employed:

$$\varepsilon^{p} = A \cdot N^{B} \tag{2}$$

where:

ε^p	[10^{-3}] permanent strain
A, B	[-] regression parameters
N	[-] number of load cycles.

3 LABORATORY PROGRAM AND TESTING

3.1 Crushed rock base (CRB)

CRB is composed of rock fragments produced by the crushing and screening of igneous, metamorphic or sedimentary source rocks. The samples used in this study were taken from a local stockpile and kept in sealed containers. They were prepared at 100% maximum dry density (MDD) of 2.27 ton/m^3 and an optimum moisture content (OMC) of 5.5%. Material properties achieve base course specifications (Main Roads Western Australia 2003).

3.2 Specimen preparation

Sample preparations were carried out using a standard cylinder mould 100 mm in diameter and 200 mm in height by the modified compaction method (Main Roads Western Australia 2007). Compaction was accomplished on 8 layers with 25 blows of a 4.9 kg rammer at a 450 mm drop height on each layer. Fully bonding conduction between the layers of each layer had to be scarified to a depth of 6 mm before for the next layer was compacted. After compaction, the basic properties of each specimen were determined after which it was carefully carried to the base platen set of the chamber triaxial cell. A crosshead and stone disc were placed on the specimen and it was wrapped in two platens by a rubber membrane and finally sealed with o-rings at both ends.

3.3 Repeated cyclic load triaxial (RLT) tests

The tests were carried out with a cyclic triaxial apparatus consisting of main set containing the load actuator and a removable chamber cell. The specimens were placed in the triaxial cell between the base platen and crosshead of the testing machine as Figure 2 shows. Controllers were used to manage the chamber, as well as the air pressure. The analogical signals detected by the transducers and load cell are received by a module where they are transformed to digital signals. A computer converts modules of the digital signals sent from the system. The system is located in the main set and facilitates the transmission of the orders to the actuator controller. User and the triaxial apparatus communication are controlled by a computer which uses convenient and precise software. This makes it possible to select the type of test to be performed as well as all the parameters, stress levels, data to be stored. The load cell, the confining pressure and the externally linear variable differential transducer (LVDT) on the top of the triaxial cell, used to measure deformations over the entire length of the specimens were measured by the control and data acquisition system (CDAS) which provided the control signals, signal conditioning, data acquisition. The CDAS was networked with a computer which provided the interfacing with the testing software and stored the raw test data. This enabled the resultant stress and strain in the sample to be determined.

This apparatus, however, is limited to laboratory samples with a maximum diameter of 100 mm and a height of 200 mm based on the standard method of Austroads APRG 00/33-2000 (Voung and Brimble 2000). Moreover, the apparatus allows the laboratory sample to be subject to cyclic axial deviator stresses but it is not feasible to vary the confining radial stresses at the same time. Confining pressure was generated air to simulate the lateral pressure acting on the surrounding materials as occurs in a pavement layer. The pressure was applied and stresses were found at different points in the granular material. The results were expressed in terms of deviator stress q= σ_1 -σ_3, mean normal stress p= (σ_1 -2σ_3)/3 and the confining pressure was simulated from the thickness of the pavement base course layer that is in common use in Western Australia. For this reason, it was decided to subject the laboratory samples to 11 different stress levels with a particular confining pressure of 40 kpa. After the confining pressure had been applied, additional dynamic vertical stress was applied. Triaxial tests were carried out with axial stress pulses reaching stress ratios of σ_1/σ_3 = 5-15. The dynamic axial stress came from a high pressure air actuator capable of accurately applying a stress pulse following the stress level. In this test, there was haversine waveform frequency of 1 Hz over a period of 1.0 sec and a load pulse of 0.1 sec duration, as illustrated in Figure 3

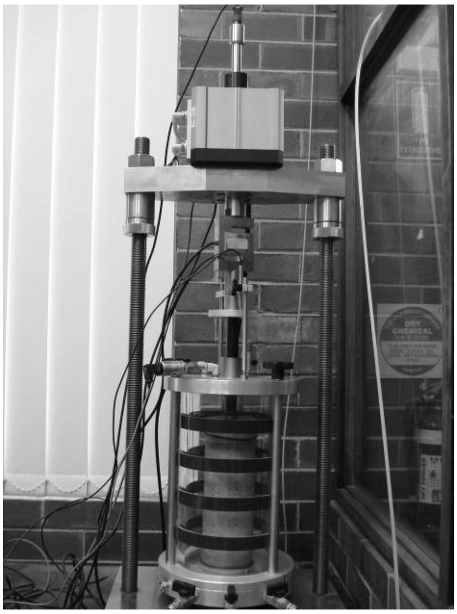

Figure 2: The repeated loads triaxial apparatus.

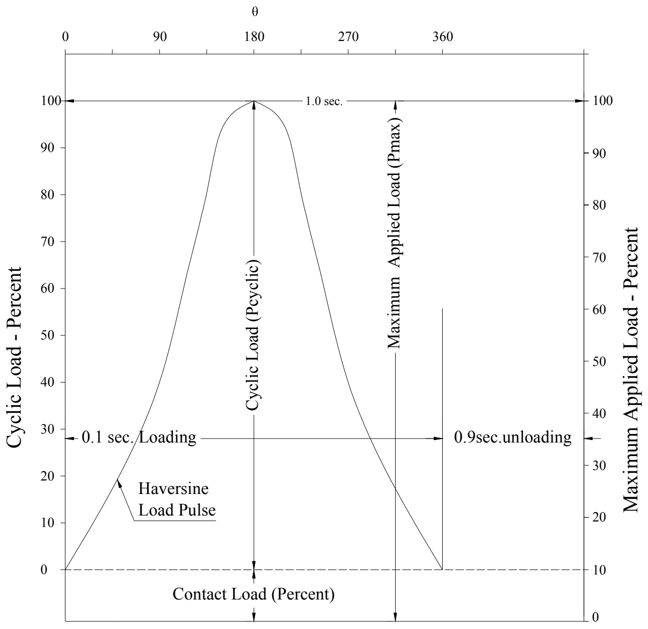

Figure 3: The vertical loading waveform.

4. RESULTS AND CONCLUSIONS

The permanent deformation accumulations were observed as shown in Figure 4 and Figure 5. As the test results show, CRB responses always produce permanent deformation during cyclic loading, hence it can describe no purely elastic behavior under repeated cyclic loads in course base materials and the multi-layer linear elastic theory is not enough to analyze the UGM layer. Permanent deformation behavior is described on the basis of internal friction between grains, particle shape, compaction, consolidation, distortion, etc and the test results can be separated into three ranges (A, B and C) based on the shakedown concept.

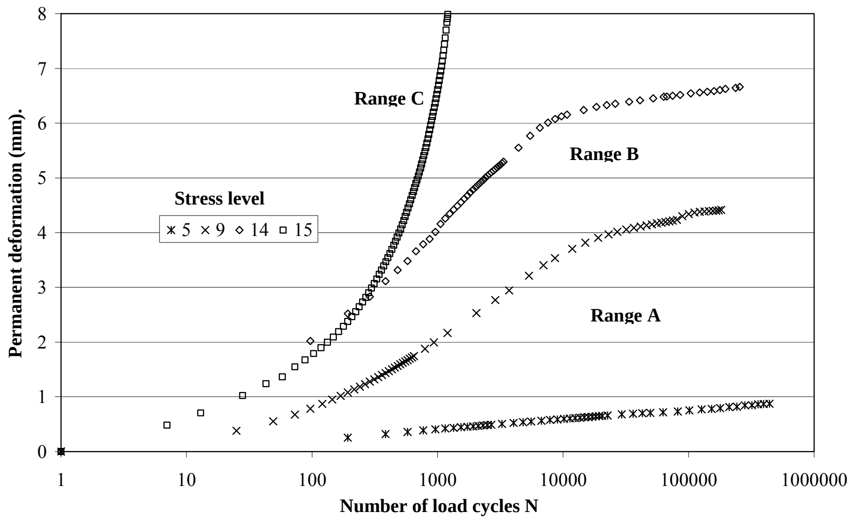

Figure 4: Permanent deformation versus number of load cycles (N).

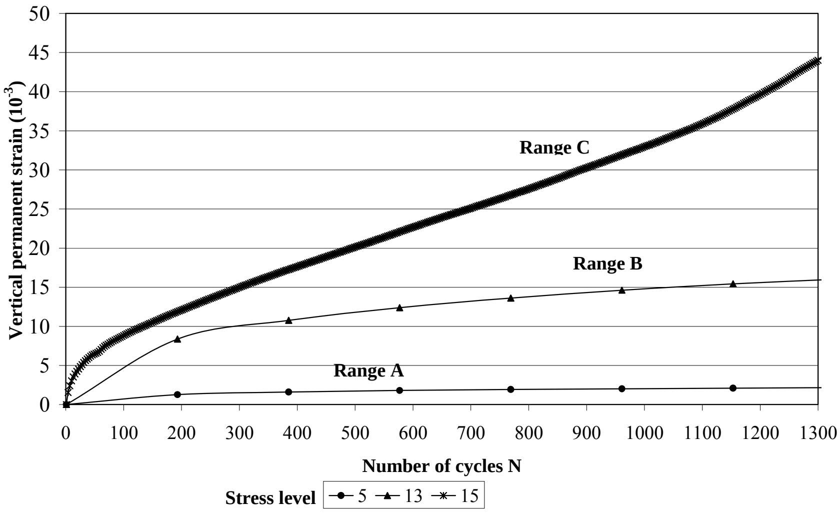

Figure 5: Ranges A, B and C permanent strains.

In this study the permanent deformation behaviors of CRB, normally used as a base course material, were investigated by RLT tests. It is generally acknowledged that the use of the shakedown concept application to UGMs in the pavement analysis is possible. The ranges defined in this study, A, B, and C, occur in CRB which under fixed stress level conditions shows a relationship between permanent strain and stress level. When a cyclic loading is applied, the sample responds by changing its permanent strain. In a continuous and gradual increase of the loading amplitude $\Delta\sigma$, the material will start by trying to change its mechanical behavior. The possibility of purely elastic approach in pavement analysis is also discarded as no purely elastic response is found in the CRB during repeated cyclic loading.

For low stress ratios, the CRB reacts corresponding to Range A at stress levels 5-9 in Figures 4. The behavior is entirely plastic for a number of cyclic load cycles but it reaches a stable state after a few cycles because some energy will have been dissipated due to viscosity. At this range of repeated loadings, the dissipated energy is independent of the loading and does not change from one cycle to another. The response becomes completely resilient and no further vertical permanent displacement occurs. The pavement will reach a shakedown limit after post-compaction deformation, with no further permanent deformation developing, and the vertical strain rate rapidly decreases. Hence Range A of CRB is accepted in pavement construction if the accumulated strains before the development of fully resilient behavior are sufficiently small. For this stage, CRB does not reach the failure state. The next step is to examine the application of material in the pavement that responds to Range B.

For higher loadings at stress levels 10-13, the energy input is first quickly dissipated by a re-arrangement of the sliding internal contacts of material, the so-called post-compaction. The dissipated energy per cycle relaxes then to a stationary value so that the vertical strain rate decreases to a constant rate depending on the loading, but also on the characteristics of the grains such as the friction or the stiffness of the contacts. A thorough investigation of the size dependence of the phenomenon would help to identify if the material is evolving on a much longer time scale to a final shakedown state in which all the energy supplied to the system is dissipated. This process may take longer in simulation than in the real experiment where more dissipative mechanisms exist. It seems that the material in Range B does not shakedown, rather it will fail at a very high number of load repetitions. Test results were observed that although the deformation is not completely resilient, permanent deformation is acceptable for the first period of the cycles. It is important to know the acceptable maximum number of load cycles that will prevent distress in the pavement from occurring. For many low-traffic road pavements where the total number of vehicles carried will be small and maintenance is ultimately required to correct inadequacies other than traffic-induced rutting, Range B behavior will probably be acceptable. A great number of failures could occur if the condition does not change and if it is maintained long enough, it deteriorates at the end as Range C. Range C behavior at stress level 15 should not be allowed to occur in the pavement. If the stress levels imposed are high enough, there is no possibility for material to re-arrange to the new state and post-compaction leads to an incremental collapse. Failure occurs with a relatively small number of load cycles when the cumulative permanent strain rate increases very rapidly after which the strain rate does not decrease again. The material is not able to dissipate enough energy without changing its configuration so it needs to modify its shape. CRB does not reach a stable state. Range C behavior in CRB would be result in the failure of the pavement by shear deformation in the base layer experienced as rutting at the road pavement surface. This range should not develop in a designed pavement standard.

Plastic strain should be used to predict whether or not a stable state occurs in the UGM layer of the road structure. It can be shown that the maximum stresses occurring in the pavement UGM are within carrying capacity. Based on pavement analysis, the approximate working stress of the road was level 11 at the CRB layer indicating that Range B behavior and possibly deterioration at a number of load repetitions. The new approach has been partially validated by the data from which those guidelines should be derived. It has been shown that the permanent strain characteristics of CRB could model each behavior range separately. Plastic strain predictions of UGMs were also presented in order to find the number of vehicle passes on the pavement using of RLT test results. Figure 3 shows the responses of plastic strain for CRB each stress ratio respectively. The limit range of CRB was stress level 14. Static failure criteria were used to determine the acceptable amount of permanent deformation in an UGM located at 2% based on triaxial shear tests as shown in Figure 6. This means that there is a strain of 2% relative to the suitable number of vehicles. Consequently, the number of vehicles that CRB is able to perform without any deterioration based on the limited range of material is shown in Figure 7.

This paper shows that having defined the strain ranges and strain predictions from laboratory results, it is possible to determine whether CRB is sufficient or whether other thicknesses of surfacing layer are required to implement satisfactory pavement performance.

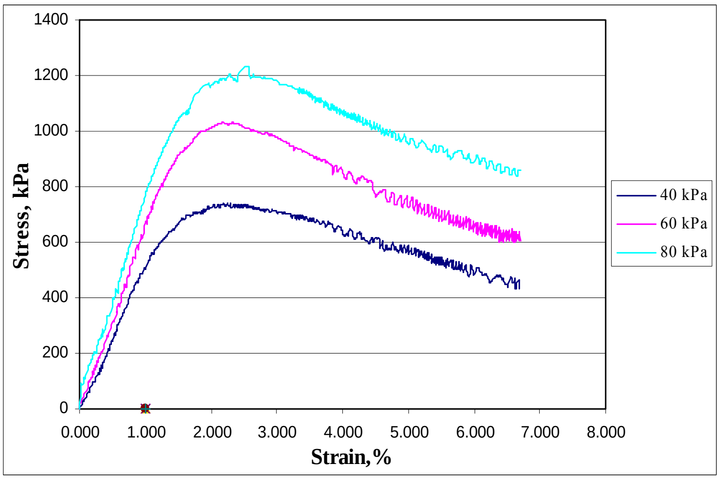

Figure 6: Triaxial shear test results.

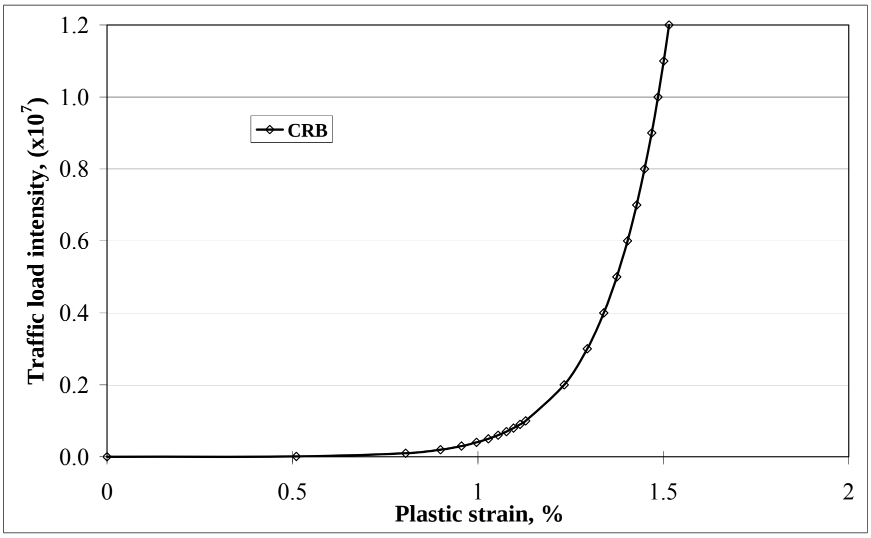

Figure 7: The amount of traffic compared with plastic strains.

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