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NOVEL APPROACH FOR CHARACTERIZATION OF UNBOUND MATERIALS

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Abstract

The current inclination towards establishing a mechanistic pavement analysis scheme to support the rehabilitation design will require the adoption of mechanical materials properties. Mechanistic properties are needed for establishing the material-mechanics link, which represents the most effective approach for accurately predicting the response of road structures and their performance. Physical characteristics served early design practices with a common understanding among users that a more robust approach is needed to effectively address rehabilitation design requirements.

This paper discusses current attempts to improve material characterization based on the outcome of mechanical tests focusing on unbound materials. The paper concluded that simplifying assumptions built in the proposed testing schemes and the manner that these properties are determined overlooked other critical behaviour indicators. Results of field and laboratory investigations highlight the need for capturing permanent deformation and the paper describes a more effective characterization technique for unbound material. Following this new approach, conventional resilient modulus and permanent deformation determinations were examined for a variety of native soils and processed material (crushed stones). Implementation of the new characterization technique in analytical models is also discussed.

Key Words**:** Unbound pavement materials characterization; resilient modulus; permanent deformation; cyclic plasticity; hardening; M_r–PD test

BACKGROUND

The Urban Road research group of the National Research Council Canada (NRC) conducted a three-year investigation with an objective to improve the characterization of unbound pavement materials. A number of field experiments were conducted to document current construction practice and to instrument road sections under rehabilitation. Collected data include stresses and strains, traffic and environmental data, and performance records. Typical data for three different materials supporting the subject of this paper are displayed in Figure 1. Records obtained for these materials provide a consistent trend where all tested sections show a relatively high rate of accumulation of permanent deformations for as long as one year followed by a slower steady rate. Deformation magnitude and rates are consistent with those established in the literature for these commonly used construction materials. These deformations take place during the initial years of service life after construction, which may lead road users to relate such performance to inadequate construction quality. Attempts to implement stage construction to offset the impact of this initial stage may also be a source for future damage when surface damage contributed by unbound layers is excessive, thereby rendering the simple overlay solution ineffective.

The complexity associated with the characterization of unbound materials forced reliance on a simplified elastic approach, which is often used to model the pavement layer in order to predict its contribution to performance. Although it represents a considerable move towards mechanistic analysis, the resilient parameters fell short of satisfying the requirements of the complete analysis that can lead to accurate performance prediction. The simplified approach being considered as a recent development in pavement design, such as the AASHTO 2002 Guide, may encourage continuation of the casual manner in which unbound layers are being handled in road design and analysis.

The difficulty in achieving an effective characterization technique seems to relate to the complexities associated with the dependency of the mechanical response on several factors:

- Physical characteristics, such as particle size distribution (gradation)
- Construction quality (density achieved by compaction)
- Local environmental conditions (moisture condition)
- Other intrinsic properties such as those associated with the mineralogical composition
- Magnitude of the applied load associated with traffic (stress dependency)

The AASHTO T292-91 (*1*) is a well-known mechanical test standard for characterizing unbound pavement materials. A review of the test procedure revealed the following results:

- The determined property, namely the resilient modulus (M_r), deals with the elastic response of the material and ignores the plastic component responsible for permanent deformation. This component of the material response is behind the majority of wheel-path rutting on roads today and deserves attention.
- The mechanical test considers the stress dependency of the determined property (15 load combinations) but limits tracing the response to a short time period (limited number of load cycles), and accordingly, does not capture the long-term trend critical to accurately characterizing the tested material.
- The repeated use of the sample ignores its loading history and the microstructural changes associated with it, which leads to non-unique values of the determined property.
- The current standard procedure includes a conditioning stage at the start of the test, which also results in microstructural changes and masks an important period in the simulated life of an unbound pavement layer. Most of the significant permanent deformation in roads occurs in the first one or two years of service life, which suggests that sample conditioning conducted after achieving the target density in the laboratory is misleading.

This paper presents the results of a laboratory investigation conducted at the National Research Council Canada (NRC) to improve the characterization of materials used to construct unbound road layers (granular base and subgrade).

LABORATORY INVESTIGATION

The NRC laboratory investigation pursued an approach aimed at encompassing the majority of components that form the overall unbound material response to external loading. Since the M_r parameter will still be needed in any mechanistic system of analysis and design, the test set-up described in the original AASHTO specification was retained. The sample preparation procedure and cyclic loading were also retained. Alternative features of the new approach pertaining to the test procedure, data collection and reduction were as follows:

- Abandoned the AASHTO initial conditioning period by capturing test results from that stage and treated them as part of the characteristic response of the material that must be considered in pavement analysis.
- Adopted a single axial load value in each test corresponding to a single stress level transmitted from the road surface. Different samples were tested using different axial loads corresponding to different stress levels applied to the road structure to account for the fact that vehicles with different weights use the road and also that unbound layers are located at different positions (depths) within the road.
- Determined the response of the material under a wide spectrum of anticipated physical conditions of compaction density and moisture by testing different samples prepared at these conditions.
- Collected data dealing with both elastic and plastic material responses. The list of parameters included:
  - Axial deviator stress
  - Axial deformation (both elastic and plastic components)
  - Confining pressure
- Implemented a data collection scheme that captures the overall response of the material at different stages of the load application.

During the current laboratory investigation, more than 600 samples of different unbound materials were tested. The database included a vast array of cohesive soils, sands and processed granular materials. Test results obtained are discussed in the following sections.

Discussion of experimental results

The outcome of the mechanical test displayed in the form of a stress–strain relationship is depicted in Figure 2. The peaks of the cyclic curve represent the accumulated total strain, ε_t, contributed by all loading cycles while the line joining the ends of the load cycles identifies the accumulated permanent strain, ε_p. The slope of the line joining the total and permanent strains for each cycle represents the magnitude of the resilient modulus parameter, M_r.

Segregation of the data into resilient and permanent material characteristics revealed interesting results. Typical results are displayed in Figures 3, 4 and 5 for crushed stone, sand and clay, respectively. The dotted lines in these figures reflect changes in M_r with an increase in the number of load cycles. The accumulation of permanent deformation with load repetitions is shown as solid lines. The M_r for crushed stone increased during the initial stage (Figure 3), reflecting the impact of microstructural changes related to aggregate particle rearrangement under the action of external loading. Flattening of the curve afterward shows a relatively constant M_r value consistent with the steady state that follows the establishment of a preferred particle orientation with no further microstructural changes. Values for sand and clay (Figures 4 and 5) show a decreasing trend at a relatively rapid rate. It is clear that all soil types experience changes in the resilient modulus at the initial stage and hence characterization in support of the analysis should account for these changes to accurately predict the response of pavement layers constructed with these commonly used materials.

Permanent deformation (PD) trends depicted in Figures 3, 4 and 5 are consistent with those established in the field (see Figure 1) based on data collected from sensors installed in layers constructed with typical materials. The tested materials initially experienced considerable permanent deformation with a lesser rate of accumulation afterward. Based on the rate of accumulation of permanent deformation, there are two unique zones that can be defined for the material at the early stage that follows construction/rehabilitation. There are differences, however, among the three material types tested in this study regarding the duration of these zones. The transition from one rate to the other for crushed stone was faster than for clay, which indicates that microstructural changes take place over a longer period for the latter material. The sand material, on the other hand, experienced a long period of microstructural changes, with no evidence of the steady state stage being reached. Permanent deformation in the sand material exhibited the highest magnitude, followed by clay and crushed stone. The PD results shown in Figures 3, 4 and 5 conform to the Shakedown behaviour stated in the theory of plasticity (*2 – 4*). In the first few loading cycles, plastic deformations develop at an accelerated rate due to changes in the microstructure and the material hardens. As this process continues, a system of residual stresses is generated in the material such that subsequent unloading and reloading always brings the state of stress back to this initial state. Consequently, permanent deformation ceases to develop further and accumulated dissipated energy in the whole material remains bounded such that the material responds purely elastically to the applied load. The observation noted earlier that the M_r parameter attains a constant value during the steady state supports this analysis. It is worthwhile mentioning that both the crushed stone and clay materials reached a hardening state. However, the sand material seems to continue accumulating permanent strain indefinitely. This phenomenon, which is called ratchetting, may ultimately cause the material to collapse under the constant applied stress.

According to the results of the laboratory investigation, a number of factors, other than material type, influence the rate of accumulation of permanent deformations. Among these is the percentage of the fractions that constitute the material gradation, especially fines content passing sieve #200 (75µm). The moisture condition also influences the rate of deformation. These factors are discussed in the following sections because of the wide variety of construction materials used in current practices including gradation, differences in targeted construction quality (density), traffic characteristics and exposure conditions associated with the environment (moisture).

Typical changes in M_r with loading (52 kPa), using results of M_r – PD tests conducted at the same moisture content (2% dry of optimum) on a granular material with different percentages of fines (7, 9, and 12% fraction passing sieve #200), are shown in Figure 6. The trend was similar to that discussed earlier in Figure 3. Higher M_r values were measured for the granular materials with the higher fines content. The M_r increased 30% when the fines content increased from 9% to 12%. Similarly, those materials with high fines content showed less permanent strain as shown in Figure 7. However, when the results of the test conducted at moisture content dry of optimum were compared with those conducted at optimum moisture condition, the granular material with 12% fines content exhibited extremely high permanent deformation potential (Figure 8), hence supporting current specifications that call for controlling this parameter in regions where the road base is exposed to fair or high moisture content levels.

The impact of fines content in the granular material discussed above relates to the sensitivity of clay size material fraction to moisture. This factor was evaluated further in this study by testing native clays. Typical results can be seen in Figure 9.a and 9.b for a clay tested at optimum moisture content and at 2.5% above optimum condition. The results highlighted the significant difference in the outcome of the evaluation between what has been established by the M_r and the permanent strain criteria. The impact was barely noticeable within the M_r results while a substantial impact was detected on this moisture-sensitive material based on the determined permanent deformation potential.

The ability of the test to effectively satisfy performance prediction requirements can be demonstrated by the results of a M_r – PD test performed on two clay types with uniquely different field performance records. The M_r values determined according to the AASHTO T292-91 test shown in Figures 10.a and 11.a may suggest that these are two clays that could be classified as similar materials. Changing the lateral confining pressure level produced no effect on the M_r trend in either clay. However, extending the evaluation process by tracing permanent deformation effectively detected known differences between these two materials as shown in Figures 10.b and 11.b. The obtained relationships demonstrated the sensitivity of Clay Type 1 to applied pressure (50% increase in permanent strain) at test moisture condition (2% wet of optimum), while Clay Type 2 remained unaffected by pressure change. The results of similar tests conducted at moisture condition dry of optimum failed to highlight the differences between these two clay types.

IMPLEMENTATION IN DESIGN AND ANALYSIS MODELS

The characteristic stress–strain response curve shown earlier in Figure 1, was redrawn to highlight the unique features that the material exhibits during cyclic loading. This is illustrated schematically in Figure 12. Cyclic hardening behaviour of the material is clearly evident during the first few loading cycles. This behaviour is attributed to microstructural changes taking place within the test specimen. During this stage, much of the energy transmitted via the cyclic load is used to rearrange the particles into their preferred orientations. This stage is characterized by non-linear irrecoverable deformation. Although the authors in this research did not perform experimental tests to quantify microstructural changes (e.g., via digital image analysis) before and after the M_r – PD test, speculations given here regarding the relationship between the microstructural evolution and the mechanical response are supported by the work of other researchers reported in the literature (*5* and *6*). Zeghal and Mohamed, for example, arrived at a similar conclusion using the method of Discrete Element Modeling. Upon attaining the most preferred particle orientations, the test specimen densifies and the strain-hardening phenomenon ceases. This stage, referred to as the zone of steady state, is characterized by a significant reduction in the rate of permanent deformation. During this stage, the material behaves virtually elastically.

The regions of material behaviour described above were found to be common to all unbound materials irrespective of their type (whether cohesive soils or granular). However, the duration of each of the deformational stages was found to be dependent on the material physical conditions and the level of applied deviator stress. This is depicted in the examples shown earlier in Figures 3 – 5.

It should be noted that the residual stress, shown in Figure 12 as curve OAC, is a test-induced condition, not a material response. The objective of applying this stress, which constitutes 10% of the magnitude of the repetitive deviator stress, is to provide a static load on top of the test specimen to avoid impact effects during the cyclic test.

To be able to implement the new characterization technique in modeling, the concepts of cyclic plasticity theory (*7 – 10*) were used to model the stress–strain response of Figure 12. This approach, which is recently being advocated by a number of researchers (*11 – 13*), can effectively be used to analyze pavement structures. The task involved postulating three relationships that uniquely describe all stages of material behaviour observed. The first relationship was a function that defined the strain-hardening phenomenon. This function is of the following form:

$$\sigma_d = \sqrt{a + b\frac{\ln \varepsilon_t}{\varepsilon_t^2}} \qquad [1]$$

where, σ_d = applied deviator stress, kPa

ε_t = percent total strain accumulated at cycle j, and j = 1, 2, 3, …

a and b = material parameters determined experimentally

The second relationship was a function that relates the permanent strain component to the total strain. This function is used to decompose the total strain into its permanent and elastic components, thus enabling the modeling of the elasto-plastic response of the material. The relationship is of the following form:

$$\varepsilon_p = c\varepsilon_t^d \qquad [2]$$

where, ε_p = percent permanent strain accumulated up to cycle j, and j = 1, 2, 3, …

ε_t = percent total strain accumulated up to cycle j, and j = 1, 2, 3, …

c and d = material parameters determined experimentally

The third relationship describes the evolution of the resilient modulus parameter as a function of the elastic strain. This relationship is of the following form:

$$M_r = f \exp\left\{-\frac{\varepsilon_e}{g}\right\} \qquad [3]$$

where, M_r = resilient modulus, MPa

ε_e = percent elastic strain at cycle j, and $\varepsilon_e = \varepsilon_t - \varepsilon_p$

f and g = material parameters determined experimentally

The above constitutive model, comprising equations 1 through 3, had been implemented in a finite element (FE) program to predict unbound material behaviour. Further details pertaining to the FE model and the verification of the characterization scheme to describe material behaviour will be the subject of a forthcoming paper.

CONCLUSIONS AND RECOMMENDATIONS

The proposal to consider permanent deformation criteria for unbound material characterization is not intended to complicate pavement analysis but rather to improve the accuracy of the prediction of performance. Measures that may facilitate implementation are already under development; including the establishment of a material library covering a wide spectrum of commonly used unbound materials. This step will encourage users to adopt the new characterization technique and a universal formula will be established in the near future. The conclusions from this paper can be summarized as:

- Characterization of unbound pavement materials discussed in light of the results of field investigations and a proposal for a new characterization technique to examine the possibility of capturing permanent deformation potential was given.
- A test procedure for combined determination of the resilient modulus and permanent deformation potential (M_r – PD), involving both elastic and plastic responses, was established.
- Unlike other standard methods for measuring the resilient modulus, the proposed M_r – PD test does not specify any limits for the amount of accumulated strain. In the authors' opinion, there should be no pre-set limits on the

amount of accumulated permanent strain as long as the sequence of microstructural changes occurs first, followed by the condition of steady state.

- Sensitivity of the proposed characterization technique to unbound materials' issues (such as gradation, moisture content, density and loading conditions) was examined and proved effective.
- A scheme for implementing the new characterization technique was discussed and a system to facilitate incorporation of the test output in analytical models was presented.

The analytical model prepared to accommodate the proposed characterization technique is currently under evaluation. A future publication will present the outcome of the model validation exercise.

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LIST OF FIGURES

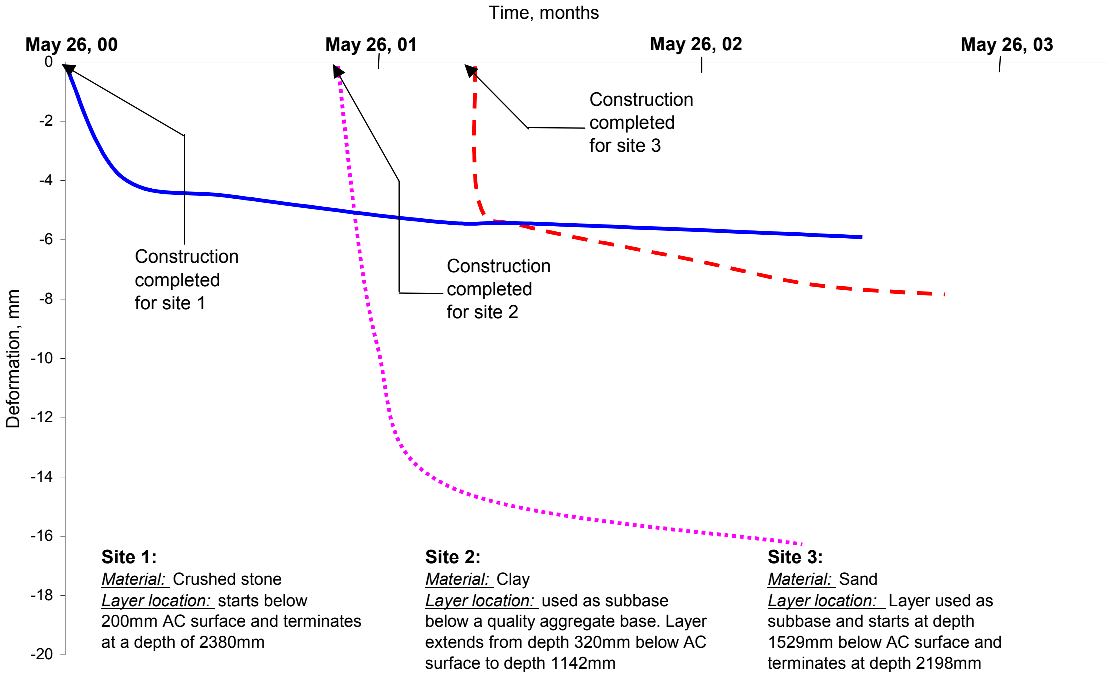

FIGURE 1 Unbound material performance from three field experiments.

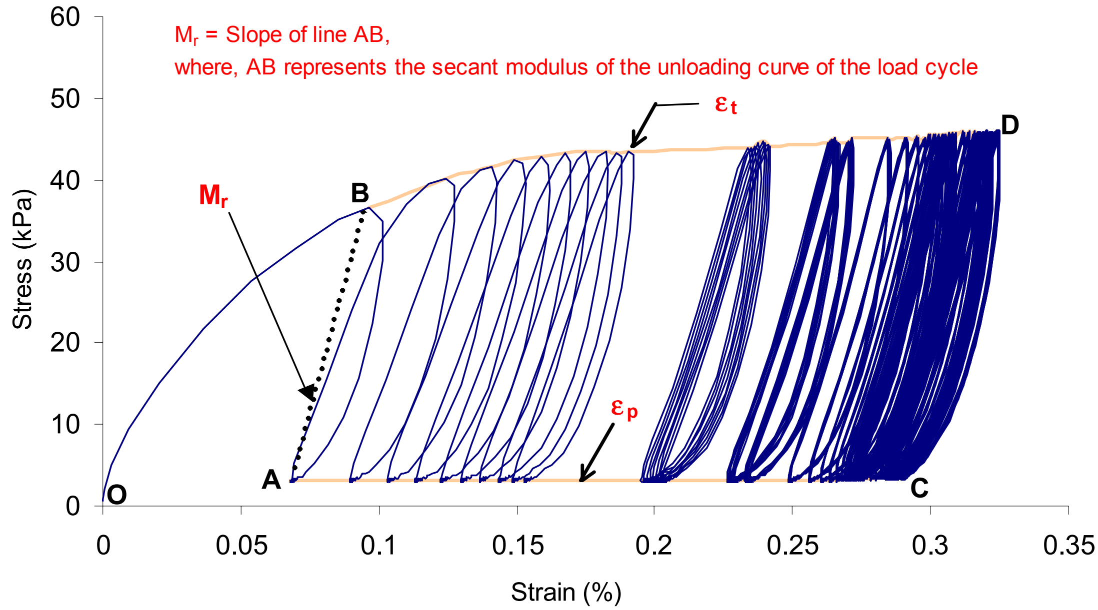

FIGURE 2 Characteristic response of unbound material.

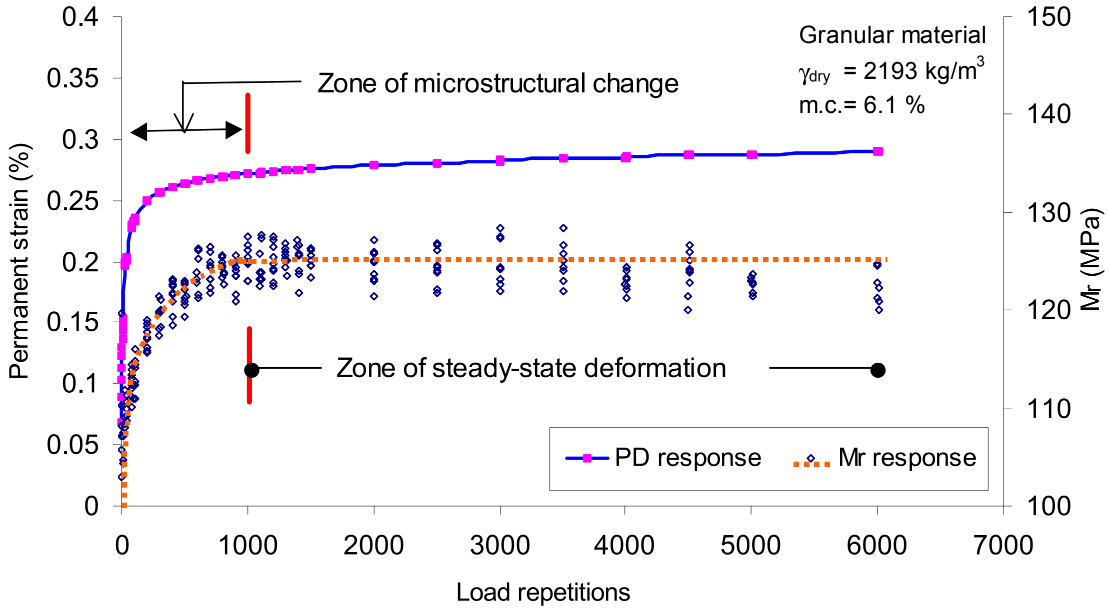

FIGURE 3 Variation in M_r and PD responses for crushed stone.

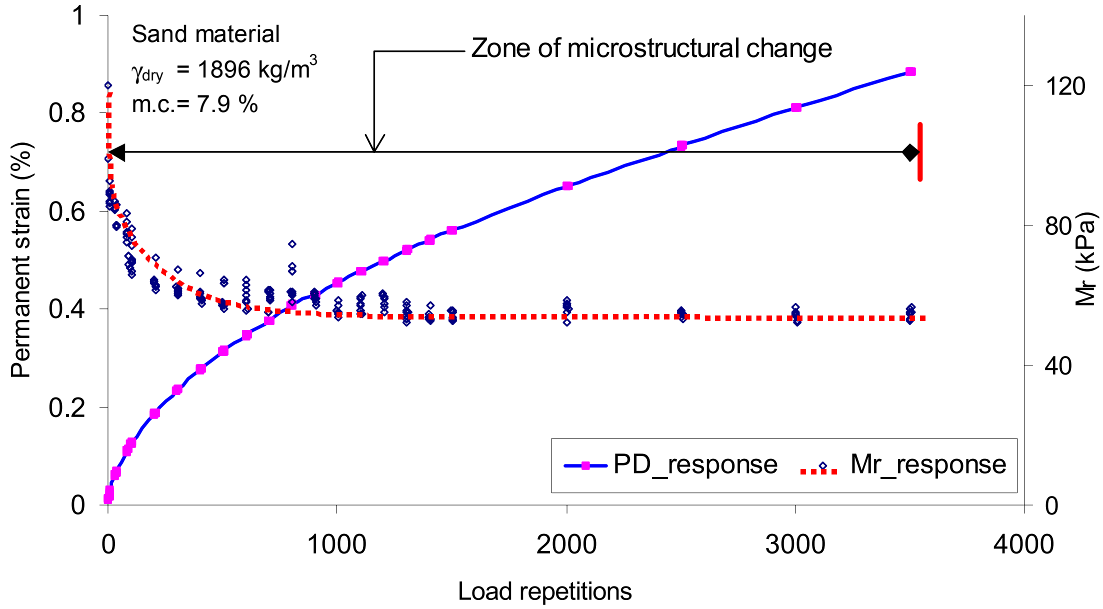

FIGURE 4 Variation in M_r and PD responses for sand.

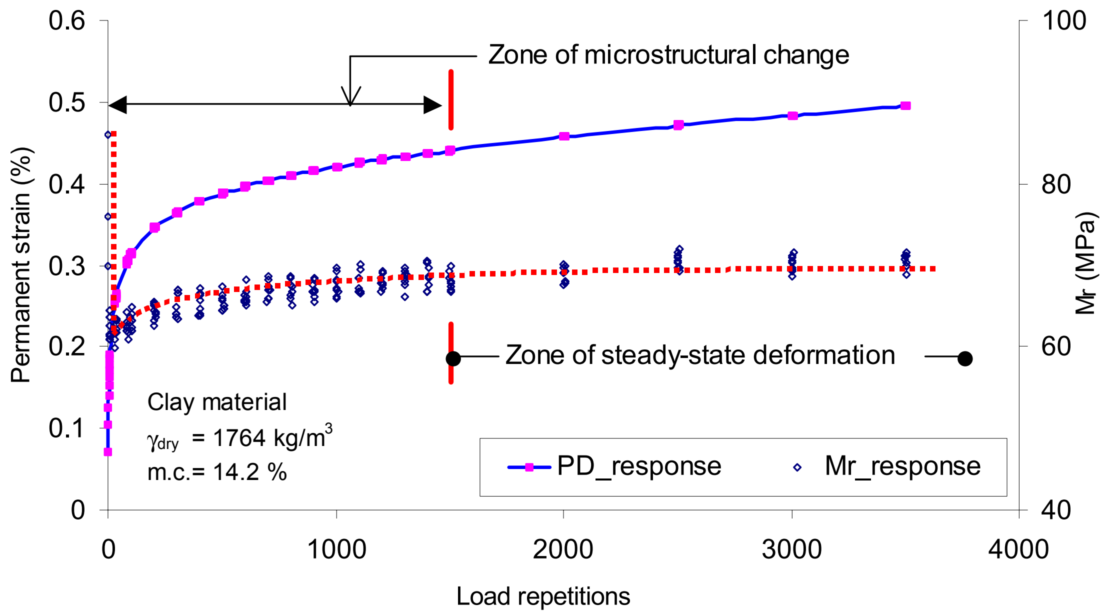

FIGURE 5 Variation in M_r and PD responses for clay.

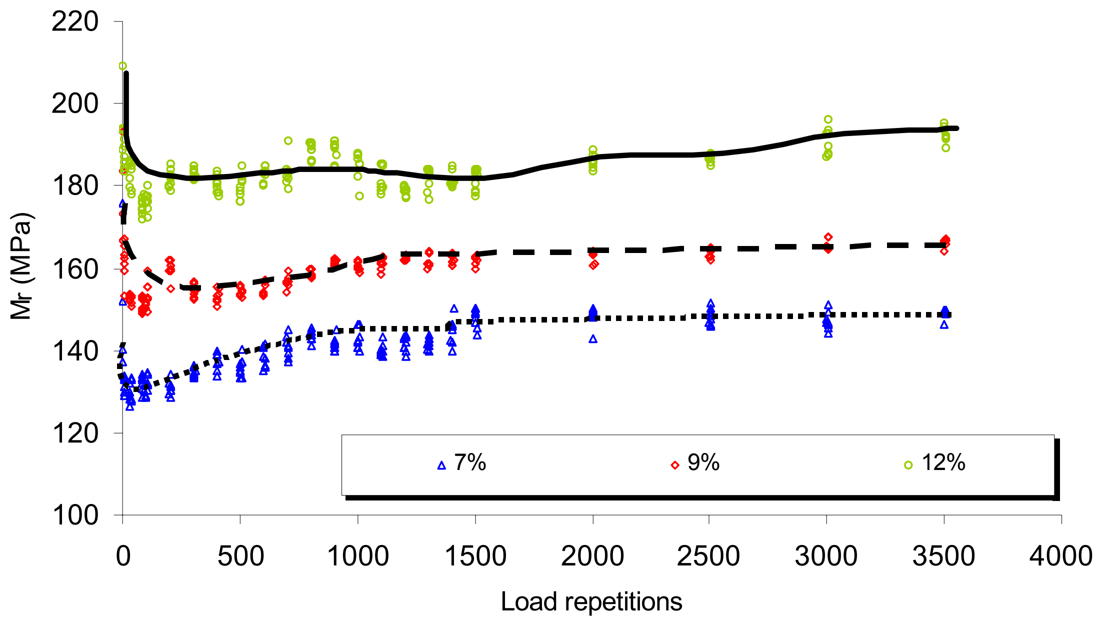

FIGURE 6 Variation in M_r response for granular material with different percentage fines.

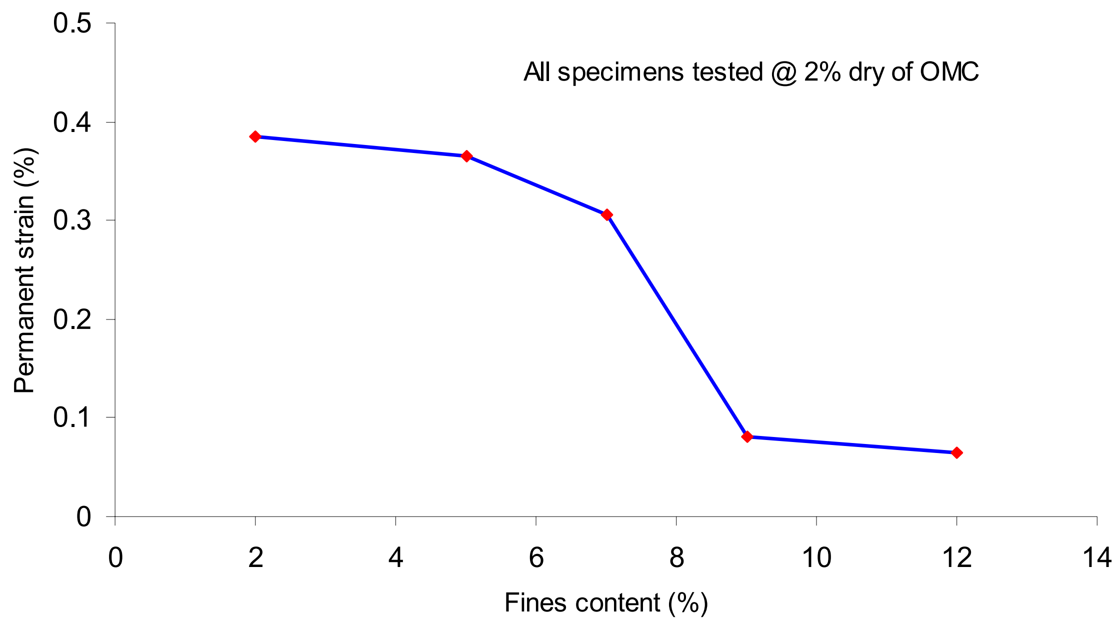

FIGURE 7 Effect of fines content on PD response for typical granular material.

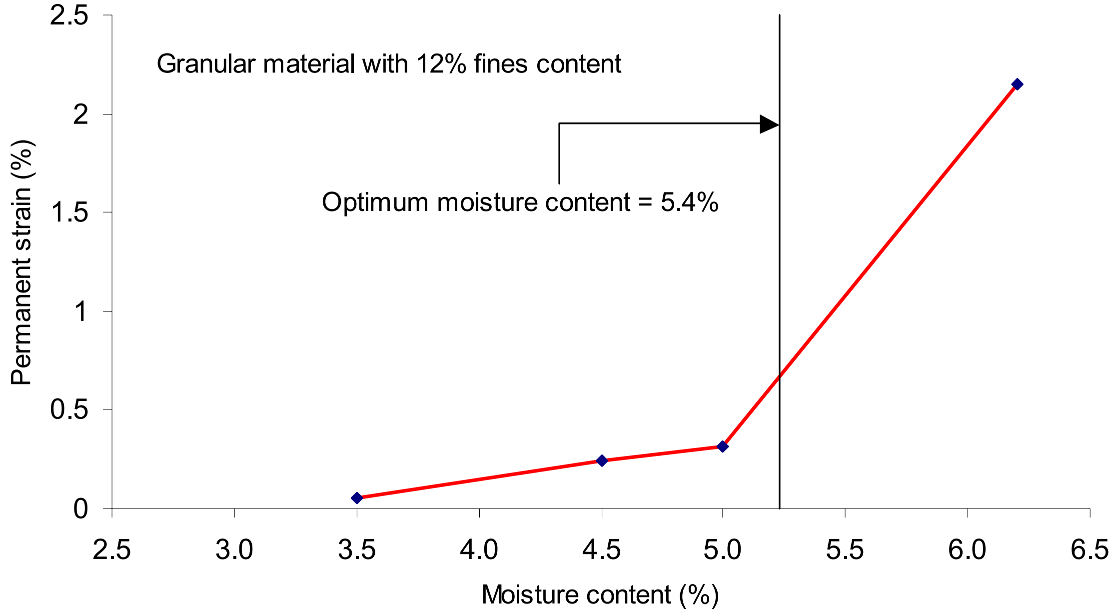

FIGURE 8 Combined effects of moisture and fines content on PD response.

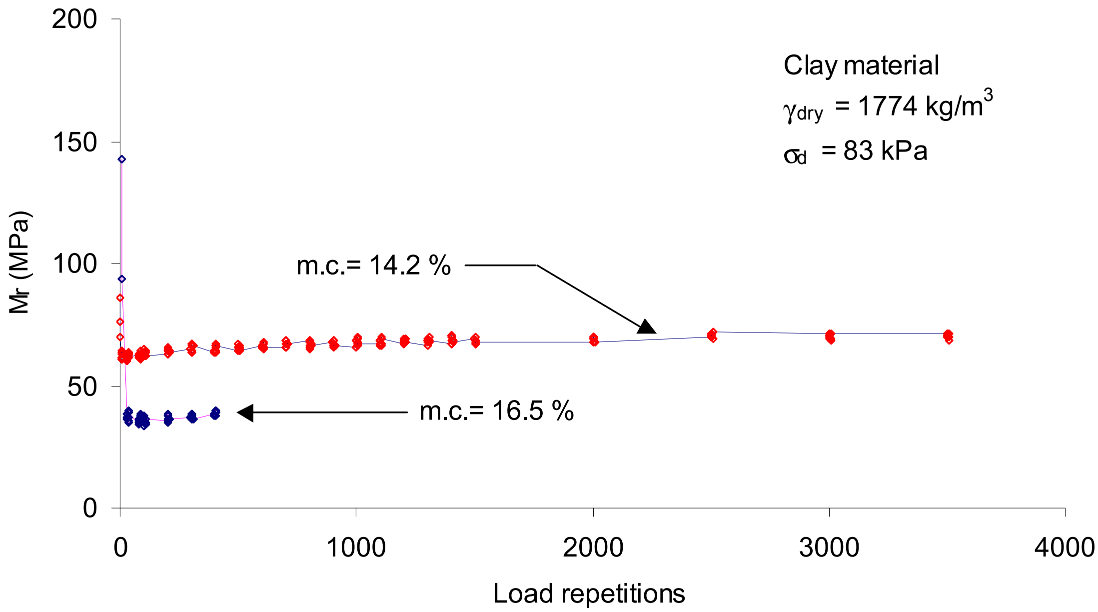

FIGURE 9(a) Effect of moisture content on M_r response for clay.

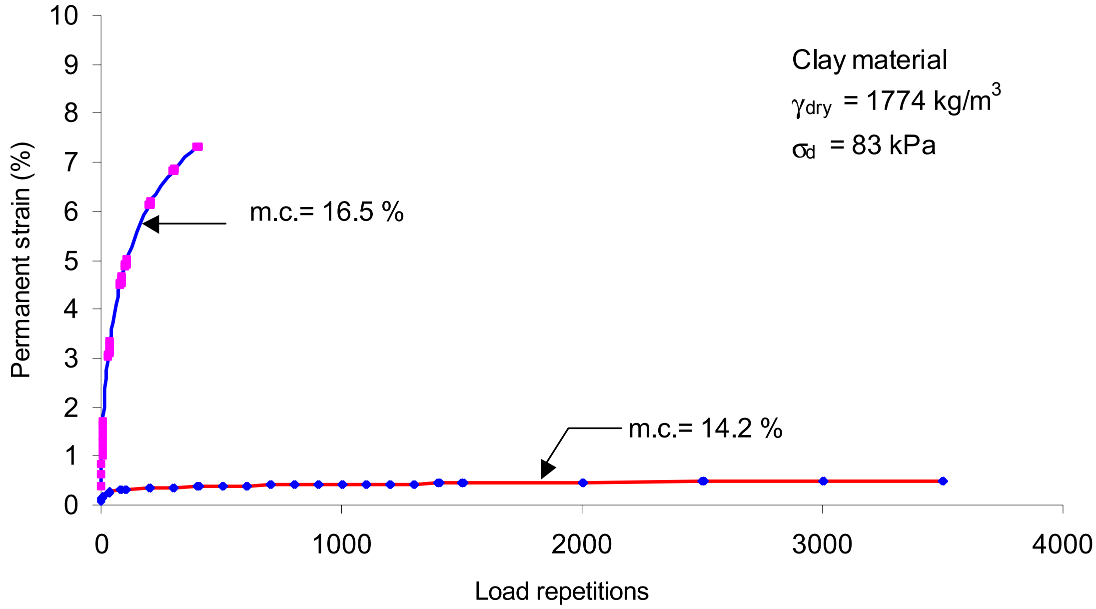

FIGURE 9(b) Effect of moisture content on PD response for clay.

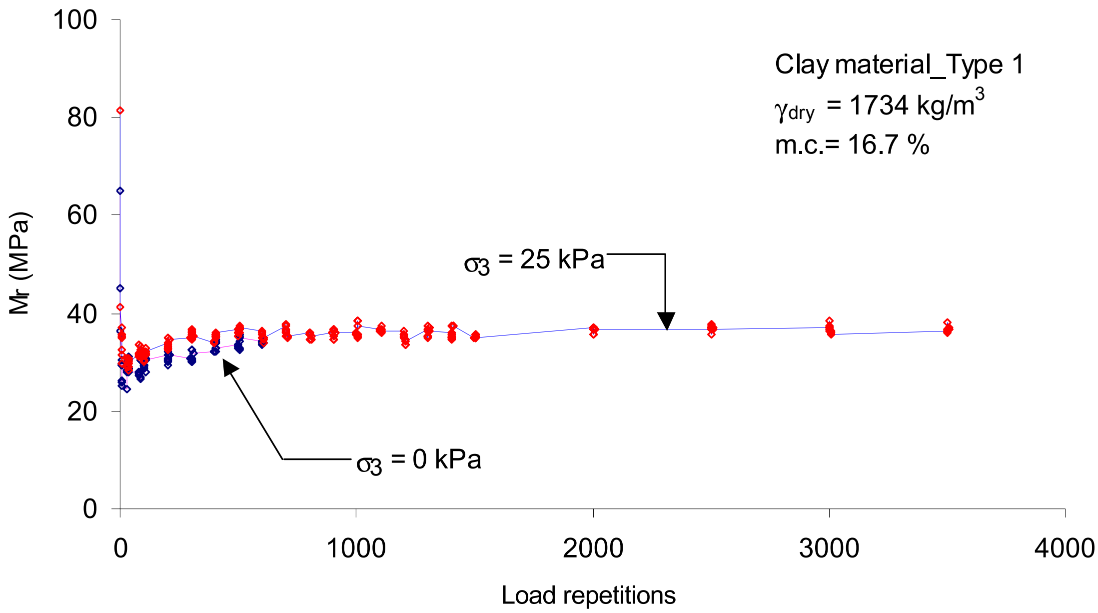

FIGURE 10(a) Effect of confining pressure on M_r response of a sensitive clay.

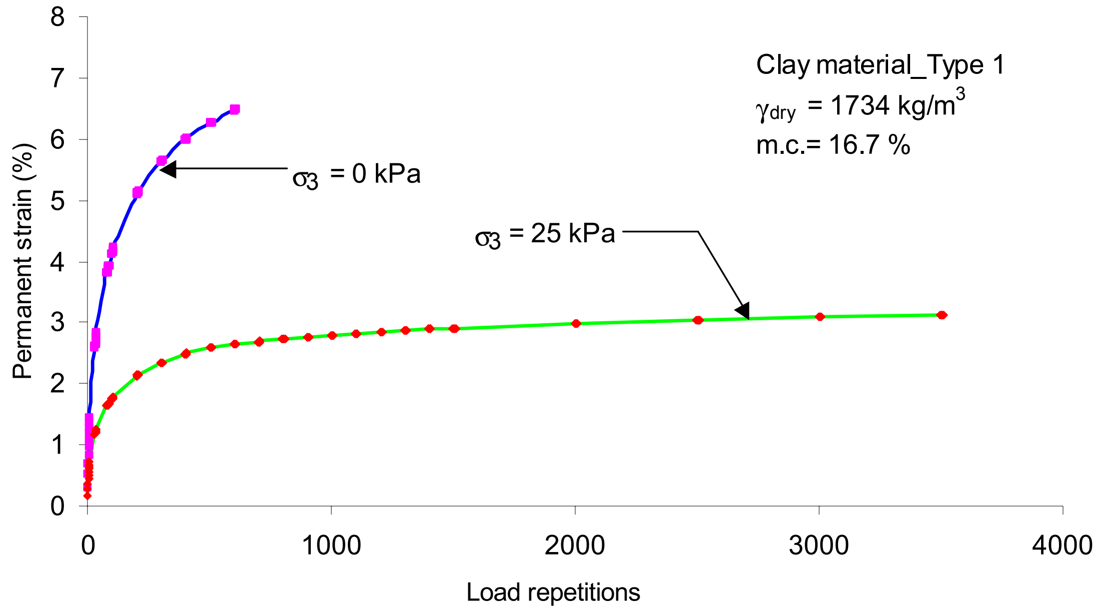

FIGURE 10(b) Effect of confining pressure on PD response of a sensitive clay.

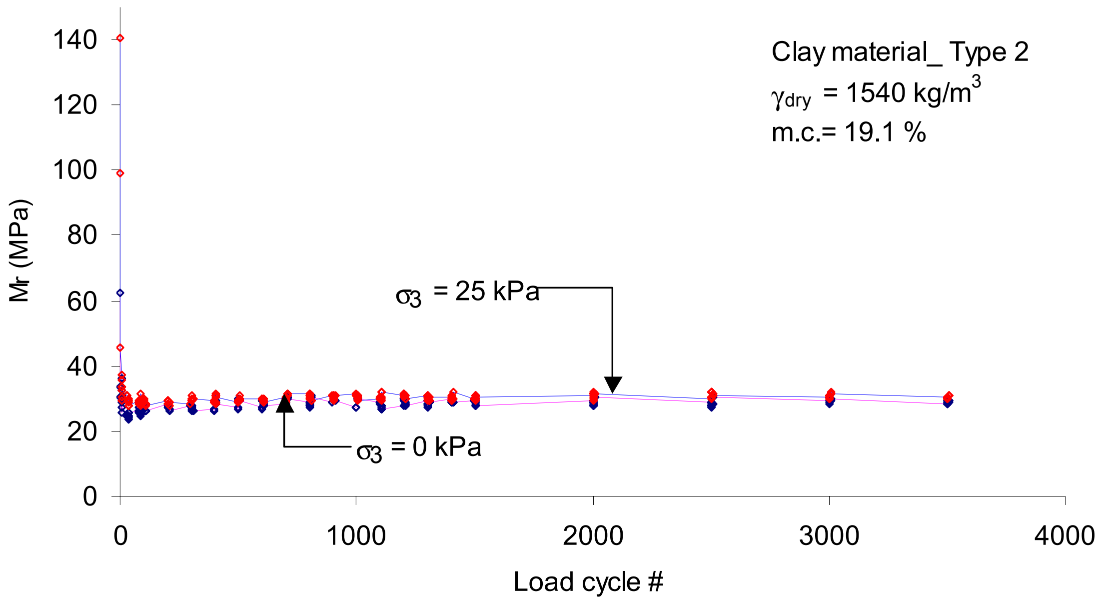

FIGURE 11(a) Effect of confining pressure on M_r response of insensitive clay.

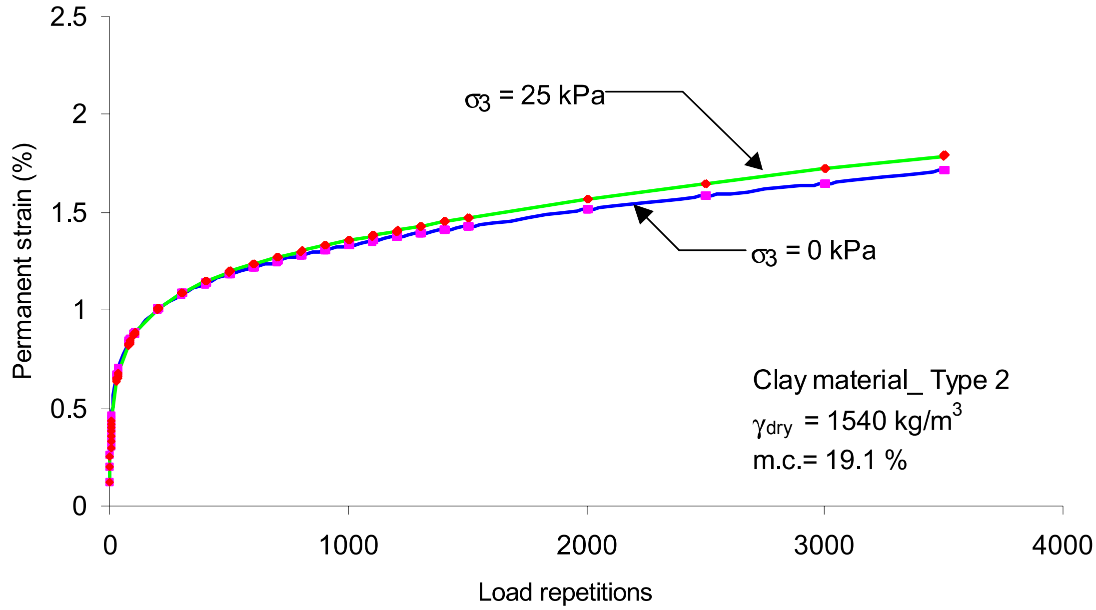

FIGURE 11(b) Effect of confining pressure on PD response of insensitive clay.

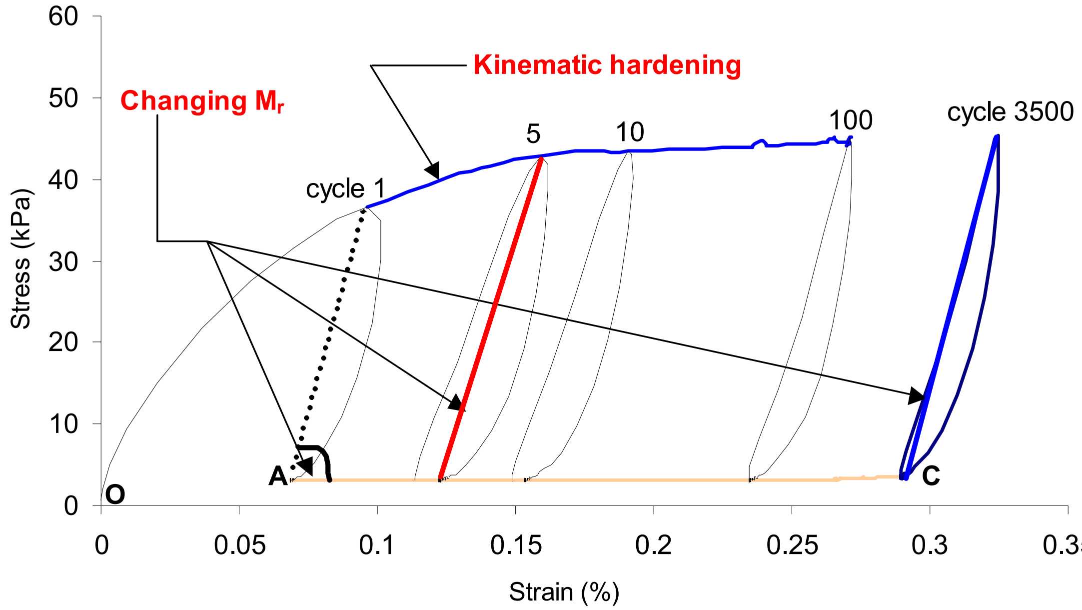

FIGURE 12 Schematic illustration of the stress–strain response showing strain–hardening phenomenon.